REPORT

ON

PROPOSED SUPPLEMENTAL TUNNEL AND AQUEDUCT

AND

ADDITIONAL RAPID SAND FILTERS

FOR THE

PROVIDENCE WATER SUPPLY SYSTEM

AUGUST 1964

CHAR[illegible]

E N [illegible] I N E E R S

BOSTON [illegible] PROVIDENCE [illegible]

The City of Providence

STATE OF RHODE ISLAND AND PROVIDENCE PLANTATIONS

CHAPTER 1649

No. 567 AN ORDINANCE REQUESTING THE SECRETARY OF STATE TO SUBMIT QUESTION TO THE VOTERS AT THE NEXT GENERAL ELECTION TO BE HELD ON THE TUESDAY NEXT AFTER THE FIRST MONDAY IN NOVEMBER, A. D. 1964.

Approved September 23, 1964

Be it ordained by the City of Providence:

Section 1. The Secretary of State is hereby requested to submit the following question to the qualified electors of the city of Providence at the general election to be held in the city of Providence on the Tuesday next after the first Monday in November, A. D. 1964, namely:

> Shall the city of Providence be authorized to issue bonds in the sum not exceeding Thirteen Million ($13,000,000) Dollars for the construction of major improvements to the Providence water supply system, including a new tunnel and aqueduct, additional filters and incidental construction necessary in connection therewith?

Sec. 2. The city clerk is directed to transmit to the Secretary of State forthwith upon the passage of this Ordinance, a certified copy thereof.

Sec. 3. This Ordinance shall take effect upon its passage and all Ordinances and Resolutions and parts thereof inconsistent herewith are hereby repealed.

IN CITY COUNCIL
SEP 3 1964
First Reading Read and Passed Back
Referred to Committee on
FINANCE
Vincent Vespia
Clerk

IN CITY COUNCIL
SEP 2 2 1964
FINAL READING
READ AND PASSED
[signature]
ACTING PRESIDENT
Vincent Vespia
CLERK

APPROVED
SEP 2 3 1964
[signature]
MAYOR

No.

CHAPTER

AN ORDINANCE requesting the Secretary of State to submit question to the voters at the next general election to be held on the Tuesday next after the first Monday in November, A. D. 1964

THE COMMITTEE ON
Finance
Approves Passage of
The Within Ordinance
Vincent Vespia
8-26-64
~~Chairman~~ Clerk

THE COMMITTEE ON
Finance
Approves Passage of
The Within Ordinance
Vincent Vespia
9-10-64
~~Chairman~~ Clerk

Mr. Wexler, by request

THE CITY OF PROVIDENCE

WATER SUPPLY BOARD

552 ACADEMY AVENUE
PROVIDENCE 8, R. I.

JOHN A. DOHERTY, CHAIRMAN
EARL H. ASHLEY
UGO RICCIO
JOHN J. TIERNEY
DAVID R. McGOVERN, EX-OFFICIO

PHILIP J. HOLTON, JR.
CHIEF ENGINEER
WILLIAM I. McDONALD
DEPUTY CHIEF ENGINEER
JOHN T. WALSH
LEGAL ADVISOR
JOHN J. DEARY
SECRETARY

August 21, 1964

Mr. Vincent Vespia
City Clerk
City Hall
Providence, Rhode Island

Dear Mr. Vespia:

I am enclosing herewith copy of the report received from Charles A. Maguire and Associates on Wednesday, August 19, 1964 regarding the Proposed Tunnel and Aqueduct and Additional Rapid Sand Filters for the Providence Water Supply System together with a copy of our letter to Mayor Walter H. Reynolds dated August 21, 1964 and also a letter addressed to Councilman O'Connor which is typical of the letter forwarded to all members of the City Council.

At the meeting of the Water Supply Board held today, it was voted to submit to the City Council for approval an ordinance requesting the Secretary of State to submit a question to the voters at the next general election seeking approval of this major project. This ordinance has been submitted to your office by our Legal Advisor, Mr. John T. Walsh, for introduction to the City Council.

Yours very truly,

Philip J. Holton, Jr.
Chief Engineer

PJH:kam

Enclosure

... Reynolds
... Rhode Island

... Reynolds:

... enclosing report ... Maguire and Associates, ... Engineers, entitled "Propo... and Aqueduct and Additional Rapid ... Filters for the Providence ... System". This engineering study ... the result of our letter ... ed May 12, 1964 in which we enclosed ... of our report describing "Ma... ditions and Reinforcements Required to Meet Future Demands ... the Provi... Water Works". On Thursday, May 21, 1964, the City Council passed a ... transferring $27,000.00 from the Water Depreciation and Extensio... as "Hydraulic Studies of Present and Pro... the meeting of the Board of Contract and S... engage Charles A. Maguire and Associates t... un...

The report recommends ... al tunnel and aqueduct along with four a... According to the report, construction should ... proposed tunnel and aqueduct will be designed to convey ... gallons daily from the plant in Scituate to a ... of West Warwick where it will eventually conver... proposed Big and Wood River development. From ... be either in tunnel or cut and cover aqueduct and ... the City of Cranston. This portion wi... carry 100 million gallons daily; 50 million gallons daily ... million gallons daily from Big and Wood River.

It will require betwee... construct these major improvements and by that ... reached the full capacity of our existing tunnel and aqued... 191... to deliver 10... gallons daily. In that ye... consumption was only 17.... million gallons and our ma... Contrast the... period with the new maxim... gallons ... on June ... 1963. If the C... connected to our system during the period, the max... have been ...

[illegible] Wo[illegible] [illegible] [illegible] p[illegible] [illegible] same [illegible] [illegible] with [illegible] supply [illegible] [illegible] loads [illegible] [illegible] well [illegible] [illegible] when [illegible] communi[illegible] [illegible]

[illegible] Engine[illegible] the [illegible] of the [illegible]. This [illegible] will have [illegible] effect [illegible] as they [illegible] financed [illegible] ments [illegible] out by the [illegible]

[illegible] the [illegible] ply Board [illegible] the [illegible] s[illegible] the [illegible] ting th[illegible] etary of Sta[illegible] to submit [illegible] ele[illegible] of the [illegible] said Cit[illegible] on Tuesday [illegible]

[illegible] Providence be authorized [illegible] issue bonds [illegible] $13,000,000.00 for the Construction of Major [illegible] the Prov[illegible] Water Works including New [illegible] and Additio[illegible] ration Capacity.

Sincerely [illegible],

[illegible signature]

Philip J. [illegible]
Chief Engineer

[illegible]

[illegible] Board

CITY OF PROVIDENCE - RHODE ISLAND - Walter H. Reynolds, Mayor

DEPARTMENT OF CITY CLERK

CITY HALL

Vincent Vespia
City Clerk

William H. Matthews
First Deputy
Dorothy K. McGinn
Second Deputy

Received Wednesday, September 23, 1964 of Vincent Vespia, City Clerk of the City of Providence, Rhode Island a duly certified copy of Chapter 1649, approved September 23, 1964 being An Ordinance requesting the Secretary of State to Submit Question to the Voters at the Next General Election to be held on the Tuesday Next After the First Monday in November, A. D. 1964 for the following:

> "Shall the city of Providence be authorized to issue bonds in the sum not exceeding Thirteen Million ($13,000,000) Dollars for the construction of major improvements to the Providence water supply system, including a new tunnel and aqueduct, additional filters and incidental construction necessary in connection therewith?

[signature]
1st Dep. Sec. State

REPORT

ON

PROPOSED SUPPLEMENTAL TUNNEL AND AQUEDUCT

AND

ADDITIONAL RAPID SAND FILTERS

FOR THE

PROVIDENCE WATER SUPPLY SYSTEM

AUGUST 1964

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE - BOSTON - HARTFORD

CHARLES A. MAGUIRE & ASSOCIATES
E N G I N E E R S

PARTNERS
HOWARD W. HOLMES
MILTON E. NELSON
HAROLD BATESON
GORDON BRONSON

ASSOCIATES
ERIC REEVES
THOMAS C. MURPHY
ROBERT L. PARE
FRANCIS C. PIERCE

CONSULTANTS
WENDELL S. BROWN
EDWARD O. GREENE
HENRY J. FITZPATRICK
DR. HENRY M. PAYNTER
PROF. HENRY CAMPBELL
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TELEPHONE AREA 617 742-0355

530 SILAS DEANE HIGHWAY
WETHERSFIELD, CONNECTICUT 06109
TELEPHONE AREA 203 529-8639

August 12, 1964

REPLY TO: Providence

Water Supply Board
City of Providence
552 Academy Avenue
Providence, Rhode Island

Gentlemen:

At the present time Providence depends entirely upon a single tunnel or aqueduct to convey the water from the Purification Works in Scituate to the Siphon Chamber in Cranston where it connects to the two primary feeders that supply the distribution system. The tunnel and aqueduct was designed in 1916 and is capable of delivering 100 million gallons daily into the district. At the time this main transmission line was designed, the average daily consumption was 17.75 million gallons and the maximum daily demand was only 22.92 million gallons. This is an outstanding example of excellent long-range planning as it has supplied the City with adequate quantities of water to meet all demands over a long period of years.

The record maximum daily demand in the Providence system was established on June 27, 1963 when the daily consumption reached 87.215 million gallons. Had East Providence been connected into the system at that time, which is now permitted under the amended Act, the maximum daily demand would have been in excess of 95 million gallons. According to these actual facts, Providence is rapidly approaching the maximum limits of the tunnel and aqueduct and in order to continue to provide ample water to meet all the requirements of residential, commercial, industrial and fire protection demands, it is urgent that the City proceed to construct a supplementary tunnel and aqueduct.

Failure to start construction within the immediate future will result in placing restrictions on the use of water throughout the whole water district whenever periods of drought occur. It would be a severe handicap to the economic life of Providence if manufacturers employing thousands of people as well as householders were ordered to curtail the use of water, with billions of gallons in storage at Scituate, due to the failure of the City to construct sufficient tunnel and aqueduct capacity. Many communities, including the Metropolitan District Commission of Massachusetts that supplies Boston, have been faced with serious shortages of water. The lead editorial in the Boston Herald on Saturday, July 27, 1957 entitled "WATER EVERYWHERE, BUT--" states:

> "The wry thing about the state's water emergency is that the state really has plenty of water. The trouble is we haven't learned how to manage it."
>
> "The emergency in the MDC system comes not from any shortage of water in the great Quabbin reservoir, but from the lack of a high pressure aqueduct all the way from Wachusett Reservoir to Chestnut Hill, where the water is distributed to the cities and towns. The low-pressure section between Wachusett and Marlboro is now being high-pressurized at a cost of $20,000,000, and in a few years this distribution bottleneck will be cleared away.".

The above is a classic example of inadequate tunnel and aqueduct capacity. Quabbin Reservoir has a storage capacity of over 400 billion gallons and the 27 communities supplied by the MDC were forced to curtail the use of water.

In order to provide ample transmission facilities capable of conveying the maximum plant output, the City will need a supplementary aqueduct to carry an additional 44 million gallons daily from Scituate into the system. For all practical purposes, this structure should be designed to carry 50 million gallons daily. By the year 2001, our study shows that the maximum daily demand will reach 173 million gallons, based on population of areas now served, and 208 million gallons if all areas entitled to water under the Act plus East Providence were served. Scituate can supply 144 million gallons daily between the present and the proposed aqueduct and the remainder will have to come from the proposed Big and Wood River Reservoirs. In designing a supplementary tunnel or aqueduct, consideration must be given to the additional reinforcement from Big and Wood Rivers so that it would be capable of carrying the maximum required volume of water. In other words, where the two converge the aqueduct or tunnel should be of proper size to handle at least 73 million gallons daily which would include the 44 from Scituate. To provide additional strength in the system, it would be prudent to base the design on a carrying capacity of 100 million gallons daily beyond that portion where the aqueducts from Scituate and Big and Wood Rivers would converge. This would match the size of the present tunnel and aqueduct designed in 1916.

When the filter plant was designed in the early twenties, the original Water Supply Board provided for 10 filters with a capacity of 44 million gallons daily. Provisions were made for the construction of 8 additional filters at some later date. Four filters which were added in 1939 and 1940 brought the plant capacity to 61.6 million gallons daily. In 1954, after years of research on experimental filters, the size of the sand in the filter beds was changed and the rate of filtration increased bringing the capacity up to 105 million gallons daily.

According to the Water Department's long-range studies, the maximum daily demand will reach the present filter capacity by the year 1967. In order to have sufficient filter capacity to meet the demands of the water district, construction should start in 1965 on the last four filters. With some modification in the rate of filtration plus the four new filters, the capacity will reach 144 million gallons daily which is the maximum potential output that can be provided on the Scituate drainage basin based on the Estimated Safe Yield less the compensatory water that must be discharged to the mills below Gainer Dam.

The proposed sequence of immediate construction follows.

1. Aqueduct from Water Purification Works to tunnel shaft in the vicinity of West Warwick.

2. Aqueduct and/or tunnel from the vicinity of West Warwick to general area of Budlong Road in Cranston.

3. Additional rapid sand filters at Water Purification Works in Scituate.

The cost of the immediate recommended program of the new tunnel and/or aqueduct along with the additional filters at the Purification Works is estimated to be $13,000,000.

It is recommended that final design plans and specifications for the proposed major improvements be prepared using the information in this report as the basis for design. Prior to preparation of final plans, it will be necessary to take borings, possibly supplemented by seismic surveys and topographic surveys along the routes of the proposed construction.

Since the construction program is to be started during 1965, it is recommended that steps be initiated immediately after authorization of the project for the taking of final surveys, borings and seismic work along with the preparation of final plans.

Respectfully submitted,

CHARLES A. MAGUIRE & ASSOCIATES

Harold Bateson

Harold Bateson - Partner-in-Charge

K. P. Devenis

K. Peter Devenis - Project Manager

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I. SUMMARY

The Providence Water Works is experiencing a phenominal growth in water demand. Since the start of the present Scituate supply in 1926, the average daily consumption has more than doubled. According to the Water Department's long-range studies, the average daily consumption will again double by the year 2001 provided all the communities entitled to water under the 1915 Act enter the system. In fact, the average daily draft in 2001, including the water diverted to the mills below Gainer Dam, will reach 116 million gallons daily, which is far in excess of the Estimated Safe Yield of 84 million gallons daily for Scituate Reservoir.

Industry continues to be the Providence Water Works' largest customer. In 1945 annual manufacturing use was 5,188.62 million gallons but increased to 7,161.01 million gallons in 1963. Commercial accounts jumped from 1,413.85 million gallons per year to 1,767.81 million gallons. Annual residential consumption was 3,488.16 million gallons in 1945 and advanced to 4,915.07 million gallons in 1963. In 1963 industrial consumption represented 44.5 percent of total water delivered into the system, next was residential with 30.5 percent, wholesale accounts 14.0 percent, and commercial 11.0 percent.

Although the engineers displayed good judgment in designing the original tunnel and aqueduct for long-range requirements and even though the filter plant capacity has been increased periodically, it is urgent that a supplementary tunnel and aqueduct be constructed and the number of filters be increased in order to avoid the curtailment of water throughout the system during periods of maximum demand.

Operational Requirements

In the design and operation of a water utility system, there are certain basic requirements relating to the collection, treatment and transmission that are mandatory in order to serve residential, commercial and industrial consumers with an adequate supply of pure water. The following is a general description of these major conditions:

1. Impounding Reservoirs - Source of Supply: Surface supplies must have sufficient reserve storage to meet the average daily demand during a period of severe drought. In other words, you cannot depend on the long-range daily yield but are limited to what is known as the "Estimated Safe Yield". This is the maximum dependable draft which can be made continually upon a source of water supply during a period of extended drought when the greatest deficiency in runoff is likely to occur.

2. Aqueduct - Impounding Reservoir to Water Purification Works: The required aqueduct capacity to convey the water from the impounding storage reservoirs to the treatment works cannot be designed on the basis of the average daily draft. This part of the system must be capable of delivering sufficient quantities to meet the peak load of the maximum day.

3. Water Purification Plant: The water purification plant must ultimately have ample filter capacity to treat the maximum daily withdrawal from the storage reservoirs based on the limitations of the "Estimated Safe Yield" less compensatory water to comply with any statutory requirement.

4. Tunnels and Aqueducts: These facilities must be designed to meet the same general requirements as outlined under the Water Purification Plant. In other words, they must be capable of conveying into the system the maximum plant potential.

General Scope of this Study

The purpose of this study was to determine a preliminary design and the cost for supplemental facilities necessary in order to develop the full potential safe yield of Scituate Reservoir in order to expand the capacity available in the existing water works.

The principal facilities recommended for immediate construction are a new aqueduct and tunnel and additional rapid sand filters.

In order to determine the requirements for the facilities scheduled for immediate construction it was necessary to review and utilize the estimates of future water consumption as determined by the Providence Water Supply Board.

A detailed hydraulic study was made of existing water works system using field tests and office computations. Hydraulic capacities and gradients were determined for existing conduits from Scituate Reservoir to Water Purification Works, through existing Water Purification Works, and for existing tunnel and aqueduct from Water Purification Works to the vicinity of Budlong Road.

Deficiencies in water supply to meet the immediate and the future water demands were analyzed. Studies were made of the safe yield of Scituate Reservoir, and the proposed Big River, Wood River and Flat River reservoirs. An evaluation of future water supply sources was necessary in order that the location and the capacity of the facilities proposed for immediate construction would be satisfactory for future progressive development of additional water resources in a flexible manner. Therefore, the facilities proposed for immediate construction consider the safe yields of water sources and the hydraulic gradients in the transmission conduits corresponding to the future ultimate development.

It was not within the scope of this study to make a preliminary design and to determine the costs of developing future water resources such as Big, Wood and Flat Rivers.

An evaluation of the effect of the proposed hydraulic capacities on sand filters and other treatment processes at the Water Purification Works and the cost of construction of additional filters at the Water Purification Works has been made by the Providence Water Supply Board.

In determining the preliminary design and the costs for the proposed facilities, alternative locations and types of construction were evaluated and a field check and geologic investigations were made for the tunnel and aqueduct routes.

Existing Water Consumption

Excellent records of water consumption have been maintained by the Providence Water Supply Board and summarized in the Annual Report of the Water Supply Board of the City of Providence.

A new record was established when the system was called upon to deliver 87.215 million gallons on June 27, 1963. This compares with the average daily demand for that year of 47.914 million gallons. In other words, the peak day was about two hundred percent of the average for the year.

During drought conditions in 1957 the peak day demand reached 84.7 million gallons while average daily water consumption was 43.7 million gallons, so that again peak day demand was about two hundred percent of the yearly average.

During a period of drought the average water consumption and the ratio of peak day demand to average rise so that for future design conditions during a period of drought the peak water supply demand can be expected to be two hundred percent of yearly average.

Estimated Future Water Consumption

Based on past water consumption records, estimates have been made by the Providence Water Supply Board of future water consumption. These projections are shown as follows:

Figure 1

Based on population of areas now served.

Figure 2

Based on population of areas now served, plus all areas entitled to water under the Act.

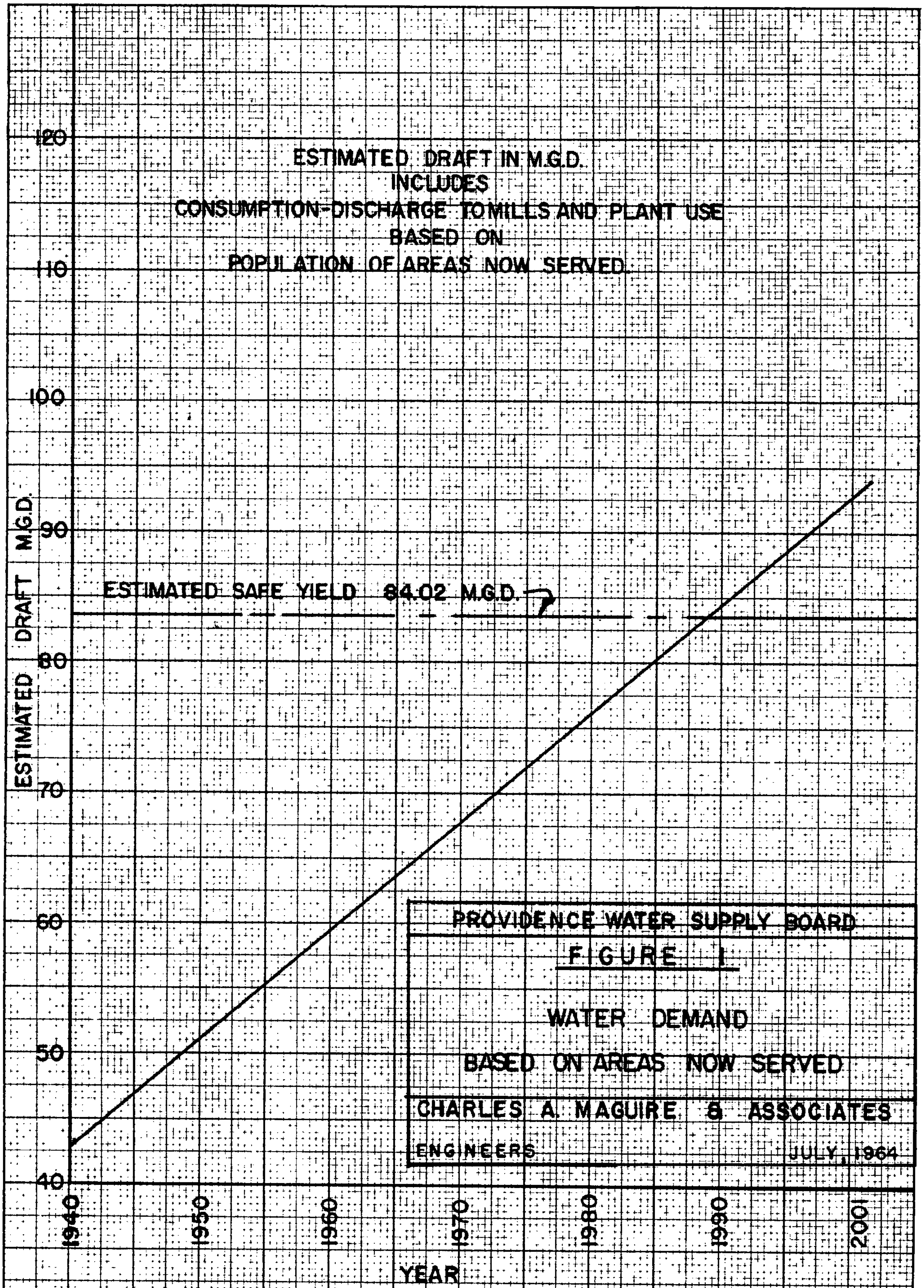
Figure 3

Based on population of areas now served, plus East Providence.

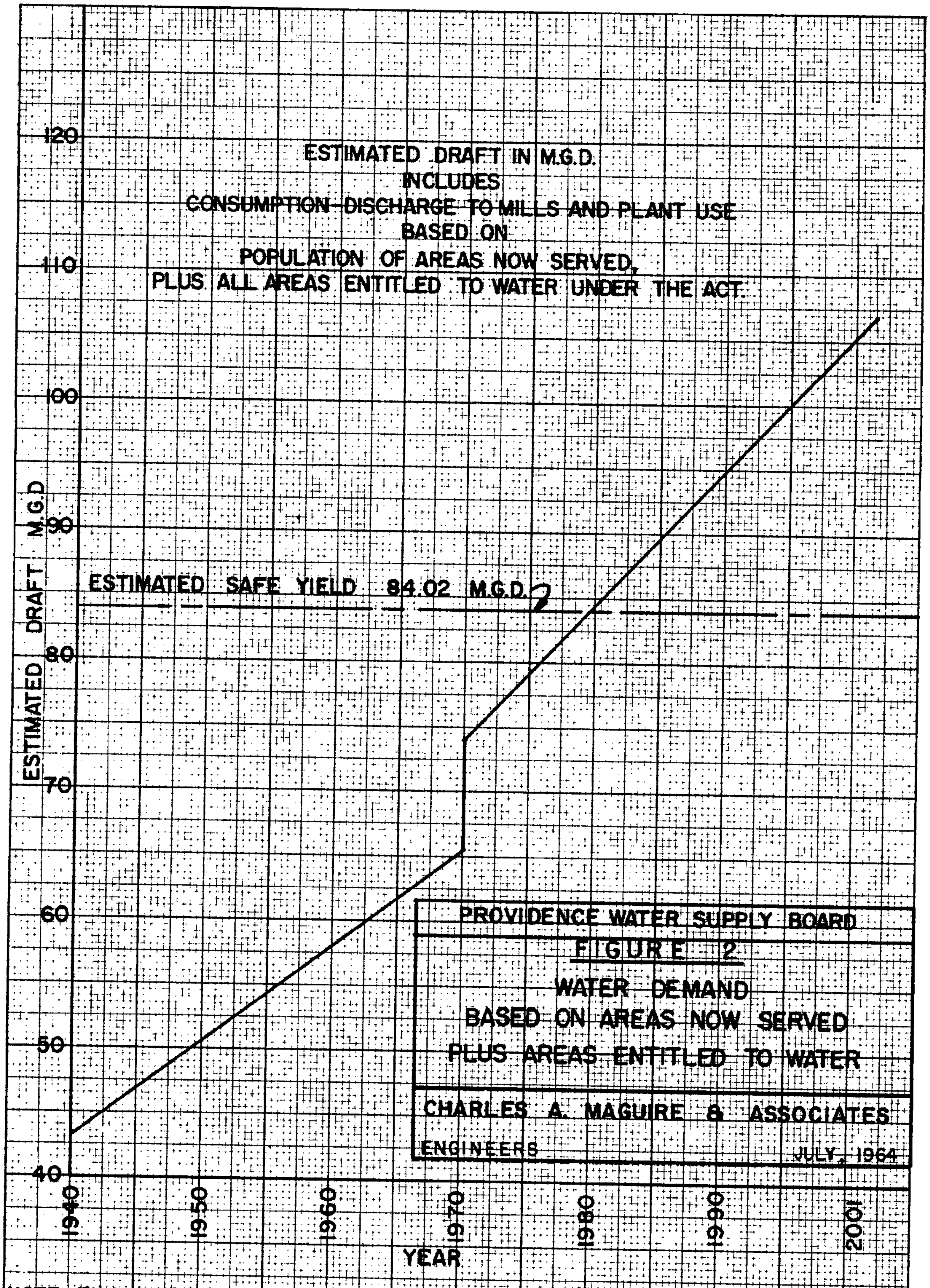
Figure 4

Based on population of areas now served, plus all areas entitled to water under the Act, plus East Providence.

Table I summarizes the future water consumption for the various assumptions of Providence Water Supply System expansion shown in Figures 1 through 4, and indicates the estimated future water consumption to be used for the design of proposed facilities.



NOTE: THIS CURVE WAS OBTAINED FROM PROVIDENCE WATER SUPPLY BOARD



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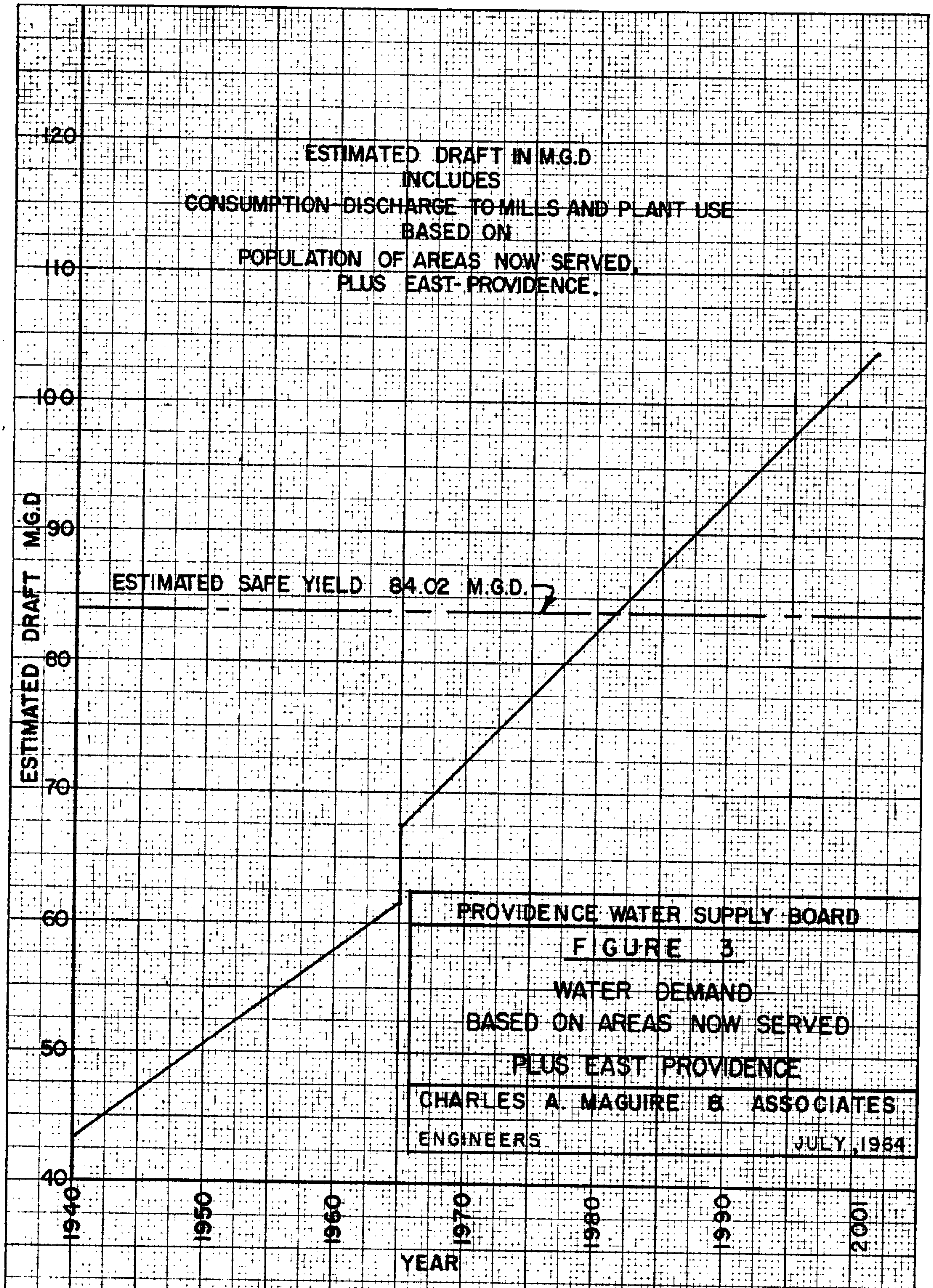
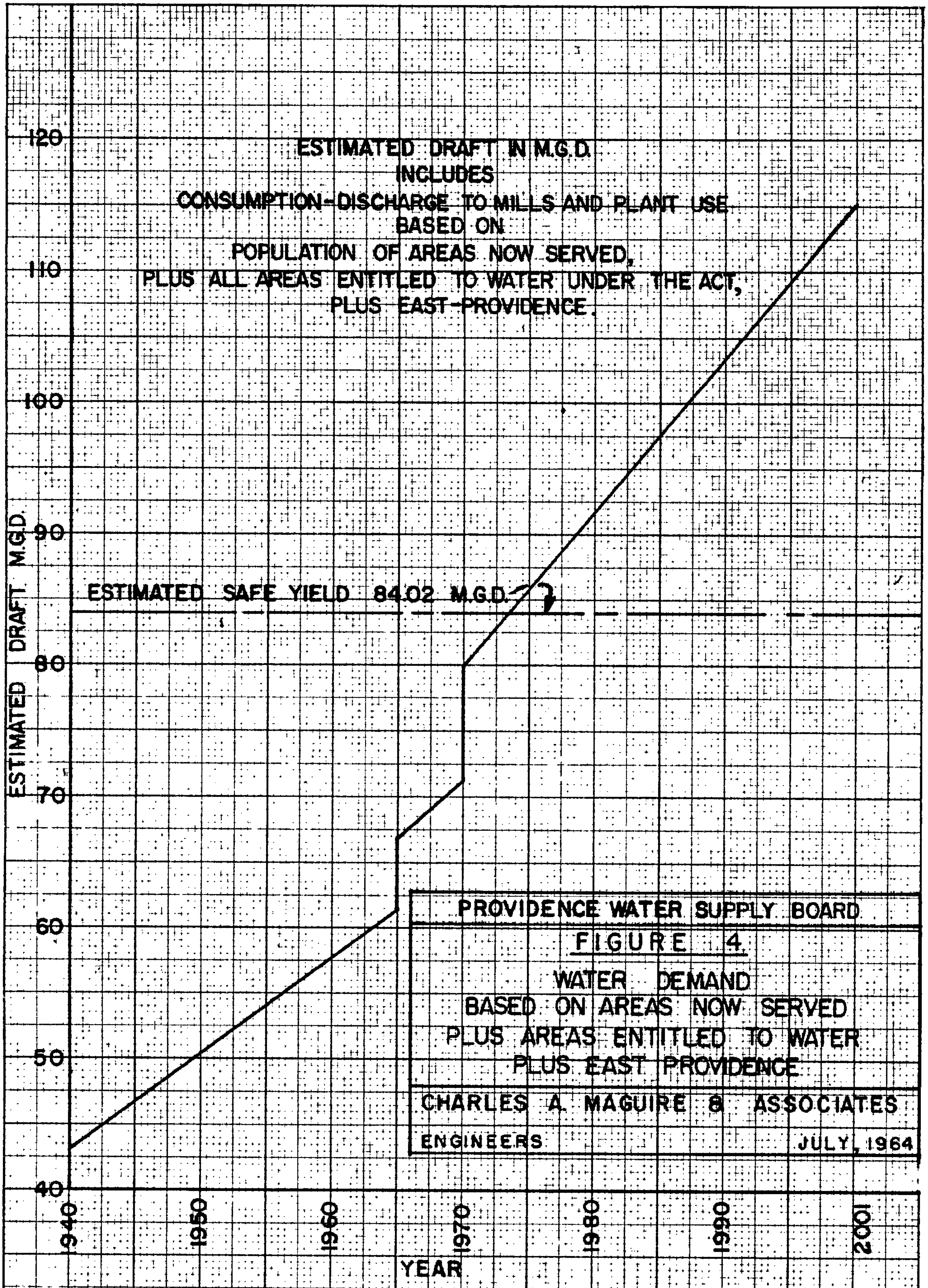


NOTE THIS CURVE HAS BEEN OBTAINED FROM PROVIDENCE WATER SUPPLY BOARD



PROVIDENCE WATER SUPPLY BOARD

FIGURE 4

WATER DEMAND
BASED ON AREAS NOW SERVED
PLUS AREAS ENTITLED TO WATER
PLUS EAST PROVIDENCE

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY, 1964

NOTE: THIS CURVE HAS BEEN OBTAINED FROM PROVIDENCE WATER SUPPLY BOARD

TABLE I

SUMMARY OF ESTIMATED AVERAGE FUTURE WATER CONSUMPTION

	Figure 1 (1)	Figure 2 (2)	Figure 3 (3)	Figure 4 (4)	(5)
1960	58.2	58.2	58.2	58.2	58.2
1965	61.8	61.8	67.4	67.4	67.4
1970	65.8	73.8	71.7	80.0	71.7
1980	74.5	83.6	81.2	90.5	81.2
1983	77.2	86.7	84.0	94.0	84.0
1990	83.5	94.2	91.5	102.3	96.0
2001	94.5	107.0	104.0	116.0	110.0

(1) Estimated draft in million gallons per day includes consumption - discharge to mills and plant use based on population of areas now served.

(2) Estimated draft in million gallons per day includes consumption - discharge to mills and plant use based on population of areas now served, plus all areas entitled to water under the Act.

(3) Estimated draft in million gallons per day includes consumption - discharge to mills and plant use based on population of areas now served plus East Providence.

(4) Estimated draft in million gallons per day includes consumption - discharge to mills and plant use based on population of areas now served, plus all areas entitled to water under the Act, plus East Providence.

(5) Estimated future water consumption to be used for design of proposed facilities.

Peak Water Consumption

Table II summarizes the peak daily future water consumption based on a ratio of 2.00 for peak water supply demand to average water supply demand.

TABLE II

SUMMARY OF ESTIMATED PEAK DAY FUTURE WATER CONSUMPTION

	Average Total Demand - mgd	Average Water Supply Demand - mgd	Peak Water Supply Demand - mgd
1960	58.2	46.2	92.4
1965	67.4	55.4	110.8
1970	71.7	59.7	119.4
1980	81.2	69.2	138.4
1983	84.0	72.0	144.0
1990	96.0	84.0	168.0
2001	110.0	98.0	196.0

The existing filter capacity of 105 million gallons per day and the existing tunnel and aqueduct capacity of 100 million gallons per day would not be adequate for the peak present water supply demands in times of extremely hot and dry weather.

The proposed immediate expansion of the water works system to 144 million gallons per day would be adequate until about the year 1983.

Necessity of New Water Supply Sources

The total safe draft which can be made continuously from the Scituate Reservoir with pumping is 84 million gallons per day of which 72 million gallons per day is available for water supply purposes. It is estimated that the total available safe yield of Scituate Reservoir will be adequate until about the year 1983. (See Figure 3.) It will be necessary to develop additional water resources for demands required beyond the year 1983.

The development of Big River Reservoir would provide an estimated additional water supply safe yield of 26 million gallons per day. The total safe yield of 84 million gallons per day from Scituate Reservoir and the water supply yield of 26 million gallons per day from Big River Reservoir would result in a 110 million gallons per day total. In year 2001 the areas presently served including East Providence will use 104 million gallons on average day and if all the communities entitled to water under the Act receive water from the Providence Water Supply System the average daily water consumption would be 116 million gallons.

For demands beyond the year 2001 if Wood River and Flat River were developed in conjunction with Big River Reservoir they would provide a combined water supply safe yield of 70 million gallons per day. The total safe yield of 84 million gallons per day from Scituate Reservoir and the additional water supply yield of 70 million gallons per day would result in a 154 million gallons per day total which would be adequate until about the year 2030.

Necessity of a Pumping Station

The maximum daily demand including East Providence will reach the present filter capacity of 105 million gallons per day by about the year 1967. In order to convey the maximum daily requirement of 105 million gallons per day to the plant, the Scituate Reservoir cannot drop below elevation 272.2. If drought conditions occurred in 1967, similar to what took place from April 1910 to October 1911, the actual level in the Scituate Reservoir would drop to elevation 266.6, 5.6 feet below the level required for gravity flow without pumping, based on average daily water supply requirement of 52.5 million gallons per day plus the 12 million gallons per day required by the mills.

The most severe drought in this area took place from June 1908 to October 1911. If we experience a similar dry spell with an average draft of 72 million gallons per day from Scituate Reservoir plus 12 million gallons per day diverted to the mills together with peak demands for the maximum day of 144 million gallons, the level in Scituate Reservoir would drop to elevation 227.7, some 56.9 feet below the required elevation of 285.6 for gravity flow.

Under these conditions,pumping would be required in order to deliver water from Scituate Reservoir to the Water Purification Works.

<u>Necessity of New Aqueduct and Tunnel</u>

The existing tunnel and aqueduct have a capacity of 100 million gallons per day from the Water Purification Works under normal flow conditions. The present demand within the Providence water supply system is approaching the tunnel and aqueduct capacity on peak days and is expected to reach 100 million gallons per day as soon as East Providence comes into the system.

The existing Scituate Reservoir has an average safe yield of 84 million gallons per day, of which 72 million gallons per day of average flow is available for water supply purposes and the remainder is discharged to the mills. An average demand of 72 million gallons per day would correspond to a peak day demand of about 144 million gallons per day. Construction of a new tunnel and aqueduct to deliver the additional 44 million gallons per day of design flow to the Providence water supply system is a necessity at the present time and should be started in 1965.

The capacity of the aqueduct for the 44 million gallons per day design flow is set at 50 million gallons per day. The tunnel from the vicinity of West Warwick to Budlong Road in Cranston will receive additional flow in the future from Big River Reservoir and therefore is designed for 100 million gallon per day capacity.

The new aqueduct and tunnel which are necessary to supply the water demands within the Providence water supply system immediately will also provide certain corollary benefits.

These benefits will be:

1. Supply of water to areas entitled to water under the 1915 act along the routes of new conduits such as West Warwick and Cranston.

2. Improvements in pressure in the Providence water supply system, particularly in the general vicinity of Budlong Road in Cranston.

3. Increased dependability of service, allowing the existing tunnel and the aqueduct to be taken out of service for inspection and repairs for short periods of time.

4. Enable extension of new future aqueduct from West Warwick to service areas to the east and south of West Warwick.

5. Availability of a conduit for part of the distance to transmit the future development of water resources at Big River and Wood River Reservoirs to points of demand.

Necessity of New Rapid Sand Filters

The existing Water Purification Works have the hydraulic and the chemical handling capacity to pass the peak day flow of 144 million gallons but will require the installation of four additional rapid sand filters.

The existing filters at the Water Purification Works have a capacity of 105 million gallons per day. Studies by the Providence Water Supply Board have shown that the capacity of the existing filters can be increased to 112 million gallons per day by modifications to permit a flow rate of 8.0 million gallons per day through each one of the fourteen existing filters instead of the present 7.5 million gallon per day flow rate. Therefore, four new rapid sand filters are necessary to provide an additional flow of 32 million gallons per day for a total flow of 144 million gallons per day, corresponding to the maximum daily draft available from Scituate Reservoir for water supply purposes.

II. IMPOUNDING RESERVOIRS - SOURCE OF SUPPLY

The safe yield of the various rivers considered in this report has been determined from runoff and evaporation records, watershed and reservoir data, and yield and drawdown studies of the various reservoirs.

The safe yield in most cases was governed by the amount of storage left in the reservoir during the most critical period of drought during the period of record. In some cases such as for Big River Reservoir, the minimum required level in the reservoir for flow through the Water Purification Works limited the amount of storage that could be used. In cases where the amount of storage is very large the safe yield is governed primarily by the average flow. In such cases the safe yield was limited not to exceed 90 percent of the average flow because of the long period required to refill the reservoir and because of possible errors due to either the measured quantity of average flow or possible variations in average flow for different portions of the same drainage area.

Runoff records were available for Scituate Reservoir from October 1915, for South Branch of Pawtuxet River at Washington, Rhode Island from October 1940, and for Wood River at Hope Valley, Rhode Island from April 1941. The records of runoff for Scituate Reservoir, Big River, Flat River and Wood River were extended back to January 1907 using the runoff records of Abbott Run at Pawtucket, Rhode Island.

The flow records show that the average runoff from Scituate Reservoir is about 7.3 percent greater than from Abbott Run in spite of the greater water surface area. If the two watersheds had the same water surface areas percentagewise, the average runoff from Scituate Reservoir would have been about 14 percent greater than from Abbott Run. Part of the reason for the greater runoff from Scituate can be explained by about 8 percent greater rainfall on the Scituate watershed as compared to the Abbott Run watershed, and the remainder due to the differences in surface cover, differences in leakage from the watershed and possible errors in measurement. During the dry periods the runoff at Scituate Reservoir and runoff at Abbott Run were approximately equal, and best agreement of past records could be obtained by considering the runoff at Scituate to equal the runoff at Abbott Run plus 7.3 percent minus the differences in evaporation from the water area during each month. The safe yield for Scituate Reservoir was determined using the above procedure for conversion of Abbott Run runoff figures to Scituate Reservoir runoff in order to create a condition similar to the 1908 to 1911 drought. The safe yield of 84 million gallons per day, which was determined in 1915 prior to the construction of Scituate Reservoir, was confirmed by present calculations. However, there is no guarantee that the next drought will not be worse than the 1908 to 1911 drought.

The average unit runoff at the U. S. Geological Survey gauging stations for the South Branch of Pawtuxet River at Washington, Rhode Island and for the Wood River at Hope Valley, Rhode Island was approximately the same during the record period and about 10 percent higher than the runoff at Scituate Reservoir when adjusted for evaporation. Therefore, the procedure for extending the records for Big, Flat and Wood Rivers was to adjust the runoff of Scituate Reservoir by the differences in evaporation due to water surface differences and to add 10 percent of flow due to higher yield.

The records of Abbott Run were very useful inasmuch as a very dry period of runoff occurred from June 1908 to October 1911, which was within the period of records for Abbott Run but not within the period of other records. In most cases the safe yield of the various reservoirs was governed by the runoff within the above period.

An analysis of long-term rainfall records by X. H. Goodnough in the Journal of New England Waterworks Association dated September 1915 shows that during a period of 165 years there were ten occasions when the drought conditions were worse than those during the 1908 to 1911 period. However, the drought conditions of 1908 to 1911 have not been equaled up to the present for the duration period of about three years considered in the safe yield studies. Therefore, the safe yield studies based on Abbott Run data can be considered to be based on a drought with a frequency of about once in 20 years.

For the purpose of this study safe yield was determined for the following combinations of water sources: Scituate Reservoir, Big River Reservoir, Wood River Reservoir, Flat River Reservoir, Big and Wood Rivers from Big River Reservoir, Big and Flat Rivers from Big River Reservoir; and Big, Wood and Flat Rivers from Big River Reservoir.

Scituate Reservoir

Data for Scituate Watershed and Reservoir is presented in Table III. Safe yield is limited by the amount of water left in storage during the critical dry period from June 1908 to October 1911. Even though the quantity left in the reservoir is small, 2.3 billion gallons at elevation 227.7 or 6.3 percent of the total reservoir volume, it will be possible to draw through the lower intake which extends from elevation 213 to 220 with a future pumping station.

Figure 5 shows the minimum levels during drought conditions in the Scituate Reservoir for the various safe yields corresponding to the system demands from 1963 to 1983.

TABLE III

SCITUATE WATERSHED AND RESERVOIR DATA

Drainage area	92.8 sq. mi.
Water area	7.13 sq. mi. Scituate Reservoir - 7.42 sq. mi. total
Percent water area	7.7% reservoir - 8.0% total
Spillway elevation	284.0
Storage capacity	37,011 million gallons reservoir - 41,268 million gallons total
Storage capacity per square mile	445 million gallons
Safe yield	84.0 mgd - .905 mgd/sq. mi.
Riparian rights	12.0 mgd
Water supply yield	72.0 mgd
Average runoff (1915-1963)	110.5 mgd - 1.190 mgd/sq. mi.
Average runoff (1915-1963)	24.98 inches
Safe yield as percent of runoff	76.1%
Conversion factor	1 inch runoff = 1,612.75 million gallons
Average runoff (1940-1962)	24.06 inches
Safe yield computation	June 1908-October 1911 - 6.3% storage left (1.073 Abbott Run - evaporation) Min. Elev. 227.7

Figure 6 shows the calculated levels in Scituate Reservoir using runoff during a period from 1908 to 1963 based on average daily draft equivalent to 84.0 million gallons per day. This represents 72.0 million gallons used for water supply purposes and 12.0 million gallons released for riparian rights.

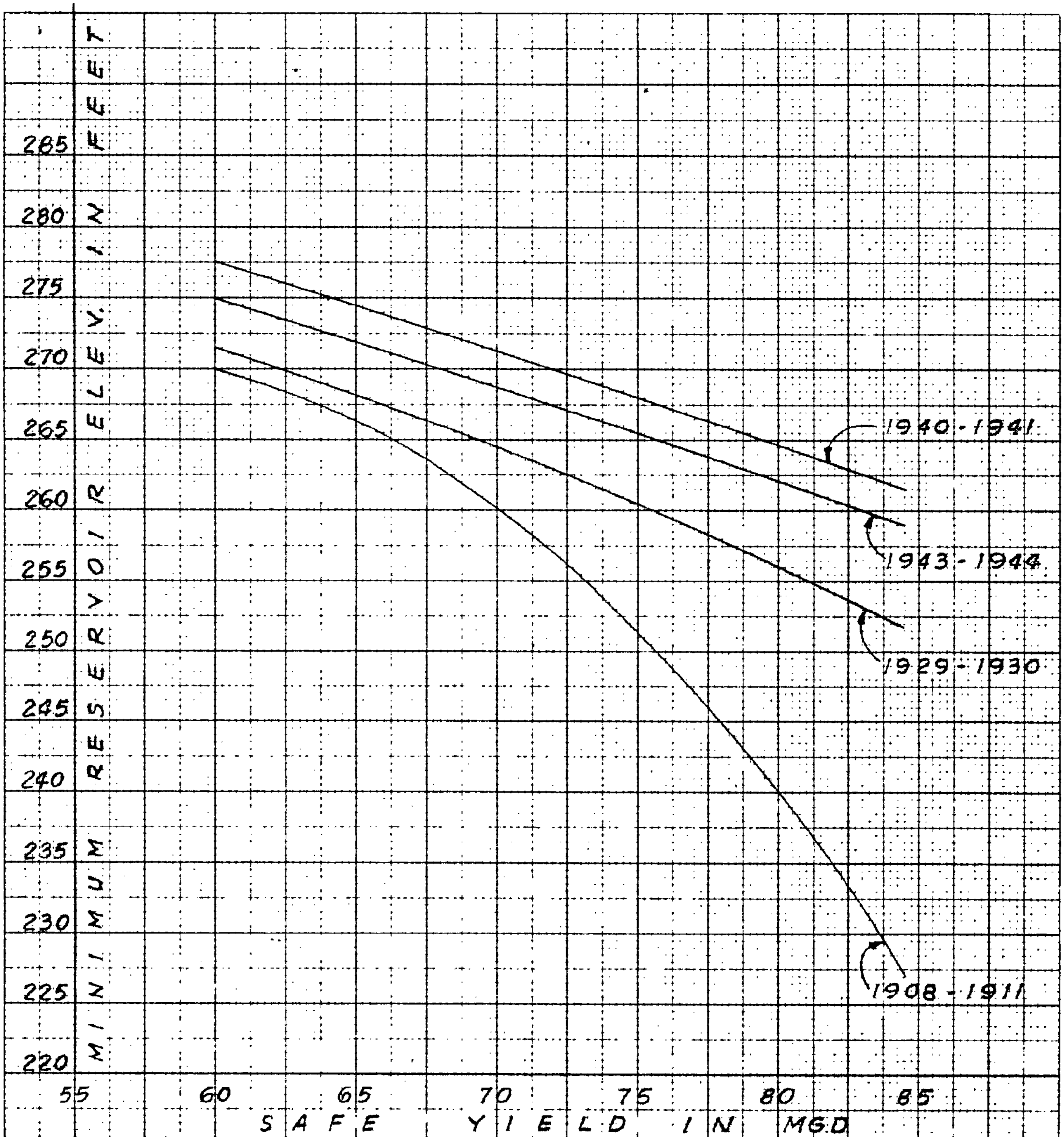


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FIGURE 5

MINIMUM LEVELS IN SCITUATE RESERVOIR FOR VARIOUS SAFE YIELDS

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ENGINEERS

JULY, 1964

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FIGURE 6

LEVELS IN SCITUATE AND BIG RIVER RESERVOIRS FOR RUN-OFF PERIOD FROM 1908 TO 1963 FOR A DEMAND EQUAL TO SAFE YIELD

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ENGINEERS JULY 1964

Big River Reservoir

Data for Big River Watershed and Reservoir are presented in Table IV. A spillway elevation of 300 was used because large amount of storage is required for future use in connection with the development of Wood and/or Flat Rivers in addition to Big River, and because of head requirements for gravity flow from the reservoir through a future water purification plant and future extension of the tunnel to the vicinity of West Warwick.

Safe yield was limited to 90 percent of average flow even though the reservoir level could have been drawn down lower to provide a somewhat higher yield. However, the reservoir would have been less than full for extremely long periods of time. The minimum level during drought conditions would be 272.0.

Figure 6 shows the calculated levels in Big River Reservoir using runoff during a period from 1908 to 1962 based on average daily draft equivalent to 30.2 million gallons per day. This represents 26.0 million gallons used for water supply purposes, and 4.2 million gallons released for riparian rights.

TABLE IV

BIG RIVER WATERSHED AND RESERVOIR DATA

Drainage area	29.66 sq. mi.
Water area	5.38 sq. mi. ± reservoir - 5.56 sq. mi. ± total
Percent water area	18.1% ± reservoir - 18.7% ± total
Spillway elevation	300.0
Storage capacity	27,300 million gallons
Storage capacity per square mile	920 million gallons
Safe yield	30.2 mgd - 1.017 mgd/sq. mi.
Riparian rights	4.2 mgd
Water supply yield	26.0 mgd
Average runoff (1940-1962)	33.6 mgd - 1.138 mgd/sq. mi.
Average runoff (1940-1962)	23.87 inches (27.87 - 4.00 = 23.87)
Safe yield as percent of runoff	90.0%
Conversion factor	1 inch runoff = 515.43 million gallons
Safe yield computation	June 1908-November 1914 - storage left 32.5%. (Scituate + evaporation) x 1.10 - evaporation. Min. Elev. 272.0

Wood River Reservoir

Data for Wood River Watershed and Reservoir are presented in Table V. Spillway elevation of 170 was used because only a small amount of storage is required for development of Wood River Reservoir in conjunction with the use of storage at Big River Reservoir. This would be a diversion reservoir, from which water would be pumped from Wood River into final storage at Big River Reservoir.

Safe yield is limited by the amount of water left in storage during the critical dry period from April 1910 to October 1911. For this drought condition 15 percent of storage volume was left in the reservoir. The minimum reservoir level during drought conditions would be 146.0.

The safe yield developed for Wood River Reservoir alone is much smaller than the yield which can be developed for Wood River in conjunction with synchronous operation of Big River Reservoir storage. The data presented for Wood River alone are for comparison purposes only because it is not recommended to develop Wood River directly without Big River Reservoir.

TABLE V

WOOD RIVER WATERSHED AND RESERVOIR DATA

Drainage area	36.0 sq. mi.
Water area	1.43 sq. mi.
Percent water area	4.0%
Spillway elevation	170.0
Storage capacity	5,965 million gallons
Storage capacity per square mile	166 million gallons
Safe yield	25.3 mgd - 0.703 mgd/sq. mi.
Riparian rights	5.3 mgd
Water supply yield	20.0 mgd
Average runoff mgd (1940-1962)	47.9 mgd - 1.330 mgd/sq. mi.
Average runoff inches (1940-1962)	27.96 (28.45 - .49 = 27.96)
Safe yield as percent of runoff	52.9%
Conversion factor	1 inch = 625.59 million gallons
Safe yield computation	April 1910-October 1911 - 15.0% storage left (Scituate + evaporation) x 1.10 - evaporation Min. Elev. 146.0

Flat River Reservoir

Data for Flat River Watershed and Reservoir are presented in Table VI. Spillway elevation of 300 was used for calculation purposes because of head requirements for gravity flow from the reservoir through a future water purification plant and tunnel and to provide additional storage for future use in conjunction with the development of Wood River. The data presented for Flat River are for comparison purposes only because the reservoir level is too high and it is not recommended to develop Flat River directly by a dam at elevation 300.

Safe yield was limited to 90 percent of average flow even though the reservoir could have been drawn down lower to provide a somewhat higher yield. The average unit runoff is very low because of the large amount of evaporation from a watershed containing about 24 percent as water surface area.

The safe yield developed for Flat River Reservoir at elevation 300 is less than the yield which can be developed from Flat River at elevation 248, and Big River at elevation 300 in conjunction with Big River Reservoir with synchronous operation.

TABLE VI

FLAT RIVER WATERSHED AND RESERVOIR DATA

Drainage area	60.45 sq. mi.
Water area	13.70 sq. mi. ± 14.50 sq. mi. ± total
Percent water area	22.7% ± reservoir - 24.5% total
Spillway elevation	300.0
Storage capacity	96,300 million gallons
Storage capacity per square mile	1,593 million gallons
Safe yield	57.9 mgd - 0.958 mgd/sq. mi.
Riparian rights	7.9 mgd
Water supply yield	50.0 mgd
Average runoff (1940-1962)	64.3 mgd - 1.065 mgd/sq. mi.
Average runoff (1940-1962)	22.3 inches (27.8 - 5.5 = 22.3)
Safe yield as percent of runoff	90.0%
Conversion factor	1 inch runoff = 1,050.55 million gallons
Safe yield computation	June 1908-October 1911 - 60.8% storage left (Scituate + evaporation) x 1.10 - evaporation. Min. Elev. 287.0

Big and Wood Rivers with Big River Reservoir

Data for Big and Wood River Watershed and Reservoirs are presented in Table VII. Spillway elevation of 300 was used for Big River Reservoir and 170 for Wood River Reservoir. The flow from Wood River diversion reservoir would be pumped into the Big River Reservoir. The flow of Wood and Big Rivers would then pass by gravity through the new water purification works and the new tunnel from the Big River Reservoir.

Safe yield is limited by the amount of water that is available at Wood River during the critical drought period while maintaining 25 percent of Wood River Reservoir storage and by the minimum withdrawal level in Big River Reservoir.

A safe yield of 61.3 million gallons per day is available for minimum Big River elevation of 275.0. About 52.0 million gallons per day can be used for water supply purposes and 9.3 million gallons per day released for riparian rights.

TABLE VII

BIG AND WOOD RIVERS WATERSHED AND RESERVOIR DATA

Drainage area	65.66 sq. mi.
Water area	6.81 sq. mi. ± reservoir 6.99 sq. mi.± total
Percent water area	10.4% ± reservoir - 10.7% ± total
Spillway elevation	300.0 Big, 170.0 Wood
Storage capacity	33,265 million gallons
Storage capacity per square mile	507 million gallons
Safe yield	61.3 mgd - 0.932 mgd/sq. mi.
Riparian rights	9.3 mgd
Water supply yield	52.0 mgd
Average runoff (1940-1962)	81.5 mgd - 1.240 mgd/sq. mi.
Average runoff inches (1940-1962)	26.00
Safe yield as percent of runoff	75.2%
Conversion factor	1 inch = 1,141.02 million gallons
Sqfe yield computations*	June 1908-October 1911 - 61.3 mgd 25.0% storage left in Wood Reservoir Min. Elev. Wood Reservoir - 150.0 25.2% storage left in Big River Reservoir Min. Elev. Big River Reservoir - 275.0

* For Min. Elev. in Big River of 270.0 - safe yield 63.2 mgd.

For Min. Elev. in Big River of 280.0 - safe yield 58.8 mgd.

Big and Flat Rivers with Big River Reservoir

Data for Big and Flat Rivers Watershed and Reservoirs are presented in Table VIII.

Spillway elevation of 300 was used for Big River Reservoir and 248 for Flat River Reservoir. The flow from existing Flat River Reservoir would be pumped into the Big River Reservoir for the above condition which was assumed in calculating safe yield. The flow from Big River Reservoir would pass through the new water purification works and the new tunnel by gravity.

Safe yield is limited by the amount of water available for development from Flat River during the critical drought period while maintaining 50 percent of existing Flat River Reservoir storage and by the minimum withdrawal level in Big River Reservoir.

A safe yield of 53.3 million gallons per day is available for minimum Big River elevation of 275.0. About 45 million gallons per day can be used for water supply purposes and 8.3 million gallons per day released for riparian rights.

TABLE VIII

BIG AND FLAT RIVERS WATERSHED AND RESERVOIR DATA

Drainage area	58.5 sq. mi.
Water area	7.13 sq. mi. ± reservoir - 7.31 sq. mi. ± total
Percent water area	12.2% ± reservoir - 12.5% ± total
Spillway elevation	300.0 Big, 248.0 Flat
Storage capacity	29,170 million gallons
Storage capacity per square mile	499 million gallons
Safe yield	53.3 mgd - 0.912 mgd/sq. mi.
Riparian rights	8.3 mgd
Water supply yield	45.0 mgd
Average runoff (1940-1962)	71.3 mgd - 1.220 mgd/sq. mi.
Average runoff inches (1940-1962)	25.60
Safe yield as percent of runoff	74.8%
Conversion factor	1 inch = 1,016.68 million gallons
Safe yield computations*	June 1908-October 1911 - 53.3 mgd 50% storage left in Flat River Reservoir Min. Elev. Flat Reservoir - 245.0 25.2% storage left in Big River Reservoir Min. Elev. Big River Reservoir - 275.0

* For Min. Elev. in Big River Reservoir of 270.0 - safe yield 55.2 mgd.

For Min. Elev. in Big River Reservoir of 280.0 - safe yield 50.8 mgd.

Big, Wood and Flat Rivers with Big River Reservoir

Data for Big, Wood and Flat Rivers Watershed and Reservoirs are presented in Table IX. Spillway elevation of 300 was used for Big River Reservoir, 170 for Wood River Reservoir, and 248 for existing Flat River Reservoir. The flow from existing Flat River Reservoir and the new Wood River Reservoir would be developed into the Big River Reservoir for the above condition which was assumed in calculating safe yield. The flow from Big River Reservoir would pass through new water purification works at Big River and the new tunnel by gravity.

Safe yield is limited by the amount of water which is available for development from Wood and Flat Rivers Reservoirs during the critical drought periods while maintaining 25 percent of Wood River Reservoir storage, and 50 percent of existing Flat River Reservoir storage and by the minimum withdrawal level in Big River Reservoir.

A safe yield of 83.0 million gallons per day is available for minimum Big River elevation of 275.0. About 70.0 million gallons per day can be used for water supply purposes and 13.0 million gallons per day released for riparian rights.

TABLE IX

BIG, WOOD AND FLAT RIVERS WATERSHED AND RESERVOIR DATA

Drainage area	94.46 sq. mi.
Water area	8.49 sq. mi.
Percent water area	9.0%
Spillway elevation	300 Big, 170 Wood and 248 Flat
Storage capacity	35,140 million gallons
Storage capacity per square mile	372 million gallons
Safe yield	83.0 mgd - 0.878 mgd/sq. mi.
Riparian rights	13.0 mgd
Water supply yield	70.0 mgd
Average runoff (1940-1962)	117.7 mgd - 1.245 mgd/sq. mi.
Average runoff inches (1940-1962)	26.05
Safe yield at percent of runoff	70.6%
Conversion factor	1 inch = 1,641.62 million gallons
Safe yield computations*	June 1908-October 1911 - 83.0 mgd 50% storage left in Flat River Reservoir Min. Elev. Flat Reservoir - 245.0
	25% storage left in Wood River Reservoir Min. Elev. Wood Reservoir - 150.0
	25.2% storage left in Big River Reservoir Min. Elev. Big Reservoir - 275.0

* For Min. Elev. in Big River Reservoir of 270.0 - safe yield 84.9 mgd.

For Min. Elev. in Big River Reservoir of 280.0 - safe yield 80.4 mgd.

Summary of Safe Yields

A summary of safe yields is presented in Table X. The actual development of Wood and Flat Rivers would be flexible so that either one could be developed in any order or both eventually. The estimates of riparian rights are approximate.

TABLE X

SUMMARY OF SAFE YIELDS

	Safe Yield mgd	Riparian Rights mgd	Water Supply mgd
Scituate Reservoir	84.0	12.0	72.0
Big River Reservoir	30.2	4.2	26.0
Big River Reservoir with Wood River	61.3	9.3	52.0
Big River Reservoir with Wood and Flat Rivers	83.0	13.0	70.0

Table XI compares the safe yield in developing the water resources in accordance with Table X with the demand in the various years. The available resources can be developed gradually in a flexible manner to meet future demands. The available safe yield, if developed totally, would be sufficient to meet the demands until about the year 2030.

TABLE XI

ADEQUACY OF SAFE YIELDS

	Available Supply(1) mgd	Demand Year	Design Period(2) Years
Scituate Reservoir	84.0	1983(3)	18
Scituate Reservoir and Big River Reservoir	110.0	2001(4)	31
Scituate Reservoir and Big River Reservoir and Wood River	136.0	2016(5)	51
Scituate Reservoir and Big River Reservoir with Wood and Flat Rivers	154.0	2029(5)	64

(1) Water supply and riparian rights for Scituate Reservoir, water supply only for others.

(2) Beyond 1965.

(3) Table I.

(4) Table I.

(5) Extending Figure 4.

Dam, Dikes and Intake at Big River - Construction 1970 to 1980

Spillway crest	Elevation 300.0
Minimum reservoir level	275.0
Initial safe yield Big River Reservoir	30.2 mgd
Estimated riparian rights	4.2 mgd
Water supply yield Big River Reservoir	26.0 mgd
Peak day water supply (Big River only)	52.0 mgd
Design future peak day flow (Big, Wood and Flat Rivers)	140.0 mgd

Wood River and Flat River Development - Construction after 1990

By developing Wood and Flat Rivers in conjunction with Big River Reservoir, the safe yield can be increased to 83 million gallons per day of which about 70 million gallons per day would be available for water supply purposes. It is estimated that about 7.5 million gallons per day could be diverted from the new water purification works to serve the areas in the general vicinity of the plant directly. The remaining average flow of 62.5 million gallons per day and peak flow of 125 million gallons per day would pass into the tunnel.

Alternate Plans for Additional Sources of Water Supply

Several alternative sources of additional water supply were considered. As a first step it is recommended to develop Big River. In the proposed immediate construction an additional flow capacity of 100 million gallons per day is provided to the vicinity of Budlong Road in Cranston. This total capacity could be utilized by flow from Big River and Scituate Reservoir alone. However, as future areas develop an extension of aqueduct will be necessary from the tunnel shaft in West Warwick eastward.

For the purposes of the immediate construction it is not necessary to determine whether Wood River or Flat River will be developed, or both. However, since both of these water resources would be needed eventually, a program to preclude further construction within areas of possible land-taking should be initiated.

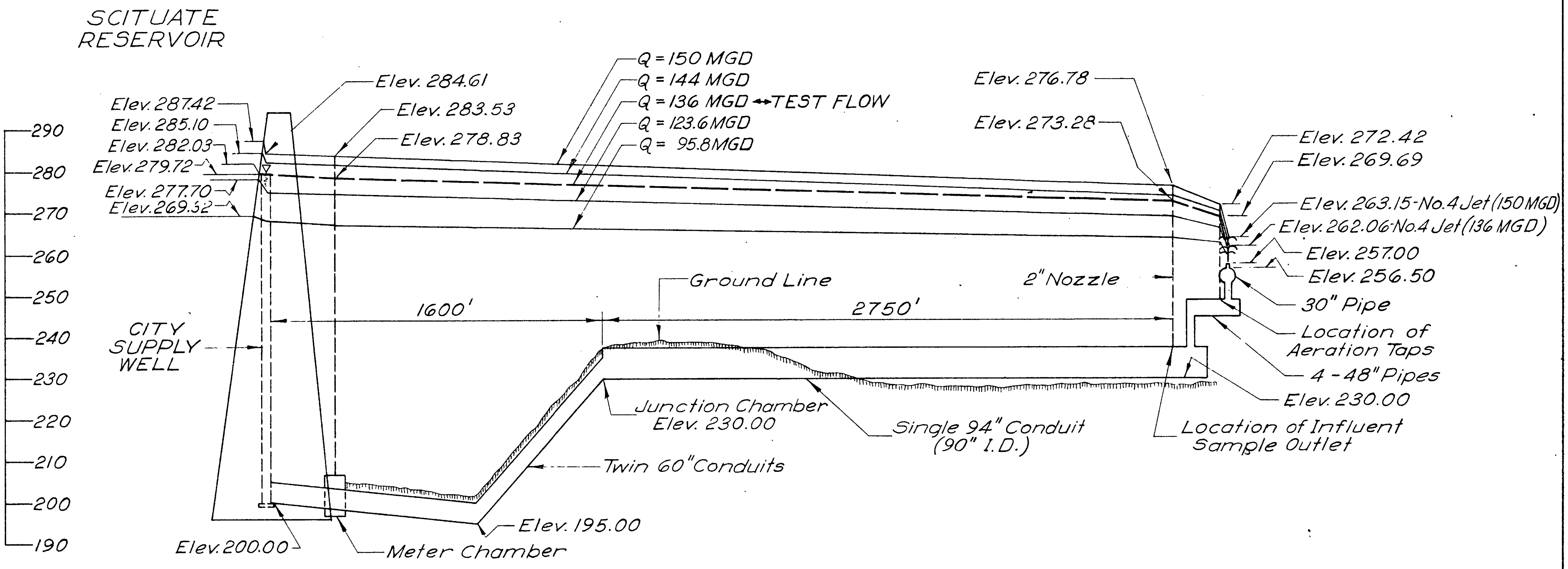
III. AQUEDUCT-IMPOUNDING RESERVOIR TO WATER PURIFICATION WORKS

The hydraulic capacity of the existing water works system from Scituate Reservoir to Water Purification Works was analyzed by office computations in conjunction with field tests. The analysis of the existing system was done with flow through influent aerators, by-passing aeration, and by-passing the plant. The requirements for a future pumping station to meet the deficiencies in the existing system for future water supply requirements are presented in this section.

Capacity of Existing Aqueduct from Scituate Reservoir to Water Purification Works

The hydraulic losses from Scituate Reservoir to Water Purification Works can be subdivided into losses through the intake at the dam, the aqueduct and the influent aerators.

There are three intakes at the gatehouse at three different levels. However, since the lowest intake produces the best quality and temperature of water, it is normally used. During field tests the lowest intake, which is at elevation 213 to 220, was open and the other two intakes were closed. The losses during the tests were high because only two of the shutters were open at the lowest intake. Figure 7 shows the hydraulic profile from Scituate Reservoir to the Water Purification Works for the flow test conditions. The losses at the intake could be reduced somewhat by installing three open shutters.



PROFILE

Scales: Horiz. 1" = 400'
Vert. 1" = 20'

NOTE

1. The above Profiles are based on a Nozzle elevation of 257.00 in the No. 4 Aerator.
The elevation of future Nozzles will be 257.50
Profiles shown must be raised equivaliantly (0.50 ft.)
2. Pressure Elevations are given for 136 MGD & 150 MGD only. Any other elevation can be obtained by Scaling directly.

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FIGURE 7

HYDRAULIC PROFILE
EXISTING AQUEDUCT
FROM SCITUATE RESERVOIR
TO WATER PURIFICATION WORKS

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From the intakes, water is conveyed to the influent chamber through twin 60-inch aqueducts and a single 94-inch aqueduct (90-inch inside diameter). Flow tests indicated that the Williams and Hazen roughness coefficient C is between 100 and 120. The exact value of the roughness coefficient for the 60-inch and the 94-inch aqueduct was not determined separately during the field tests because no pressure tap was available during the testing period and only the total loss was required.

From the influent chamber the flow passes through four 48-inch pipes to the influent aerator. The flow through each pipe is controlled by a gate valve and a butterfly valve placed adjacent to each other. The valves are 36 inches in diameter and produce a substantial head loss. At the influent aerator the flow is distributed by three pipe rings and one central pipeline to nozzles from which jets of water shoot upward. The highest loss occurs through the outer ring. All the butterfly and gate valves were open for the test flow conditions. With Scituate Reservoir level at elevation 282.03 a flow of 136 million gallons per day was obtained. Water jets ranged from 5 to 13 feet in height. The top of the nozzles, which are 2 inches in diameter on the three pipe rings, was at elevation 257.0 during the field tests as shown in Figure 7. For future conditions, the nozzles will be raised to elevation 257.5. The overflow weir at the aerators should also be raised for future flow conditions. The head loss curves in Figure 8 were determined for future conditions using the revised nozzle arrangement.

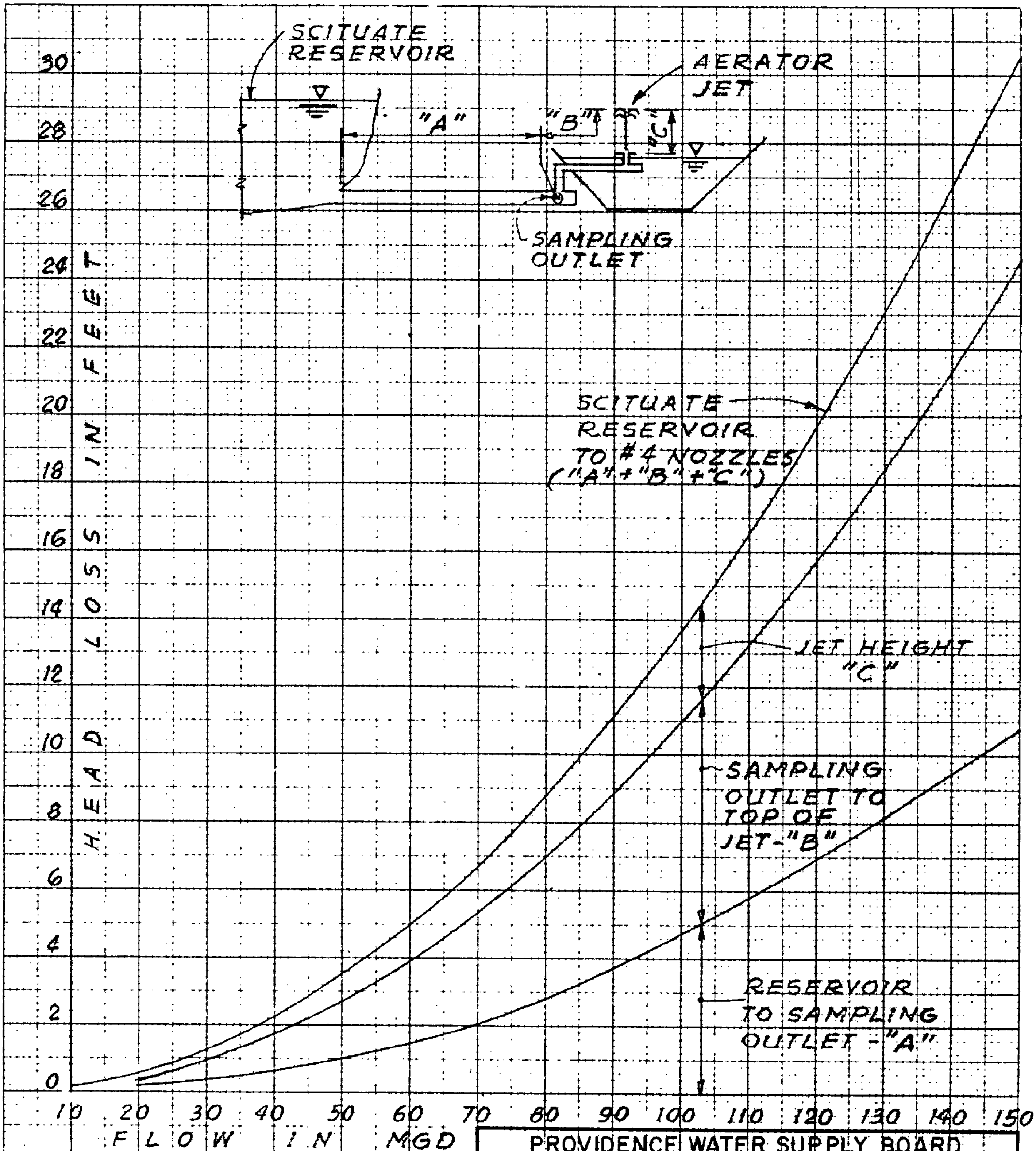

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FIGURE 8

HEAD LOSS CURVES-EXISTING AQUEDUCT FROM SCITUATE RESERVOIR TO WATER PURIFICATION WORKS THROUGH AERATION

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Under extreme peak loads for existing water works system aeration can be by-passed occasionally in order to increase the flow capacity from Scituate Reservoir to Water Purification Works. Figure 9 shows the flow conditions from Scituate Reservoir to coagulation basin if influent aerators were by-passed by closing the butterfly valves to the aerators. The required Scituate Reservoir elevation is reduced appreciably and a flow of 144 million gallons per day can be passed with a head loss of 14.4 feet to the coagulation basin instead of a head loss of 28.4 feet to the influent aerator pool. This means that with Scituate Reservoir at elevation 268.4 the plant can still operate, but will require some adjustment in chemical dosage rates for removal of carbon dioxide.

Figure 10 shows the flow conditions from Scituate Reservoir in the event the entire Water Purification Works were by-passed and the flow was diverted into the existing tunnel and aqueduct. Under these conditions a flow of 144 million gallons per day could be passed with slightly lower levels in the Scituate Reservoir than for the condition of direct flow to coagulation basins. Since aeration, coagulation and filtration would be by-passed completely and disinfection by chlorination only would be provided, such a procedure would be used only in case of emergency.

Figure 11 shows the elevation in Scituate Reservoir for various rates of flow with flow through the aerators, by-passing aerators to coagulation basin, and by-passing the plant directly into the existing effluent aqueduct. In order to pass 144 million gallons per day the required level in Scituate Reservoir would be elevation 285.6 with flow through influent aerator, elevation 268.4 by-passing aeration, and elevation 266.6 by-passing the plant.

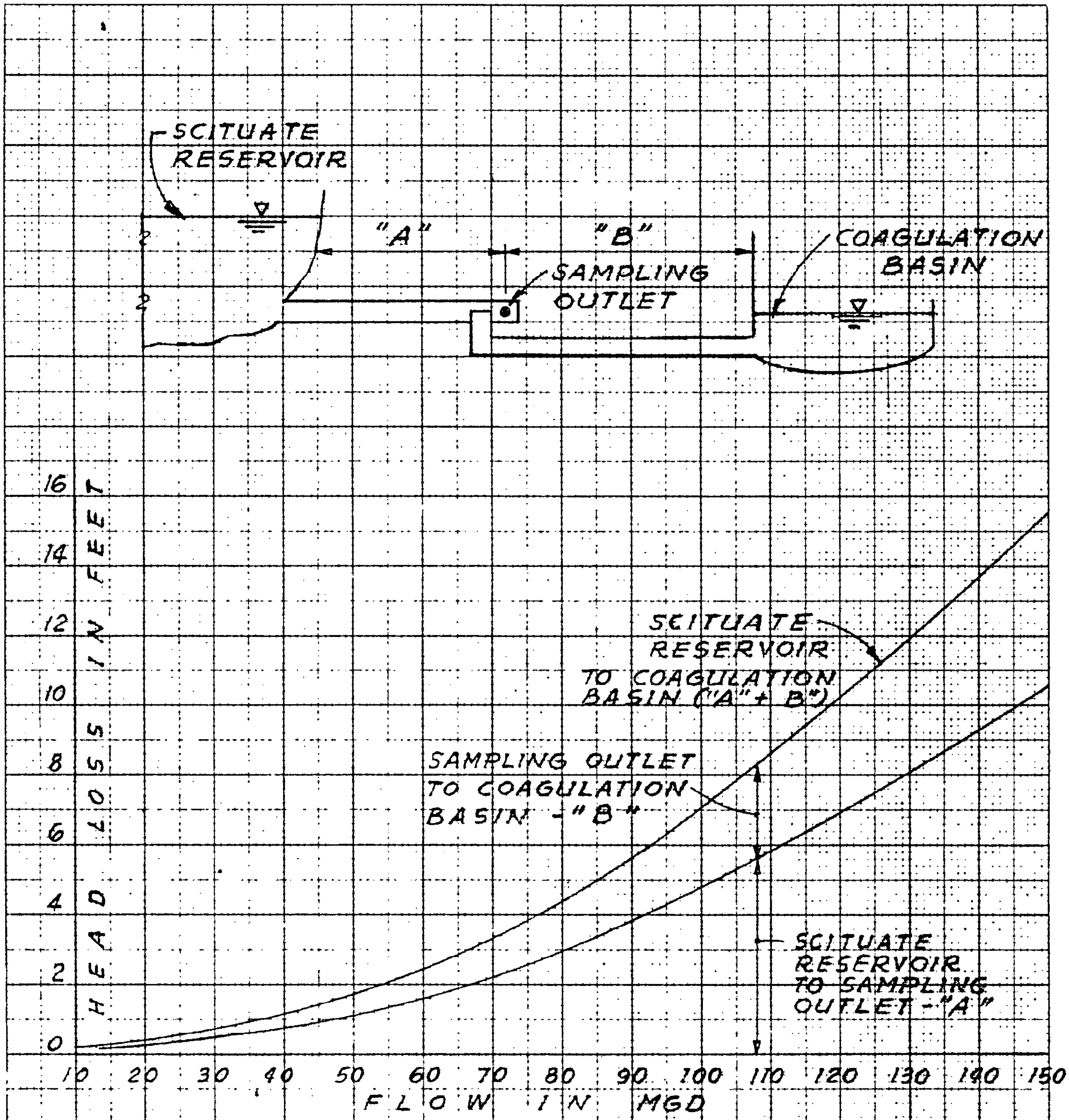

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FIGURE 9

HEAD LOSS CURVES-EXISTING AQUEDUCT FROM SCITUATE RESERVOIR TO WATER PURIFICATION WORKS BY-PASSING AERATION

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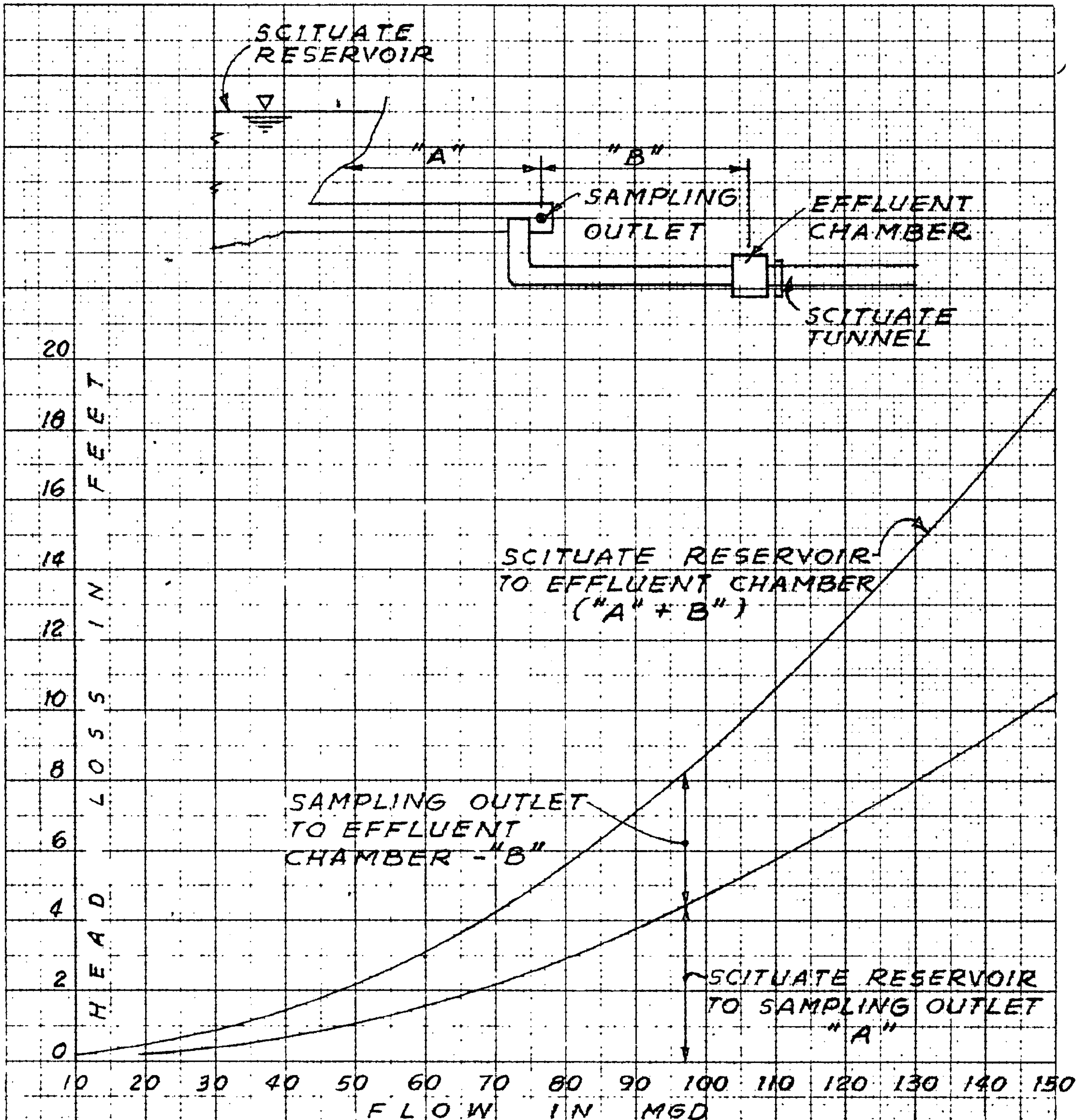

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FIGURE 10

HEAD LOSS CURVES EXISTING AQUEDUCT FROM SCITUATE RESERVOIR TO WATER PURIFICATION WORKS BY-PASSING PLANT

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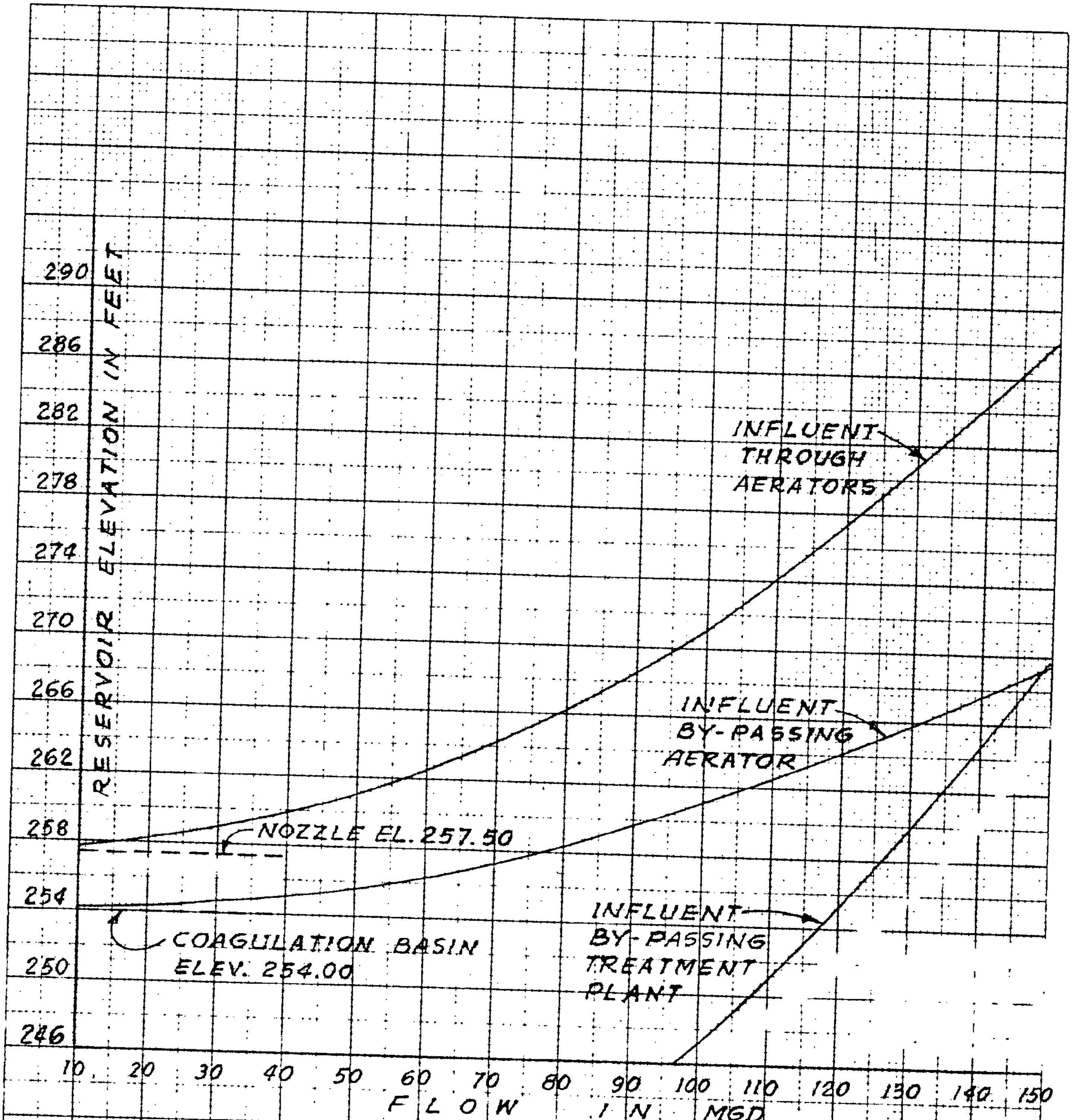

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FIGURE 11

HEAD LOSS CURVES-SUMMARY EXISTING AQUEDUCT FROM SCITUATE RESERVOIR TO WATER PURIFICATION WORKS

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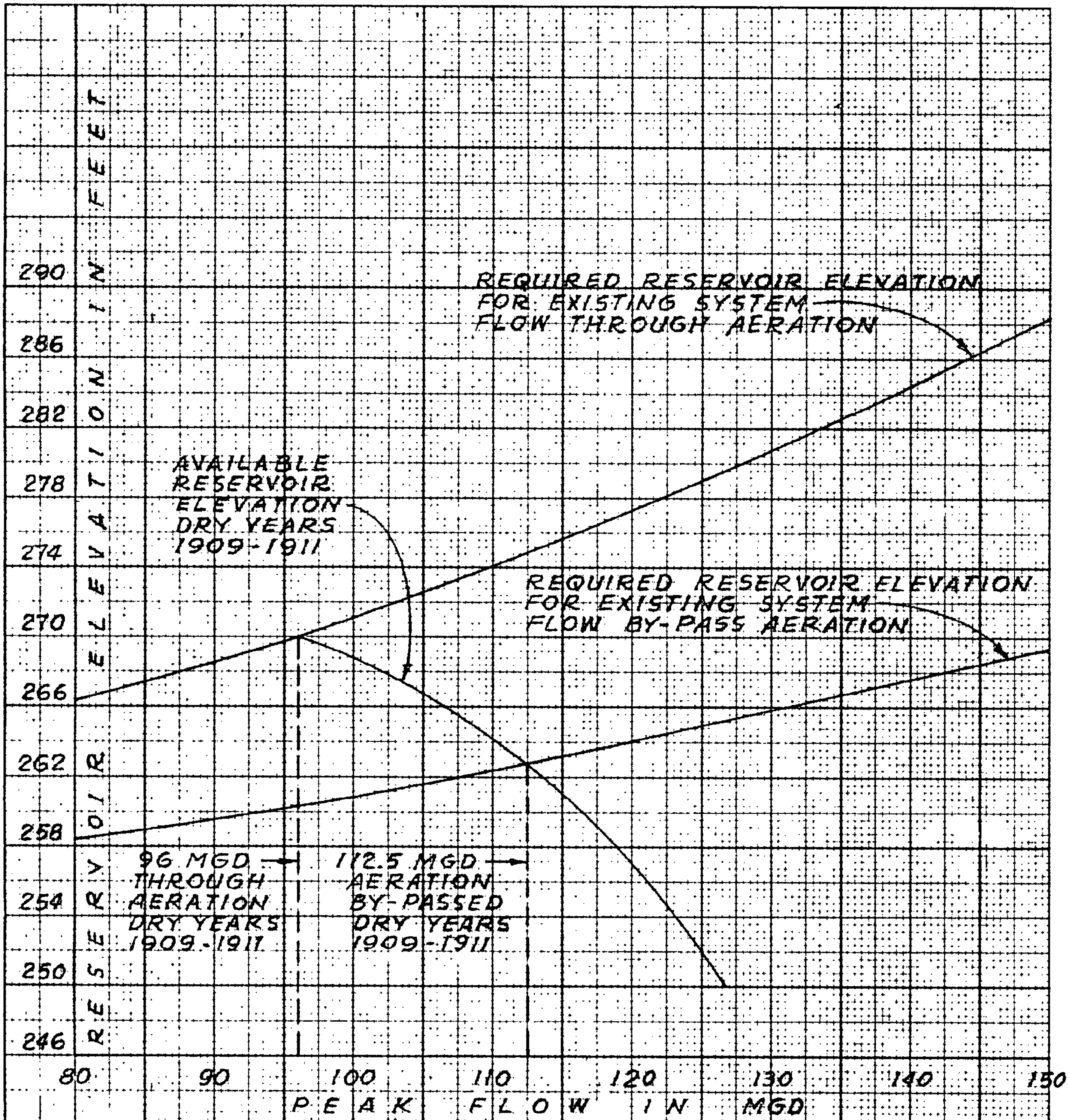
Pumping Station at Scituate Reservoir - Construction in 1968

Design flow	144 mgd
Capacity	150 mgd
Number of pumps	4
Design criteria	Total design flow taken by any two pumps
Capacity of each pump	75 mgd
Minimum level in pump sump	223.8
Hydraulic gradient in discharge conduit	282.8
Approximate pumping station losses	5.0
Maximum total dynamic head	64.0
Horsepower of each motor	1,000 HP
Type of pump drives	3 pumps with electric motors 1 pump with diesel engine
Minimum reservoir level for maximum flow	227.7
Location of pumping station	At the downstream side of Gainer Dam, east of meter chamber

Figure 12 shows the required and available levels in Scituate Reservoir for gravity flow to Water Purification Works.

Figure 13 shows the hydraulic profile for existing aqueduct from Scituate Reservoir to Water Purification Works with pumping.



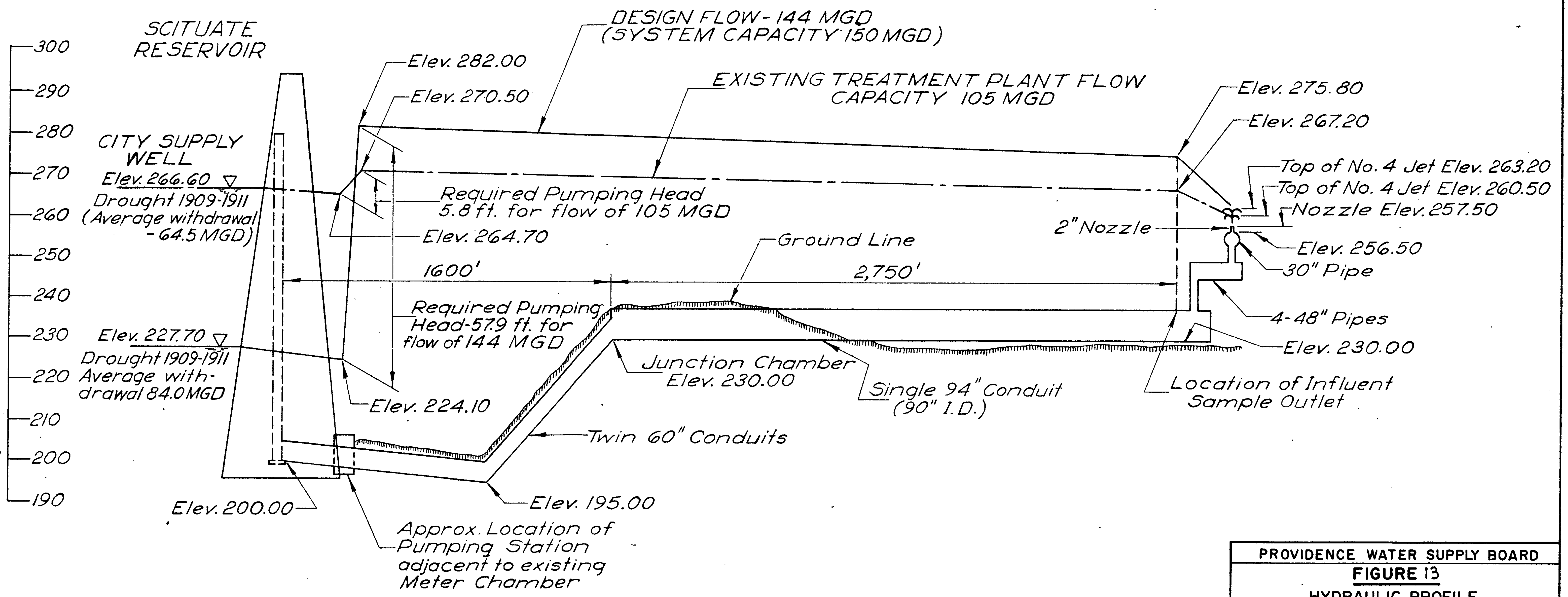
PROVIDENCE WATER SUPPLY BOARD

FIGURE 12

REQUIRED AND AVAILABLE LEVELS IN SCITUATE RESERVOIR FOR GRAVITY FLOW TO WATER PURIFICATION WORKS

CHARLES A. MAGUIRE & ASSOCIATES

ENGINEERS JULY 1964



PROFILE

Scales: Horiz. 1" = 400'
Vert. 1" = 20'

PROVIDENCE WATER SUPPLY BOARD

FIGURE 13

HYDRAULIC PROFILE
EXISTING AQUEDUCT
FROM SCITUATE RESERVOIR TO WATER PURIFICATION WORKS - WITH PUMPING

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
JULY 1964

Alternate Plans for Pumping Station at Scituate Reservoir

Consideration was given to construction of a pumping station at the Water Purification Works, adjacent to influent aerator. However, it is recommended to construct the pumping station at the Scituate Reservoir, adjacent to the meter chamber. At this location the power source for the pump motors is much closer and the intake elevation is lower, allowing pumping with lower levels in Scituate Reservoir during critical drought periods.

IV. WATER PURIFICATION PLANT

The hydraulic capacity of the existing Scituate Water Purification Works was analyzed by office computations in conjunction with field tests. The requirements for additional rapid sand filters at Scituate Water Purification Works and for new Water Purification Works at Big River Reservoir to meet the deficiencies in the existing system for future water supply requirements are presented in this section.

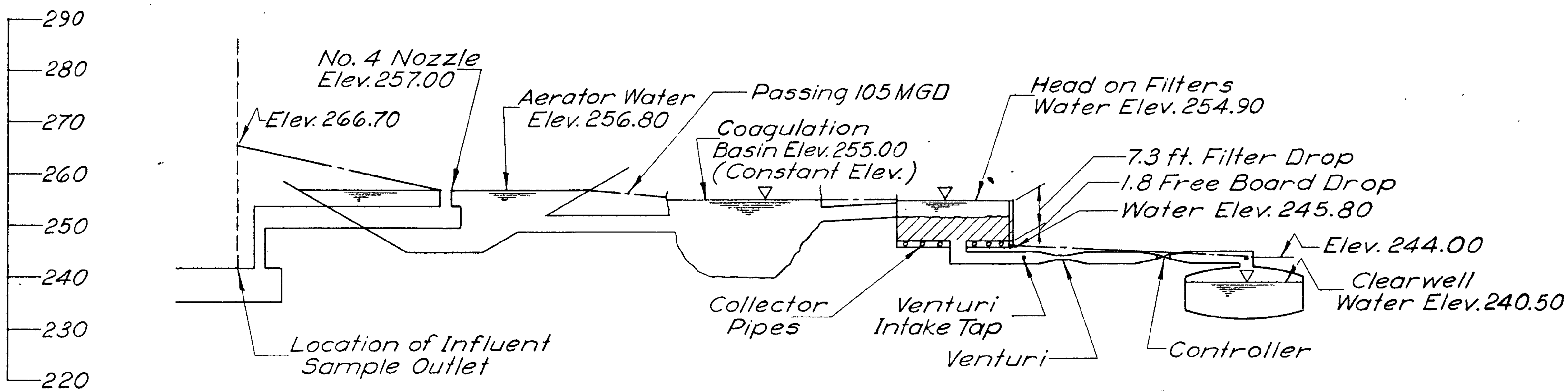
Capacity of Existing Scituate Water Purification Works

The hydraulic losses through the Water Purification Works from the influent aerator to the clear well can be subdivided into losses from influent aerator to coagulation basin, from coagulation basin to sand filters, through sand filters, from sand filters to clear well and through the clear well.

The losses from the influent aerator to the coagulation basin are such that the elevation in the aerator pool rises to the level of existing nozzles at elevation 257.0 for a flow of 112 million gallons per day for a coagulation basin elevation of 255.0. For 150 million gallons per day the aerator pool would rise to elevation 258.5, one foot above the future level of aeration nozzles at elevation 257.5 with coagulation basin at elevation 255.0. Therefore, for a flow of 150 million gallons per day the coagulation basin should be drawn down to elevation 254.0 in order not to back up the aeration nozzles.

Figure 14 shows hydraulic profiles through existing Water Purification Works and Figures 15 and 16 show the head loss curves through the various components of the treatment plant. The losses from the coagulation basin to the filters are minor. The losses through the sand filter are variable and amount to about 1.5 feet for a clean filter and to about 7.5 feet just before backwashing. From the filters to the clear well the losses are appreciable due to the venturi meters, flow controllers and exit loss into the clear well. The flow controllers throttle the flow when the sand filters are clean and open up gradually as they clog, in order to keep a constant rate of flow of 7.5 million gallons per day through each filter. In the future, after the flow is increased to 8.0 million gallons per day through each filter, the hydraulic capacity from the filters to the clear well will still be adequate.

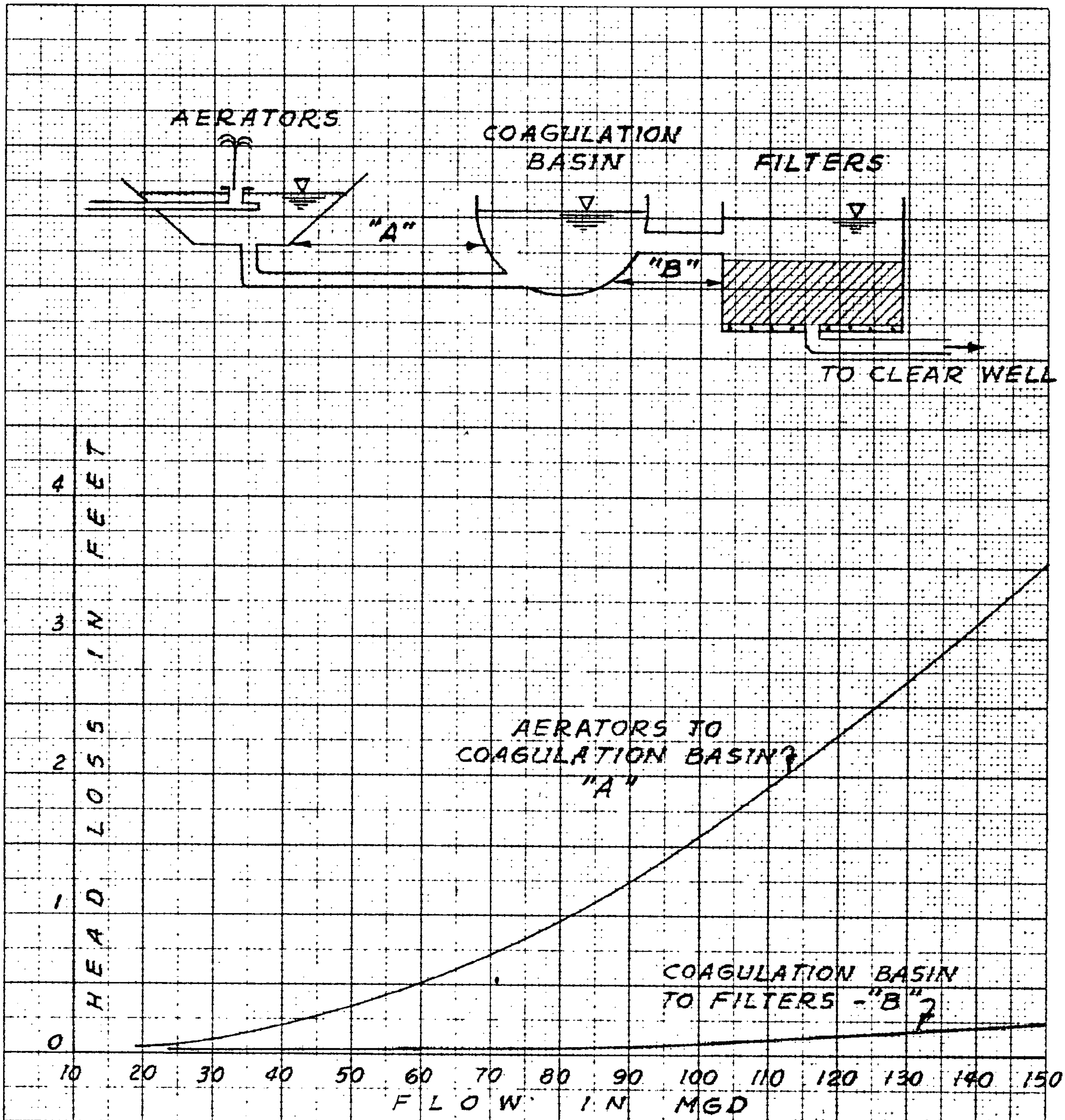
The water level in the clear well is controlled entirely by the stop-log level at the downstream end of the clear well for flows less than 100 million gallons per day and by the back-up from the aqueduct for flows above 114 million gallons per day. Even with no back-up at the siphon chamber at the downstream end of the aqueduct, the back-up at the upstream end of the aqueduct at the Water Purification Works for a flow of 114 million gallons per day would cause the level in the clear well to climb up to the crown of the clear well chamber. Since any back-up of the aqueduct at the siphon chamber or surging of levels in the clear well would cause damage to the sand filters at the 114 million gallons per day flow, it is considered essential to provide some freeboard in the clear well between the effluent pipe outlet and the water level in the clear well. A flow of 100 million gallons per day under normal operations can be considered satisfactory and would give 2 feet of freeboard.



PROFILE

Scales: Horiz. 1" = 400'
Vert. 1" = 20'

PROVIDENCE WATER SUPPLY BOARD
FIGURE 14
HYDRAULIC PROFILE EXISTING WATER PURIFICATION WORKS
CHARLES A. MAGUIRE & ASSOCIATES ENGINEERS JULY 1964



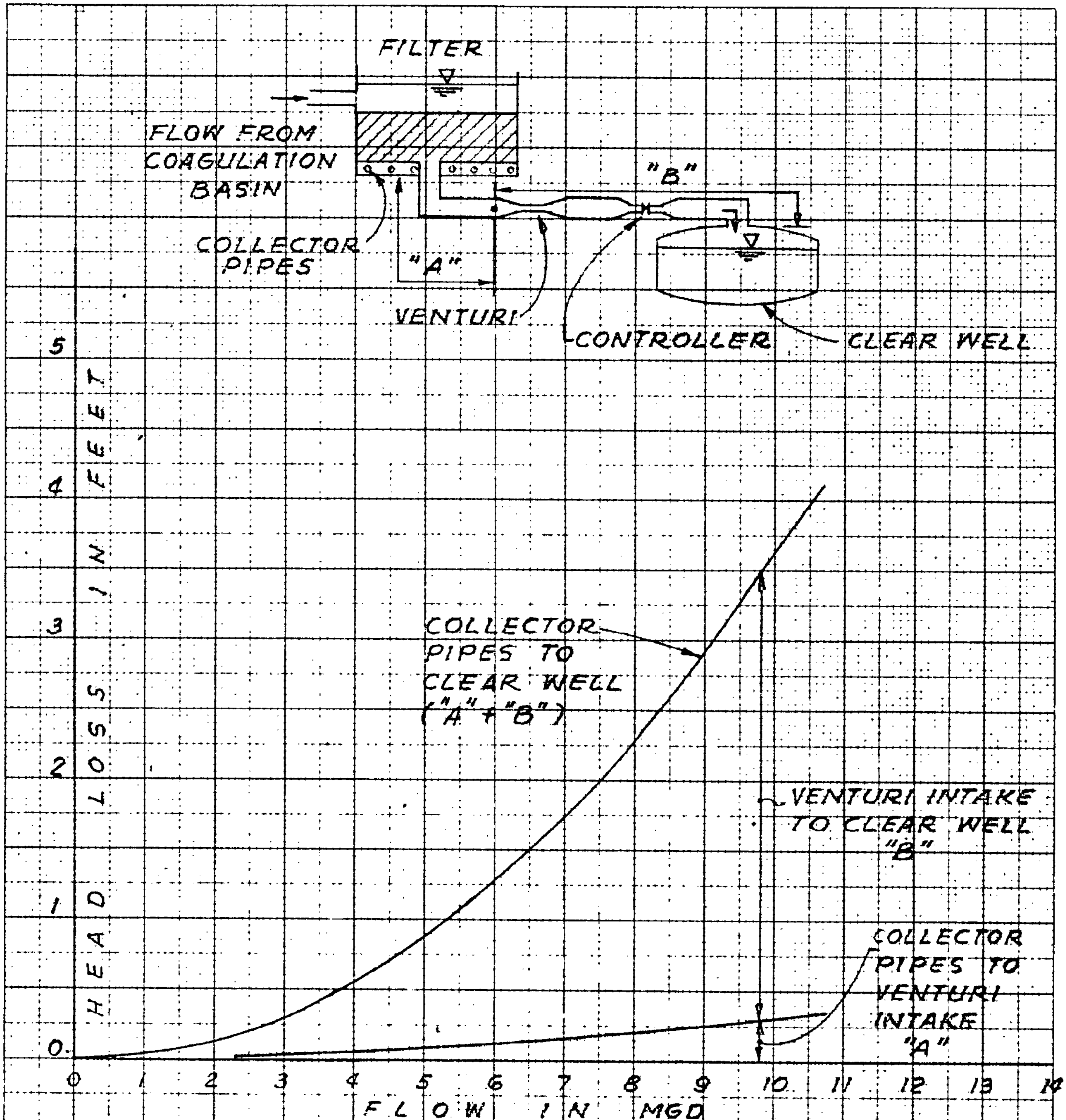
PROVIDENCE WATER SUPPLY BOARD

FIGURE 15

HEAD LOSS CURVES
EXISTING WATER PURIFICATION WORKS

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS

JULY, 1964



PROVIDENCE WATER SUPPLY BOARD

FIGURE 16

HEAD LOSS CURVES
EXISTING SAND FILTERS
AND PIPING SYSTEM

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY, 1964

Additional Sand Filters at Water Purification Works - Construction in 1965

Design flow	32 mgd
Capacity of each filter	8.0 mgd
Number of new filters	4
Total number of filters	18
Total filter capacity	144 mgd
Elevation downstream of the clear well	238.2
Elevation inside the clear well at upstream end	241.2
Location of filters	North of existing filters

Alternate Plans for Additional Sand Filters at Water Purification Works

Consideration was given to constructing six new filters with a capacity of 7.5 million gallons per day each instead of four new filters with a capacity of 8.0 million gallons per day each. However, since 144 million gallons per day would be available for 18 filters at 8.0 million gallons per day capacity each instead of 150 million gallons per day for 20 filters at 7.5 million gallons per day each, the limitations of space requirements and the additional cost ruled out the 20 filters and 18 are recommended.

Water Purification Works at Big River - Construction 1970 to 1980

Design initial peak flow	52 mgd
Design future peak flow	140 mgd
Elevation downstream of clear well	242.7
Elevation coagulation basins	257.5
Minimum allowable Big River Reservoir level	275.0
Location of Water Purification Works	In the vicinity of Big River Reservoir

V. TUNNELS AND AQUEDUCTS

The hydraulic capacity of existing tunnel and aqueduct from Scituate Water Purification Works to the vicinity of Budlong Road in Cranston was analyzed by office computations in conjunction with field tests. The requirements for additional immediate aqueduct and/or tunnel construction and for future tunnel construction to meet the deficiencies in the existing system for future water supply requirements are presented in this section.

Tunnel and Aqueduct from Scituate Water Purification Works to Budlong Road

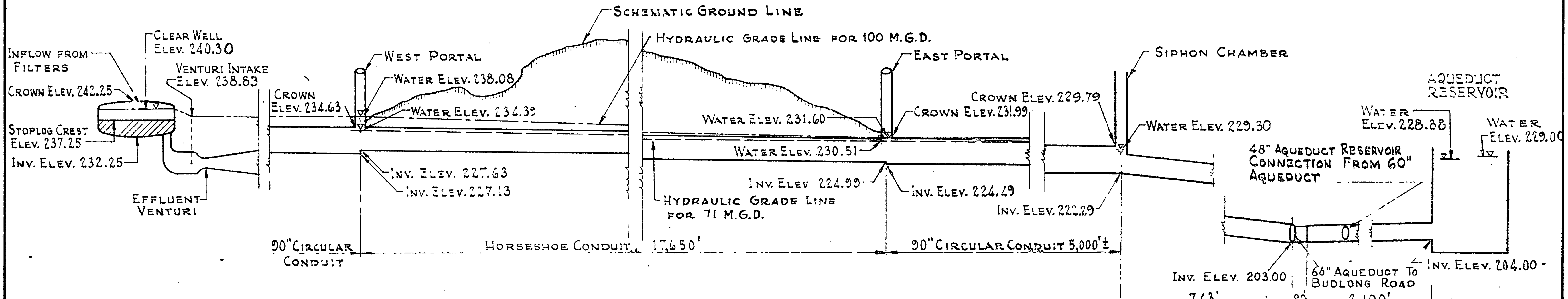
The tunnel from the Water Purification Works is concrete-lined horseshoe section which is equivalent hydraulically to a pipe 7 feet 6 inches in diameter. The aqueduct from the end of the tunnel to the siphon chamber is 7 feet 6 inches in diameter.

At the siphon chamber the flow divides into a 66-inch main known as Scituate Aqueduct, which runs to Budlong Road, and a 60-inch lock joint concrete main known as Neutaconkanult Conduit. Near the beginning of the 66-inch steel main a 60-inch connection runs to Aqueduct Reservoir. The Aqueduct Reservoir is normally maintained near elevation 229.0 and controls the gradient in the siphon chamber.

Field tests were conducted to determine the capacity of the existing tunnel and aqueduct from the Water Purification Works. Water levels were measured in the east and west portals and flow recorded at the plant. Figure 17 shows the hydraulic profile of the existing tunnel and aqueduct from the clear well to the siphon chamber. The tests were run at two flows, 71.0 million gallons per day and 99.5 million gallons per day. The Williams and Hazen roughness coefficient C for the tunnel was 133.

Test conducted on June 23, 1964
Test Area – Filters to East Portal

– – – – – – Test at 1:15 P.M. Flow = 71 M.G.D.
——— - ——— Test at 3:00 P.M. Flow = 99.5 M.G.D.



PROFILE

Scales: Horiz. 1" = 400'
Vert. 1" = 20'

LOSS BETWEEN PORTALS

Time	Flow	Head Loss in feet
1:15 P.M.	71 M.G.D.	3.90
3:00 P.M.	99.5 M.G.D.	6.50

PROVIDENCE WATER SUPPLY BOARD

FIGURE 17

HYDRAULIC PROFILE
FLOW TEST DOWNSTREAM OF TREATMENT PLANT TO AQUEDUCT RESERVOIR

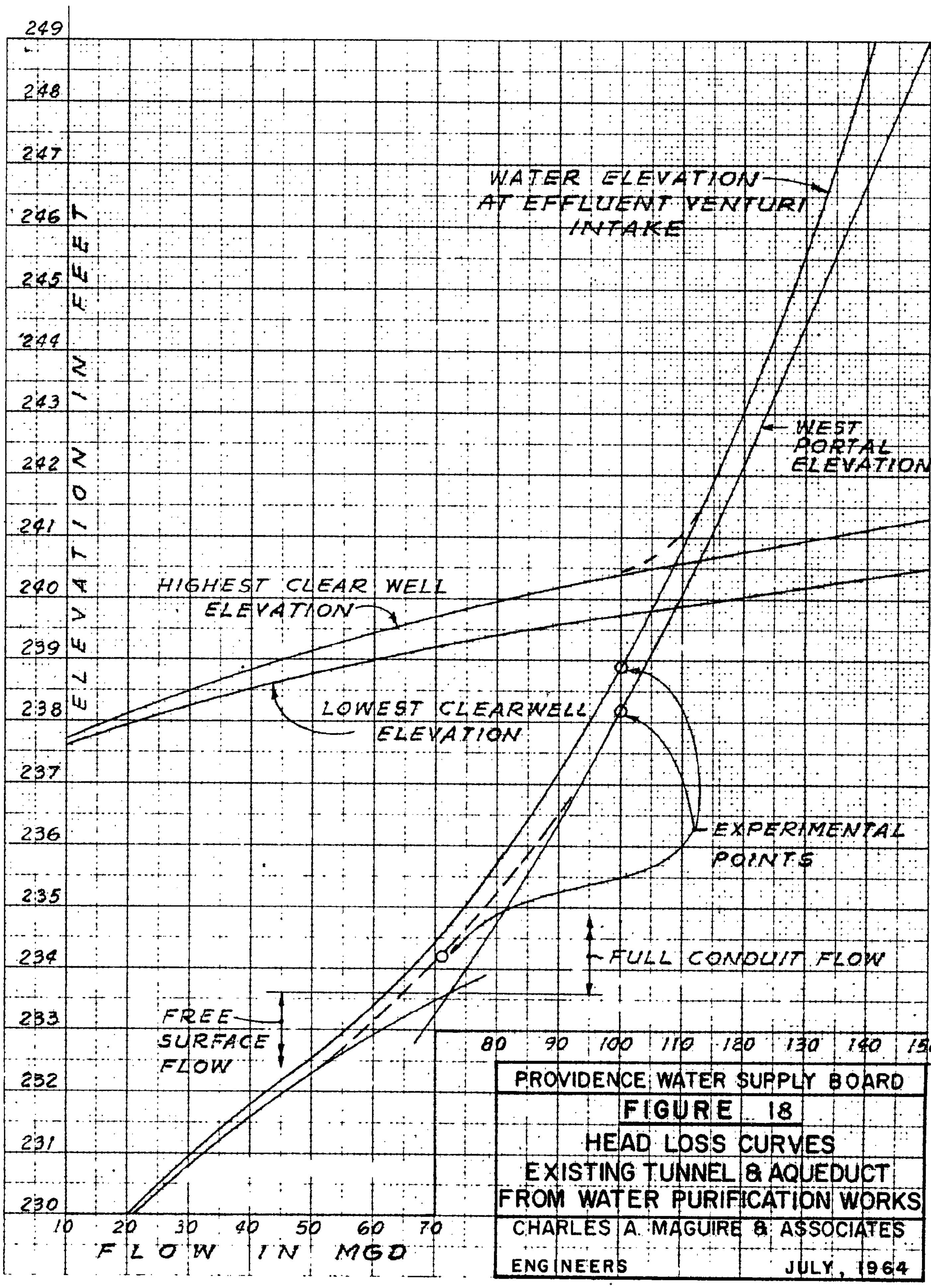
CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY 1964

The tunnel and aqueduct flows under pressure for flows above 100 million gallons per day and as a free surface conduit for flows less than 75 million gallons per day. For flows between 75 million gallons per day and 100 million gallons per day part of the tunnel as well as the entire aqueduct would flow as a free surface conduit. Figure 18 shows the back-up levels of the existing tunnel and aqueduct at the clear well and at west portal.

The capacity of the tunnel is 100 million gallons per day for normal operations. This capacity is controlled by back-up of water level in the clear well to elevation 240.5, 2 feet below the effluent pipe outlet at the crown of the clear well. If water level was allowed to submerge the effluent pipe outlet proper functioning of flow controllers would be lost. Even with water levels below the effluent pipe outlet damage to sand filters could occur if surges in the clear well were transmitted through the effluent piping.

The aqueduct between the siphon chamber and Budlong Road flows under pressure. Figure 19 shows the head loss curves for this 66-inch conduit.

Figure 20 shows a composite hydraulic profile from Scituate Reservoir to Budlong Road for future flows with improvements to allow 144 million gallons per day flow through the Water Purification Works.

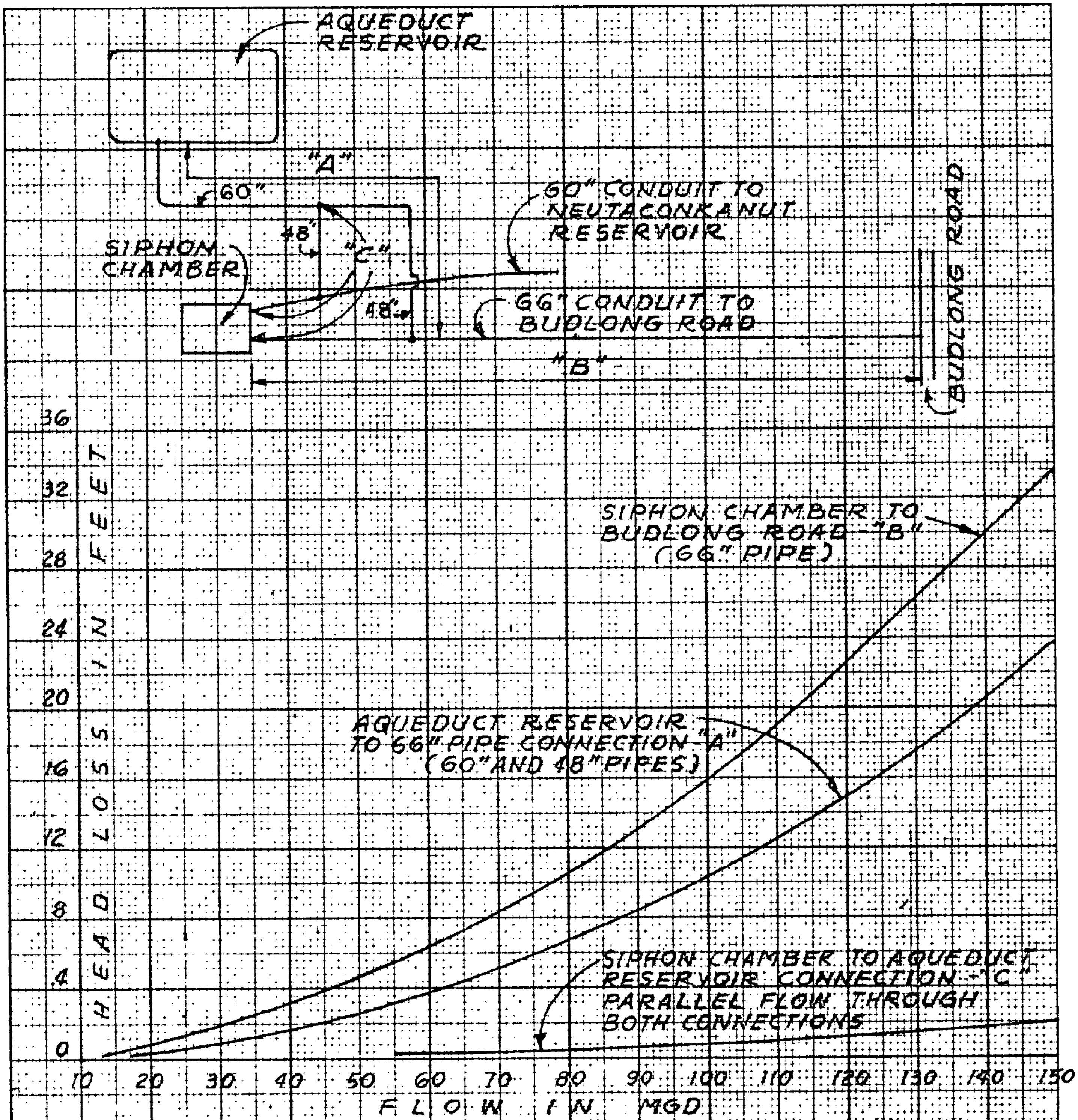


PROVIDENCE WATER SUPPLY BOARD

FIGURE 18

HEAD LOSS CURVES
EXISTING TUNNEL & AQUEDUCT
FROM WATER PURIFICATION WORKS

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY, 1964

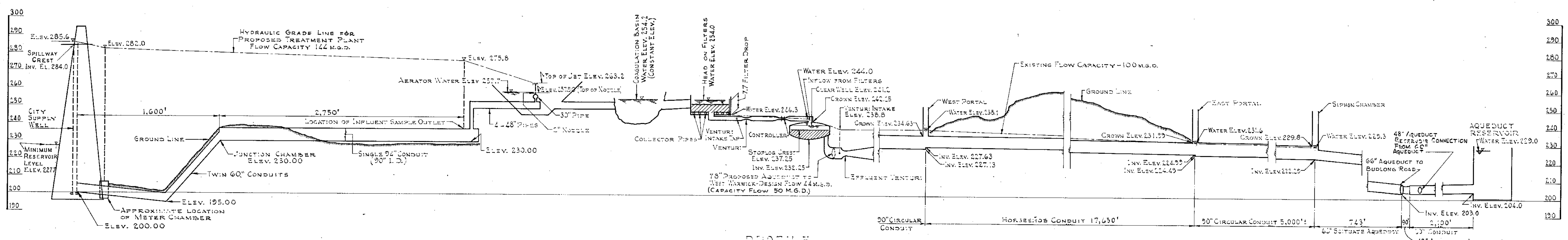


PROVIDENCE WATER SUPPLY BOARD

FIGURE 19

HEAD LOSS CURVES
SIPHON CHAMBER TO AQUEDUCT
RESERVOIR AND TO BUDLONG ROAD

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY 1964



PROFILE

SCALES: HORIZ. 1" = [illegible] VERT. 1" = [illegible]

NOTE: SPILLWAY CREST AT SCITUATE RESERVOIR IS AT ELEVATION 284.0. THEREFORE, A FLOW OF 144 MGD WOULD NOT BE POSSIBLE WITHOUT PUMPING.

PROVIDENCE WATER SUPPLY BOARD

FIGURE 20

COMPOSITE HYDRAULIC PROFILE FROM SCITUATE RESERVOIR TO BUDLONG ROAD FOR FUTURE FLOW

CHARLES A. MAGUIRE & ASSOCIATES ENGINEERS JULY 1964

Proposed Aqueduct from Scituate Water Purification Works to West Warwick - Construction in 1965

Design flow	44 mgd
Capacity flow	50 mgd
Diameter	6'-6"
Type of construction	Cut and cover conduit

Proposed Route to Juncture with Tunnel to Budlong Road

Length	18,000 feet (to a tunnel shaft in West Warwick for a tunnel to Budlong Road)

Head loss - C = 120; HL/1,000 ft. = 0.231; HL = 4.2 ft.

Maximum elevation of hydraulic gradient at West Warwick	234.0
Maximum elevation of hydraulic gradient at Water Purification Works	238.2
Invert at West Warwick tunnel shaft	220.0
Invert at Water Purification Works	228.0

Proposed Alternate Route to Juncture with Aqueduct to Budlong Road

Length - 22,750 feet (for an alternate route with aqueduct from West Warwick to Budlong Road)

Head loss - C = 120; HL/1,000 ft. = 0.231; HL = 5.3 ft.

Maximum elevation of hydraulic gradient at West Warwick	233.2
Maximum elevation of hydraulic gradient at Water Purification Works	238.5
Invert at West Warwick aqueduct junction	220.0
Invert at Water Purification Works	228.0

Proposed Conduit from West Warwick to Budlong Road - Construction in 1965

Design flow	96 mgd
Capacity flow	100 mgd

Proposed Tunnel Route

Type of construction	Deep rock tunnel
Diameter	8'-0"
Length	23,000 feet

Head loss - C = 120; HL/1,000 ft. = 0.304; HL = 7.0 feet

Maximum elevation of hydraulic gradient at Budlong Road	227.0
Maximum elevation of hydraulic gradient at West Warwick	234.0

Proposed Alternate Aqueduct Route

Type of construction	Cut and cover aqueduct
Diameter	8'-6"
Length	27,300 feet

Head loss - C = 120; HL/1,000 ft. = .225; HL = 6.2

Maximum elevation of hydraulic gradient at Budlong Road	227.0
Maximum elevation of hydraulic gradient at West Warwick	233.2
Invert at West Warwick	218.5
Invert at Budlong Road	56.0

Because the distance is longer by 9,000 feet from Budlong Road to the Water Purification Works if aqueduct is constructed from West Warwick instead of tunnel, the diameter has to be increased from 8 feet to 8 feet 6 inches in order to have similar elevations at the Water Purification Works.

Figure 21 shows the head loss curves for proposed tunnel and aqueduct and Figure 22 shows the head loss curves for proposed alternate aqueduct route.

Proposed Tunnel from Big River to West Warwick - Construction 1970 to 1980

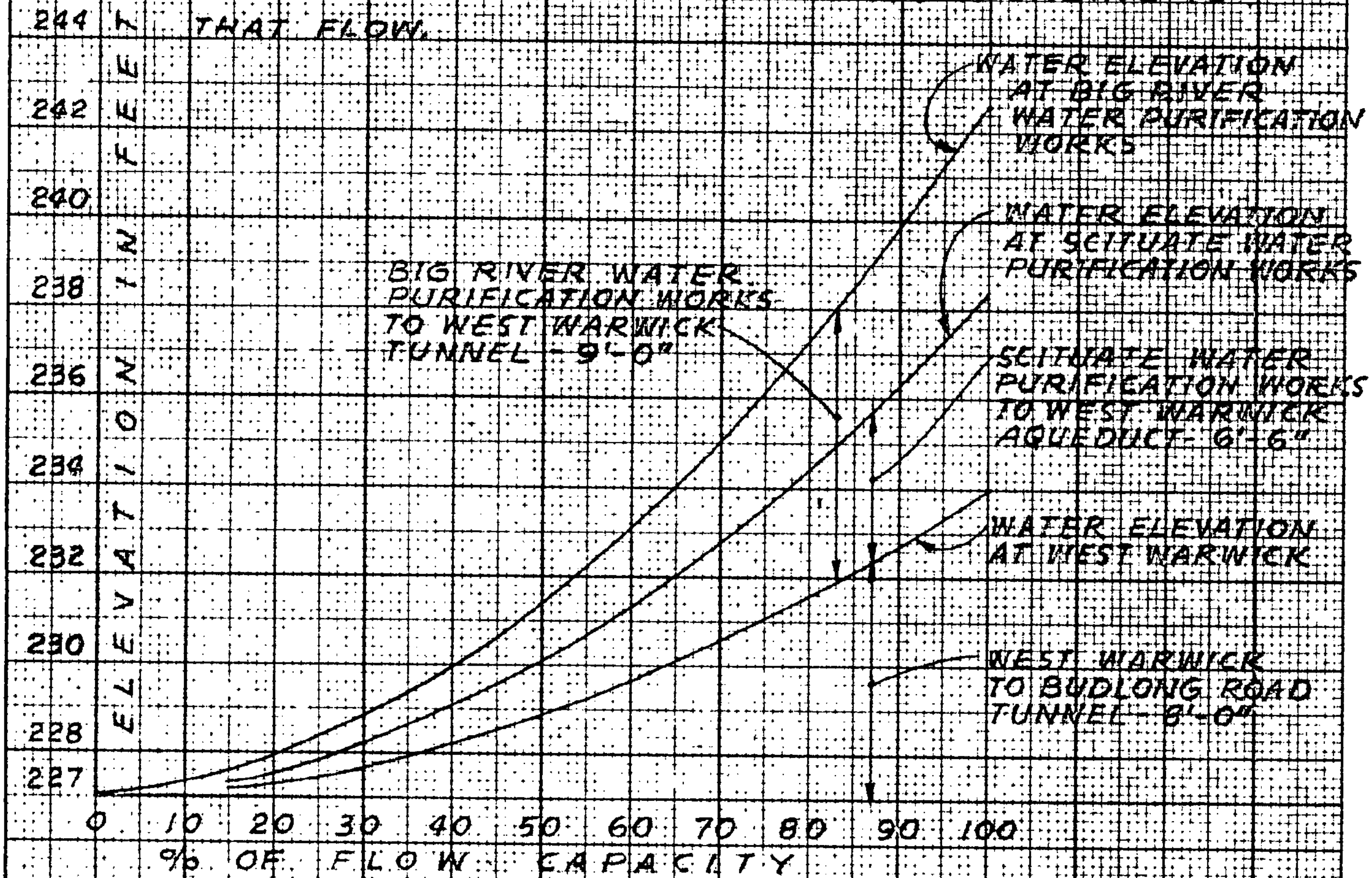
Design flow	125 mgd
Diameter	9'-0"
Type of construction	Deep rock tunnel
Length	34,000 feet
Head loss - C = 120; HL/1,000 ft.	= 0.256; HL = 8.7 feet
Maximum elevation of hydraulic gradient at West Warwick	234.0
Maximum elevation of hydraulic gradient at Water Works	242.7

Alternate Plans for Aqueduct from Scituate Water Purification Works to West Warwick

For the aqueduct from the Scituate Water Purification Works to the vicinity of West Warwick construction of a tunnel as well as a cut and cover aqueduct were considered. However, for the small 6-foot 6-inch diameter size tunnel construction would not be economical and cut and cover aqueduct is recommended. Variations in aqueduct location may be made following topographic surveys and subsurface soil investigations. In the vicinity of the Water Purification Works connection may be made either at the venturi intake chamber by tunneling a very short distance under the existing conduits or by connecting to the west portal after dewatering the coagulation basin.

NOTES:

1. SCITUATE WATER PURIFICATION WORKS TO WEST WARWICK FLOW CAPACITY 50 MGD.
2. BIG RIVER WATER PURIFICATION WORKS TO WEST WARWICK FLOW CAPACITY 125 MGD.
3. WEST WARWICK TO BUDLONG ROAD - FLOW CAPACITY 100 MGD.
4. FLOW IN ANY CONDUIT IS DETERMINED BY MULTIPLYING PERCENT CAPACITY BELOW BY CORRESPONDING FLOW CAPACITY
5. HEAD LOSS FOR ANY FLOW IS DETERMINED BY DEDUCTING ELEV. 227.0 FROM ELEVATION CORRESPONDING TO THAT FLOW.



PROVIDENCE WATER SUPPLY BOARD

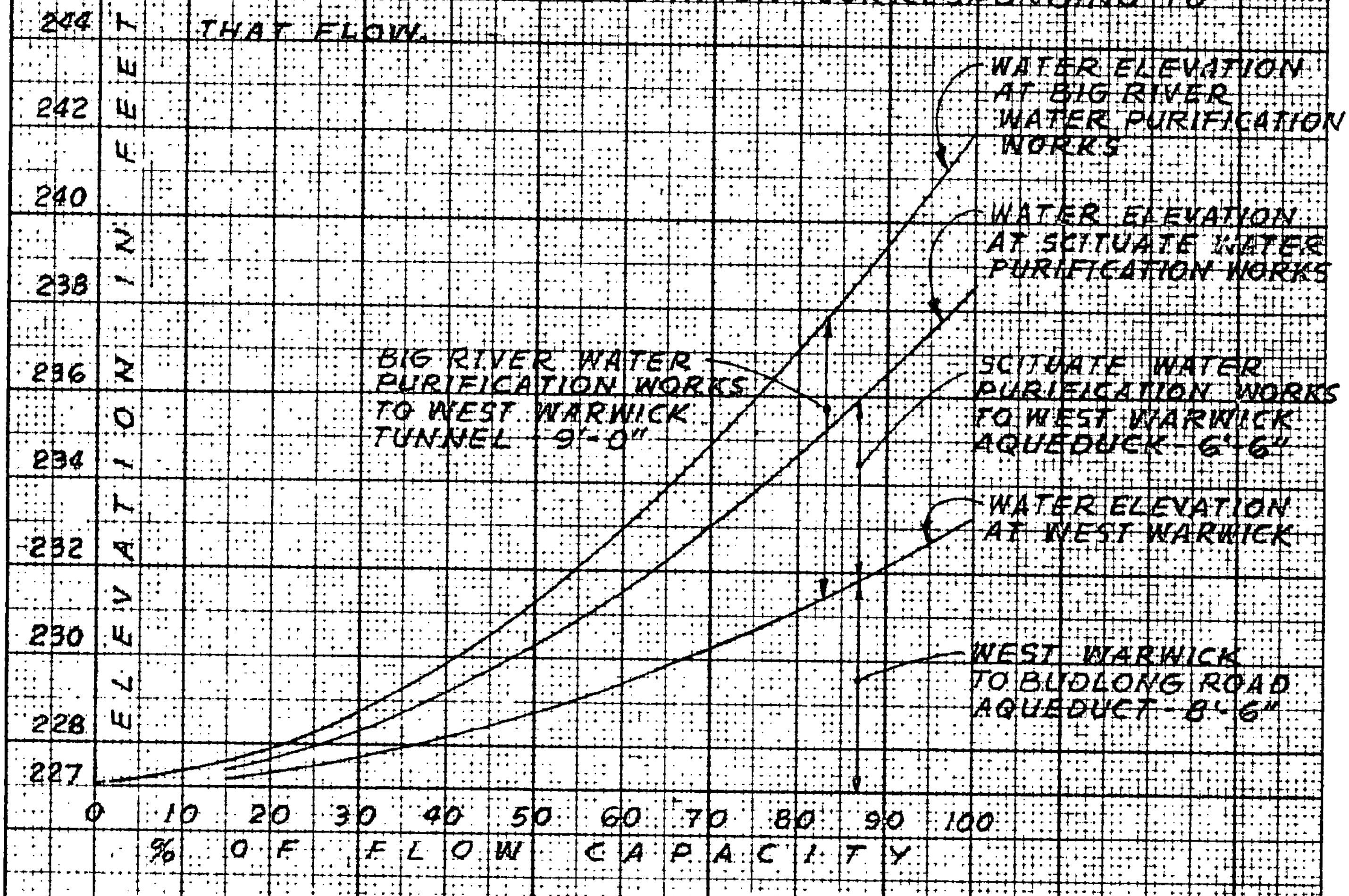
FIGURE 21

HEAD LOSS CURVES
PROPOSED TUNNEL & AQUEDUCT

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS — JULY 1964

NOTES:

1. SCITUATE WATER PURIFICATION WORKS TO WEST WARWICK FLOW CAPACITY 50 MGD.
2. BIG RIVER WATER PURIFICATION WORKS TO WEST WARWICK FLOW CAPACITY 125 MGD.
3. WEST WARWICK TO BUDLONG ROAD - FLOW CAPACITY 100 MGD.
4. FLOW IN ANY CONDUIT IS DETERMINED BY MULTIPLYING PERCENT CAPACITY BELOW BY CORRESPONDING FLOW CAPACITY
5. HEAD LOSS FOR ANY FLOW IS DETERMINED BY DEDUCTING ELEV. 227.0 FROM ELEVATION CORRESPONDING TO THAT FLOW.



PROVIDENCE WATER SUPPLY BOARD

FIGURE 22

HEAD LOSS CURVES
PROPOSED ALTERNATE AQUEDUCT

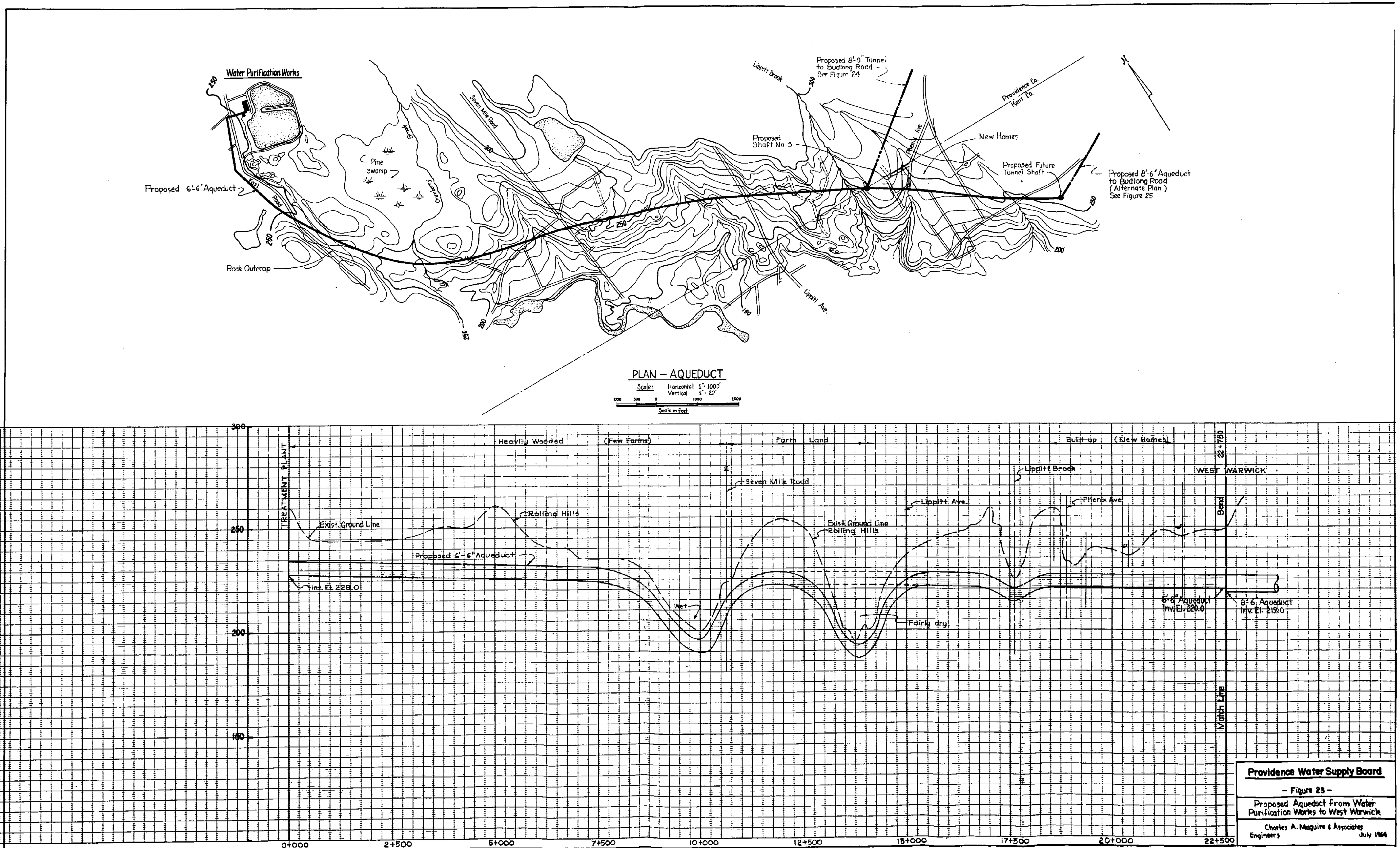
CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS JULY 1964

For final design a detailed comparison should be made of the two alternative routes to the two different points of connection. Figure 23 shows the plan and profile of the aqueduct from the venturi intake chamber to the tunnel shaft for flow from Big River Reservoir. The tunnel shafts would be at about Station 18+000 for a tunnel from the vicinity of West Warwick to Budlong Road and at about Station 22+750 for an aqueduct from the vicinity of West Warwick to Budlong Road.

Alternate Plans for Conduit from West Warwick to Budlong Road

Figure 24 shows the plan and profile for a tunnel from the shaft in the vicinity of West Warwick at Station 18+000 on the 6-foot 6-inch aqueduct to a tunnel shaft in the vicinity of Budlong Road near Woodridge School. The tunnel which is 8 feet in diameter is set deeply, as described in the geologic report, in order to insure that it is in sound rock. An intermediate shaft is for construction purposes, but it would also be useful in servicing adjacent areas.

Figure 25 shows the plan and profile for an aqueduct from the vicinity of West Warwick at Station 22+750 on the 6-foot 6-inch aqueduct to the vicinity of Budlong Road near Woodridge School. Figure 26 shows proposed tunnel and aqueduct routes on an aerial photograph. The aqueduct is 8 feet 6 inches in diameter in order to compensate for the greater head losses due to longer length. This aqueduct location would have the advantage of servicing the areas in West Warwick and Cranston along the route and would result in a shorter future extension of aqueduct to the east from the tunnel shaft in the vicinity of West Warwick.

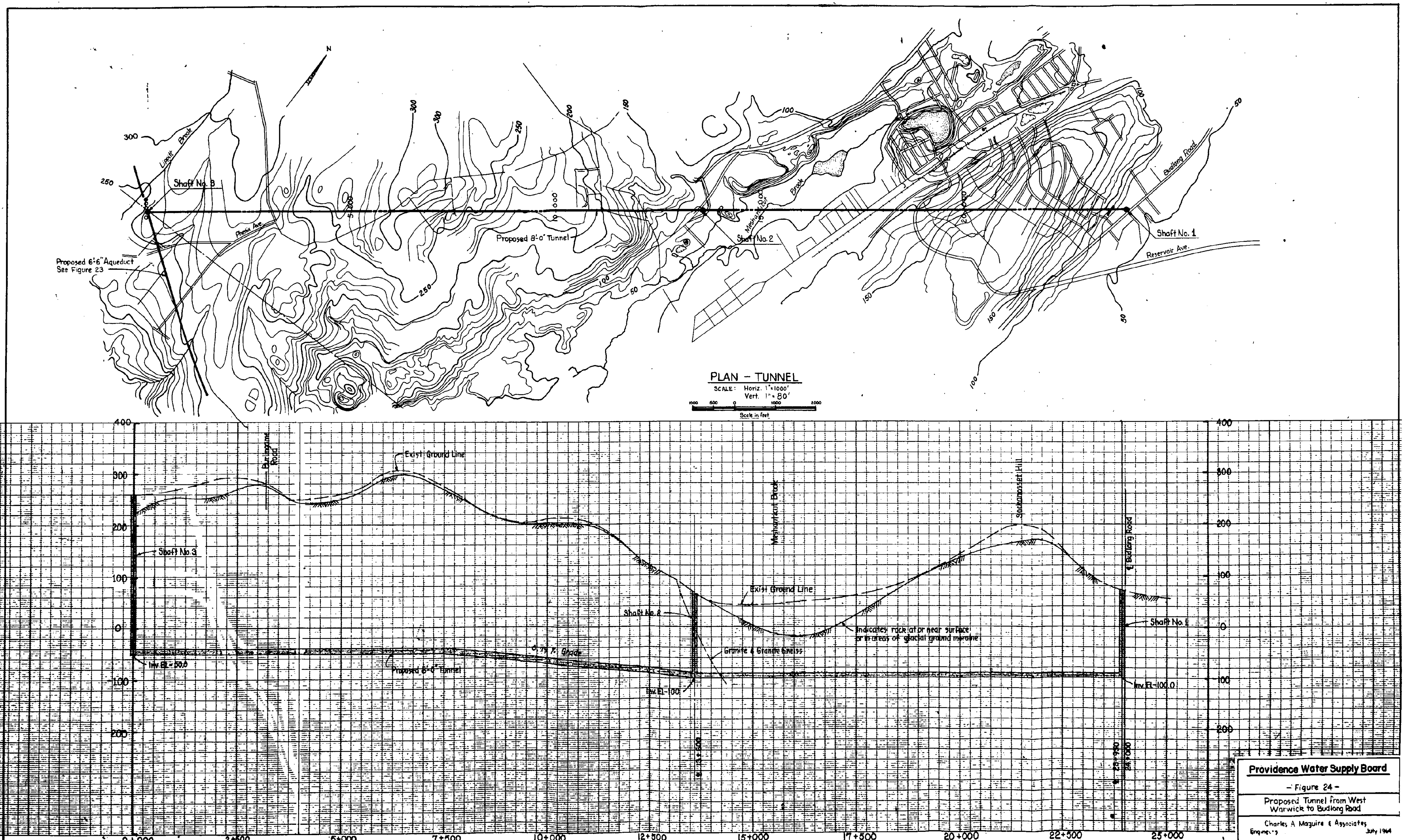


Providence Water Supply Board

– Figure 23 –

Proposed Aqueduct from Water Purification Works to West Warwick

Charles A. Maguire & Associates
Engineers
July 1964

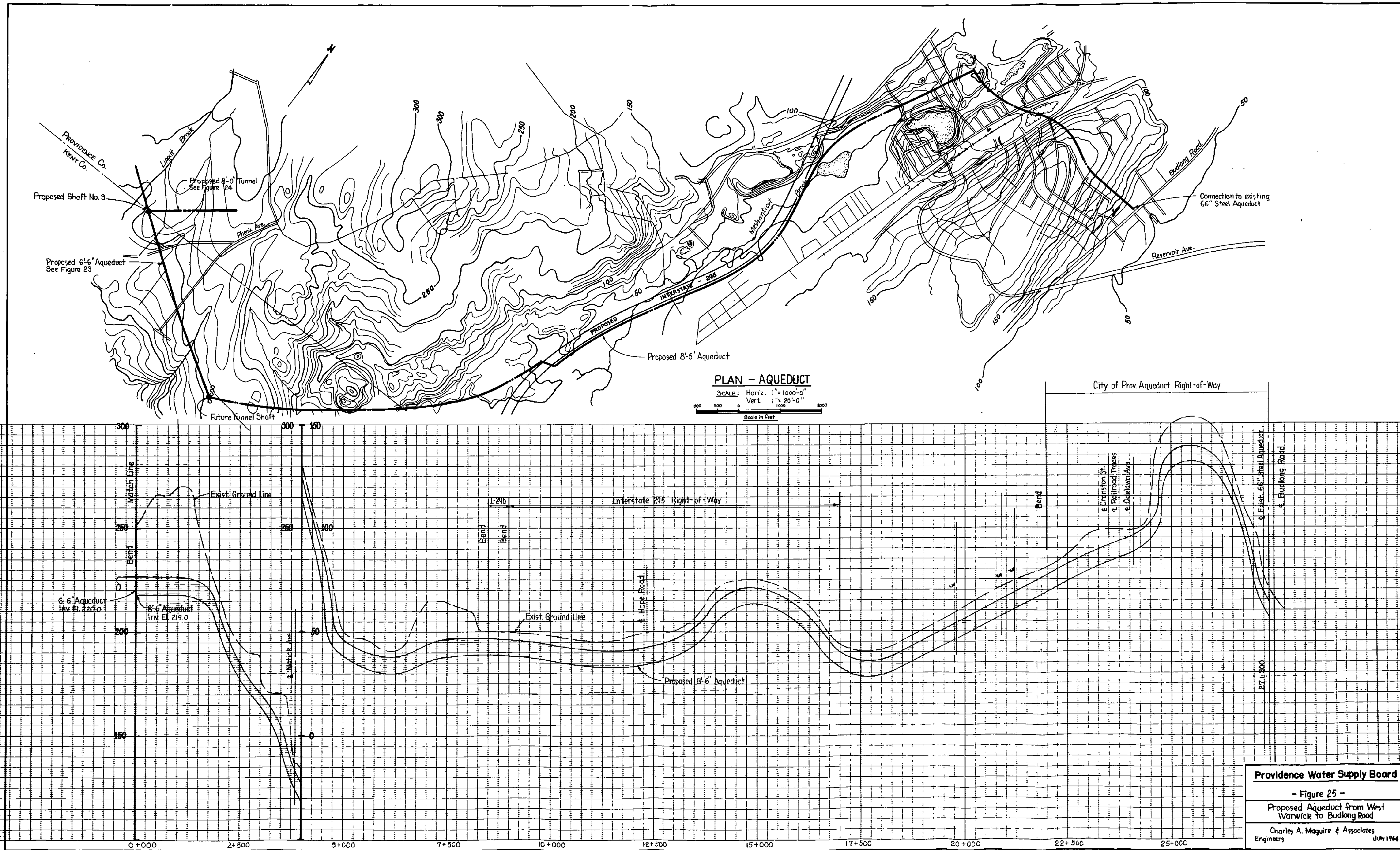


Providence Water Supply Board

- Figure 24 -

Proposed Tunnel from West Warwick to Budlong Road

Charles A. Maguire & Associates
Engineers
July 1964



Providence Water Supply Board

- Figure 25 -

Proposed Aqueduct from West Warwick to Budlong Road

Charles A. Maguire & Associates
Engineers
July 1964

City of Providence Water Supply Board
AQUEDUCT STUDY
LEGEND
Proposed 6'-6" Aqueduct
Proposed 8'-6" Aqueduct
Proposed ℄ Interstate 295
Proposed 8'-0" Tunnel
N
Charles A. Maguire & Associates — Engineers — Providence, Rhode Island
SCALE
1400 700 0 1400 2800
— IN FEET —
WATER PURIFICATION WORKS
6'-6" AQUEDUCT
CENTERLINE OF PROPOSED I-295
8'-6 AQUEDUCT
SHAFT NO. 1
8'-0" TUNNEL
SHAFT NO. 2
SHAFT NO. 3
6'-6" AQUEDUCT
8'-6" AQUEDUCT
I-295
FUTURE TUNNEL SHAFT
Lippitt Ave.

— FIGURE 26 —

The aqueduct is more advantageous than the tunnel from the service point of view. However, since the costs of the two alternates are comparable and could vary appreciably according to rock excavation, ground water conditions, and final topographic surveys as determined during the final design, it is recommended to investigate both routes for final design and if both routes still appear comparable to take alternate bids.

Another tunnel route was considered from existing water purification works to Budlong Road along the route of the existing tunnel. This route is considered to be unsatisfactory for the following reasons:

1. No additional areas could be served with water from aqueduct or tunnel shaft locations along the existing tunnel and aqueduct route, whereas the proposed tunnel and aqueduct serve large tributary populations along the route.

2. Danger of damage to the existing tunnel due to blasting of a new tunnel located relatively close to the existing tunnel. (Location within the existing right-of-way is not practical and a distance of at least 300 feet to the existing tunnel is recommended.)

3. In case of a rock fault slide due to an earthquake, both the existing and the proposed tunnel would have a greater chance to be damaged simultaneously if they are located relatively close to each other.

4. Tunnel from Big River to Scituate Water Purification Works would be longer than a tunnel to the vicinity of West Warwick in the future.

VI. ESTIMATES OF COST

For immediate construction scheduled to start in 1965 preliminary cost estimates are as follows:

TABLE XII

SUMMARY OF COSTS - TUNNEL PLAN

Aqueduct from Water Purification Works to vicinity of West Warwick - 6'-6" diameter	
18,000 feet at $160/ft.	$ 2,880,000
Tunnel from vicinity of West Warwick to Budlong Road - 8'-0" diameter	
23,000 feet at $270/ft.	6,210,000
Additional sand filters	2,500,000(1)
Total Construction Cost	$11,590,000
10% Engineering and Contingencies	1,159,000
Total Cost	$12,749,000
Say	$13,000,000

(1) Cost estimate furnished by Providence Water Supply Board.

For an alternate plan with an aqueduct instead of a tunnel from the vicinity of West Warwick to Budlong Road the cost estimates are as follows:

TABLE XIII

SUMMARY OF COSTS - AQUEDUCT ALTERNATE PLAN

Aqueduct from Water Purification Works to vicinity of West Warwick - 6'-6" diameter	
22,750 feet at $160/ft.	$ 3,640,000
Aqueduct from vicinity of West Warwick to Budlong Road - 8'-6" diameter	
27,300 feet at $224/ft.	6,120,000
Additional sand filters	2,500,000 (1)
Total Construction Cost	$12,260,000
10% Engineering and Contingencies	1,226,000
Total Cost	$13,486,000
Say	$13,500,000

(1) Cost estimate furnished by Providence Water Supply Board.

For budgeting purposes it is recommended to allocate 13 million dollars. The actual decision as to whether a tunnel or aqueduct is to be built should be withheld until detailed subsurface and topographic information becomes available for a more accurate comparison.

The above engineering cost estimates are based upon currently prevailing labor and materials costs for the nature of construction involved. In the event the project construction is delayed beyond January 1965, an allowance of about 5 percent per annum is recommended for budgeting funding purposes.

VII. SEQUENCE OF CONSTRUCTION

The proposed sequence of construction consists of the following:

A. Immediate Construction

1. Aqueduct from Water Purification Works to tunnel shaft in the vicinity of West Warwick (1965).

2. Tunnel from vicinity of West Warwick to vicinity of Budlong Road in Cranston (1965).

3. Additional sand filters at Water Purification Works (1965).

B. Future Construction (1968 to 1980)

1. Pumping station at Scituate Reservoir to pump the flow to Water Purification Works (1968).

2. Big River Reservoir including dam, dikes and intake to new Water Purification Plant.

3. Water Purification Plant in the vicinity of Big River Reservoir.

4. Tunnel from the new Water Purification Plant to tunnel shaft in the vicinity of West Warwick.

C. Distant Future Construction (beyond 1996)

1. Development of supplemental safe yield from Wood River and/or Flat River in conjunction with Big River Reservoir.

VIII. CONCLUSIONS AND RECOMMENDATIONS

1. Subsurface soil explorations and topographic surveys should be initiated immediately, during 1964, for the two alternate routes of aqueduct and tunnel outlined in this report.

2. Preparation of final plans and specifications should be undertaken during 1964 in conjunction with the analysis of subsurface and topographic information.

3. Construction of aqueduct tunnel and sand filters should be initiated during 1965 in accordance with the proposed sequence of construction outlined in this report.

4. Pumping station construction at Scituate Reservoir should be initiated during 1968, after the start of construction of tunnel, aqueduct and sand filters.

5. If the State fails to proceed with the proposed development of Big River Reservoir, City of Providence should then take the necessary action.

6. The construction of dam, dikes and intake for Big River Reservoir; water purification works near the reservoir; and a tunnel from Big River to the vicinity of West Warwick should be scheduled in accordance with future needs sometime between 1970 and 1980.

7. The development of additional water sources, such as Wood and possibly Flat Rivers, should be considered in conjunction with Big River Reservoir for needs beyond about 1996.

APPENDIX A

SCITUATE RESERVOIR STORAGE CURVE

APPENDIX B

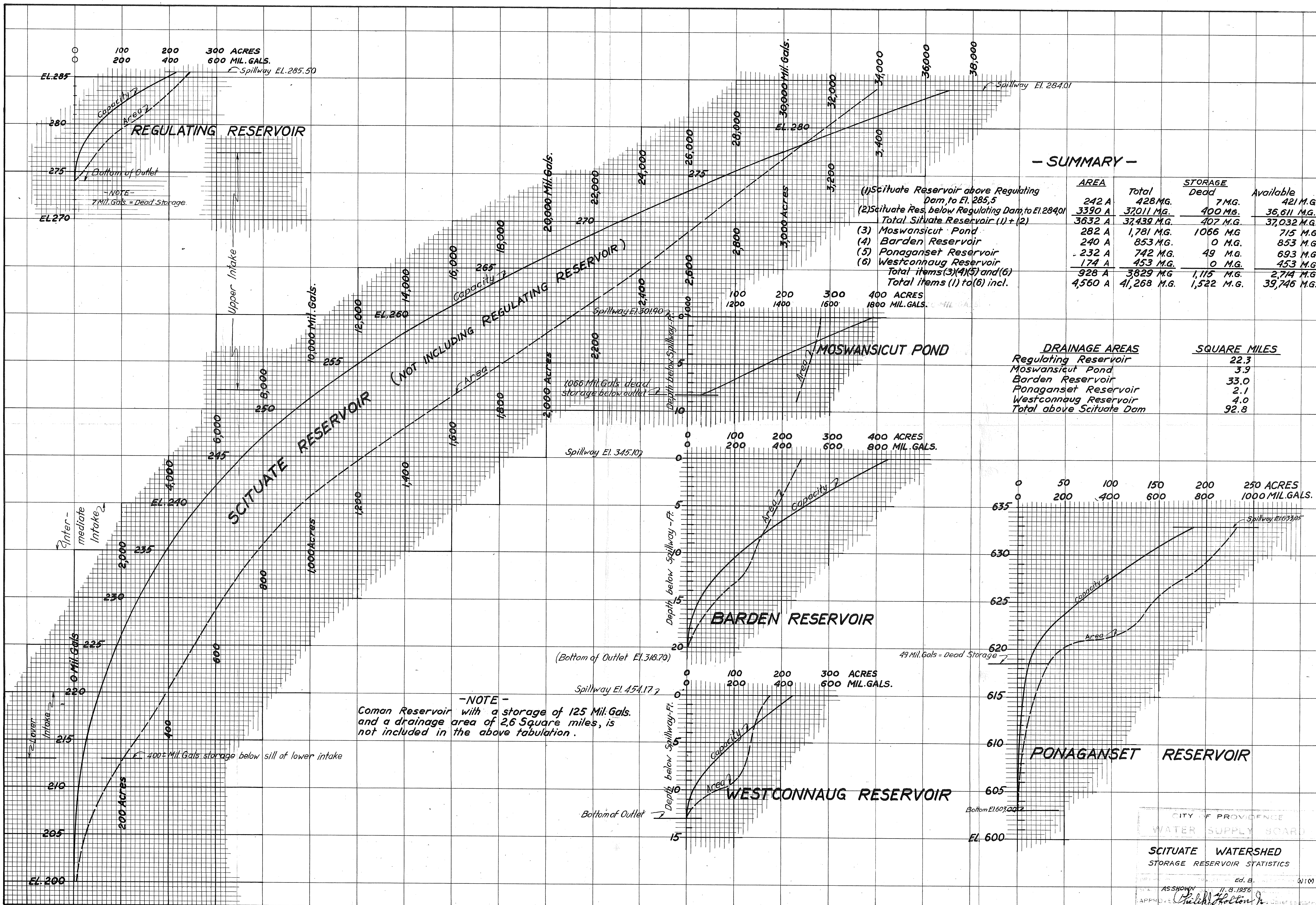
GEOLOGY

APPENDIX C

SAFE YIELD AND RESERVOIR DRAWDOWN STUDIES

APPENDIX D

FLOW CAPACITIES AND GRADIENTS FOR EXISTING WATER WORKS SYSTEM



- SUMMARY -

	AREA	STORAGE Total	Dead	Available
(1) Scituate Reservoir above Regulating Dam, to El. 285,5	242 A	428 M.G.	7 M.G.	421 M.G.
(2) Scituate Res. below Regulating Dam, to El. 284,01	3390 A	37,011 M.G.	400 M.G.	36,611 M.G.
Total Situate Reservoir (1) + (2)	3632 A	37,439 M.G.	407 M.G.	37,032 M.G.
(3) Moswansicut Pond	282 A	1,781 M.G.	1066 M.G	715 M.G.
(4) Barden Reservoir	240 A	853 M.G.	0 M.G.	853 M.G.
(5) Ponaganset Reservoir	232 A	742 M.G.	49 M.G.	693 M.G.
(6) Westconnaug Reservoir	174 A	453 M.G.	0 M.G.	453 M.G.
Total items (3)(4)(5) and (6)	928 A	3,829 M.G	1,115 M.G.	2,714 M.G.
Total items (1) to (6) incl.	4,560 A	41,268 M.G.	1,522 M.G.	39,746 M.G.

DRAINAGE AREAS	SQUARE MILES
Regulating Reservoir	22.3
Moswansicut Pond	3.9
Barden Reservoir	33.0
Ponaganset Reservoir	2.1
Westconnaug Reservoir	4.0
Total above Scituate Dam	92.8

-NOTE-
Coman Reservoir with a storage of 125 Mil. Gals. and a drainage area of 2.6 Square miles, is not included in the above tabulation.

CITY OF PROVIDENCE
WATER SUPPLY BOARD

SCITUATE WATERSHED
STORAGE RESERVOIR STATISTICS

Ed. B. WIM
AS SHOWN 11.8.1956
APPROVED Philip J. Holton Jr. Chief Engineer

APPENDIX "A"