THE DESIGN OF A DAM AT THE JUNCTION OF THE UPPER AND LOWER MYSTIC LAKES WINCHESTER MASS.

By

JOHN L. VAUPEL.

1922

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May 29th 1922.

Prof. A.M.Merrill,
Secretary of the Faculty,
Mass. Inst. of Technology,
Cambridge, Massachusetts.

Dear Sir:

In accordance with the Faculty Rules for graduation I hereby submit my thesis "The Design of a Dam at the Junction of the Upper and Lower Mystic Lakes, Winchester Massachusetts."

Respectfully,

Acknowledgements

Thanks are due to all members of the Institute Instructing Staff for their logical and thorough supervision in the work prescribed for me as an undergraduate. I wish, especially, to thank Mr. H. K. Barrows for the helpful suggestions made to me in the preparation of this thesis and, also, Mr. Whittaker of the Metropolitan Water Board, Boston, Massachusetts for his successful efforts to gain the data necessary to compile this work.

J. L. V.

Brookline, Mass.

May 25th 1922.

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PLANS and DESIGNS

LARGE SIZE DRAWINGS

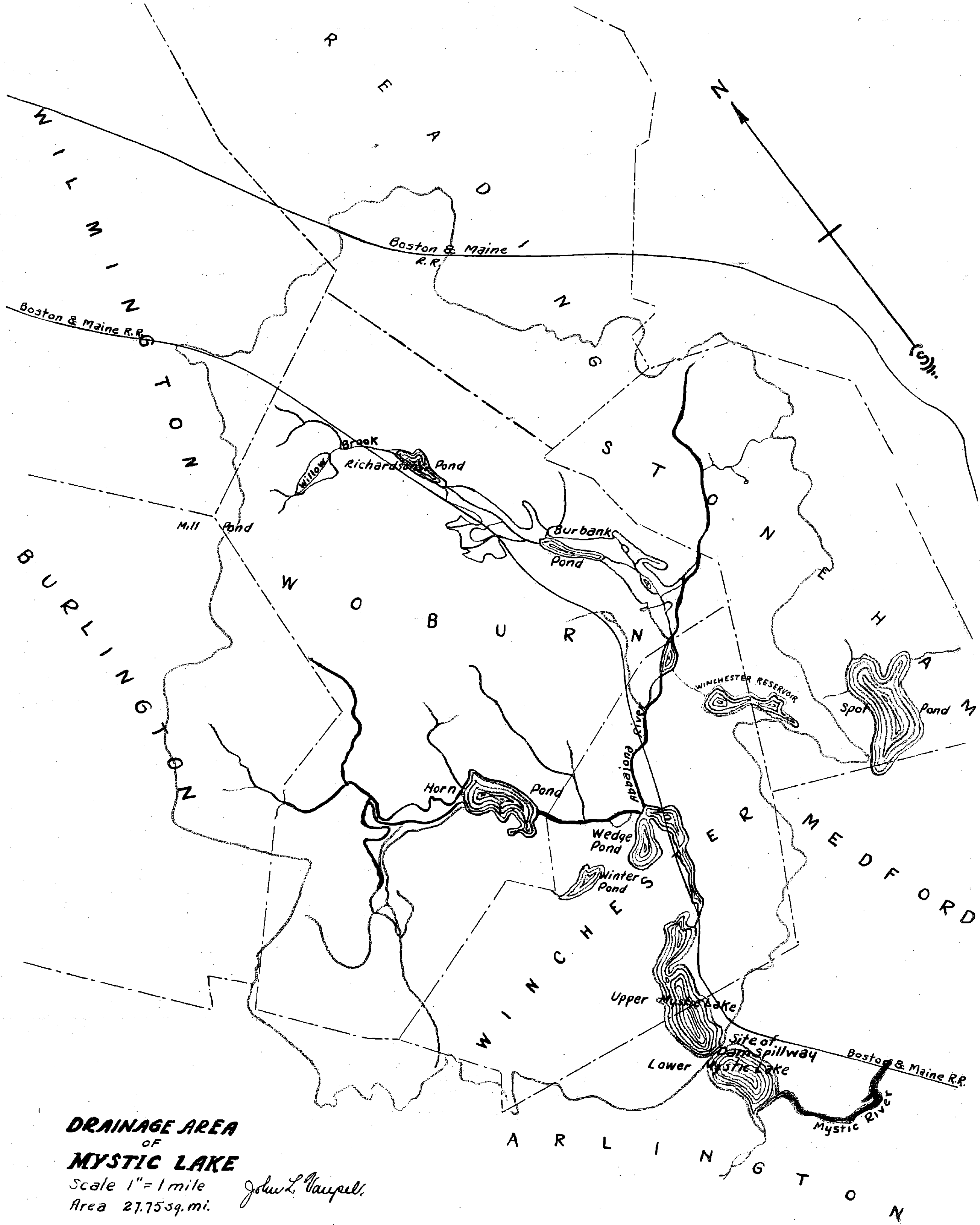
PURPOSE.

Without stating the various purposes fulfilled by dams in general the object of this dam is to back up the waters of the Upper Mystic Lake, thereby preserving the aesthetic appearance which was contributed in 1862 by the construction of the Mystic Reservoir as a source of water supply for the then, town of Charlestown.

LOCATION AND WATERSHED.

The location of the dam, as shown on the following page, is at the junction of the Upper and Lower Mystic Lakes. The Upper Mystic is fed by the waters of the Aberjona river, which river together with its tributaries drains the Mystic Watershed comprising 27.75 square miles, 3% of which is water surface, but which is divided as follows:-* woodland 9.30 square miles; cultivated land 11.67 square miles; buildings and adjacent grounds 6.78 square miles. The principal ponds

* Data obtained from the Metropolitan Water Works. Boston, Mass.



DRAINAGE AREA OF MYSTIC LAKE
Scale 1" = 1 mile
Area 27.75 sq. mi.
John L. Vaupel

are - Horn, Burbank, Wedge and Richardson, small in size; incapable of sufficient storage to make them an accountable factor towards retarding the flood flow.

CHARACTER OF THE SOIL AND TOPOGRAPHY.

The watershed included in the towns of Winchester, Woburn, part of Stoneham, Arlington and Reading is of intermediate character, neither entirely hilly nor entirely flat. The greater part is gravel and sand underlaid with diorite and trap rock. This nature of watershed give rise to a rapid absorption of rainfall, a minimum amount of surface evaporation, while the water is gradually and equally conveyed to the water courses which feed the Lake. A district of this nature is favorable to a rapid runoff. The Upper Mystic is advantageously situated for drainage due to its location at the extreme southerly end of the Mystic Valley. These Lakes empty into the Mystic River at Medford, which river flows thru the locks at Craddock's Bridge and thence encounters the tidal waters of the Atlantic Ocean.

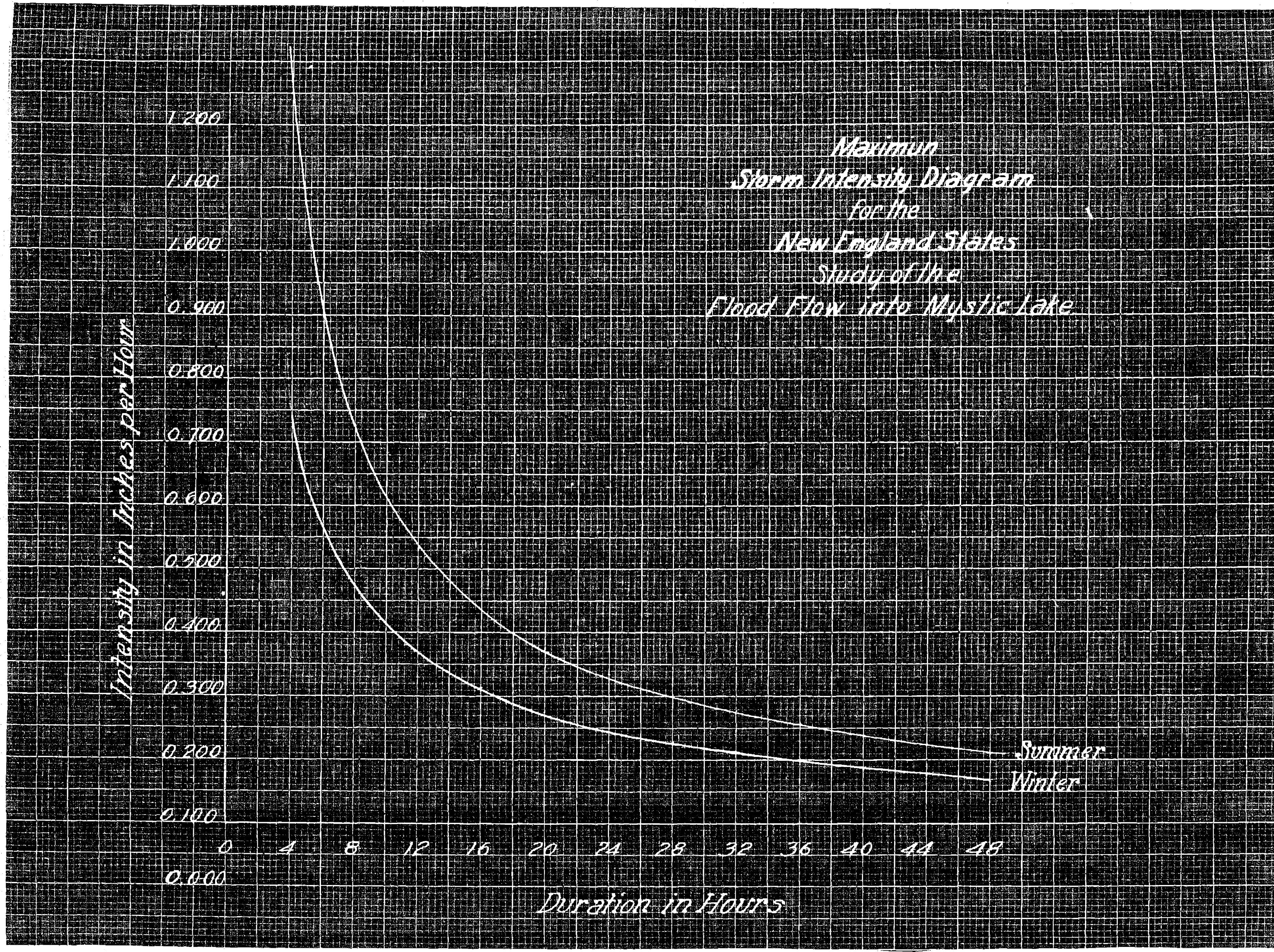
RAINFALL.

The rainfall statistics for Eastern Massachusetts, obtained from the U.S.Weather Bureau for a period of 103 years, gives the annual rainfall in Boston as 43.75 inches. The greater part of the rainfall occurs during the late winter and early spring. Thus from December to May the rainfall is recorded as being 22.11 inches; from June to November it is recorded as 21.66 inches, over the Mystic Watershed. Thus unlike other drainage areas, the yearly precipitation over the Mystic Valley is approximately equally divided between the winter-spring and summer-autumn periods. This information is of but slight value in the determination of the amount of flood-flow to be expected, for it gives no indication as to whether 10 inches fell in one day, two days or a month; it serves simply to show that the Mystic watershed receives a good quantity of rainfall, equally distributed throughout the year and that droughts are not common; it fails to show whether the rainfall was local or spread over the entire area;

then the area over which the rain falls, the duration and depth are important considerations in the determination of flood flows.

During the year of 1913 great floods occurred at Dayton,Ohio, on the Miami river. Mr. A.E.Morgan in designing protection works for the Miami valley made extensive time, area, and depth studies of rainfall and storms over the United States east of the 103d meridian. In his reports of Storm Rainfall of the Eastern United States* valuable information concerning rainfall over Massachusetts is presented. The diagram on the following page shows the maximum storm intensities of the New England states and are self explanatory. These data are for summer and winter storms. The accompanying table upon which the graph is based shows that the depth of rainfall during the winter months is about two inches less than that during the summer. The melting of snow and ice later may be considered as adding 2"

* Miami Conservancy District Report #5 1917


Maximun
Storm Intensity Diagram
for the
New England States
Study of the
Flood Flow into Mystic Lake
Intensity in Inches per Hour
1.200
1.100
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
0 4 8 12 16 20 24 28 32 36 40 44 48
Duration in Hours
Summer
Winter

NEW ENGLAND

Cumulative

Maximum Probable Rainfalls Two day Storms

	Summer		In Winter	
Duration in hours	Depth in inches	Intensity Rate per Hour	Depth in inches	Intensity Rate per Hour
4	5.0	1.250	2.9	0.725
8	5.8	0.725	3.8	0.475
12	6.4	0.534	4.4	0.366
16	7.0	0.438	5.0	0.312
20	7.5	0.375	5.5	0.275
24	8.0	0.334	6.0	0.250
28	8.4	0.300	6.4	0.228
32	8.9	0.278	6.8	0.212
36	9.2	0.256	7.2	0.200
40	9.5	0.237	7.4	0.185
44	9.8	0.223	7.7	0.177
48	10.1	0.210	8.0	0.167

Obtained from Monthly Weather Review presented in Turneaure and Russells Public Water Supplies.

to the winter rainfall thus raising it to the summer average. It is peculiar that New England is in the direct path of most storms originating in the areas of low pressure in the United States. The storms originating in the west and southwest proceed easterly and northeasterly respectively and tend to pass directly through the New England States. Those of the south Atlantic follow along the coast. Thus New England is in a position to receive rainfall from the majority of storms originating in the United States and hence storms of frequency occur in Massachusetts. The New England states are not a storm center; the barometric pressure at Boston is reasonably constant, varying from 28.86" to 31.03" with a mean of 29.99". This means that cyclonic and hurricanial conditions are absent and hence great cloudbursts and heavy intensive rainfalls are not present. The floods of New England are caused, then, by the spring rains, accompanied by the melting of snow.

Thus the rainfall over Massachusetts is equally distributed throughout the year; there are not present storms of excessive intensity when compared with those of the low pressure areas and the spring freshets are the accountable factor in the determination of the flood flow.

COEFFICIENT OF RUNOFF.

The most difficult study in this flood flow is the determination of the proper coefficient of runoff. As previously stated, the nature of the soil, the topography, and the situation of the Upper Mystic Lake all tend to develop a rapid discharge. In dry years the amount of rainfall collected was 40%. The accompanying table based on actual rainfall collected during the years of 1878-1890 inclusive gives a mean yearly coefficient of 52%. Mr.Fteley determined by measurement (1871) the value of 43%. The Boston Water Board determined a coefficient of 45%. Thus between 40% to 50% of the rainfall is runoff. But as the Mystic Valley is subject to floods during the spring the conditions prevalent at that time must be considered. During the winter the ground is covered

	Month											
Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1878	.47	.90	.83	.36	2.10	.34	.10	.14	.16	.10	.19	.54
1879	.44	.54	.85	.60	2.0	.15	.17	.10	.26	.37	.15	.16
1880	.46	.62	.60	.53	.59	.68	.09	.18	.19	.11	.19	.14
1881	.11	.44	.69	1.31	.36	.26	.29	.28	.12	.10	.13	.23
1882	.37	.64	1.25	.46	.30	.79	.09	.15	.27	.25	.32	.21
1883	.19	.52	.80	.57	.23	.25	.11	.56	.12	.06	.20	.12
1884	.24	.68	.80	.80	.45	.21	.13	.12	.74	.85	.11	.26
1885	.33	.60	1.78	.61	.51	.23	.32	.07	.20	.12	.41	1.14
1886	.33	1.10	1.22	1.90	.41	.41	.22	.06	.12	.12	.24	.39
1887	.65	.98	.94	1.4	1.10	.64	.24	.44	.49	.22	.32	.33
1888	.63	1.4	1.3	1.6	.55	.31	.22	.08	.19	.76	1.0	1.2
1889	1.09	1.2	1.3	0.7	.5	.68	.23	.52	.33	.36	.50	1.7
1890	.79	.97	.91	1.3	.67	.87	.41	.17	.12	.46	2.0	.67
Mean	.47	.81	.97	.93	.75	.45	.20	.22	.25	.30	.36	.54

Mean yearly coefficient 52%

Obtained from the Metropolitan Sewage Works Nov.1921 by snow, which, thawing during the early spring, seeps into the sandy soil and accumulates over the diorite as ground water storage. At such times the ground presents a relatively impervious character, promoting a rapid discharge, which results in a short period of concentration due to the small size of the drainage area.

THE FLOOD FLOW.

After considering the drainage area, the rainfall and the runoff, the determination of the flood flow on the area is necessary to adequately design the spillway; so that the spillway will provide for the passage of this flood flow to the extent that neither the rigidity of the dam nor the normal condition of all property will be endangered.

It is impossible to derive a general formula for flood flows in terms of the drainage area, rainfall and runoff and arrive at better than an approximate result. Conditions encountered on one drainage area are seldom, if ever, duplicated. Authorities state that exact duplication is impossible since the factors involved are so numerous as to preclude the possibility. An individual study of the drainage area is necessary to gain satisfactory results as to the flood flow on that area.

On December 3, 1921, directly following a severe ice storm and rainfall from Nov.27,1921 to Dec.2, 1921, an investigation of the surface velocities of the Aberjona river and its tributaries together with several streams emptying into the Upper Mystic showed the velocity to range from 2 to 3 feet per second. The data is presented on the following page. This time was propitious because of the imperviousness of the ground contributed to the runoff. The ground was well stored with seepage and the streams in a condition of fair flood flow.

The source of the Aberjona river in Stoneham is six miles from the point of entrance to the Upper Mystic Lake. The time of concentration then is four hours, which, allowing for simosities of the smaller channels, may be increased to five hours.

DETERMINATION OF SURFACE VELOCITIES

Stream	Surface Vel.	Topography
(1) Smith Brook	2.5 ft/sec.	Fields, wooded area, gentle smooth slopes.
(2) Tributary Mill Brook	2.5 "	Hilly, wooded farm land, steep slopes
(3) Mill Brook at Lexington and Russell Streets	1.0 "	Gentle slope, wide stream farm land, wooded area.
(4) Mill Brook at Lexington Street	2.0 "	Farm land.
(5) Greenlief Brook at Bedford Street	2.5 "	Farm land, wooded area.
(6) Aberjona River at Salem Street	3.0 "	Marsh land, farm land wooded areas.
(7) Sweetwater Brook at Montvale Street	2.2 "	Farm land, cemetery, hilly gentle slopes.
(8) Aberjona River Washington Street	2.0 "	Residential area.
(9) Willow Brook	2.3 "	Residentail section built-up areas.

The ponds wherein storage might ordinarily exist and thus equalize the flood flow and hence retard the time of concentration are assumed as full. This condition is consistent with a maximum rate of discharge and altogether desirable as an assumption. Then from the rainfall intensity diagram on page 6 a storm lasting five hours contributes a rainfall of 0.85 inches per hour. The relation between these two factors is such that a storm lasting to the time of concentration yields a certain intensity which contributes the maximum flood at the point where the spillway is to be constructed. Considering the nature of the soil, reviewed in the previous paragraph on the "Character of the Soil and Topography" rapid absorption is expected and considerable ground storage is gained. The topography, being neither entirely hilly nor entirely flat, but with gentle slopes and cultivated areas and woodlands forming the greater part of the surface, likens itself to gradual and equal conveyance of waters to the streams. A coefficient of 40% - 50% may be assumed applicable to this area as was determined in the discussion on "Coefficient of Runoff". Evaporation is neglected as the surface waters are small and the time of concentration so small as to make this assumption rational. Applying this data, the following results are gained, consistent with the previous discussion. The source of the river is 3200 ft. from the spillway. Time of concentration then

is $\frac{32000}{2.5 \times 3600}$ = 3.5 hours say 5 hours.

The diagram determines the average intensity as 0.85 inches per hour. The coefficient of runoff varies between 40% - 50% consistent with the investigations made by Mr. Fteley, Dr. Cunningham and the data obtained by calculating the coefficient over a period of 13 years. Then the maximum flood expected is

$$\frac{27.75 \times 5280 \times 5280 \times 0.85 \times .40}{60 \times 60 \times 12} = 6075 \text{ sec.ft.}$$

Coefficient of 45% = 6840 sec.ft.

" " 50% = 7600 sec.ft.

Mr. W.E.Fuller made a study of flood flows which may be expected in various areas throughout the United States, the presentation of which is made in the Trans. of the A.S.C.E. Vol. 77, 1914. Without going into the details of his report, which is too lengthy to adequately and justly review here, a summary of the resulting flood flow is set forth below. Mr. Murphy also derived a formula during his connection with the U.S. Geological Survey; the application of which gives a value of 4000 sec.ft. Mr. Fanning's formula is somewhat less than any of the values obtained previously. Whereas the values gained range from 3000 sec.ft. to 7000 sec.ft. the most probable value is 5000 sec.ft. for the Mystic drainage area.

Fuller's Method

$$Q = C A^{.8} (1 + 0.8 \log T) (1 + 2A^{-.3})$$

Q = maximum 24 hour rate of flow in sec.feet

C = a coefficient dependent upon the drainage area, topography, soil.

A = area of drainage area in square miles

T = period of years in which the Q is expected

Years	C	Q		C	Q	
15	50	2520	cu.ft. sec.	75	3780	cu.ft./sec.
25	50	2745	"	75	3618	"
50	50	3060	"	75	4590	"
100	50	3375	"	75	6063	"

Fanning's Method

$$Q = 200\ A^{5/6}$$

Q = maximum flood in sec.ft.

A = drainage area in sq.miles

$$\therefore Q = 200 \times (27.75)^{5/6} = 3166 \text{ sec.ft.}$$

Murphy's Method

$$Q = \left[\frac{46790}{A + 320} + 15\right] A$$

Q = Maximum flood in sec.ft.

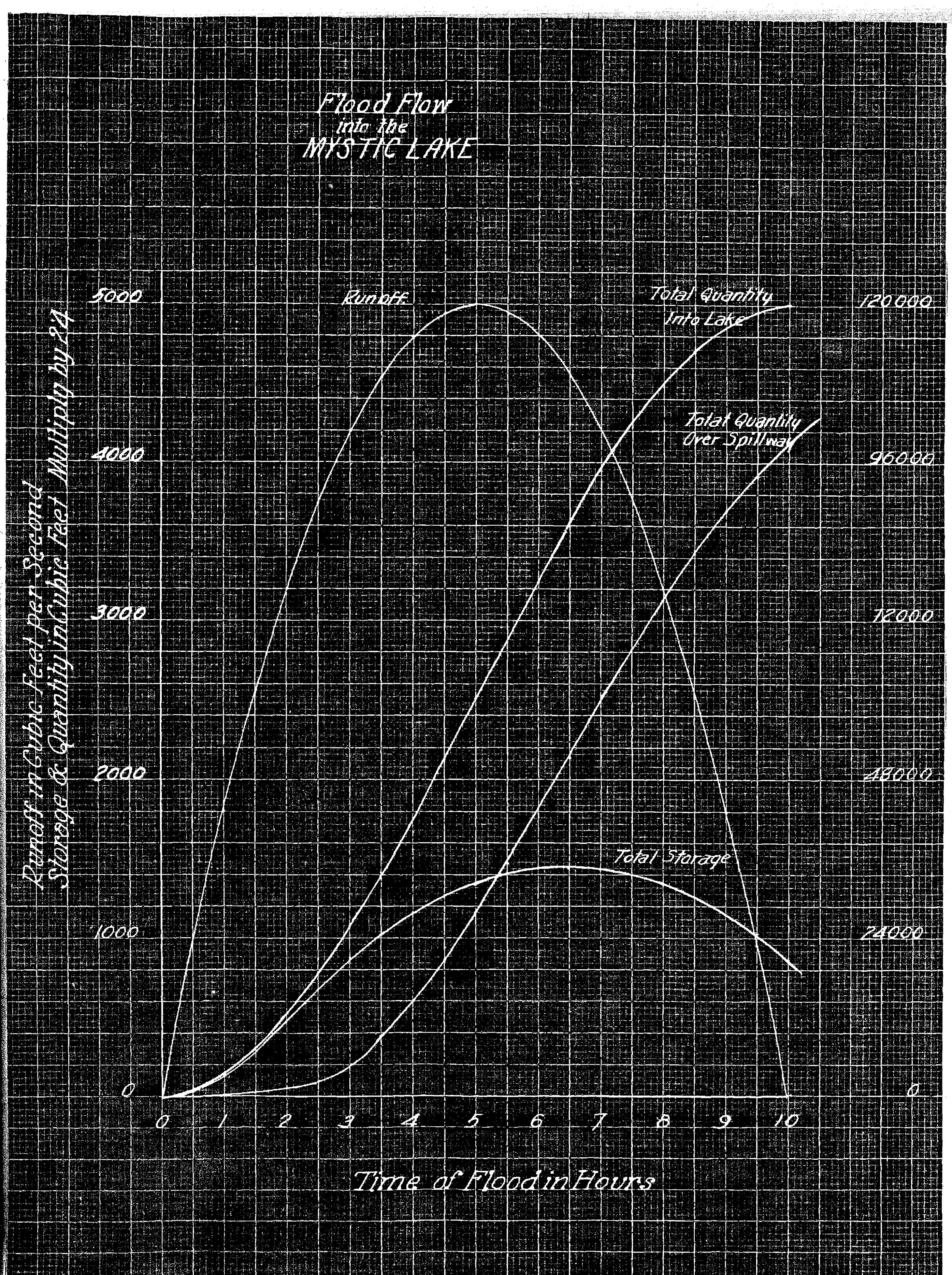
A = drainage area in sq.miles

$$Q = \left[\frac{46790}{27.75 + 320} + 15\right] A = 4125 \text{ sec.ft.}$$

HOW THE SPILLWAY CONTROLS THE FLOOD.

The accompanying graph on the following page is based on the assumption that the run off follows the curve of a parabola with the flood peak of 5000sec.ft. at the vertex. By integration the total quantity of water in cubic feet emptying into the Upper Mystic is shown, the ordinate of the curve giving the quantity in cubic feet at any time. Due to the large storage 580,000,000 gallons at high water mark, the spillway need not be designed to care for the total inflow; but for sufficient capacity to prohibit excessive rise in the lake, which would encroach on adjacent property. A semi-circular spillway of 150' length will care for the flood as shown on the graph; the quantity in cubic feet passed by the spillway at any time is given by the ordinate to the curve. The storage in the lake is the difference between the quantity which enters the lake and that which passes over the spillway. The table on the following page, from which the

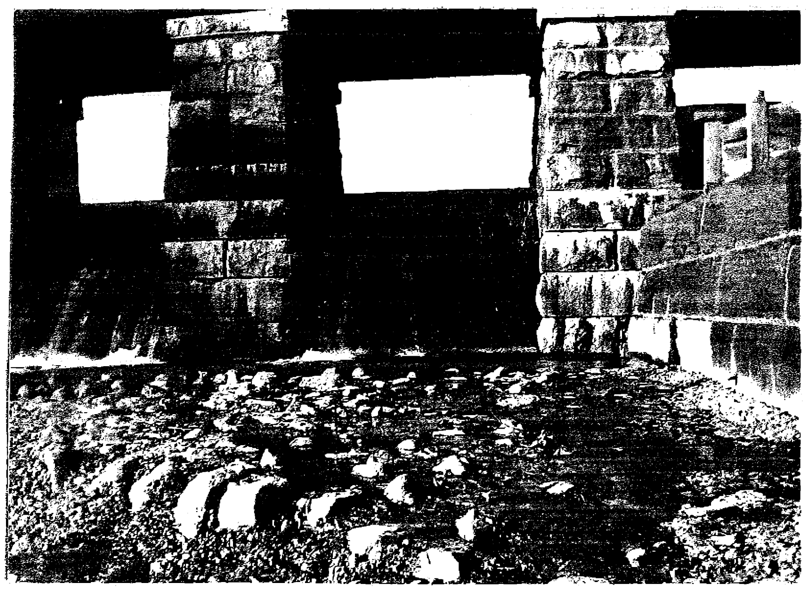
Time period of Flood in Hours	Totat Quantity into Lake in 1000 cu.ft. at close of Period	Increase in Quantity over Quantity of previous Period	Quantity in 1000 cu.ft. over Spillway at Close of each Period	Total Storage in Lake in 1000 cu.ft at close of each Period	Total Rise of Lake in feet at close of each Period	Total Quantity over Spillway in 1000 cu.ft. at close of Period
0.5	870	870	1.30	868.7	0.106	1.3
1.0	3360	2490	51.80	3306.9	0.405	53.1
1.5	7290	3930	219.5	6947.4	0.850	342.6
2.0	12480	5190	821.6	11315.8	1.39	1164.2
2.5	18750	6270	1380.	16205.8	1.98	2544.2
3.0	25920	7170	2782	20593.8	2.54	5236.2
3.5	33810	7890	3969	24514.	3.00	9296
4.0	42240	8430	5097	27847	3.42	14393
4.5	51030	8790	6136	30501	3.74	20529
5.0	60000	8970	6956	32515	3.98	27485
5.5	687910	8970	7606	33879	4.15	35091
6.0	77760	8790	8036	34633	4.25	43127
6.5	86190	8430	8297	34766	4.25	51424
7.0	94080	7890	8229	34427	4.23	59653
7.5	101250	7170	7987	33600	4.13	67640
8.0	107520	6270	7737	32142	3.93	75377
8.5	112710	5190	7115	30217	3.70	82493
9.0	116640	3930	6468	27679	3.39	88961
9.5	119130	2490	5548	24621	3.02	94509
10.0	120000	870	4726	20765	2.84	99235


Flood Flow
into the
MYSTIC LAKE
Runoff
Total Quantity
Into Lake
Total Quantity
Over Spillway
Total Storage
5000
4000
3000
2000
1000
0
120000
96000
72000
48000
24000
0
0
1
2
3
4
5
6
7
8
9
10
Time of Flood in Hours
Runoff in Cubic Feet per Second
Storage & Quantity in Cubic Feet Multiply by 24

graph was constructed, shows that the maximum rise in the lake is 4.'25 above elevation 15 the present height of the existing overflow above mean sea level. The area of the lake at elevation 15 is 8,167,000 sq.ft. or 187.26 acres.* This rise in the lake is permissible as it does not encroach upon property or endanger the existing dam.

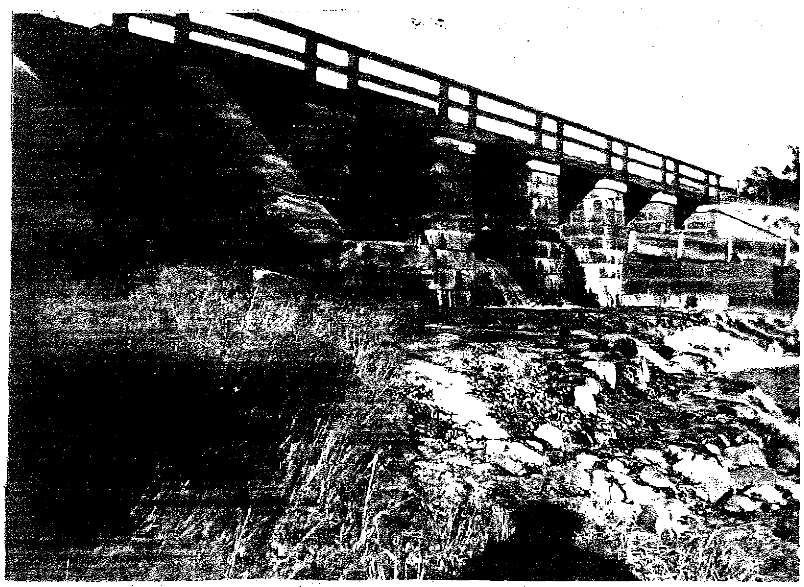
THE EXISTING STRUCTURE.

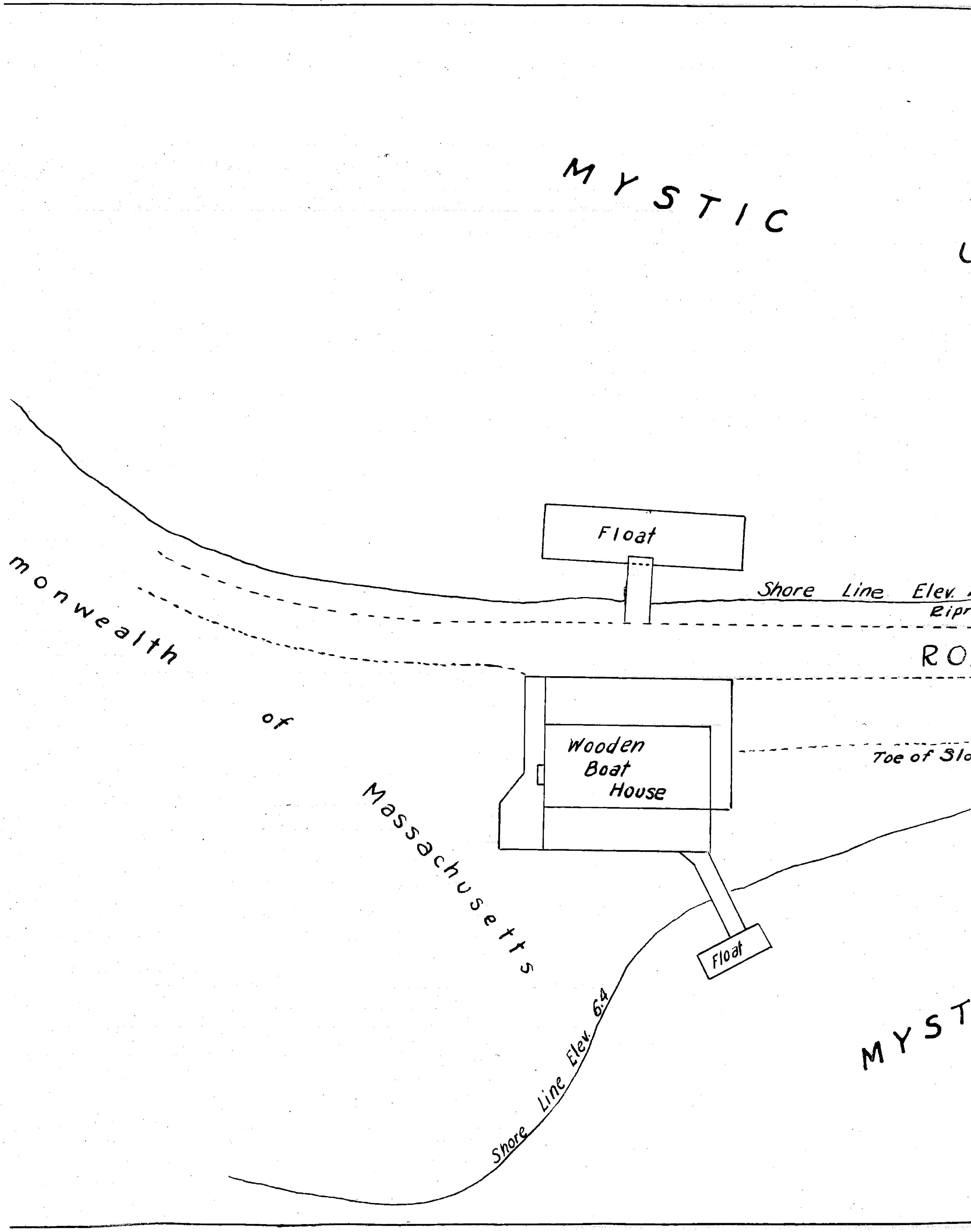
The Overfall is composed of two abutments and five piers. Let into grooves in the abutments and



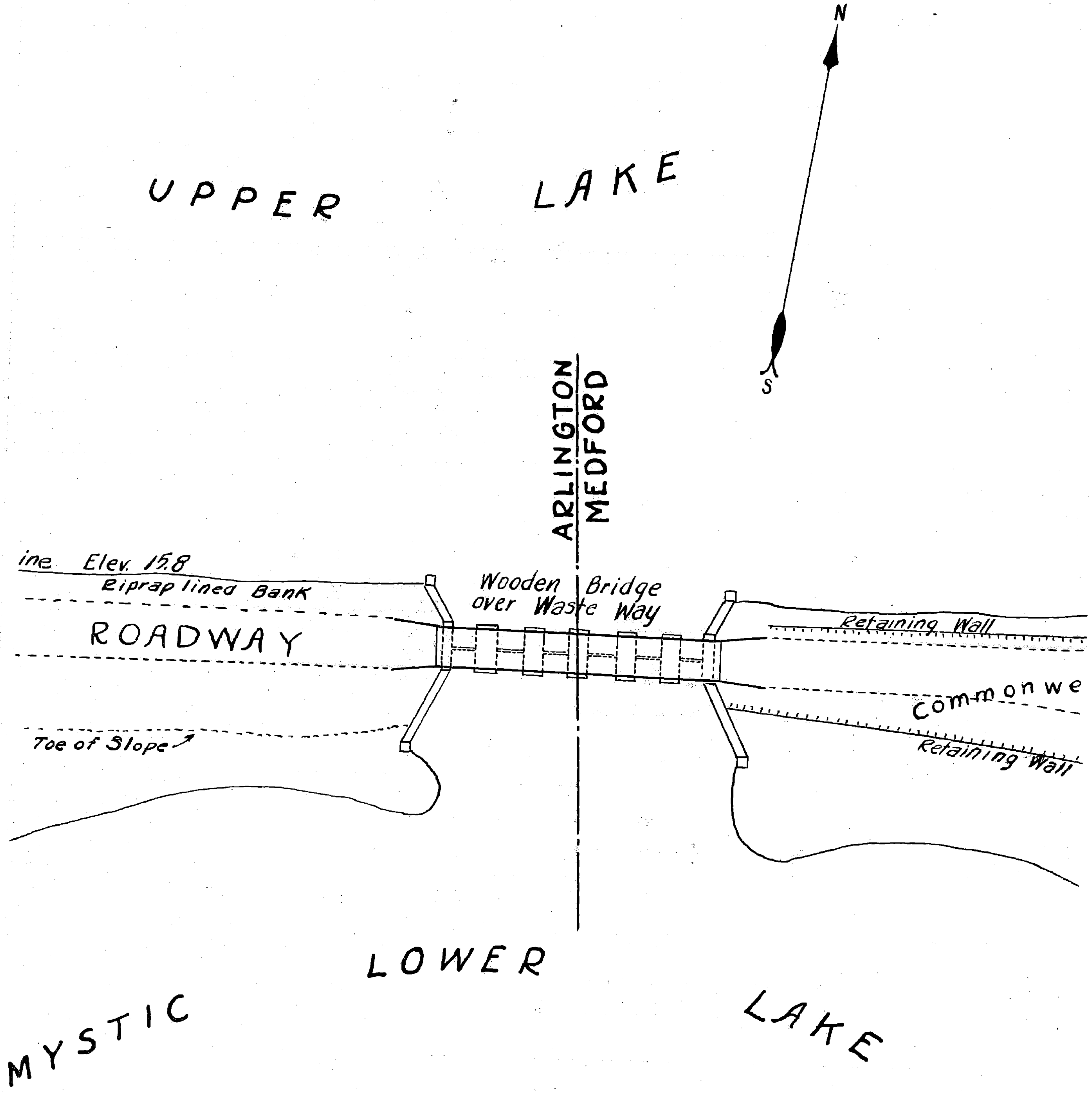
* Reports and records of Mystic Reservoir Metropolitan Water Works Boston Mass.

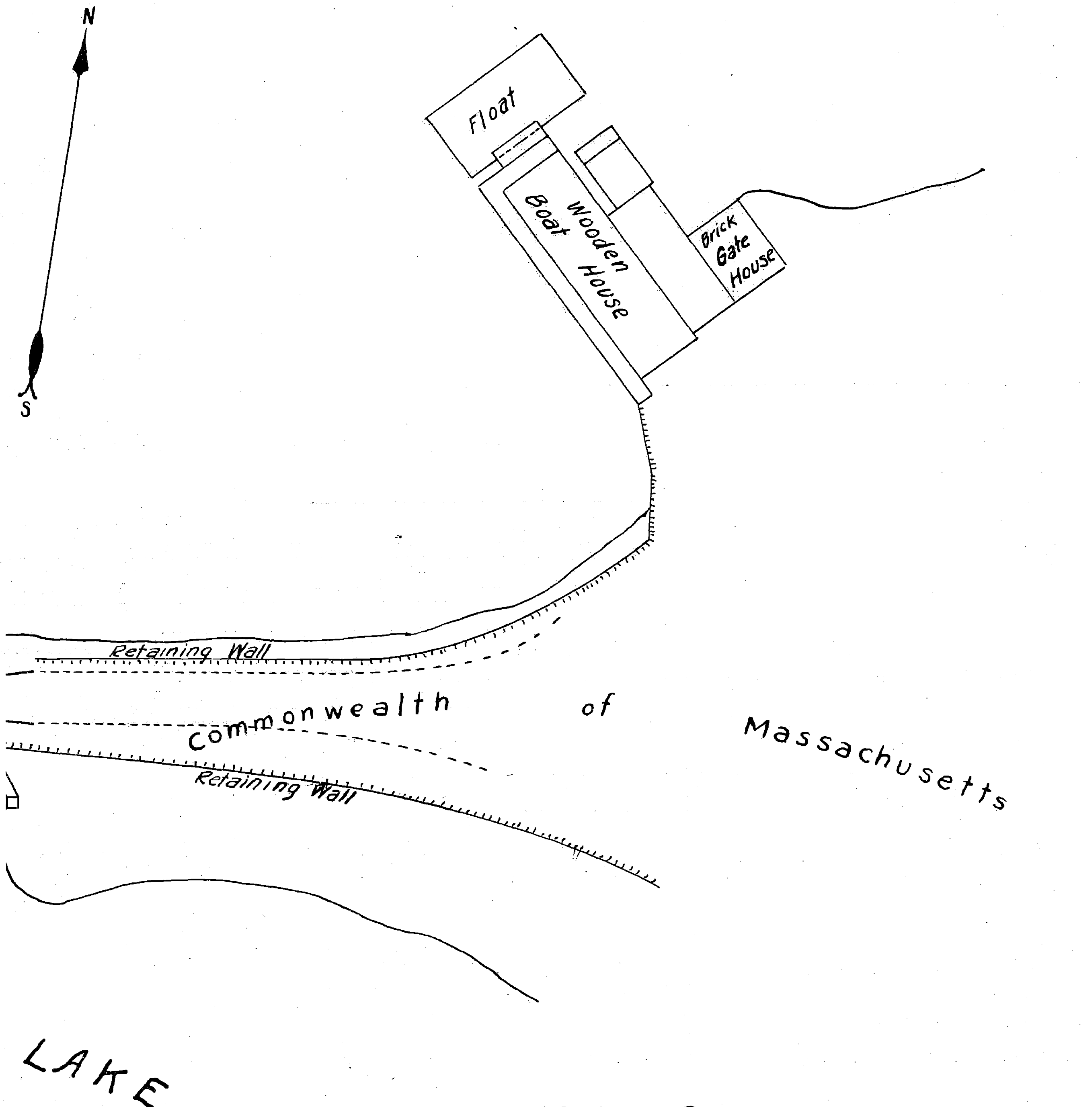
piers are two rows of wooden bulkheads or stop-planks, rising to a height of 16.14 above mean sea level. To care for floods at present it is necessary to visit the overflow and take out the planks in the bays until sufficient overflow is gained to care for the flood. During the past two years the elevation of the water surface in the lake has varied so that average elevation of the overflow crest has been 15.65. The abutments are of granite block stone and are composed of a face and two wings, all of which rest upon the foundation. The upper or shorter wall has a splay backwards of 30° from the prolongation of the line of the face, and the lower or longer wing, has a splay of 30° also, measured on the bottom of the foundation.





MYSTIC
monwealth
of
Massachusetts
Float
Shore Line Elev.
Ripr
RO
Wooden
Boat
House
Toe of Slo
Float
Shore Line Elev. 6.4
MYST


UPPER LAKE
N
S
ARLINGTON
MEDFORD
ine Elev. 15.8
Riprap lined Bank
Wooden Bridge over Waste Way
ROADWAY
Retaining Wall
Commonwe
Toe of Slope
Retaining Wall
LOWER
MYSTIC
LAKE



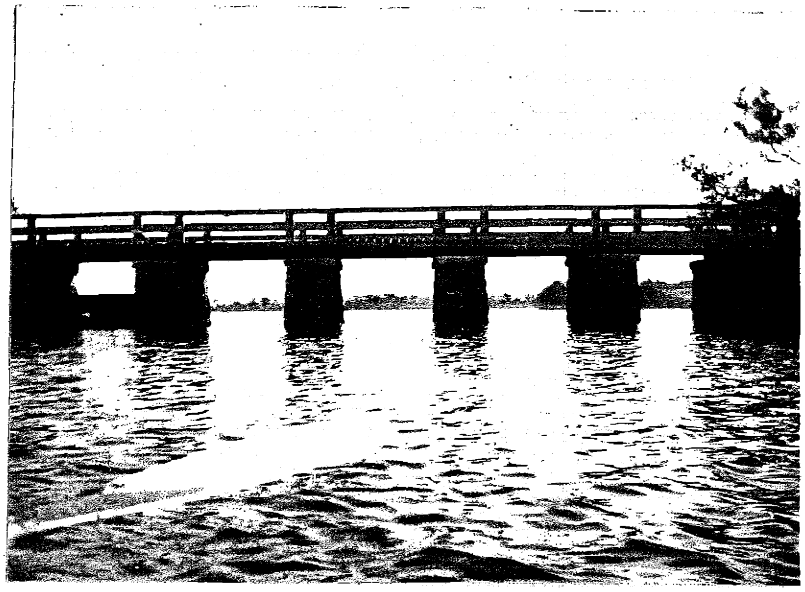
PRESENT PLAN OF MYSTIC DAM AND VICINITY

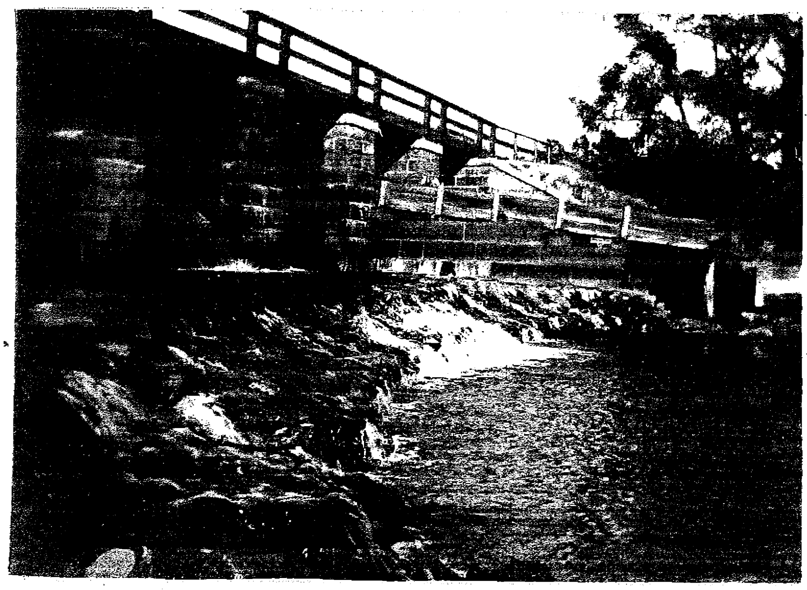
MEDFORD AND ARLINGTON

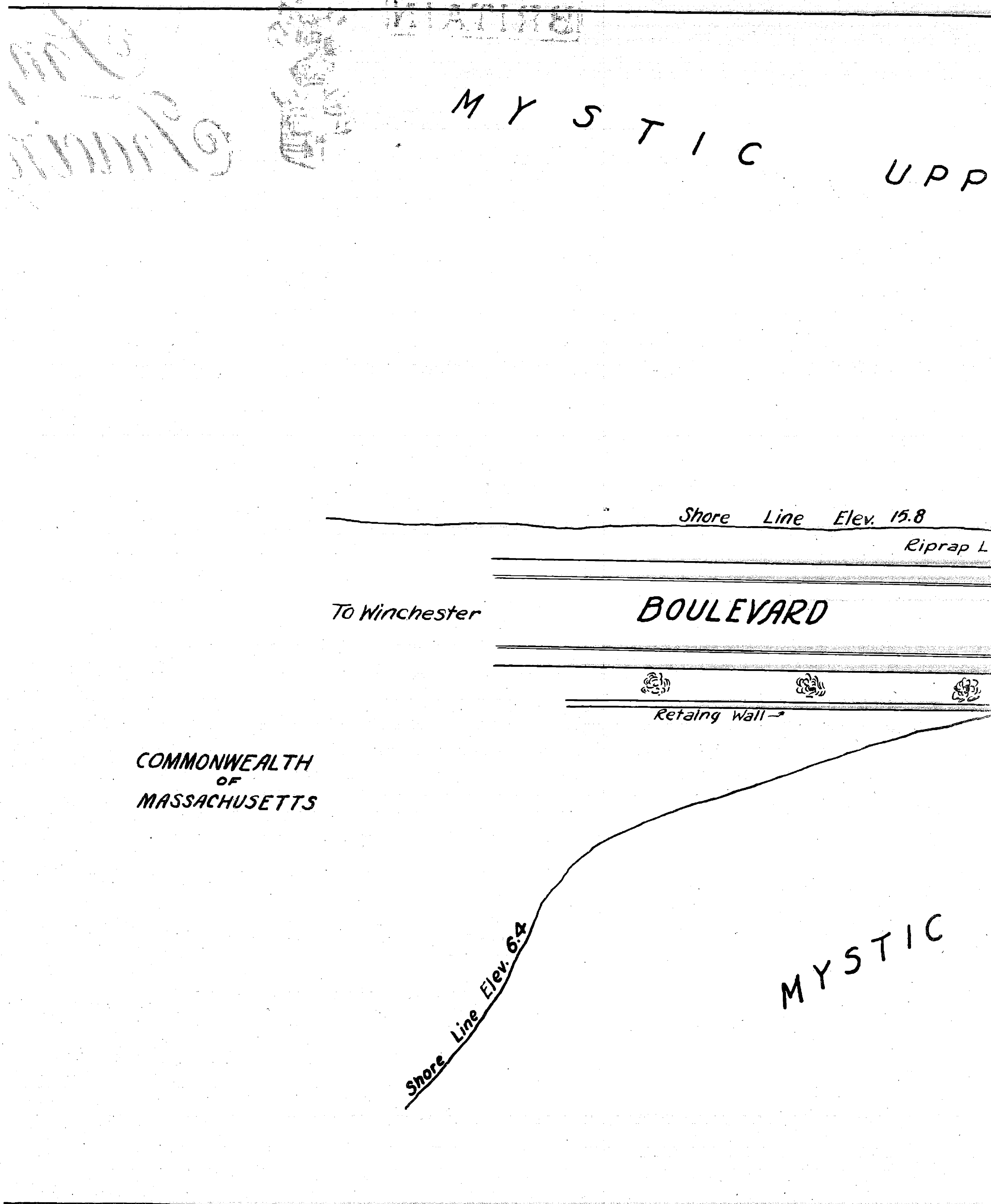
SCALE 1" = 40'   March 1922

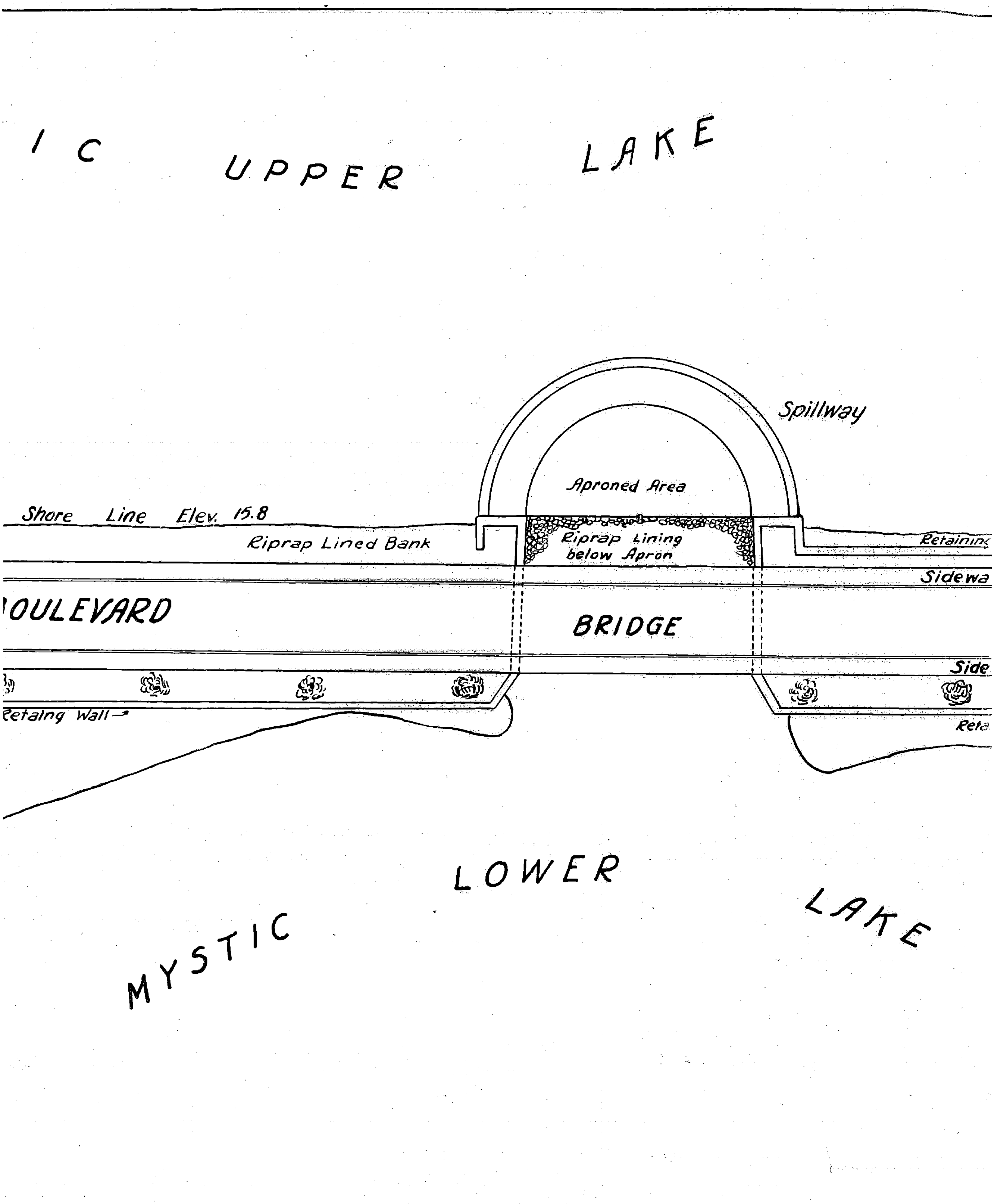
The piers are five in number, spaced 9' inside edge of one to inside edge of the other. They measure 5' x 12' at the base. The four faces have a batter of ½" to the foot. This makes the pieces immediately under the coping 11'-0¼" x 4-0¼". The coping is 9" thick and project 3" on the underside beyond each face. The top of the coping is level with the top of the abutments.

The bulkheads are of white oak 6" thick and 9" wide. They extend from the foundation to the

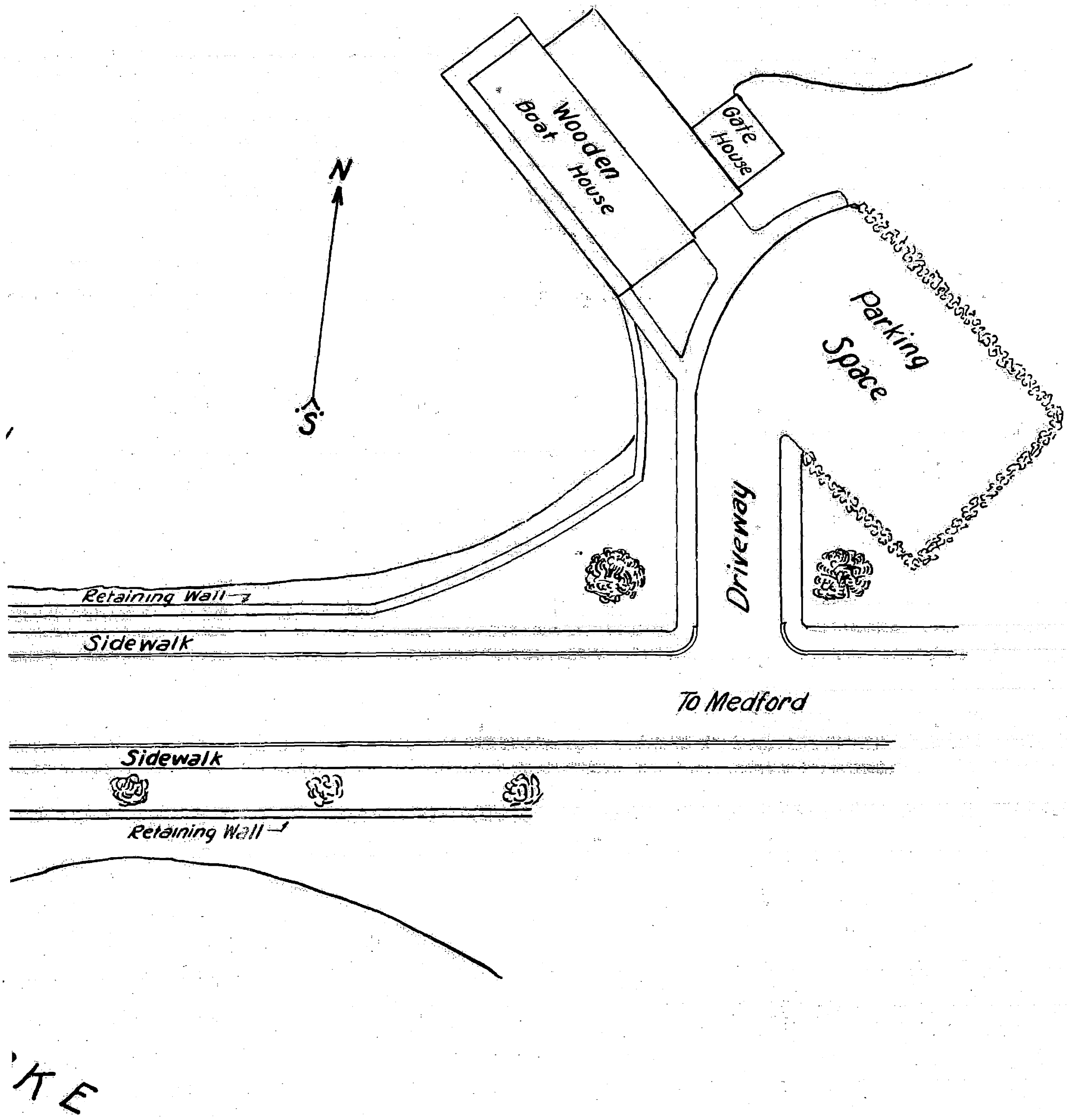





MYSTIC
UPP
Shore Line Elev. 15.8
Riprap L
To Winchester
BOULEVARD
Retaing Wall
COMMONWEALTH OF MASSACHUSETTS
Shore Line Elev. 6.4
MYSTIC



I C
UPPER
LAKE
Spillway
Aproned Area
Shore Line Elev. 15.8
Riprap Lined Bank
Riprap Lining below Apron
Retaining
Sidewa
OULEVARD
BRIDGE
Side
Retaing Wall
Reta
LOWER
LAKE
MYSTIC



REVISION OF

MYSTIC DAM AND VICINITY

MEDFORD AND ARLINGTON

SCALE 1" = 40' John L. Vaupel

level of the Upper Lake. The distance between piers and piers and abutments are all 9'.

GENERAL CONDITIONS OF NEW STRUCTURE.

Consistent with the existing structure just described the following general revisions have been made. Following the accompanying plan, the circular spillway is to join the face of the retaining wall of the Upper Lake side. It is to be constructed of concrete and dependent upon its weight to resist the external forces when the maximum flood is passing over the spillway. The foundation is treacherous, as sand and gravel are materials which do not lend themselves to support a masonry structure of considerable weight; however as this spillway is not to be of excessive height and weight, due attention being given to upward pressure and percolation of the water together with the pile foundation, the masonry spillway provides a durable structure of pleasing appearance. The type of spillway is known as the Ogee; with a concrete apron extending 36' beyond the toe of any radial section. At the extremity of the apron, riprap of two feet thickness extends for a distance of approximately 25-40 feet.

CONSIDERATIONS IN DESIGN.

This spillway has been designed as a gravity structure although circular in plan. A circular or arch dam of considerable height does not resist the hydraulic forces by virtue of its weight alone; the arch action produces a thrust at the abutments which also resists overturning. This thrust action in the case of low spillways is so ineffective as to be negligible. M.S.H.Woodward in discussing the Lake Cheeseman dam, Colorado, which dam is 225 feet high, curved in plan, but designed as a gravity dam, found by calculations submitted in Trans.of A.S.C.E. Vol.53,1904 that at a section 135 feet from the crest the arch action was 2/10 per cent.

The Ogee type was decided upon because the impact of the overflowing water is lessened by the gradual turning of the water from a direction nearly at right angles to the ground to one parallel to the ground and in the direction of flow. This gives an increased velocity at the base; but the length of the apron and the resistance of the rip rap lining coupled with the interference offered by the overflow meeting within the aproned area offsets the eroding influence of the swiftly moving water.

PERCOLATION

The length of the path of percolation was determined as eight-five feet, thirty-six of which is cared for by cut off walls at the heel of the spillway and extremity of the apron, the remaining forty-nine being cared for by the apron.

UPWARD PRESSURE

No tension was permitted in the concrete. At the base, upward water pressure was considered as acting at the upstream face with an intensity of two-thirds the hydrostatic head and diminishing uniformly to zero at the assumed downstream end.

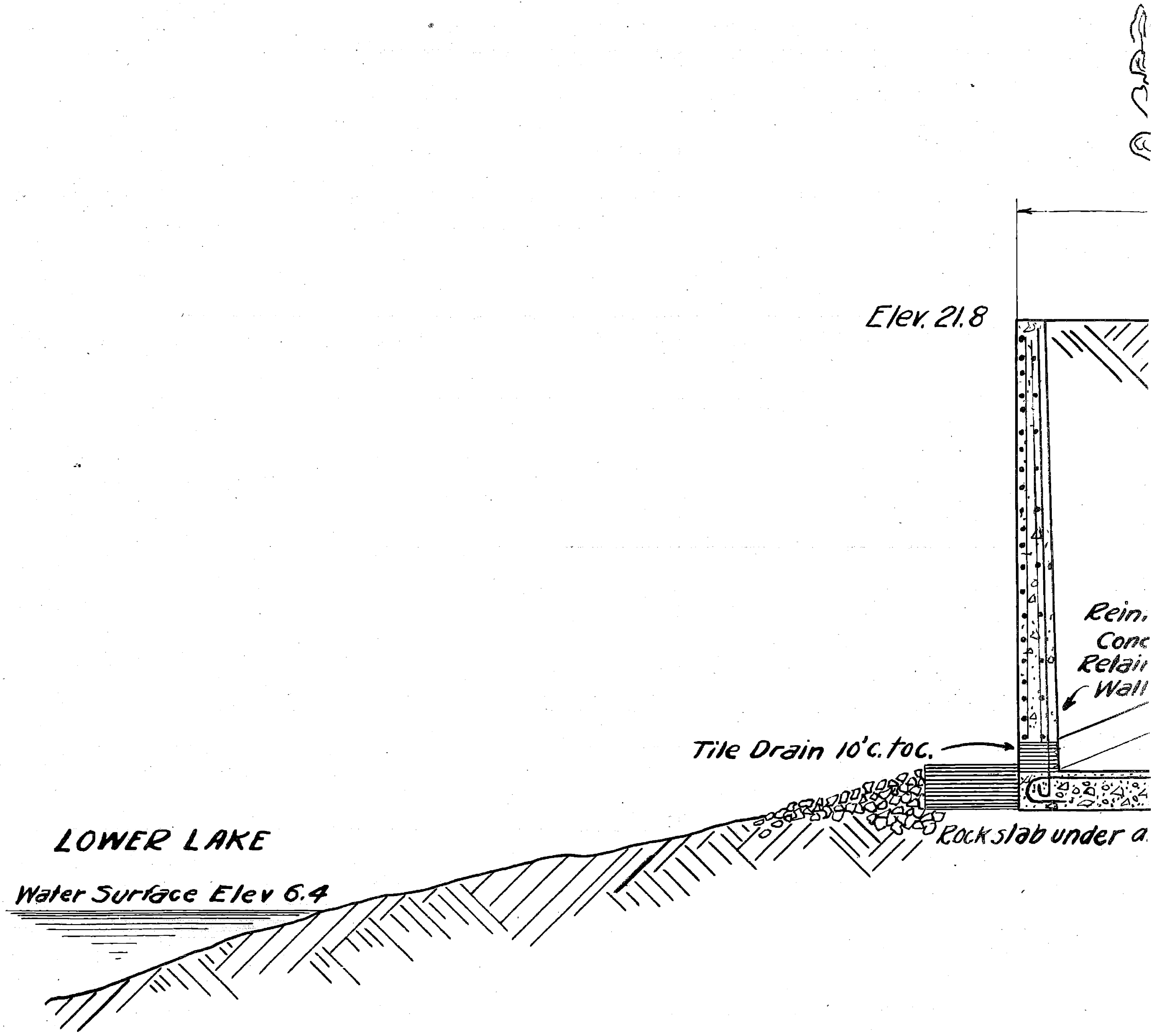
APRON.

The apron was figured as a beam subjected to the upward pressure of the water from the upstream face of the spillway under a hydrostatic head equal to the height of the spillway and varying uniformly to zero at the end of the apron. The computations of the stability of the spillway are presented in the appendix in the back of the book.

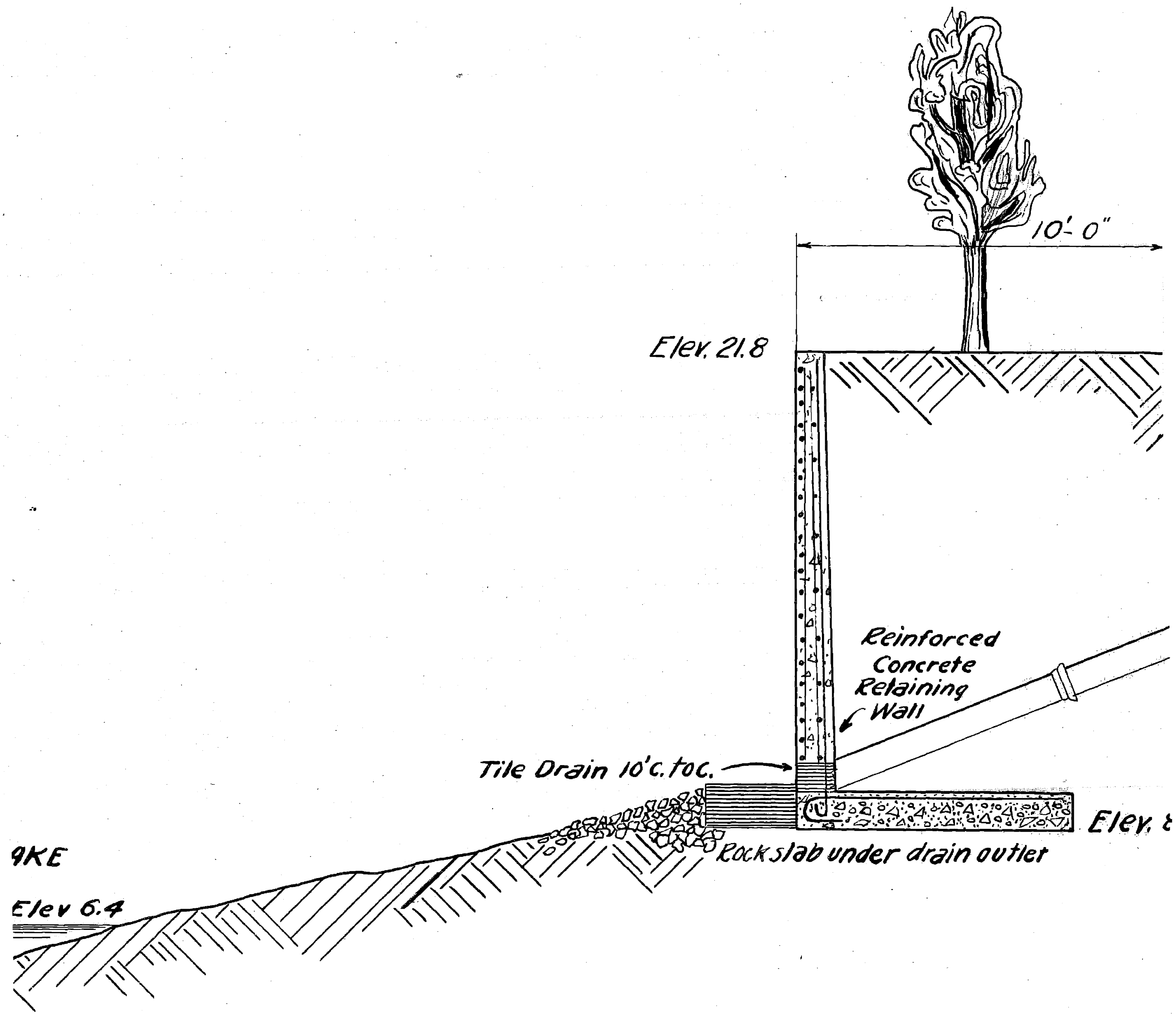
CROSS SECTION OF DAM.

Referring to the Revised Plan of the Mystic Dam and Vicinity a section through the dam at a point twenty feet from the bridge abutment is shown on the accompanying section AB. The boulevard is twenty five feet in width, balanced by sidewalks five feet in width making the overall width thirty five feet. The Massachusetts Metropolitan Park Commission have adopted a standard parkway of thirty-six foot roadway and nine foot pathway; but to maintain this standard it would be necessary to increase the width of the dam throughout its length by twenty feet thereby giving rise to the necessity of filling in some 300,00 cu.feet of earth. From an economical view point this practice seems inadvisable.

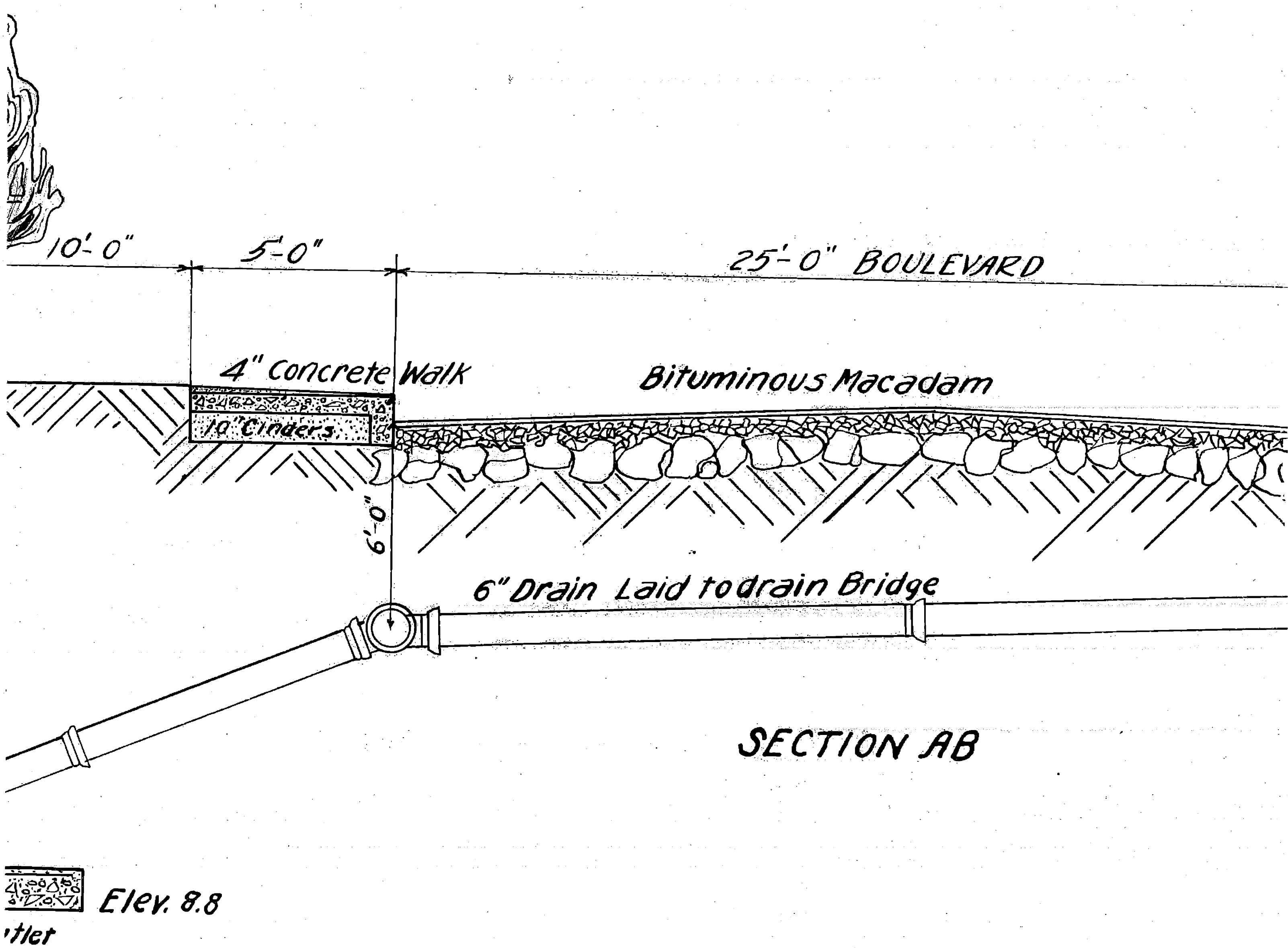
The boulevard is of bituminous macadam, on a Telford foundation. The longitudinal clay pipes connect the catch basins of the bridge with the intercepting drains which carry off the water as shown. These drains, laid with open joints provide for the drainage of seepage water from the boulevard. The tile drains of the retaining wall are not connected every ten feet along the boulevard with intercepting drains, but only on each side of the bridge; these tile drains are for the purpose of preventing water to accumulate against the retaining wall and exerting a pressure against the inside face which pressure would make the wall unstable and cause it to fail.



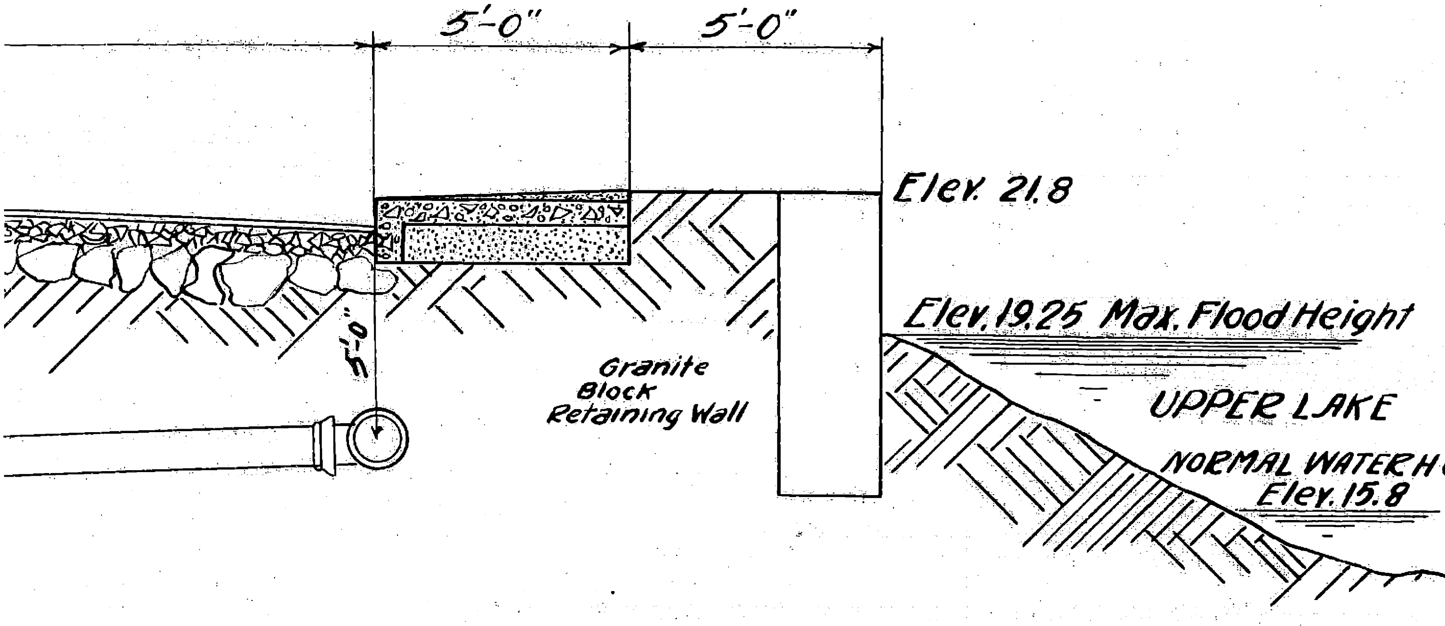
Elev. 21.8
Rein.
Conc
Retai
Wall
Tile Drain 10'c. to c.
Rock slab under a
LOWER LAKE
Water Surface Elev 6.4



10'- 0"
Elev. 21.8
Reinforced Concrete Retaining Wall
Tile Drain 10'c. to c.
Elev.
Rock slab under drain outlet
AKE
Elev 6.4



10'-0"
5'-0"
25'-0" BOULEVARD
4" Concrete Walk
Bituminous Macadam
10" Cinders
6'-0"
6" Drain Laid to drain Bridge
SECTION AB
Elev. 8.8
tlet



SECTION
OF
MYSTIC DAM

Scale $\frac{1}{4}$"=1'

John L. Naupel.

The computations of the stability of the retaining wall are shown in the appendix.

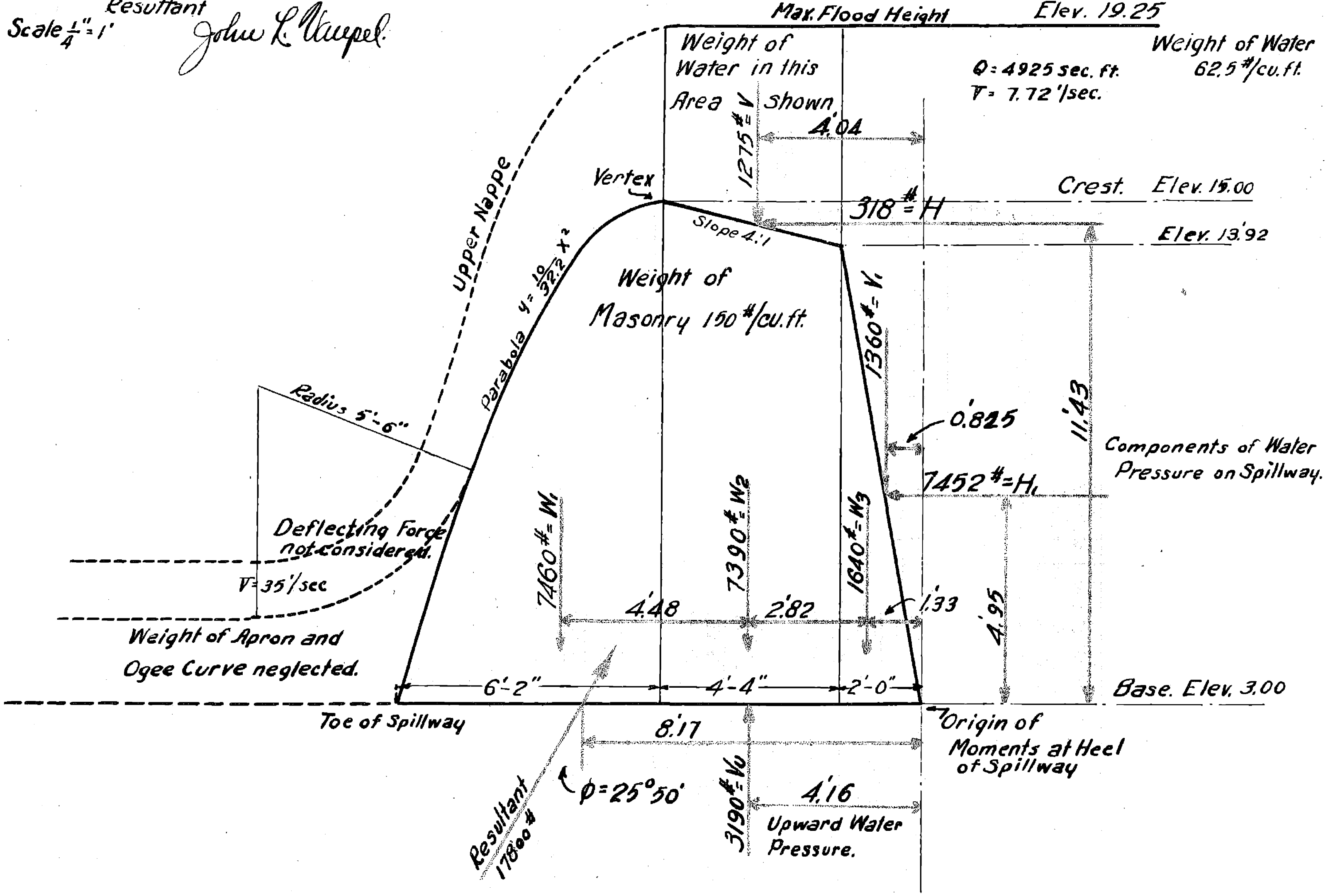
APPENDIX

Elevation of Spillway

Showing forces acting on Dam which determine the position of Resultant

Scale 1/4" = 1'

John L. Vaupel



Computations of Stability of Spillway

Force	Expression	Weight	Lever Arm	Moment
W_1	$\frac{2}{3} \times 6.22 \times 12 \times 150$	− 7460	8.63	− 64400
W_2	$\left(\frac{12+10.92}{2}\right) 4.3 \times 150$	− 7390	4.15	− 30750
W_3	$2 \times \frac{10.92}{2} \times 150$	− 1640	1.33	− 2180
V_1	$2 \times 62.5 \times 10.89$	− 1360	0.825	− 1120
V	$62.5 \times 4.3 \times 4.75$	− 1275	4.04	− 5150
	Total	− 19125		
V_U	$\frac{2}{3} \times 62.5 \times 6 \times 12.52$	+ 3120	4.16	+ 13100
	Resultant V =	−16005		
H_1	$62.5 \times 10.92 \times 10.89$	7425	4.95	− 36900
H	$62.5 \times 1.08 \times 4.75$	318	11.43	− 3630
				−144130
				+ 13100
		Resulting Moment		−131030

$x_0 = -131030 \div -16005 = 8.17$ ft from heel

$8.32 > 8.17 > 4.17$ Resultant acts within the middle third

II Coefficient of Friction

$$C = \frac{H}{V} = \frac{7743}{16000} = .484 < .50$$

$$\phi = 25^\circ 50'$$

III Resultant of Forces.

$$R = \sqrt{H^2+V^2} = \sqrt{(7743)^2+(16000)^2}$$

$$= 17800\#$$

IV Stresses in Concrete

At heel.

$$f = \frac{P}{A} - \frac{My}{I} = \frac{16000}{12.5} - \frac{16000 \times 1.92 \times 6.25 \times 12}{(12.5)^3}$$

$$= 100 \#/sq.ft$$

At toe

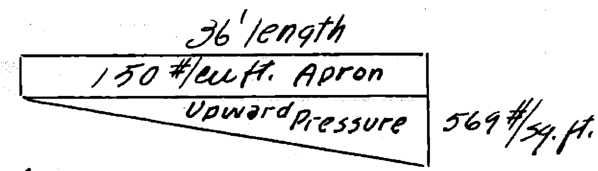
$$f = \frac{P}{A} + \frac{My}{I} = 2360 \#/sq ft < 4000 \#/sq ft.$$

V Determination of Path of Percolation

$$L = C \times H$$

$$= 9 \times (15.8 - 6.4) = 84.6 ft \text{ say } 85 ft.$$

VI Thickness of Apron



Shear.

Assume apron 2' thick

$$\frac{569 \times 36}{2} = 10250\#$$

$$150 \times 36 \times 2 = 10800\#$$

$$\text{Shear} = \frac{550}{24 \times 12} = 1.9 \#/sq in < 40 \#/sq in$$

Moment.

$$150 \times 36 \times 2 \times \frac{36}{2} = 194000$$

$$569 \times \frac{36}{2} \times \frac{36}{3} = \frac{123000}{71000}\ ' \#$$

$$\frac{71000 \times 12}{12 \times (20.5)^2} = R = 169 \text{ too great}$$

$$\frac{71000 \times 12}{12 \times (24)^2} = R = 123$$

f_c in concrete = 650 #/sq in

Area steel = .0115 X 12 X 24 = 3.3 sq in

Use 2 1/8 "φ spaced 12" c. to c.

VII Foundation

Sand and gravel Allowed 2 tons/sq ft.

Resultant vertical force = 16000 #

$$\frac{16000}{2 \times 2000} = 4 \text{ sq ft needed}$$

12.5 sq ft. available

Use 4 piles spaced 3' c. to c. on radial section

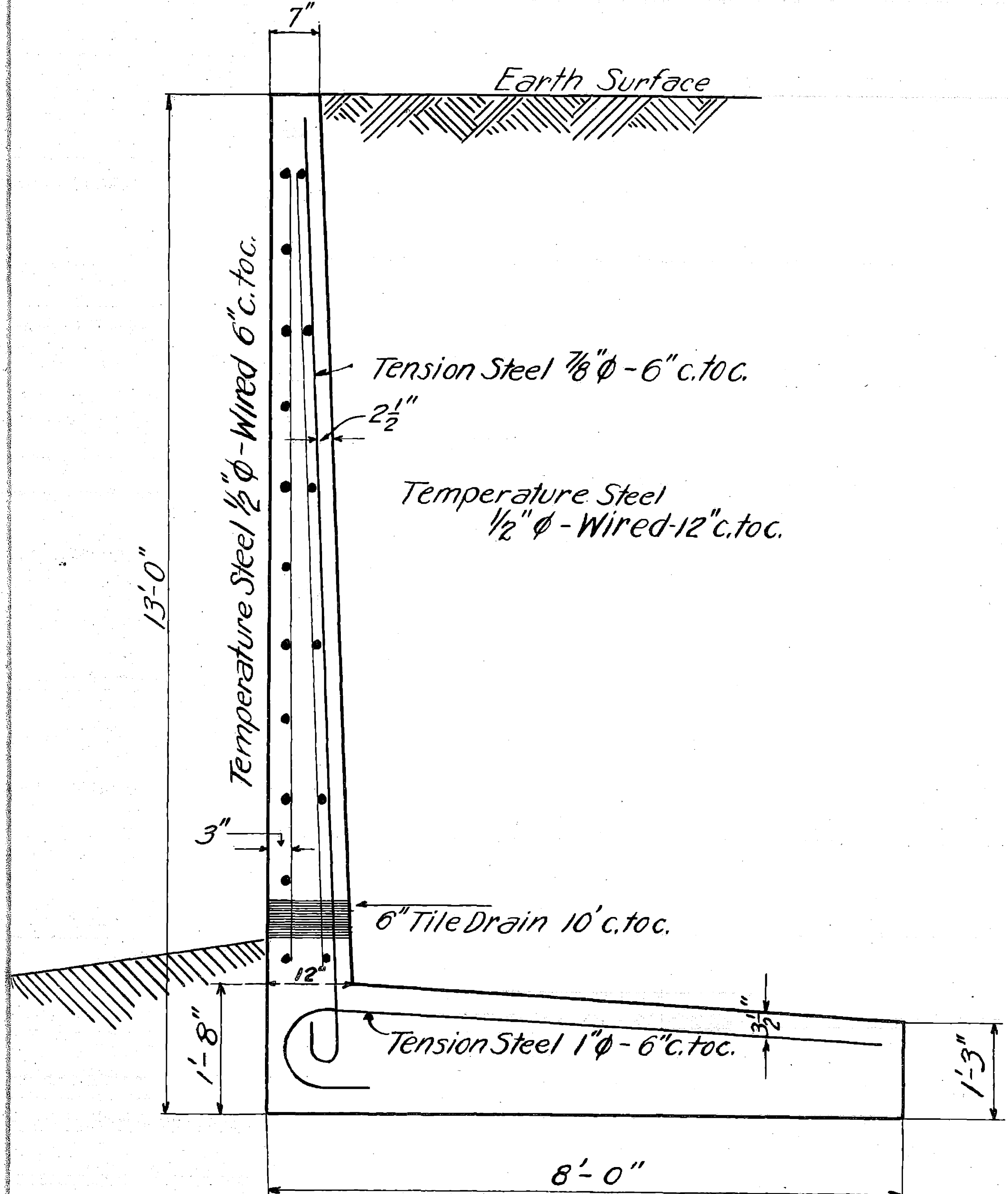
VIII Depth of piles into concrete

Shearing force = H = 7743 #

Bearing on 10" diam. piles = 350 #/sq in

$$\text{Depth} = \frac{7743}{10 \times 350} = 2.21 \text{ inches} < 6"$$

Imbed piles 6" in. concrete.



CROSS SECTION
REINFORCED CONCRETE RETAINING WALL
Scale 1/2" = 1'

John L. Vaupel.

Computations of Retaining Wall.

Weight of Earth = 100 #/cu.ft.

" " Concrete = 150 #/cu.ft.

Angle of repose of Earth = 30°

Allowable fibre stresses

Concrete 650 #/sq. in.

Steel 16000 #/sq. in.

Rankin's Formula Used.

$$P_a = \frac{100 \times 13 \times 13}{2} \times 1 \times \frac{1-\sqrt{1-0.75}}{1+\sqrt{1-0.75}} = 2815^{\#}$$

Moment at base.

$$M = 2815 \times \frac{13}{3} = 12200'^{\#}$$

$$d^2 = \frac{12200 \times 12}{107 \times 12} = 114 \quad \therefore d = 10.7''$$

Depth overall = 10.7 + 1.5 = 12.2" Say 12"

Area of Tension Steel.

$$A_s = 10.7 \times 12 \times .0077 = 0.99 \square''$$

$$\frac{0.99}{2} = 0.495 \square''$$ Use 7/8" ϕ spaced 6" c to c.

Area available = 0.6013 □"

Moment at $\frac{1}{2}$ depth.

$$M = 435 \times \frac{6.5}{2} \times \frac{6.5}{3} = 3050'^{\#}$$

$$d^2 = \frac{3050 \times 12}{107 \times 12} = 28.5 \quad d = 5.34$$

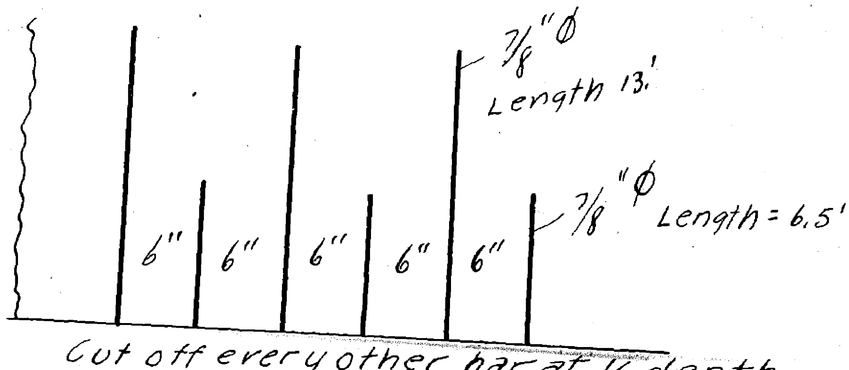
Depth overall = 5.34 + 1.5 = 6.84 say 7"

Make top of wall 7" also.

Area of steel at half depth

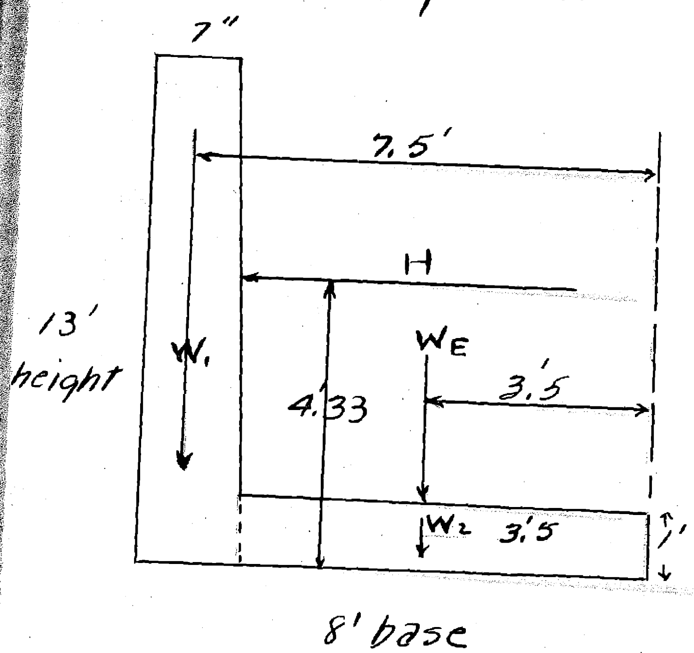
$A_s = .0077 \times 12 \times 5.34 = 0.494 \square''$ needed

Where to cut off steel.



Place 7/8"φ as shown and cut off as shown.

Stability of Wall.



$W_1 = 1950 \times 7.5 = 14600$

$W_2 = 1050 \times 3.5 = 3520$

$W_E = 8400 \times 3.5 = 29200$

11400

$H = 2815 \times 4.3 = 12100$

59420

$X_0 = \frac{59420}{11400} = 5.'2$

$5.32 > 5.'2 > 2.66$

Resultant cuts Middle Third.

Stress at Heel and Toe.

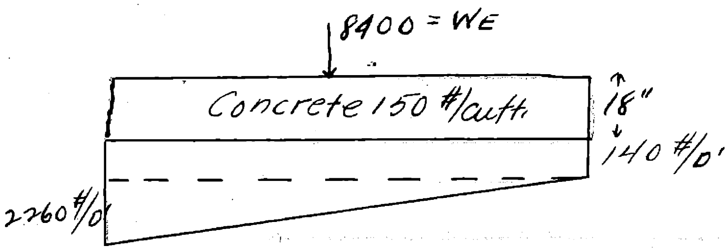
$$f \text{ at heel} = \frac{11400}{8} - \frac{11400 \times 1.2 \times 4 \times 12}{8 \times 8 \times 8}$$

$$= 1430 - 1290 = 140 \ \#/\square'$$

$$f \text{ at toe} = 1430 + 1290 = 2720 \ \#/\square' < 4000 \ \#/\square'$$

Design of Heel.

Assume depth of Heel as 18"



$$M = 9000 \times 3.5 - 1850 = 29650$$

$$R = \frac{29650 \times 12}{12 \times 16.5 \times 16.5} = 108 \text{ OK.}$$

A_s = Area of tension steel

$$= 16.5 \times 12 \times .0077 \times 1 = 1.52 \ \square''$$

Use 1" ϕ spaced 6" c. to c.

Temperature Steel $\frac{1}{2}$" ϕ spaced 6"

$\frac{1}{3}$ Area in back $\frac{2}{3}$ Area in Front.

Area needed = 3% X sectional area of Wall

$$= 3\% \times 12 \times 23.5 = 8.5 \ \square''$$

Front	Back
$\frac{6}{12} = \frac{1}{2}$ □"/ft. of wall	$\frac{3}{12} = \frac{1}{4}$ □"/ft. of wall.
Use $\frac{1}{2}$" ϕ spaced 6" ℄. to ℄.	Use $\frac{1}{2}$" ϕ spaced 12" ℄. to ℄.

Wire Temperature steel.

Test for shear.

$$Shear = 9000 - 266 \times 3.5$$
$$= 8210\#$$

$$\frac{8210}{12 \times 18} = 38\#/□" < 40\#/□"$$

Make Heel 20" at toe

" 15" at Heel

to allow for Reinforcing Steel.