US Army Corps
of Engineers
New York District

General Design Memorandum

Streams in Westchester County, N.Y.

Flood Control Project for the Mamaroneck and Sheldrake Rivers Basin in the Village of Mamaroneck, N.Y.

Volume 2 of 6 - Appendix A, Hydrology, and
Appendix B, Hydraulics

January 1989
Final Report

WESTCHESTER COUNTY STREAMS
MAMARONECK AND SHELDRAKE RIVERS
FLOOD CONTROL PROJECT
VILLAGE OF MAMARONECK

GENERAL DESIGN MEMORANDUM

APPENDIX A

HYDROLOGY

JANUARY 1989

MAMARONECK AND SHELDRAKE RIVERS
MAMARONECK, NEW YORK

HYDROLOGIC SUMMARY

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APPENDIX A - HYDROLOGY

I - GEOGRAPHY AND TOPOGRAPHY

A1. DESCRIPTION OF AREA. Mamaroneck is a village that lies in the southeastern part of Westchester County, New York. It is located on the Mamaroneck and Sheldarake Rivers as shown on Figure A1. Drainage areas and stream bed elevations of the main stream and its tributaries are given in Table A1.

A2. WATERSHED. MAMARONECK AND SHELDRAKE RIVERS. The combined watershed of the Mamaroneck and Sheldrake Rivers, located entirely in New York State, has a total drainage area of 23.6 square miles. The leaflike, two stem watershed is roughly rectangular shaped, with a maximum length of 9 miles in a north-south direction and with a width that varies from 2 to 3 miles. The terrain is gently rolling, lightly wooded in the upper portion and generally cleared in the lower valley. The ridges extend generally in a north-south direction, as shown on Figure A2.

A3. The Mamaroneck River rises downstream of Rye Lake, in the northern section of Harrison at an elevation of 520 feet above mean sea level. The river flows generally south for a distance of about 11 miles to Long Island Sound, which it enters through Mamaroneck Harbor. The average slope of the Mamaroneck is approximately 10 feet per mile, as shown on Figure A2.

A4. The Sheldrake River rises in the northeast portion of Scarsdale, New York, at an elevation of 300 feet above mean sea level. The river flows generally south-southeast for a distance of about 7.0 miles and joins the Mamaroneck River at a point about 0.6 miles above its mouth. One major tributary, known as the East Branch, enters the Sheldrake River at a point 1.8 miles upstream of its junction with the Mamaroneck River. The average slope of the Sheldrake River is approximately 25 feet per mile, as shown on Figure A2.

A5. Several ponds, artifical reservoirs and lakes are located on the Mamaroneck and Sheldrake Rivers and their tributaries. A water supply reservoir operated by the Westchester Joint Water Works and serving the Village of Mamaroneck is located on the Mamaroneck River 2 to 3 miles above its mouth. It has a storage capacity of 107 acre-feet at a spillway crest of 40 feet mean sea level. The Mamaroneck Dam was constructed as a "run of river" dam, and the entire dam length is an overflow section with a crest elevation of 40 feet (M.S.L.). Two uncontrolled outlets were subsequently constructed in 1978 at an elevation of 33 feet (M.S.L.) These openings are 6.3 feet x 3.0 feet and have a computed discharge of 560 C.F.S. with a head of 9.5 feet (water surface at top of embankment dam). The discharge over the overflow section with water surface at 44.0 feet (top of embankment) is 4,240 C.F.S. Four

Table A-1
Drainage Areas, Mamaroneck and Sheldrake Rivers Basin, New York

Stream and Locality	Distance from Mouth of Mamaroneck River (miles)	Stream Bed Elevation (Ft. above m.s.l)	Total Drainage Area (sq. mi.)
Mamaroneck River			
Mouth (Boston Post Road)	0	-9.0	23.6
U.S.G.S. gaging station #01301000 (Halstead Avenue)	0.5	10.5	23.4
Below confluence with Sheldrake River	0.7	9.0	23.1
Above confluence with Sheldrake River	0.7	9.0	17.3
U.S.G.S. gaging station #01300800 (Winfield Avenue)	2.1	26.5	15.4
Westchester Joint Waterworks (Mamaroneck Reservoir) Dam	2.2	28.5	15.4
Downstream of confluence with West Branch	3.5	39.5	14.7
Upstream of confluence with West Branch	3.5	39.5	14.0
West Branch Mamaroneck River (mouth)	3.5	39.5	0.7
Sheldrake River			
Mouth	0.7	9.0	6.2
Downstream of Fenimore Road	1.3	17.0	5.8
Upstream of Fenimore Road (site of proposed diversion tunnel)	1.3	16.1	5.6
Downstream of East Branch	1.9	21.0	5.2
Upstream of East Branch	1.9	21.0	3.4
East Branch Sheldrake River (mouth	1.9	21.0	1.8
Sheldrake River at Sheldrake Lake Dam	3.6	90.0	2.6

water supply reservoirs, operated by and serving the Village of Larchmont, are located on the Sheldrake River, from 2.9 to 5.0 miles upstream of the mouth. The total capacity is 555 acre-feet of which 500 acre feet are in the largest, the Larchmont Water Company Dam No.2. The original purpose for the dam was for water supply use. Presently, the dam provides off line, standby public water supply capabilities. Its present use appears to be mainly that of a recreational and conservation area for the community. The Larchmont Dam No. 2 is not adequate to pass the 1/2 PMF. However, the rock ogee spillway is in excellent condition and the facility is reasonably well maintained. A number of small lakes and ponds are located in the headwaters of the watershed. The largest are Silver and Forest Lake and Croker Pond with water surface areas of 42, 11 and 6 acres, respectively. The combined water surface area of all the ponds and reservoirs is approximately 120 acres. The location of the dams are shown on Figure A2.

II - CLIMATOLOGY

A6. GENERAL. The climate in the study areas is moderate with an average temperature of 51 degrees, Fahrenheit. The extreme temperatures observed, based on the available data for all stations, were 18 degrees Fahrenheit below zero and 105 degrees Fahrenheit above zero at Scardale, New York. The average growing season is 184 days in the Mamaroneck and Sheldrake Rivers watershed. The relative humidity averages about 67 percent. The prevailing winds are from the northwest with an average velocity of 14 miles per hour.

A7. PRECIPITATION. Data on precipitation is obtainable from five stations in and surrounding the Mamaroneck and Sheldrake River Basins, as shown on Figure A1. Of these, three stations are equipped with automatic recording rainfall gages and two with standard non-recording gages, which are read once daily.

A8. ANNUAL AND MONTHLY PRECIPITATION. The average annual precipitation in the Mamaroneck and Shedrake River Basins is approximately 44.2 inches. The observed extreme annual values were 66.98 inches at Bedford Hills Station, New York (1901), and 25.83 inches at White Plains Maple Moor Station, New York (1965). The monthly extremes are 16.64 inches in October 1955, and a trace in November 1917, at the Bedford Hills Station, New York. The distribution of precipitation throughout the year is fairly uniform with higher amounts occurring during the summer months. The average annual snowfall recorded at Scarsdale, New York, is approximately 39 inches with a water equivalent of 4 inches.

A9. STORM TYPES. The past floods of the greatest magnitude in the Mamaroneck and Sheldrake watersheds have been caused by (1) intense rain accompanying the transcontinental type storms, (2) localized thunderstorms, (3) hurricane-like disturbances of West Indian origin or (4) less intense rains of long duration falling on snow-covered frozen or saturated ground. Those storms which have resulted in the worst floods in the watersheds are described in the following paragraphs.

A10. PAST STORMS. Flood-producing storms over the Mamaroneck and Sheldrake Basin have occurred most frequently in the spring and fall seasons. Some of the notable storms which have caused flood conditions in the basins occurred during the following periods: July 1889, October 1903, March 1936, July 1938, September 1938, July 1942, August 1942, September 1944, March 1953, August 1955, October 1955, August 1960, April 1961, March 1962, August 1971, June 1972, September 1975, November 1977, and April 1980. Some of the notable storms over the basins are briefly described in the following pargraphs.

A11. Storm of 28-31 July 1889. This was a severe summer storm of limited extent which centered west of the Mamaroneck and Sheldrake River basins. The average rainfall over this area was about 10 inches. A total rainfall of 21.73 inches was recorded at Yonkers, of which 9.57 inches fell in 21 hours and 16.65 inches fell in two days. At White Plains, a total of 10.21 inches of rainfall was recorded for the storm, of which a maximum of 2.8 inches fell in one day.

A12. Storm of 7-12 October 1903. This was an unsusal storm of cloudburst type and was of great areal extent. The rainfall was centered over Paterson, New Jersey about 18 miles west of the Mamaroneck and Sheldrake River basins. During this storm an average rainfall of 7.8 inches fell on the study areas and caused one of the worst floods in history. The maximum 24-hour rainfall recorded in the vicinity was 5.49 inches at Bedford, New York, from 3:00 P.M. of the 7th of October to 1:00 P.M. of the 8th of October.

A13. Storm of 9-22 March 1936. This was a general transcontinental storm throughout the northeastern United States which centered over both the Ohio and Connecticut River basins. During the period 10-12 March, an average rainfall of 3.0 to 3.5 inches fell over the study areas. The maximum daily rainfall recorded in the vicinity was 3.60 inches at Valhalla, New York. The precipitation from this storm was augmented by melting snow with a water equivalent of about two inches.

A14. Storm of 21-24 July 1938. This storm was a severe summer storm of limited extent which covered southeastern New York, Connecticut and Massachusetts. During this period an average rain of about 5.4 inches to 6.0 inches fell on the study areas of which about 3.7 inches fell in 7 hours. This storm was preceded by 3 days of moderate rains. During this storm period, total rainfall of 6.51, 5.99 and 4.52 inches was recorded at Putnam Lake, Conn., Scarsdale, NY and Bedford Hills, NY respectively, of which 5.04 inches fell within 24 hours at Putnam Lake.

A15. Storm of 19-22 September 1938. This storm, which covered the Atlantic coastal states and centered over Connecticut and Massachusetts, was the result of a tropical hurricane which originated in the West Indies and moved northward along the Atlantic Coast and across the New England States. During this storm an average rainfall of about 8.7 inches fell on the study areas. The maximum daily rainfall recorded in the vicinity was 6.55 inches at Scarsdale, New York. This storm was preceded by 2 days of moderate rainfall.

A16. Storm of 9-10 August 1942. This was a severe local storm of cloudburst intensity which centered over the southern portion of Westchester County. During this period, the total average rainfall over the study areas was about 6 inches. After a rainfall of 4.59 inches during the day of 9 August 1942, a heavy rain of 2.53 inches was recorded at Larchmont between 12 noon and 2:00 P.M. on 10 August 1942. This sudden downpour caused severe flooding.

A17. Storm of 12-14 September 1944. This storm was the result of a severe hurricane which originated in the West Indies and traveled northward along the Atlantic Coast. Although the hurricane was about equal in intensity to that of September 1938, the rainfall intensity was less severe. During this storm, a total of about 9.4 inches of rain fell over the study areas. The maximum daily rainfall recorded in the vicinity was 6.00 inches at Valhalla, New York.

A18. Storm of 14-18 October 1955. A cold front moved into eastern Pennsylvania and southern New York on the morning of 13 October 1955 and became stationary with a coastal wave moving northward accompanied by moderate to heavy rains on the 14th and 15th of October. The center drifted slowly northward bringing an abundance of rains which continued in the northeast through the 16th. Concurrently, progress of an extratropical storm accompanied by heavy rainfall extended through the 17th of October. The maximum daily rainfall recorded in the vicinity was 4.64 inches at Scarsdale, New York. During the storm period total rainfall of 9.66, 7.92 and 9.01 inches was recorded at Putnam Lake, Conn., White Plains, N.Y. and Pleasantville, N.Y. respectively, of which 6.68 inches fell within 24 hours at Putnam Lake.

A19. Storm of 26-29 August 1971. Tropical storm "Doria" originated in the Bahama Islands and moved northward along the North Atlantic Coast. As she crossed the coastal portions of North Carolina and Southeastern Virginia, her speed increased to 20-25 knots. The storm center reached southwestern Connecticut by 0800 on 28 August. At the Scarsdale rain gage adjacent to the Sheldrake River Basin, a total rainfall of 6.54 inches was recorded between the 27th and 28th of August.

A20. Storm of 16-22 June 1972. Tropical storm "Agnes" was the result of a tropical storm depression that originated south of the Gulf of Mexico and moved northward over land carrying massive amounts of moist air. At the Scarsdale rain gage in the Mamaroneck and Sheldrake River Basins, a total of 4.83 inches was recorded on the 18th and 19th of June. The Mamaroneck and Sheldrake River basin-wide total rainfall was 4.52 inches. The hyetograph and mass rainfall curve for this storm are shown on Figure A3.

A21. Storm of 19-27 September 1975. Hurricane "Eloise" was the result of a tropical storm depression that originated east of Puerto Rico and moved northward over land carrying massive amounts of moist air. The duration of the storm in the study areas was from the morning of the 19th to the early morning hours of the 27th. The Mamaroneck River basin wide rainfall was 4.89 inches. The hyetograph and mass rainfall curve for this storm are shown on Figure A4.

A22. Storm of 8 November 1977. A tropical depression merged with a weak extratropical storm off the New Jersey coast. This resulted in a heavy down pour with 4.75 inches on 8 November over the Mamaroneck River Basin with a maximum hourly intensity of 0.80 inches per hour. The hyetograph and mass rainfall curve are shown on Figure A5.

A23. Storm of 9-10 April 1980. This storm originated in the midwest and moved toward the Great Lakes. It was caused by a typical frontal system with an occluded warm and cold front. This resulted in a heavy down pour of up to 4.07 and 4.65 inches on 9 April, at the Scarsdale and Pleasantville Stations with a maximum hourly intensity of 1.00 inch per hour at the Scarsdale Station. The hyetograph and mass rainfall curve are shown on Figure A6.

A24. Storm of 10 April 1983. This storm originated in the midwest and moved toward the Great Lakes. It was caused by a typical frontal system with an occluded warm and cold front. This resulted in a heavy down pour of from 2.4 to 3.20 inches in the Westchester County area with an average of 2.74 inches over the Mamaroneck River Basin. The hyetograph and mass rainfall curve are shown on Figure A7.

A25. STANDARD PROJECT RAINFALL. The "Standard Project Storm" represents the most severe flood producing rainfall depth-area-duration relationship and isohyetal pattern of any storm that is considered reasonably characteristic of the region in which the drainage basin is located. The rainfall used in determining the Standard Project Flood in the basin was obtained from the Civil Works Bulletin No. 52-8, dated 26 March 1952 and reprinted in June 1964 as EM 1110-2-1411 and revised March 1965, entitled "Standard Project Flood Determination." The 200 square mile, 24-hour precipipation index for the basin is 10.4 inches. The interpolated total storm rainfall for the Mamaroneck River Basin with a drainage area of 23.6 square miles using a transposition coefficient of 1.0 was 14.61 inches. The Standard Project Rainfall distribution on a half-hour basis is shown on Table A2. A high soil moisture content and ground water conditions due to previous rains was assumed at the start of the storm. Therefore, losses of rainfall through the ground would be small, so that the infiltration losses were assumed as 0.15 inch per hour. The resulting rain excess graphs are shown on Figure A19.

A26. HYPOTHETICAL STORM RAINFALL. The hypothetical storm rainfalls for the 500, 100, 10, 2, and 1-year storms were developed using procedures and plates in Technical Paper No. 40, "Rainfall Frequency Atlas of the United States." Average point rainfalls were taken from the isopluvial maps for the Mamaroneck River area. The values derived are in a partial duration series. The depths are tabulated in Table A-3 for each storm for durations from 1 to 24 hours. The point rainfall depths were converted to 23.6 square mile rainfall depths using the area-depth curves of Figure 15 of Technical Paper No. 40. A rainfall distribution similar to that for the Standard Project Storm was used where the largest hour of precipitation is preceded by the second largest and followed by the third largest. Because of the relatively small sizes of the subareas utilized in the hydrological model, in the range of 1 square mile, a 1/2 hour time distribution was used. The final hypothetical rainfall distributions are shown in Table A-4.

III - RUNOFF AND STREAM FLOW

A27. RUNOFF RECORDS. Runoff records are available for the stream gage operated by the United States Geological Survey on the Mamaroneck River at Mamaroneck. The gage is located 113 ft downstream from the bridge on Halstead Avenue and 700 ft downstream from the Sheldrake River confluence. The drainage area for the gage is 23.4 square miles. Data for the station is given in Table A5.

A28. ANNUAL RUNOFF. The average annual discharge for the Mamaroneck at Mamaroneck stream gage for a period of record dating back to 1943 is 34.6 C.F.S. which is equivalent to about 20 inches of runoff or about 45 percent of the estimated average rainfall. Additional stream data are contained in Table A5. A tabulation of the annual peak discharges for the gage is shown on Table A6.

Table A2

Standard Project Storm Rainfall Distribution
Mamaroneck and Sheldrake Rivers Basin, New York

Half Hour Ending	Day 1	Day 2	Day 3	Day 4
0.30	0	0	.04	0
1.00	0	0	.04	0
	0	0	.04	0
2	0	0	.04	0
	0	0	.04	0
3	0	0	.04	0
	0	0	.04	0
4	0	0	.04	0
	0	0	.04	0
5	0	0	.04	0
	0	0	.04	0
6	0	0	.04	0
	0	.02	.12	.01
7	0	.02	.12	.01
	0	.02	.12	.01
8	0	.02	.12	.01
	0	.02	.12	.01
9	0	.02	.12	.01
	0	.02	.12	.01
10	0	.02	.12	.01
	0	.02	.12	.01
11	0	.02	.12	.01
	0	.02	.12	.01
12	0	.02	.12	.01
12:30	.01	.06	.47	.02
13	.01	.06	.47	.02
	.02	.08	.56	.03
14	.02	.08	.56	.03
	.02	.09	.70	.04
15	.02	.09	.70	.04
	.03	.11	.85	.04
16	.08	.36	2.70	.14
	.02	.09	.65	.03
17	.02	.09	.65	.03
	.02	.07	.51	.03
18	.02	.07	.51	.03
18:30	0	.01	.07	0
19	0	.01	.07	0
	0	.01	.07	0
20	0	.01	.07	0
-	0	.01	.07	0
21	0	.01	.07	0
	0	.01	.07	0
22	0	.01	.07	0
	0	.01	.07	0
23	0	.01	.07	0
	0	.01	.07	0
24	0	.01	.07	0

Storm Total = 14.61 in.

TABLE A3

MAMARONECK RIVER

POINT RAINFALL DEPTHS IN INCHES

FOR HYPOTHETICAL STORMS[1]

HYPOTHETICAL STORM	DURATION--HOURS 1	2	3	6	12	24
1 YEAR	1.20	1.46	1.61	1.91	2.35	2.64
2 YEAR	1.35	1.75	1.92	2.32	2.80	3.31
10 YEAR	2.00	2.63	2.88	3.58	4.25	5.08
50 YEAR	2.67	3.31	3.71	4.48	5.38	6.38
100 YEAR	2.96	3.72	4.19	5.21	6.23	7.19
500 YEAR[2]	3.90	4.90	5.60	6.95	8.20	9.25

1. Data taken from isopluvial maps contained in Technical Paper No. 40, Rainfall Frequency Atlas of the United States, U.S. Dept. of Commerce, Washington, D.C., 1961.

2. Determined by extrapolation according to procedures in Technical Paper No. 40 for return periods longer than 100 years.

Table A4
Mamaroneck Hydrologic Model
Half-Hour Precipitation Increments in Inches
For Hypothetical Storms

Half Hour Ending	1 Yr.	2 Yr.	10 Yr.	50 Yr.	100 Yr.	500 Yr.
030	.01	.02	.03	.03	.03	.04
100	.01	.02	.03	.04	.03	.04
	.01	.02	.03	.04	.03	.04
200	.01	.02	.03	.04	.04	.04
	.01	.02	.03	.04	.04	.04
300	.01	.02	.03	.04	.04	.04
	.01	.02	.03	.04	.04	.05
400	.01	.02	.04	.04	.04	.05
	.01	.02	.04	.05	.04	.05
500	.01	.02	.04	.05	.05	.05
	.02	.03	.04	.05	.05	.06
600	.02	.03	.04	.05	.05	.06
	.03	.03	.05	.06	.07	.09
700	.03	.04	.05	.07	.08	.09
	.03	.04	.05	.07	.08	.10
800	.04	.04	.06	.08	.09	.11
	.04	.04	.06	.08	.10	.12
900	.04	.05	.07	.09	.11	.13
	.04	.06	.10	.11	.14	.19
1000	.05	.06	.11	.12	.16	.21
	.06	.08	.13	.15	.19	.25
1100	.07	.08	.12	.19	.23	.33
	.12	.18	.29	.29	.35	.46
12	.26	.31	.48	.65	.72	.92
	.81	.90	1.32	1.75	1.93	2.57
13	.15	.22	.35	.37	.43	.57
	.08	.10	.14	.22	.26	.38
14	.06	.08	.14	.16	.21	.28
	.05	.07	.12	.13	.17	.23
15	.04	.06	.10	.11	.15	.20
	.05	.05	.07	.10	.11	.14
16	.04	.05	.07	.09	.10	.12
	.04	.04	.06	.08	.09	.11
17	.04	.04	.06	.07	.08	.10
	.03	.04	.05	.07	.08	.10
18	.03	.03	.05	.06	.07	.09
	.02	.03	.05	.06	.05	.06
19	.02	.03	.04	.05	.05	.06
	.01	.03	.04	.05	.05	.05
20	.01	.02	.04	.05	.05	.05
	.01	.02	.04	.05	.04	.05
21	.01	.02	.04	.04	.04	.05
	.01	.02	.03	.04	.04	.04
22	.01	.02	.03	.04	.04	.04
	.01	.02	.03	.04	.04	.04
23	.01	.02	.03	.04	.04	.04
	.01	.02	.03	.04	.03	.04
24	.01	.02	.03	.03	.03	.04
Total	2.57	3.22	4.94	6.21	7.19	9.00

TABLE A5
STREAM DISCHARGE DATA

Stream Gage	Mamaroneck River at Mamaroneck, N.Y.
Drainage Area (sq.mi.)	23.4
Period of Record	Water Years 1944-1981
Annual Discharge	
Maximum Water Year	1978
c.f.s.	59.9
c.s.m.	2.56
inches	34.74
Minimum Water Year	1950
c.f.s.	16.70
c.s.m.	0.71
inches	9.69
Average Year	
c.f.s.	34.6
c.s.m.	1.48
inches	20.07
Monthly Discharge	
Maximum Month	January 1979
c.f.s.	208
c.s.m.	8.89
inches	10.05
Minimum Month	Sept. 1965
c.f.s.	0.95
c.s.m.	0.941
inches	0.05
Daily Discharge	
Maximum Day	19 June 1972
c.f.s.	2,340.0
c.s.m.	100.0
inches	3.72
Minimum Day	30 Sept. 1965
c.f.s.	0.10
c.s.m.	0.004
inches	0.00016
Peak Discharge	
Day	26 September 1975
c.f.s.	3,700
c.s.m.	158.1
inches	5.88

TABLE A6
MAMARONECK RIVER AT MAMARONECK
DRAINAGE AREA = 23.4 SQUARE MILES

PERIOD OF RECORD 1944 - 1982
RECORDED YEARLY PEAK DISCHARGE

WATER YEAR	DATE	PEAK DISCHARGE (C.F.S.)	ORDER OF MAGNITUDE
1944	15 SEPTEMBER 1944	1760	14
1945	7 AUGUST 1945	738	33
1946	27 MAY 1946	1200	27
1947	5 APRIL 1947	795	32
1948	1 APRIL 1948	660	37
1949	6 JANUARY 1949	723	35
1950	23 MARCH 1950	232	39
1951	31 MARCH 1951	1550	18
1952	1 JUNE 1952	1270	24
1953	13 MARCH 1953	1620	15
1954	11 SEPTEMBER 1954	900	31
1955	13 AUGUST 1955	1370	22
1956	15 OCTOBER 1955	1940	12
1957	1 NOVEMBER 1956	711	36
1958	28 FEBRUARY 1958	1260	25
1959	2 JANUARY 1959	723	34
1960	19 AUGUST 1960	1490	21
1961	16 APRIL 1961	1500	20
1962	12 MARCH 1962	1500	19
1963	10 NOVEMBER 1962	975	30
1964	29 NOVEMBER 1963	1150	29
1965	8 FEBRUARY 1965	1310	23
1966	13 FEBRUARY 1966	1170	28
1967	7 MARCH 1967	1260	26
1968	29 MAY 1968	1840	13
1969	25 MARCH 1969	1600	16
1970	10 FEBRUARY 1970	1990	10
1971	28 AUGUST 1971	2260	8
1972	19 JUNE 1972	3550	2
1973	2 FEBRUARY 1973	2290	7
1974	3 SEPTEMBER 1974	2840	5
1975	26 SEPTEMBER 1975	3700	1
1976	10 AUGUST 1976	1570	17
1977	25 FEBRUARY 1977	2000	9
1978	8 NOVEMBER 1977	3240	4
1979	21 JANUARY 1979	3410	3
1980	10 APRIL 1980	2790	6
1981	9 SEPTEMBER 1981	653	38
1982	4 JANUARY 1982	1970	11

A29. FLOODS OF RECORD. Information concerning the occurrence and magnitude of historic floods in the Mamaroneck and Sheldrake River basins is meager. From newpaper accounts and reports and from miscellaneous sources that were investigated for data concerning the occurrence of floods in the basins, it was found that high discharges resulting from the storms of July 1889, October 1903, July 1938 and September 1938. Recent floods for which data is available occurred on 15 September 1944, 13 March 1953, October 1955, August 1971, June 1972, September 1975, November 1977, April 1980 and April 1983.

A30. Flood of 24 July 1938. Actual discharge measurements for the flood of 24 July 1938 are not available.

A31. Flood of September 1938. This storm caused severe flooding on the Mamaroneck and Sheldrake Rivers.

A32. Flood of October 1955. The storm of October 1955 caused major flooding on the Mamaroneck and Sheldrake Rivers. A peak discharge of 1940 C.F.S. was recorded at the Mamaroneck River at Mamaroneck stream gage.

A33. Flood of August 1971. The storm of August 1971 caused major flooding on the Mamaroneck and Sheldrake Rivers. A peak discharge of 2,260 C.F.S. was recorded at the Mamaroneck River at the Mamaroneck stream gage.

A34. Flood of June 1972. The storm of June 1972 caused the second highest flood of record on the Mamaroneck River at Mamaroneck and was estimated at 3,550 C.F.S. based on U.S. Geological Survey indirect computations and on our own hydraulic computations using flood marks. At Winfield Avenue on the Mamaroneck River above the Sheldrake River, in the vicinity of the Mamaroneck Reservoir a discharge of 2560 C.F.S. was estimated with the HEC-1 model. This flood was routed down to a point above the Sheldrake River and the peak discharge was determined to be 2,594 C.F.S. For the Sheldrake River at its mouth, the peak discharge was estimated as 957 C.F.S. A reconstitution of the June 1972 flood at the Mamaroneck River at Mamaroneck gage is shown on Figure A3.

A35. Flood of September 1975. The storm of September 1975 caused the highest flood of record on the Mamaroneck River at Mamaroneck gage with a peak discharge of 3,700 C.F.S. The reproduced flood hydrograph at the Mamaroneck gage is shown on Figure A4.

A36. Flood of November 1977. The storm of November 1977 caused the fourth highest flood of record on the Mamaroneck River at Mamaroneck gage with a peak discharge of 3,240 C.F.S. The reproduced flood hydrograph at the Mamaroneck gage is shown on Figure A5.

A37. Flood of April 1980. The storm of April 1980 caused the sixth highest flood of record on the Mamaroneck River at Mamaroneck gage with a peak discharge of 2,790 C.F.S. The reproduced flood hydrograph at the Mamaroneck gage is shown on Figure A6.

A38. Flood of April 1983. The storm of April 1983 caused a peak discharge of 1810 C.F.S. at the Mamaroneck gage. The reproduced flood hydrograph at the Mamaroneck gage is shown on Figure A7.

A39. FLOOD FREQUENCY. At the Mamaroneck stream gage, the peak discharge versus frequency analysis was developed in accordance with the methods contained in the "Guidelines for Determining Flood Flow Frequency, United States Water Resources Council, Washington, D.C. March 1976". The basic procedure is the Pearson Type III distribution with log transformation of the flood data for defining the annual flood series. This method assumes that the logarithms of the annual peak discharges are normally distributed and that statistical procedures are applicable, thus minimizing personal judgement in plotting and extrapolating data, and thus providing consistent results for economic studies. Because of the urbanization trends in this basin, the period of record annual peaks and precipitation records were analyzed and it was determined that a major storm occurred in September 1944. The precipitation intensities during this storm were the highest since 1940.

A detailed review of the land use and population data indicates that the Mamaroneck River Basin did not experience any significant urbanization since 1955. Because of this a detailed analysis was made of all storms from the 1940's to 1955. The only significant floods in this period were those of September 1944 and October 1955. Lesser flood magnitudes with peaks in excess of 1000 c.f.s. were the following:

Flood	Peak (c.f.s.)
27 May 1946	1200
31 March 1951	1550
1 June 1952	1270
13 March 1953	1620
13 August 1955	1370
15 October 1955	1940

These floods were investigated with the current HEC-1 model, and were reproduced with constant loss rates in the range of 0.2 inch per hour. The reproductions of the March 1951, June 1952, and March 1953 floods are shown on Figures A8, A9, and A10 respectively. This indicates that because of the lower intensities and runoff associated with these stroms, that urbanization impacts on these stroms are not significant. Because of this, the only adjusted peak used in the peak discharge versus frequency analysis was that of September 1944. This flood was reproduced with extremely high constant loss rates of 0.90 inch per hour, as shown on Figure A12. However, if the September 1944 strom occurred today with the current conditions of urbanization, and with an initial loss of 0.87 inches and a constant loss rate of 0.27 inches per hour, loss rates that were used to reproduce the conditions of the similar September 1975 strom, the reproduced peak would be 3,999 c.f.s., as shown on Figure A13. This is significantly higher than the observed peak flood discharge at the gage of 1760 c.f.s.. The precipitation distributions during the similar September 1944 and September 1975 storms are shown on Table A7.

Table A7

Precipitation Distributions
Sept. 1944 Storm - Sept. 1975 Storm
Comparison
Mamaroneck and Sheldrake Rivers Basin

Date	Half Hour Ending	Precipitation Inches	Date	Half Hour Ending	Precipitation Inches
14 Sept. 1944	1600	0	(a) 26 Sept. 1975	0800	.01
	1630	.13		0830	.15
	1700	.21		0900	.26
	1730	.41		0930	.42
	1800	.60		1000	.48
	1830	.65		1030	.45
	1900	.65		1100	.43
	1930	.55		1130	.66
	2000	.50		1200	.57
	2030	.35		1230	.31
	2100	.25		1300	.21
	2130	.15		1330	.07
	2200	.07		1400	.05
	2230	.03		1430	.15
	2300	.03		1500	.20
				1530	.12
	Total:	4.58 in.		1600	.06
				1630	.01
				1700	.03
				1730	.03
				1800	.01
				1830	0
				1900	0

(a) : Total September 1975 storm not shown

A further review of the urbanization trends indicated by the list of annual peaks indicated that another major flood magnitude was that of October 1955 with a peak discharge of 1940 C.F.S. The HEC-1 model was able to reproduce this event with a peak discharge of 1940 C.F.S. utilizing reasonable basin loss rates with a inital loss of 1 inch and a constant loss rate of .2 in/hr, as shown on Figure A11. Therefore we made the assumption that the major period of development or urbanization took place in the years 1944 to 1955. This was confirmed in discussions with local officials. Therefore the statistical analysis of the recorded yearly peak discharges was run with the adjusted September 1944 flood and the remaining flood peaks as they were observed. The statistically derived parmeters were a mean of 3.173, a standard deviation of 0.225 and a computed skew of 0.14. A regional skew of 0.4 was developed for this area using the, "Regional Frequency Study, Upper Delaware and Hudson River Basins, New York District," as developed by the Hydrologic Engineering Center in November 1974. The weighted adopted skew was determined to be 0.253. The mean square error of the generalized skew was determined to be 0.185. The confidence limits for 5% and 95% levels of significance were developed and are shown on the derived frequency curve shown on Figure A14.

The peak discharge versus frequency curve was adjusted to include flows of magnitude less than the annual peak discharge regardless of interval of occurrence as developed by a partial duration series described by W.B. Langbein on pages 879-881 of the Transactions of the American Geophysical Union, Volume 30, No. 6, December 1949. In addition, Weibull plotting positions were applied to 152 peaks above a base flow of 550 c.f.s. to determine the lower end of the frequency curve shown on Figure A15. The two methods of analysis, as shown on the curve, yielded similar answers for the lower end of the frequency curve.

A40. HYDROLOGIC MODEL. Because of the complexity of the Mamaroneck River basin, a hydrologic model was developed to more accurately calibrate the runoff chracteristics of the basin. The sub-area breakdown of the model and the network of 21 sub-areas and 29 nodes is shown on Figure A16. Because of the relatively small sub-areas used in the model, a 1/2 hour time step was used for modeling purposes.

A41. MODELING TECHNIQUE. The basic modeling tool selected for this study was the Flood Hydrograph Package, HEC 1, of the Hydrologic Engineering Center. The program is capable of performing a variety of hydrologic modeling tasks. The particular capability used for the Mamaroneck River basin was the generalized modeling of the runoff and routing processes to simulate the hydrologic response of the Mamaroneck River basin to a precipitation event. The HEC-1 program was used to optimize specified parameters of the precipitation, runoff and routing processes to achieve a best fit with respect to an observed hydrograph and known precipitation. The objective is to minimize the weighted squared deviations between the observed and computed hydrographs.

A42. SIMULATION. In the process of modeling this basin, the program provided several techniques with which to input and distribute the precipitation, compute infiltration losses, and determine subbasin outflow hydrographs from unit hydrograph methods. The outflow hydrographs for each subbasin are then combined with those of other subbasins at appropriate confluences or nodes. For this model, the runoff response for all main stream incremental areas was divided such that half entered at the upstream end and half entered at the downstream end. This allows for a more accurate depicition of the runoff response for the subarea. Then stream routing is applied to bring a hydrograph at an upstream location to a downstream location. The model output is the response of the basin in terms of discharge to the input storm event. The Mamaroneck River Basin map with the subarea and nodal network is shown on Figure A16.

A43. DESCRIPTION OF HEC-1 PROCEDURES. HEC-1 is a lumped parameter model of each subarea under consideration. The parameters or computations for a particular subarea are assumed to apply for its entire area.

A44. PRECIPITATION. The basic driving input to the model is a time history of precipitation for each subbasin. Within and surrounding the basin there are a few non-recording rainfall gages and a recording gage.

The location of these gages is shown on Figure A1. Because of the limited number of precipitation gages, there are a few pockets in the basin which do not have adequate precipitation coverage. Therefore, in order to predict the rainfall volume in ungaged areas for a particular storm event, rainfall isohyets were developed from the existing gages which allow for the complete aerial coverage of the basin. A summary of the precipitation data used within the model for the floods of record is shown in Table A8.

A45. INFILTRATION LOSSES. The infiltration losses are subtacted from the rainfall record to produce a rainfall excess record. Although more complex equations which relate loss rates to rainfall intensity and to accumulated loss are available, the Mamaroneck River model utilized the initial and uniform loss concept. As a first trial, the total runoff amount determined from flood hydrograph analysis was subtracted from the total storm rainfall to determine the total losses. These losses were evenly distributed during the storm period after an adjustment was made for the initial loss.

The final loss rates were based on matching the observed hydrographs at the Mamaroneck gage and on reconstituting estimated peak flows (from high water marks) at other locations within the basin. The initial (STRTL) and final constant (CNSTL) loss rates used within the model are shown in Table A8. In addition, the percent of urbanized area in each subarea was estimated from U.S. Geologic Survey 7 1/2 minute topographic maps and other available local mapping based on the type of predominant urbanization, (RTIMP). The HEC-1 program used this variable to reduce the computed basin average loss rate and the resulting value is used for the entire subarea. A summary of these values is shown in Table A8.

A46. DIRECT RUNOFF. Once a rainfall excess record has been created, a discharge or runoff hydrograph can be computed by the use of unit hydrograph theory. The HEC-1 program allows the user to input Snyder or Clark unit

TABLE A8
MAMARONECK RIVER MODEL

SUBBASIN RAINFALL DEPTHS, LOSS DATA
AND HYDROGRAPH AND RECESSION CHARACTERISTICS FOR FIVE (5) STORMS AND FLOODS ANALYZED

SUBAREA	DRAINAGE AREA SQ. MI.	PERCENT IMPERVIOUS	JUNE 1972 SUBBASIN PRECIPITATION (IN.)	26-29 SEPT. 1975 SUBBASIN PRECIPITATION	7-9 NOV. 1977 SUBBASIN PRECIPITATION	9-11 APRIL 1980 SUBBASIN PRECIPITATION	APRIL 1983 SUBBASIN PRECIPITATION
A	1.74	20	5.40	4.82	4.97	2.75	2.58
B	1.18	22	4.45	4.81	5.00	4.07	2.82
C	0.32	13	4.45	4.81	5.00	3.51	2.72
D	2.95	4	6.04	5.08	4.96	2.63	2.56
E	2.94	10	4.50	4.81	4.98	2.76	2.58
F	0.92	8	4.45	4.81	5.00	3.87	2.78
G	1.32	16	4.45	4.81	5.00	4.07	2.82
H	0.43	6	4.44	4.81	5.00	4.07	2.82
I	2.24	20	4.60	4.87	5.00	4.07	2.82
J	0.63	5	4.65	4.97	5.00	4.07	2.82
K	0.68	6	3.94	4.86	5.00	4.07	2.82
L	0.83	8	4.10	4.84	5.00	4.07	2.82
M	0.68	36	3.94	4.86	5.00	4.07	2.82
N	0.44	13	3.94	4.86	5.00	4.07	2.82
AA	2.63	28	4.82	4.99	4.98	4.07	2.82
BB	0.73	30	3.94	4.86	5.00	4.07	2.82
CC	1.84	10	4.13	4.89	5.00	4.07	2.82
DD	0.37	40	3.94	4.86	5.00	4.07	2.82
EE	0.24	16	3.94	4.86	5.00	4.07	2.82
FF	0.35	34	3.94	4.86	5.00	4.07	2.82
O	0.17	40	3.94	4.86	5.00	4.07	2.82
Average Precipitation			4.65	4.89	4.99	3.64	2.74
	STRTL, in.		0.50	1.27	1.50	0.50	0.60
	CNSTL, in/hr.		0.09	0.27	0.10	0.12	0.17
	STRTQ		5 cfs/sq.mi.	6 cfs/sq. mi.	6 cfs/sq. mi.	3 cfs/sq. mi.	5 cfs/sq. mi.
	QRCSN		20% of Peak Q	20% of Peak Q	20% of Peak Q	20% of Peak Q	20% of Peak Q
	RTIOR		1.08	1.08	1.08	1.08	1.08

hydrograph parameters and the program will internally compute the unit hydrograph ordinates. For the Mamaroneck River model we selected the Clark method. The basic input parameters for Clark's Method are:

Tc = Time in hours from the end of a burst of rainfall excess to the inflection point on the recession limb of the resulting direct runoff hydrograph.

R = Discharge at the inflection point on the recession limb of the direct runoff hydrograph divided by the slope of the recession limb at that point, in hours.

The development of the Clark unit hydrograph parmeters was accomplished through the use of regression equations that related the Clark unit hydrograph parameter to the physical charcteristics of the basin. A tabulation of the physical parameters and the final unit hydrograph parameters is shown on Figure A16 and on Table A9.

A47. REGRESSION ANALYSIS. A multiple regression routine (Generalized Computer Program 704-G1-L2020 developed by the Hydrologic Engineering Center) was used in an effort to correlate Tc and R with various physiographic characteristics of a drainage basin. Imperviousness was included as one of the physical parameters because of the high degree of development in parts of the basin. The physical parameters of the basin that were used in the analysis were:

D.A. = Drainage area in square miles

L = Longest Length

S = Water course slope, in feet per mile, defined as the average slope of the watercourse between points 10 and 85 percent of the distance upstream from the runoff site to the watershed boundary.

I = Index of impervious cover in percent of total land area.

St = Surface storage, in percent of drainage area occupied by lakes and swamps.

TABLE A9
MAMARONECK RIVER
HEC-1 HYDROLOGIC MODEL
SUB-BASIN PHYSICAL PARAMETERS
AND CLARK UNIT HYDROGRAPH PARAMETERS

SUB-BASIN	D.A. SQ. MI.	L MILES	SLOPE FT./MI.	STORAGE %	STORAGE + 1%	tc=R
A	1.74	2.80	119	10	11	3.14
B	1.18	2.18	64	1	2	2.02
C	0.32	1.07	163	0	1	.68
D	2.95	3.90	82	20	21	5.58
E	2.94	2.77	64	1	2	2.44
F	0.92	2.02	116	1	2	1.52
G	1.32	2.12	116	5	6	2.16
H	0.43	1.34	72	0	1	1.10
I	2.24	3.16	76	1	2	2.52
J	0.63	2.14	66	5	6	2.70
K	0.68	1.32	33	15	16	3.20
L	0.83	1.55	99	0	1	1.08
M	0.68	1.63	16	0	1	2.26
N	0.44	1.27	165	3	4	1.14
O	0.17	0.80	87	0	1	.70
AA	2.63	4.60	28	4	5	3.18
BB	0.73	1.65	89	2	3	1.62
CC	1.84	3.16	57	1	2	2.82
DD	0.37	1.42	56	1	2	1.54
EE	0.24	0.74	224	1	2	.56
FF	0.35	1.04	13	0	1	1.74

Note: Slope = Watercourse slope, in feet per mile, defined as the average slope of the watercourse between points 10 and 85 percent of the distance upstream from the runoff site to the watershed boundary.

Storage = Surface storage, in percent of drainage area occupied by lakes and swamps.

tc = Time in hours from the end of a burst of rainfall excess to the inflection point on the recession limb of the resulting direct runoff hydrograph (Clark Method).

R = Discharge at the inflection point on the recession limb of the direct runoff hydrograph divided by the slope of the recession limb at that point, in hours (Clark Method).

Four gages in the Westchester County Area, one gage in the Wallkill Basin, two gages in the Passaic River Basin and 15 gages in the Raritan River Basin were analyzed. The physical parameters of these gaged basins were determined and the unit hydrograph parameters were developed through HEC-1 optimization procedures. A summary of the gaged basins used, their physical parameters and the Clark Tc is shown in Table A10. The analysis was performed with the basin characteristics as the independent variable and Tc, R, Tc+R and R/Tc+R as the dependent variables. Numerous forms of regression equations were investigated and attempts were made to segregate the gages into similar sub-groups to improve the correlation coefficients. However, no significant increases in correlation coefficient were noted. The drainage area and the impervious cover (I) contributed the least to the regression equation and were consequently deleted from the analysis. The regression equation was developed and the final Clark characteristics, Tc, was computed to be:

$$Tc = 4.5\ L^{0.76}\ S^{-.38}\ STOR^{.28}$$

The correlation coefficient, R, for the regression equation was determined to be 0.81. The associated standard error of estimate was 0.065.

Since a regression equation to define R did not give good results, the ratio

$$\frac{R}{Tc+R} = .5 \quad \text{or } R = Tc$$

was adopted. It is noted that this ratio usually remains constant within a given hydrologic region. The final Clark unit hydrograph parameters developed with these equations were utilized in the unit hydrograph development and subsequent flood reproductions within the HEC-1 model and yielded excellent results.

A48. BASE FLOW AND RECESSION. In HEC-1, base flow is described by an exponential decay of flow from the preceding discharge. The functional form is:

$$Q = Qo\ (RTIOR)^{n}$$

Qo = Flow at start of interval t, STRTQ
n = Number of time intervals since recession was initiated
RTIOR = Ratio of recession flow to that one interval later

The total flow at any time is computed by adding the direct runoff (from the unit hydrograph procedure) to the base flow. When this total is below a recession threshold flow (QRCSN), it is not permitted to recede any faster than the original base flow decay rate. At the beginning of the simulation run, the total outlflow from a basin is set equal to STRTQ, an input variable. A QRCSN of 20 percent and a RTIOR of 1.08 was used for the calibration and the hypothetical flood runs.

The flow at the end of the first step is then computed as described by the above equation. The STRTQ utilized for each of the actual flood reproductions varied between 3 and 6 CFS per square mile. The final hypothetical runs used a STRTQ of 5 C.F.S. per square mile. The (STRTO), (QRCSN) and (RTIOR) values used in the model for the major storms analyzed are shown in Table A8.

TABLE A10 MAMARONECK RIVER REGRESSION ANALYSIS-GAGED DATA
Physical Parameters vs Tc

Number	Gaged Basin	PHYSICAL PARAMETER				
		Drainage Area (Sq. Mi.)	Length (Mi.)	Stor	L/ Slope$^{1/2}$	Clark Tc
1	Mamaroneck R. at Mamaroneck, NY	23.4	11.5	3.1	2.7	7.5
2	Bronx R. at Bronxville, NY	26.5	14.6	11.9	3.6	9.4
3	Hutchinson R. at Pelham, NY	5.8	7.5	4.0	1.3	3.5
4	Blind Brook at Rye, NY	9.2	7.7	2.0	1.0	8.2
5	Quaker Creek at Florida, NY	9.7	5.9	6.0	0.9	6.6
6	Whippany R. at Morristown, NJ	29.4	11.8	2.0	1.7	17.7
7	Hohokus Bk. at Hohokus, NJ	16.4	8.5	6.5	1.6	3.5
8	So. Br. Raritan R. nr. High Brg., NJ	65.3	23.0	2.7	4.4	11.6
9	Neshanic R. at Reaville, NJ	25.7	4.5	1.1	0.9	1.0
10	No. Br. Raritan R. nr. Far Hills, NJ	26.2	8.4	1.8	1.3	2.5
11	Lamington R. nr. Pottersville, NJ	32.8	15.8	5.8	3.3	1.6
12	Upper Cold Bk. nr. Pottersville, NJ	2.2	2.1	1.8	0.2	0.5
13	No. Br. Raritan R. nr. Raritan, NJ	190.0	21.7	2.0	5.0	7.8
14	Millstone R. at Plainsboro, NJ	65.8	18.8	4.5	7.4	24.5
15	Stony Bk. at Princeton, NJ	44.5	16.7	1.2	5.2	8.2
16	Stony Bk. at Watchung, NJ	5.5	2.9	1.7	0.3	0.7
17	Lawrence Bk. at Farrington Dam, NJ	34.4	6.9	6.0	2.9	6.2
18	Manalapan Bk. at Spots Wood, NJ	40.7	19.5	7.2	7.2	36.2
19	South R. at Old Bridge, NJ	94.6	16.3	5.1	5.8	25.5
20	Raritan R. at Manville, NJ	490.0	58.2	1.7	16.7	16.5
21	Millstone R. at Blackwells Mills, NJ	258.0	32.3	2.9	16.5	21.6
22	Raritan R. at Bound Bk., NJ	785.0	60.4	2.1	17.7	18.5

Notes: L = Main channel length in miles. Stor = Percent of area occupied by water increased by 1%. Slope = Main channel slope in ft/mi, defined as the average slope of the water course between points 10 and 85 percent of the distance upstream from the runoff site to the watershed boundery. Tc = Time in hours from the end of a burst of rainfall excess to the inflection point on the recession line of the resulting direct runoff hydrograph (Clark Method).

A49. CHANNEL ROUTING. There are various procedures for flood routing available within HEC-1. Selected for use in this study were the Muskingum Method and The Modified Puls Method. The Modified Puls Method which is based on continuity and a unique relationship between storage and outflow. The storage versus outflow data was based on a comprehensive assessment of the full channel and overbank storage in the Mamaroneck and Sheldrake Valleys from the mouth of the Mamaroneck River to above the Mamaroneck Reservoir and on the Sheldrake River from the mouth to the New England Thruway. The final storage versus discharge relations were then developed for the various routing reaches through the use of hydraulic flood profile runs. In the upstream reaches of the Sheldrake and Mamaroneck Rivers the Muskingum Method was used. This method uses two parameters, K= estimated reach travel time in hours and the X= weighting factor, which expresses the relative importance of inflow and storage in determining the outflow. In order to determine the Muskingum parameters the following approach was used. The time of concertation of the Mamaroneck and Sheldrake Rivers basin was found to be 7.48 hours from the unit hydrograph optimizations. The HEC-2 model of the Mamaroneck River showed that the mean travel time from Mamaroneck Reservoir Dam to the mouth to be about 3.50 hours for a range of flows. The difference of these two times, 4.00 hours, was assumed to be the sum of the travel times of the Muskingum routing reaches of the Mamaroneck River from the outlet of the most upstream sub-basin to the upstream end of Mamaroneck Reservoir. The individual travel times were found by assuming them to be proportional to L/ S where L is the reach length and S the square root of the reach slope. This factor of proportionality between travel time and L/ S was applied to the Muskingum routing reaches of the Sheldrake River. The Muskingum weighting factor X is, for each reach, a judgment based on the size and storage of the flood plain of each reach, as estimated from a topographic map. A summary of the flood routing paramenters and the storage versus discharge relations within the hydrologic model is shown in Table A11.

A50. CALIBRATION AND VERIFICATION. The overall strategy of the study was as follows:

1. For the recent flood of record, that of September 1975, the total rainfall over the basin was developed by a Thiessen analysis, shown on Figure A4, to establish the total storm amounts over each subarea. The Larchmont, Scarsdale and White Plains Maple Moor recording gages were used as distributors to establish the pattern of precipitation. In addition, Westchester County Airport was used as a daily station. The total precipitation during this storm over the basin was 4.89 inches, with a constant loss rate of 0.27 inches per hour. The computed peak of 3699 C.F.S. matched the observed peak of 3700 C.F.S. very well. The distribution of precipitation and the computed and observed hydrographs are shown on Figure A4.

2. For the second highest flood of record, that of June 1972, the total rainfall over the basin was developed by a Thiessen analysis, shown on Figure A3, to establish the total amounts over each subarea. The Larchmont, Scarsdale and White Plains Maple Moor recording gages were used as distributors to establish the pattern of precipitation. In addition, White Plains Airport was used as a daily station. The total precipitation during this storm over the basin was 4.65 inches, with a total runoff of 3.15

TABLE A11

ROUTING PARAMETERS — MAMAROONECK AND SHELDRAKE RIVERS (a) — MAMARONECK RIVER BASIN WESTCHESTER COUNTY, NY

Muskingum Routing

Routing Reach	Parameter	Coefficient
E Spring Lake Br. to Hutchinson River Pkwy.	NSTPS	3
	AMSKK	1.66
	X	0.2
F Along Parkway to Service Area	NSTPS	3
	AMSKK	1.65
	X	0.2
H Service Area to West Br. Mamaroneck	NSTPS	2
	AMSKK	0.70
	X	0.2
BB Sheldrake Reservoir to East Br.	NSTPS	1
	AMSKK	0.54
	X	0.2

Modified Puls Routing

Routing Reach	Storage AC-FT	Outflow CFS
K Mamaroneck Detention Reservoir	0	0
	41	308
	114	588
	217	819
	266	1,093
	313	2,028
	334	2,557
	350	2,624
	371	3,563
	404	4,508
	501	7,005
	567	8,818
L Mamaroneck Detention Reservoir to Thruway	0	0
	12	845
	18	1,126
	39	2,083
	52	2,626
	53	2,680
	80	3,657
	107	4,628
	173	7,172
	222	9,017
M Mamaroneck R. Thruway to Sheldrake River	0	0
	43	871
	58	1,159
	145	2,095
	214	2,597
	231	2,636
	441	3,433
	640	4,223
	1,011	6,688
	1,239	8,329

Modified Puls Routing

Routing Reach	Storage AC-FT	Outflow CFS
AA Main Sheldrake Reservoir	0	0
	21	190
	43	537
	65	987
	87	4,028
	109	9,218
	132	15,824
	154	23,583
	177	32,339
	200	41,990
DD Sheldrake R. East Br. to Fenimore Road	0	0
	23	571
	38	986
	82	1,612
	158	1,952
	282	2,349
	455	2,975
	563	3,455
FF Sheldrake R. Fenimore Road to Mouth	0	0
	15	522
	23	678
	92	1,057
	516	1,544
	615	1,765
	765	2,347
	901	2,883
O Mamaroneck R. Sheldrake R. to Mouth	0	0
	36	1,248
	40	1,765
	53	3,099
	59	3,574
	60	3,720
	79	4,938
	101	5,995
	164	9,065
	201	11,250

(a) Reach name corresponds to that of adjacent sub-area (see schematic, figure A9). Two letter names denote Sheldrake sub-area.

inches. The initial loss rate was 0.5 inches with a constant loss rate of 0.09 inches per hour. The computed peak of 3539 C.F.S. matched the observed peak of 3550 C.F.S. very well. The distribution of precipitation and the computed and observed hydrographs are shown on Figure A3.

3. For the November 1977 flood, the total rainfall over the basin was developed by a Thiessen analysis, shown on Figure A5, to establish the total amounts over each subarea. The Pleasantville and Scardale recording gages were used as distributors to establish the pattern of precipitation. In addition, Westchester County Airport was used as a daily station. The total precipitation during this storm over the basin was 4.99 inches, with a total runoff of 2.97 inches. The initial loss rate was 1.50 inches with a constant loss rate of 0.10 inches per hour. The computed peak of 3141 C.F.S. matched the observed peak of 3241 C.F.S. very well. The distribution of precipitation and the computed and observed hydrographs are shown on Figure A5.

4. For the April 1980 flood, the total rainfall over the basin was developed by a Thiessen analysis, shown on Figure A6, to establish the total amounts over each subarea. The Pleasantville and Scarsdale recording gages were used as distributors to establish the pattern of precipitation. In addition, Westchester County Airport was used as a daily station. The total precipitation during this storm over the basin was 3.64 inches, with a total runoff of 1.87 inches. The initial loss rate was 0.50 inches with a constant loss rate of 0.12 inches per hour. The computed peak of 2762 C.F.S. matched the observed peak of 2790 C.F.S. very well. The distribution of precipitation and the computed and observed hydrographs are shown on Figure A6.

5. For the April 1983 flood, the total rainfall over the basin was developed by a Thiessen analysis, shown on Figure A7, to establish the total amounts over each subarea. The Scarsdale recording gage was used as a distributor to establish the pattern of precipitation. In addition, Westchester County Airport was used as a daily station. The total precipitation during this storm over the basin was 2.74 inches, with a total runoff of 1.31 inches. The initial loss rate was 0.60 inches with a constant loss rate of 0.17 inches per hour. The computed peak of 1633 C.F.S. agreed fairly well with the observed peak of 1810 C.F.S. The distribution of precipitation and the computed and observed hydrographs are shown on Figure A7.

The volume and timing comparisons are shown on Figures A3 to A7 for the above calibration and verification floods. It is emphasized that the model parameters chosen for the simulation of these five storms are the same, except for the initial subarea flows. It is noted that one of the major problems in model calibration is the lack of ample data to define the rainfall distribution which actually occurs over each sub-basin within the study area. The only recording gage in close proximity to a particular sub-basin may not be indicative of the rainfall distribution which occurred in that area for a particular storm event.

A51. HYPOTHETICAL FLOODS. In order to develop the hypothetical flood discharges throughout the basin, the hypothetical storm rainfalls, summarized in Table A4, were applied to the calibrated model. The loss rates used in these runs were influenced by the loss rates used in the calibration runs which varied between 0.09 and 0.27 inches per hour. The constant loss rate varied from 0.37 inches per hour for the 10-year flood to 0.03 inches per hour for the 500-year flood. These loss rates were used to calibrate the peak discharges into the peak discharge versus frequency curve for the Mamaroneck River gage. The final discharges are shown on Table A11. The flood hydrographs and the associated hyetographs for the Sheldrake River at its mouth, Mamaroneck River above the Sheldrake and the Mamaroneck River below the Sheldrake River for the 10,100 and Standard Project Floods are shown on Figures A17 to A19, respectively. The peak discharge versus frequency curves for the Mamaroneck, above the Sheldrake River confluence, at the mouth of the Mamaroneck, and for the Sheldrake River at its mouth are shown on Figure A20. The confidence limits were also developed for the Mamaroneck upstream of the Sheldrake and for the Sheldrake at its mouth existing condition peak discharge versus frequency curves in order to provide a measure of the uncertainty in the discharge for a selected exceedence probability. The confidence limits for 5% and 95% levels of significance are shown on the derived frequency curves on Figure A20a and A20b. In addition the peak discharge versus drainage area plots are shown for the 2, 10, 100 and Standard Project Floods for the Mamaroneck and Sheldrake Rivers on Figures A21 and A22 respectively.

A52. STANDARD PROJECT FLOOD. The Standard Project Flood (SPF) is intended as a practicable expression of the degree of protection that should be sought, whenever possible, in the design of flood control works. Since these estimates are based on generalized studies of meteorological and hydrologic conditions in the region, the SPF estimate provides a basis for comparing the selection of the design flood and giving a comparable degree of protection for similar classes of property. The Standard Project Flood was synthesized from the Standard Project Rainfall and the hydrologic model previously discussed. This method of analysis showed the Standard Project Flood peak to be 9579 C.F.S. at the Mamaroneck gage. The associated flood peaks for the Mamaroneck above the Sheldrake and for the Sheldrake at its mouth were 7191 C.F.S. and 2442 C.F.S. respectively. The Standard Project Flood peak for the Sheldrake River at Fenimore Road, the proposed site of the tunnel diversion was 3092 C.F.S., under existing conditions. The Standard Project Flood hydrographs are shown on Figure A19.

IV. DESIGN CONDITIONS

A53. GENERAL. The proposed improvements for the Mamaroneck River Basin consist of the following: (1) improvement of the Mamaroneck River Channel between Winfield Avenue and the New York Thruway; (2) improvement of the Mamaroneck River channel between the Thruway and Tompkins Avenue; (3) improvement of the Sheldrake River channel between Rockland Ave. and Fenimore Road and construction of a diversion tunnel from the downstream end of the improvement (Fenimore Road) to the west basin of Mamaroneck Harbor; and (4) modification of the Sheldrake River channel downstream of Fenimore Road to accommodate flow entering or originating below Fenimore Road.

The minimum reasonable degree of protection that should be considered would be against the largest flood of record (September 1975 flood). However, such a degree of protection is a low percentage of the Standard Project Flood (32 percent) and has only a 5 percent exceedence frequency at the Halstead Avenue gaging station. The Standard Project Flood represents the objective towards which the design of flood protection works is normally directed, but topographic and economic limitations did not permit the complete attainment of this objective in all reaches. The selected design frequency for the considered improvements is 0.5 percent with one exception. The Sheldrake River improvements (including the diversion tunnel), will accommodate the Standard Project Flood.

TABLE A12

MAMARONECK & SHELDRAKE RIVERS BASIN
HEC-1 MODEL - EXISTING CONDITIONS

Flood (Actual)	Infiltration Loss Rate-in./hr	Sheldrake D.S. of E. Br. D.A. = 5.20mi^2	Sheldrake at Fenimore D.A. = 5.57mi^2	Sheldrake at Mouth D.A. = 6.16mi^2	Mamaroneck Res. Dam D.A. = 15.35mi^2	Mamaroneck U.S. Sheldrake D.A. = 17.30mi^2	Mamaroneck D.S. Sheldrake (gaging station) D.A. = 23. 4 sq mi computed	observed	Mamaroneck at Mouth D.A.=23.63 sq mi
June 1972	0.09	908	962	958	2557	2594	3540	3550	3547
Sept 1975	0.27	1102	1154	1087	2624	2630	3702	3700	3711
Nov 1977	0.10	954	1011	946	2124	2275	3141	3241	3154
April 1980	0.12	1038	1087	1039	1765	1791	2764	2790	2778
April 1983	0.17	686	726	733	1036	1100	1635	1810	1642
(Hypothetical)									
1 Yr.	0.29	474	507	550	726	784	1223	1225	1229
2 Yr.	0.24	699	739	742	1152	1205	1691	1680	1705
10 Yr.	0.37	1119	1159	1077	2076	2116	3055	3000	3074
50 Yr.	0.11	2015	1860	1362	3963	3674	5031	4950	5045
100 Yr.	0.09	2524	2117	1518	4813	4491	6006	6000	6034
500 Yr.	0.03	3616	2851	2257	7010	6862	9067	9050	9092
SPF	0.15	3920	3092	2442	7371	7191	9579	-	9606

A54. HURRICANE TIDE LEVELS. Because of the lack of specific tidal information in Mamaroneck Harbor, a review was made of the tidal records at Willets Point, New York and Stamford, Connecticut. In addition, the results of a tidal surge model developed for the coastline of New York City by Camp Dresser and McKee in November 1981 were analyzed in the vicinity of Mamaroneck Harbor. This model was calibrated to all available tidal information for the New York City coastline. The elevations for the 10, 50, 100, 200, 500 and 1000 year hypothetical tides taken from the tidal surge model report, based on node 105 in the vicinity of Mamaroneck Harbor, are shown on Figure A 25. The partial duration portion of the frequency curve was based on a linear interpolation between Willets Point, New York and Stamford, Connecticut, of the 1 year tide based on tidal flood profiles along Long Island Sound, shown on Figure A 24. The partial duration curve at Willets Point was based on a Weibull plotting position analysis of 435 events above a stage of 4.7 ft. m.s.l., for the years 1931 to 1968, and is shown on Figure A 23. The 1 year tide at Willets Point was determined to be 7.0 ft. m.s.l. Based on a correlation analysis between Willets Point and Stamford, the 1 year tide at Stamford was determined to be 6.5 ft. m.s.l. Based on interpolation of the 1 year tide profile between Willets Point and Stamford, the 1 year tide at Mamaroneck Harbor was estimated to be 6.7. ft. m.s.l. The final tidal stage vs. frequency analysis for Mamaroneck Harbor is shown on Figure A 25.

V. DOWNSTREAM IMPACTS

A55. GENERAL. The plans of improvement were investigated for downstream impacts, and as a result, it was determined that by eliminating the natural overbank flood storage through a project reach, a reduction in normal attenuation would take place, and flood peaks would increase downstream of the project. It is noted that a loss in storage does not imply a proportional increase in discharge, because the storage changes are reflected all along the rising limbs of the affected hydrographs, not just around their peaks. In addition, the channel improvements increase the speed of the flood wave. The location of the project is shown on Figure A37. The basic improvement plan consists of channel work on the Mamaroneck River from its mouth to the Westchester County Joint Water Works Dam and a diversion tunnel on the Sheldrake River. In order to analyze the discharges associated with these improvements, the hydrologic model was rerun using the storage versus discharge relations from the improved condition hydraulic runs. A summary of the storage versus discharge relations under existing and improved conditions is shown in Table A12. The flood discharges associated with the impacts of the construction is shown on Table A13.

As shown on Figures A26 to A29, the 2400 foot Sheldrake channel improvement plan from the Rockland Avenue to Fenimore Road will increase and speed up the flood wave. For the Standard Project Flood, shown on Figure A29, the discharge will increase from 3090 c.f.s. under existing conditions to 4040 c.f.s. under improved conditions on the Sheldrake River at the Sheldrake Diversion Tunnel Inlet. Since the tunnel is designed for the Standard Project Flood, the peak at the mouth of the Sheldrake River was reduced to 370 c.f.s., which is the residual peak generated by the sub area below the tunnel. When this peak is combined with the contribution of the Mamaroneck River upstream of the Sheldrake River, the Standard Project Flood flow below the Sheldrake confluence will be reduced from 9580 c.f.s. under existing conditions to 7930 c.f.s. under improved conditions. For the 200 year flood, shown on Figure A28, the discharge will increase from 2400 c.f.s. under existing conditions to 2960 c.f.s. under improved conditions on the Sheldrake River at the tunnel inlet. Since this peak is diverted through the tunnel the residual peak generated by the sub area below the tunnel, at the mouth of the Sheldrake River, is 290 c.f.s. When this peak is combined with the contribution of the Mamaroneck River upstream of the Sheldrake River, the 200 year flood flow below the Sheldrake confluence will be reduced from 7240 c.f.s. under existing conditions to 6200 c.f.s. under improved conditions.

TABLE A13

MAMARONECK AND SHELDRAKE RIVERS - ROUTING PARAMETERS (STORAGE vs DISCHARGE RELATIONS) - EXISTING and IMPROVED CONDITIONS

EXISTING CONDITIONS

Notes	Routing Reach	Parameter												
a,c	ROUT-E/Spring Lk Br to Hutchinson R Pkwy		NSTPS= 3;	AMSKK=1.66;	X=0.2									
a,c	ROUT-F/Along Pkwy to Service Area		NSTPS= 3;	AMSKK=1.65;	X=0.2									
a,c	ROUT-H/Service Area to West Br Mamaroneck		NSTPS= 2;	AMSKK=0.70;	X=0.2									
b,c	RESOUT/Mamaroneck Detention Reservoir	STORAGE	0	41	114	217	266	313	334	350	371	404	501	567
		OUTFLOW	0	308	583	819	1093	2028	2557	2624	3563	4508	7005	8818
b,c	ROUT-L/Mamaroneck Detention Reservoir to Thruway	STORAGE	0	12	18	39	52	53	80	107	173	222		
		OUTFLOW	0	845	1126	2083	2626	2680	3657	4628	7172	9017		
b,c	ROUT-M/Thruway to Sheldrake River	STORAGE	0	43	58	145	214	231	441	640	1017	1239		
		OUTFLOW	0	871	1159	2095	2597	2636	3433	4223	6688	8329		
b,d	RESFLO/Main Sheldrake Reservoir	STORAGE	0	21	43	65	87	109	132	154	177	200		
		OUTFLOW	0	190	537	937	4028	9218	15824	23583	32339	41990		
a,d	ROUTBB/Sheldrake Reservoir To East Branch		NSTPS=1;	AMSKK= 0.54;	X=0.2									
b,d	RTUSDD/East Branch to Thruway	STORAGE	0	7	9	17	23	40	72	93				
		OUTFLOW	0	571	986	1612	1952	2349	2975	3455				

TABLE A13

EXISTING CONDITIONS (continued)

Notes	Routing Reach	Parameter										
b,d	RTDSDD/Thruway to Fenimore Road	STORAGE	0	16	29	65	135	242	383	470		
		OUTFLOW	0	571	986	1612	1952	2349	2975	3455		
b,d	ROUTFF/Fenimore Road To Mouth	STORAGE	0	15	23	92	516	615	765	901		
		OUTFLOW	0	522	678	1057	1544	1765	2347	2888		
b,c	ROUT-O/Sheldrake River To Mouth	STORAGE	0	36	40	53	59	60	79	101	164	201
		OUTFLOW	0	1248	1765	3099	3574	3720	4938	5995	9065	11250
						IMPROVED CONDITIONS (with Tunnel)						
	ROUT-E		Same as Plan #1 (Existing Conditions)									
	ROUT-F		Same as Plan #1 (Existing Conditions)									
	ROUT-H		Same as Plan #1 (Existing Conditions)									
	RESOUT	STORAGE	0	90	124	304	376	403	425	513		
		OUTFLOW	0	790	1150	2080	3960	4810	5700	7380		
	ROUT-L	STORAGE	0	8	9	13	32	39	45	84		
		OUTFLOW	0	800	1190	2140	4070	4930	5860	7550		
	ROUT-M	STORAGE	0	23	32	47	78	96	128	185	351	
		OUTFLOW	0	773	1220	2201	4183	5079	5990	7394	9361	
	RESFLO		Same as PLAN #1 (Existing Conditions)									
	ROUTB3		Same as PLAN #1 (Existing Conditions)									
	RTUSDD		Same as Plan #1 (Existing Conditions)									
	RTDSDD	STORAGE	0	8	10	14	20	25	30	40	56	
		OUTFLOW	0	511	748	1204	2133	2669	3140	3867	4815	
	ROUTFF		Same as Plan #1 (Existing Conditions)									
	ROUT-O	STORAGE	0	35	37	43	58	65	73	86	109	
		OUTFLOW	0	806	1267	2282	4320	5239	6230	7617	9670	

TABLE A14
MAMARONECK & SHELDRAKE RIVERS
IMPROVED CONDITIONS - HYDROLOGIC MODEL
DISCHARGES IN C.F.S.

LOCATION	S.P.F. Exist	S.P.F. Improv.	200 YEAR Exist	200 YEAR Improv.	100 YEAR Exist	100 YEAR Improv.	50 YEAR Exist	50 YEAR Improv.	10 YEAR Exist	10 YEAR Improv.	2 YEAR Exist	2 YEAR Improv.
Mamaroneck at Mouth	9610	7970	7270	6220	6030	5250	5040	4330	3070	2880	1700	1250
Mamaroneck D.S. Sheldrake	9580	7930	7240	6220	6010	5215	5030	4300	3055	2260	1690	1245
Mamaroneck U.S. Sheldrake	7190	7820	5400	6150	4490	5151	3670	4250	2120	2240	1200	1230
Mamaroneck U.S. Jefferson Ave.	7100	7720	5350	6010	4430	5070	3620	4180	2080	2200	1180	1210
Mamaroneck U.S. New England Thruway	7550	7550	5860	5860	4930	4930	4070	4070	2140	2140	1190	1190
Mamaroneck at Mamaroneck Res	7370	7370	5700	5700	4810	4810	3960	3960	2080	2080	1150	1150
Sheldrake at Mouth	2440	370	1760	290	1520	260	1360	220	1080	140	740	90
Sheldrake below Tunnel Fenimore Rd.	2410	180	1730	140	1495	130	1340	110	1060	70	730	40
Sheldrake above Tunnel Fenimore Rd.	3090	4040	2400	2960	2120	2550	1860	2110	1160	1210	730	750

NOTES: 1. The improved plan consists of a 40 foot channel on the Sheldrake River, a tunnel, a 60 foot channel to Jefferson Avenue and a 50 foot channel to Winfield Avenue on the Mamaroneck River.

2. The improved condition runs were made with the same constant infiltration loss rates as the existing condition runs and are as follows:

S.P.F. = 0.03 in/hr	50 yr = 0.11 in/hr
200 yr = 0.04 in/hr	10 yr = 0.37 in/hr
100 yr = 0.09 in/hr	2 yr = 0.24 in/hr

It is noted that the flood discharges associated with the improved condition runs were used to develop the peak discharge versus frequency curves for the Mamaroneck River upstream and downstream of the Sheldrake River and for the Sheldrake River above the diversion tunnel and at its mouth, shown on Figures A30, A31, A32, and A33, respectively. The improved condition curve for the Mamaroneck River upstream of the Sheldrake River, shown on Figure A30, reinforces the fact that there is a minimal change in discharge for the improvement plan at this location. The improved condition curve for the Mamaroneck River downstream of the Sheldrake River, shown on Figure A31, demonstrates the reduction in flow as a result of the diversion tunnel. The improved condition curve for the Sheldrake River at the diversion tunnel shown on Figure A32, demonstrates the increase in flow caused by the channel plans. The improved condition curve for the Sheldrake River at its mouth, shown on Figure A33, demonstrates the significant reduction in flow caused by the diversion tunnel. The confidence limits for the improved condition frequency curves for the Mamaroneck River upstream and downstream of the Sheldrake River and for the Sheldrake River above the proposed diversion are shown on Figures A34, A35 and A36 respectively.

VI. SENSITIVITY RUNS

A56. GENERAL. Since the flood hydrographs and the gaged frequency curve with a period of record of 40 years were reproduced using reasonable unit hydrographs and loss rate parameters, the number of sensitivity runs were kept to a minimum.

VII. FUTURE DEVELOPMENT

A57. GENERAL. Any future development would tend to increase flood flows immediately below the development works because of the decrease in the natural infiltration capacity of the soil cover. In order to analyze the future development trends, the 1977 population and land use report written by the Westchester County Department of Planning was used as a basis for analyzing the urbanization trends in the Mamaroneck and Sheldrake River Basins. This report presented a summary of the methodology and results of the study of existing land use and population projections. Local zoning ordinances, zoning maps and community master plans, as well as the Westchester Generalized Zoning Map were also assembled in the analysis of future development potential. In addition, general discussions were held with Westchester Planning Board Members. Based on the review, it was concluded that there would be no population change in the forecast period. Similarly, land use changes in this suburban basin will be a minimum. Therefore, we do not anticipate significant increases in the design discharges because of future development.

VIII. FLOOD WARNING SYSTEM

A58. GENERAL. A flood warning system for the considered Mamaroneck improvement would be oriented toward reducing catastrophic losses and social disruptions that could occur with flood events exceeding the design capacity of the improvement. The principal components of the flood warning plan would be:

A. early recognition of flood threat

B. dissemination of flood warnings

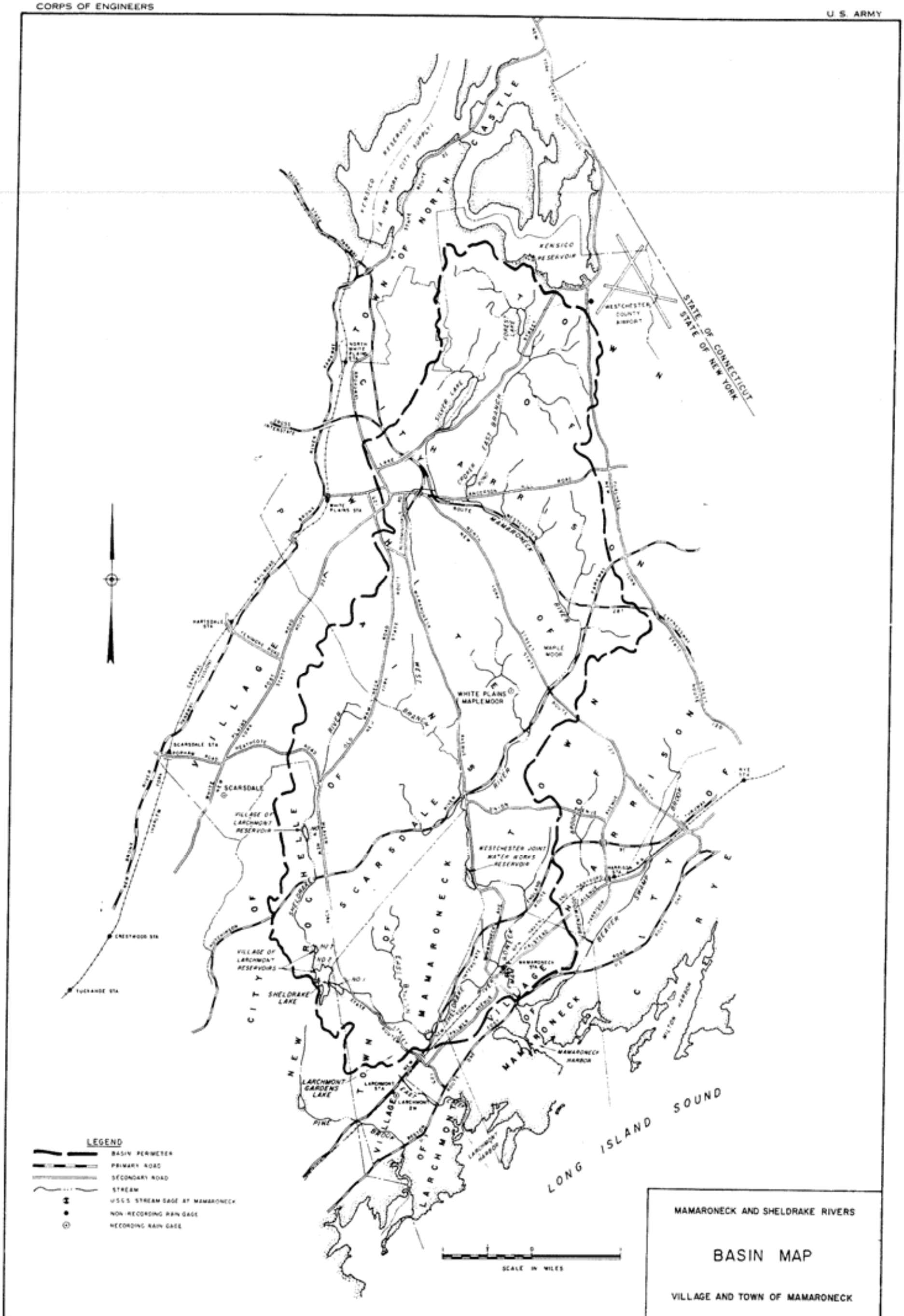
C. emergency response actions

D. recovery and reoccupation of the flood areas

E. continuous management of the flood preparedness plan

A flood warning system for the considered Mamaroneck improvement would require monitoring the Mamaroneck River Basin. The National Weather Service Northeast River Forecast Center in Hartford, Connecticut in association with the Westchester County Department of Public Works has developed an "alert" flood warning system, Automated Local Evaluation Real Time for the Mamaroneck River Basin within the Westchester County area. As part of this system, they have communication links to the Scarsdale, Harrison and White Plains precipitation gages, and the Mamaroneck River recording gage. This precipitation and stream gage data is automatically transmitted to the Westchester County Department of Public Works, where they make decisions regarding the severity of the flood threat.

CORPS OF ENGINEERS U.S. ARMY

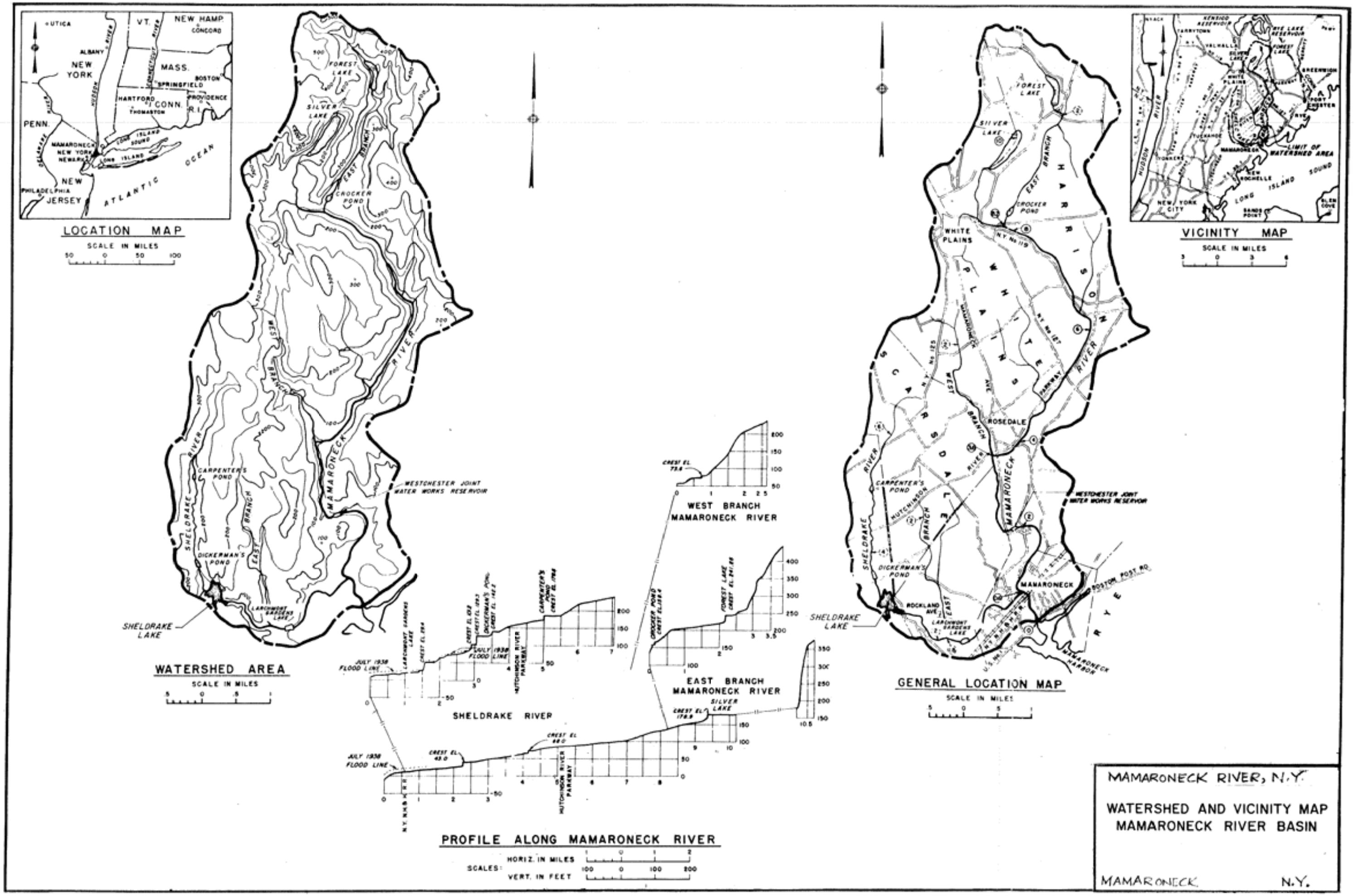


MAMARONECK AND SHELDRAKE RIVERS

BASIN MAP

VILLAGE AND TOWN OF MAMARONECK

FIGURE A1



LOCATION MAP
SCALE IN MILES
WATERSHED AREA
SCALE IN MILES
PROFILE ALONG MAMARONECK RIVER
HORIZ. IN MILES
SCALES:
VERT. IN FEET
SHELDRAKE RIVER
WEST BRANCH MAMARONECK RIVER
EAST BRANCH MAMARONECK RIVER
GENERAL LOCATION MAP
SCALE IN MILES
VICINITY MAP
SCALE IN MILES
WESTCHESTER JOINT WATER WORKS RESERVOIR
SHELDRAKE LAKE
MAMARONECK RIVER, N.Y.
WATERSHED AND VICINITY MAP
MAMARONECK RIVER BASIN
MAMARONECK N.Y.

FIGURE A 2

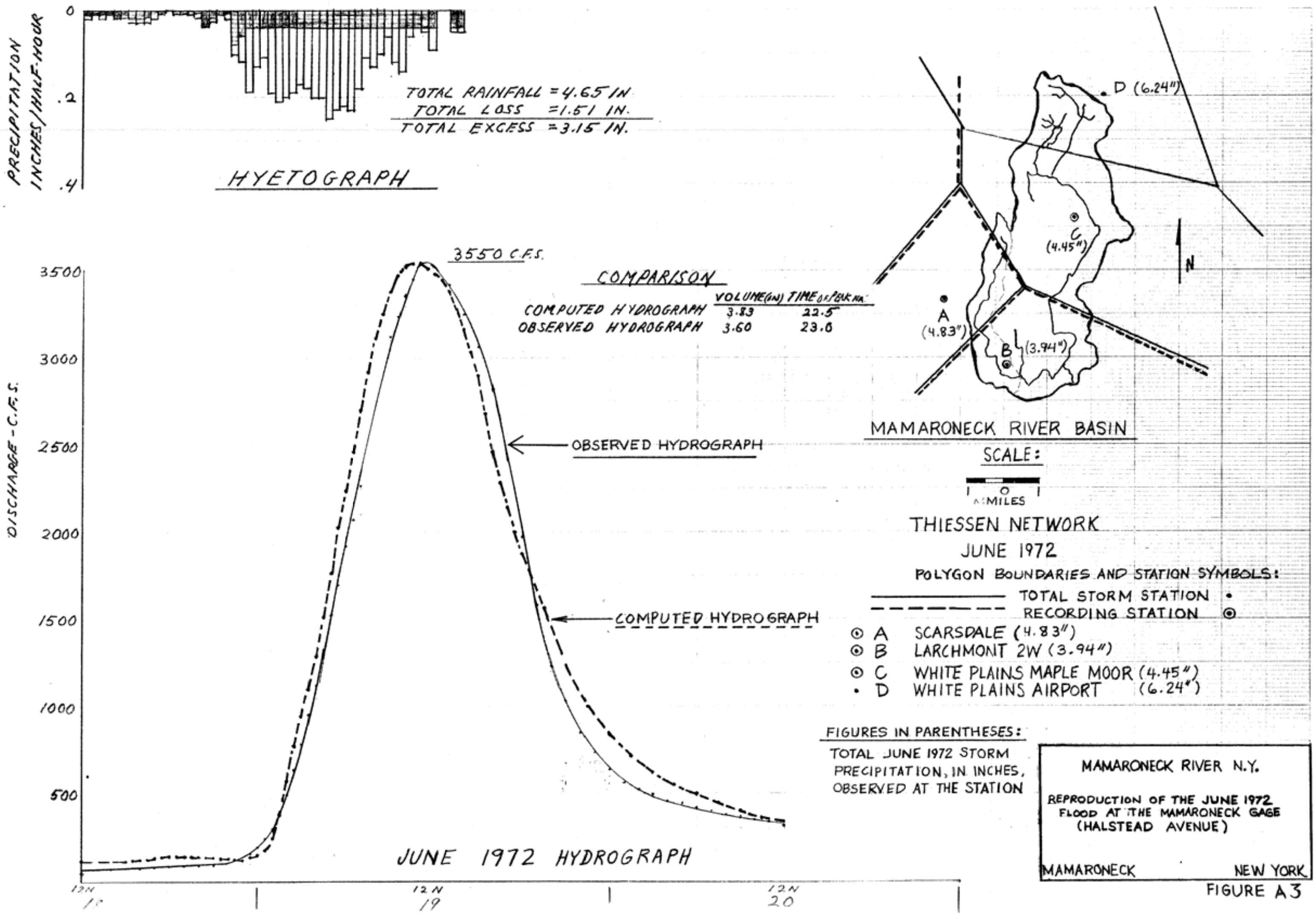
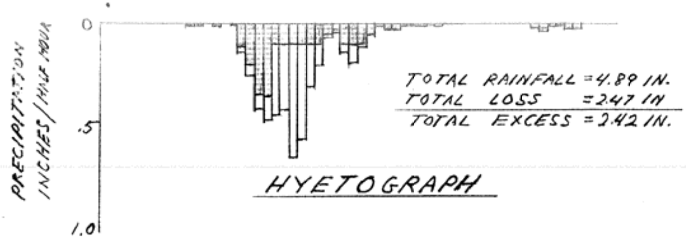
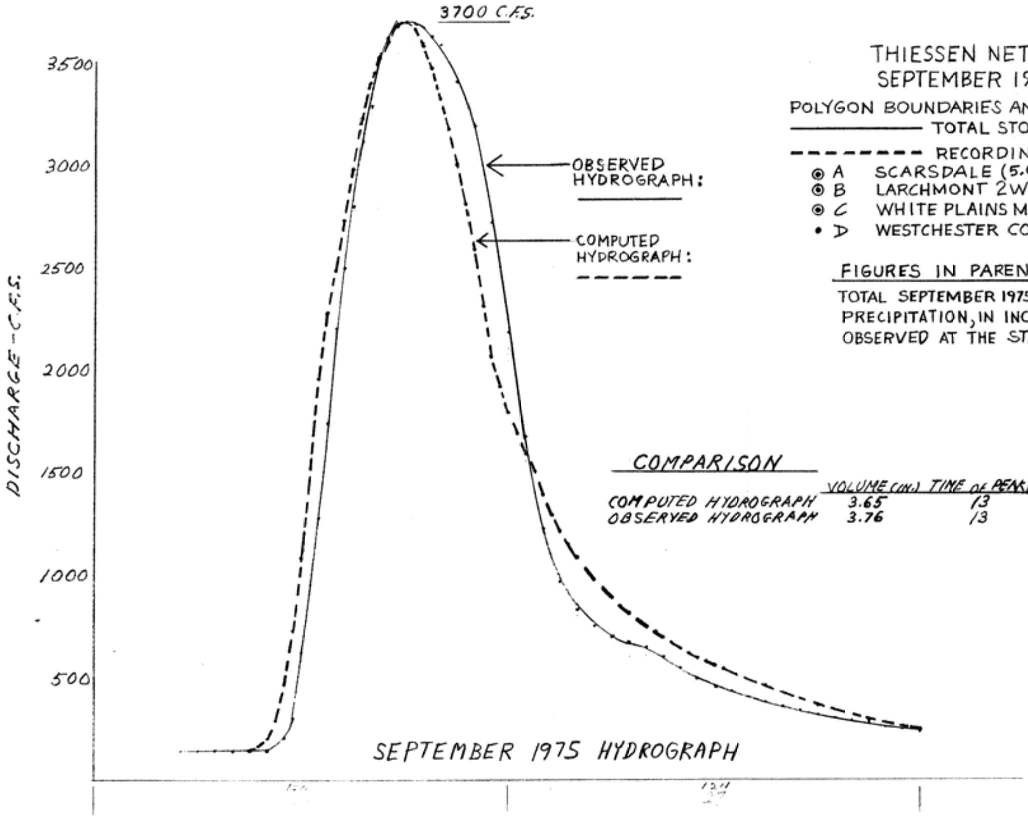

PRECIPITATION INCHES/HALF-HOUR
0
.2
.4
TOTAL RAINFALL = 4.65 IN.
TOTAL LOSS = 1.51 IN.
TOTAL EXCESS = 3.15 IN.
HYETOGRAPH
3550 C.F.S.
DISCHARGE - C.F.S.
3500
3000
2500
2000
1500
1000
500
COMPARISON
VOLUME(IN) TIME OF PEAK HR.
COMPUTED HYDROGRAPH 3.83 22.5
OBSERVED HYDROGRAPH 3.60 23.0
OBSERVED HYDROGRAPH
COMPUTED HYDROGRAPH
JUNE 1972 HYDROGRAPH
12N 18
12N 19
12N 20
D (6.24")
C (4.45")
A (4.83")
B (3.94")
N
MAMARONECK RIVER BASIN
SCALE:
MILES
THIESSEN NETWORK
JUNE 1972
POLYGON BOUNDARIES AND STATION SYMBOLS:
TOTAL STORM STATION
RECORDING STATION
A SCARSDALE (4.83")
B LARCHMONT 2W (3.94")
C WHITE PLAINS MAPLE MOOR (4.45")
D WHITE PLAINS AIRPORT (6.24")
FIGURES IN PARENTHESES:
TOTAL JUNE 1972 STORM PRECIPITATION, IN INCHES, OBSERVED AT THE STATION
MAMARONECK RIVER N.Y.
REPRODUCTION OF THE JUNE 1972 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)
MAMARONECK NEW YORK

FIGURE A3





THIESSEN NETWORK
SEPTEMBER 1975

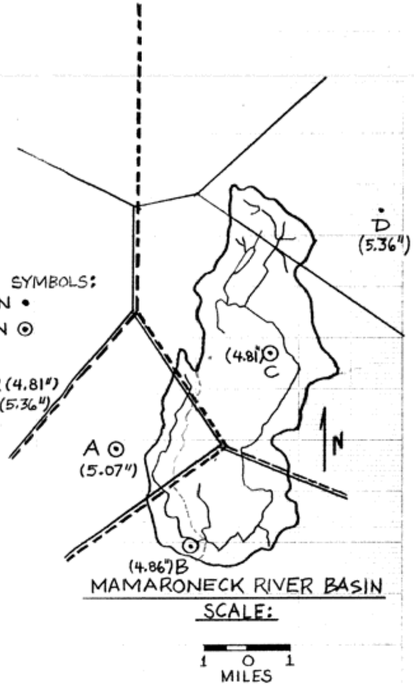
POLYGON BOUNDARIES AND STATION SYMBOLS:

——— TOTAL STORM STATION •

- - - - RECORDING STATION ⊙

⊙ A SCARSDALE (5.07")
⊙ B LARCHMONT 2W (4.86")
⊙ C WHITE PLAINS MAPLE MOOR (4.81")
• D WESTCHESTER CO. AIRPORT (5.36")

FIGURES IN PARENTHESES:
TOTAL SEPTEMBER 1975 STORM PRECIPITATION, IN INCHES, OBSERVED AT THE STATION

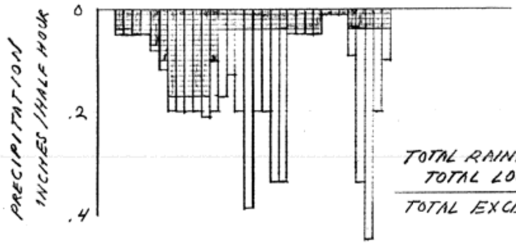


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE SEPTEMBER 1975 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

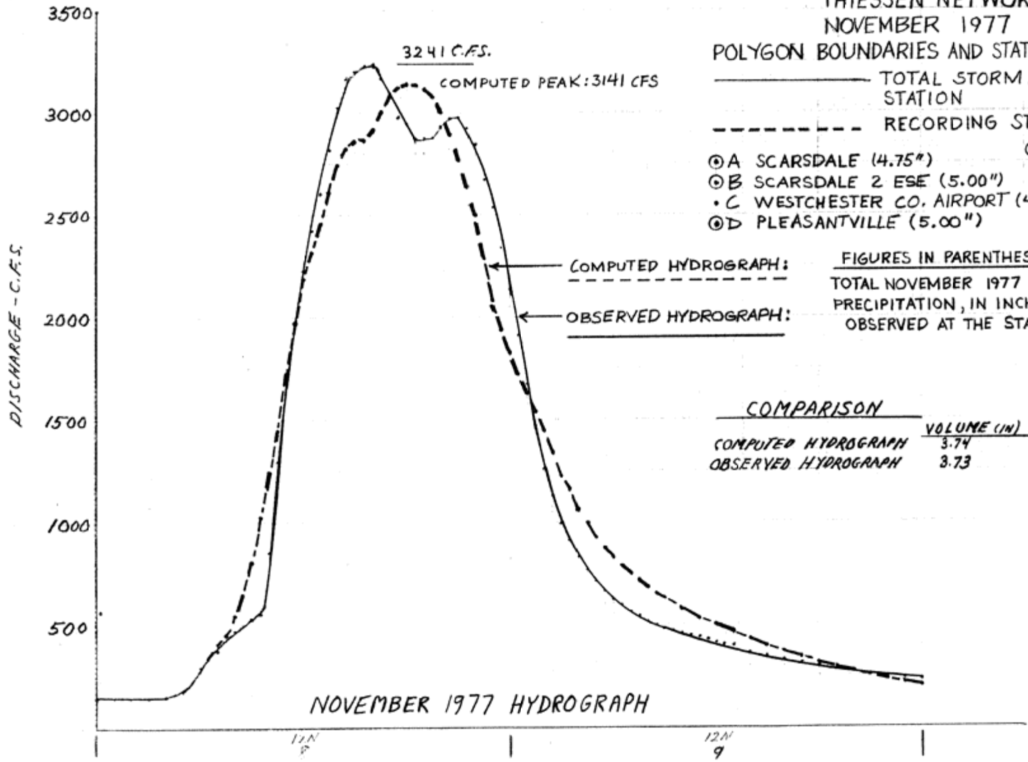
MAMARONECK NEW YORK

FIGURE A4



TOTAL RAINFALL = 4.99 IN
TOTAL LOSS = 2.02 IN.
TOTAL EXCESS = 2.97 IN.

HYETOGRAPH



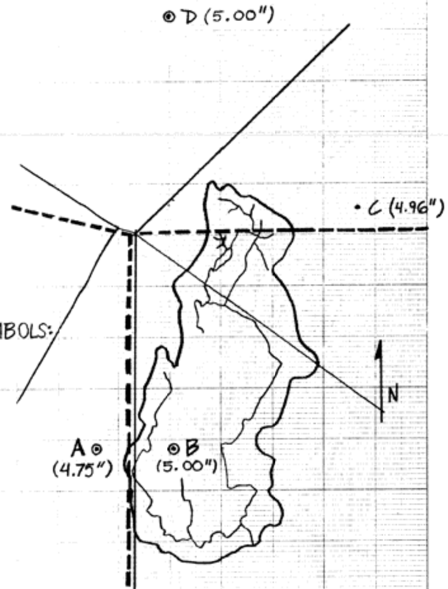
NOVEMBER 1977 HYDROGRAPH

THIESSEN NETWORK
NOVEMBER 1977
POLYGON BOUNDARIES AND STATION SYMBOLS:

TOTAL STORM STATION •
RECORDING STATION ⊙

⊙A SCARSDALE (4.75")
⊙B SCARSDALE 2 ESE (5.00")
• C WESTCHESTER CO. AIRPORT (4.96")
⊙D PLEASANTVILLE (5.00")

FIGURES IN PARENTHESES:
TOTAL NOVEMBER 1977 STORM PRECIPITATION, IN INCHES, OBSERVED AT THE STATION



MAMARONECK RIVER BASIN
SCALE:
1 0 1
MILES

COMPARISON

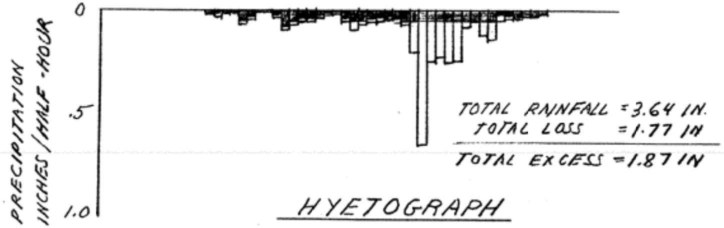
	VOLUME (IN)	TIME OF PEAK (HR)
COMPUTED HYDROGRAPH	3.74	18.5
OBSERVED HYDROGRAPH	3.73	16.0

MAMARONECK RIVER N.Y.

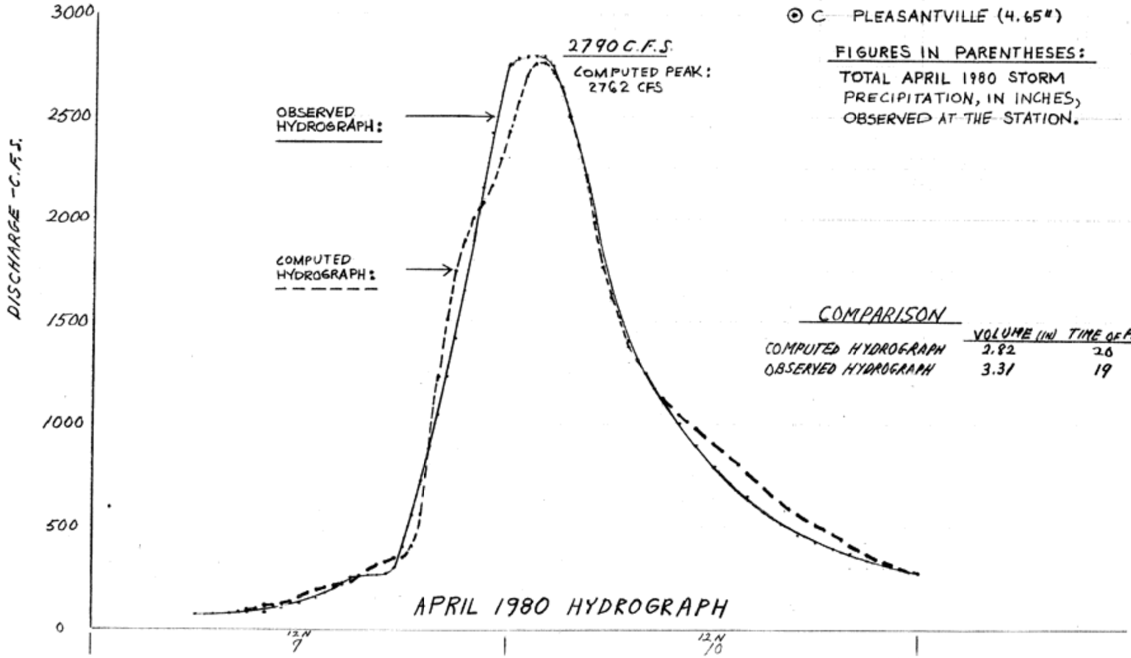
REPRODUCTION OF THE NOVEMBER 1977 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A5



HYETOGRAPH

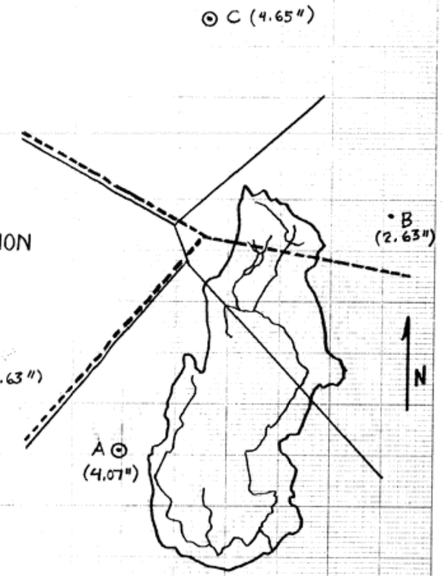


APRIL 1980 HYDROGRAPH

THIESSEN NETWORK
APRIL 1980
POLYGON BOUNDARIES AND STATION SYMBOLS:

——— TOTAL STORM STATION •
- - - - RECORDING STATION ⊙
⊙ A SCARSDALE (4.07")
• B WESTCHESTER CO. AIRPORT (2.63")
⊙ C PLEASANTVILLE (4.65")

FIGURES IN PARENTHESES:
TOTAL APRIL 1980 STORM PRECIPITATION, IN INCHES, OBSERVED AT THE STATION.



MAMARONECK RIVER BASIN
SCALE:
0
MILES

COMPARISON

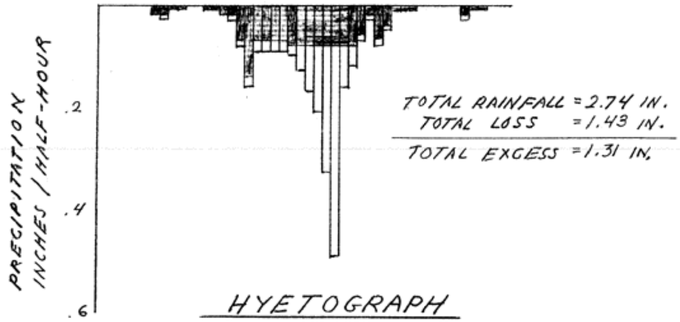
	VOLUME (IN)	TIME OF PEAK (HR.)
COMPUTED HYDROGRAPH	2.82	20
OBSERVED HYDROGRAPH	3.31	19

MAMARONECK RIVER N.Y.

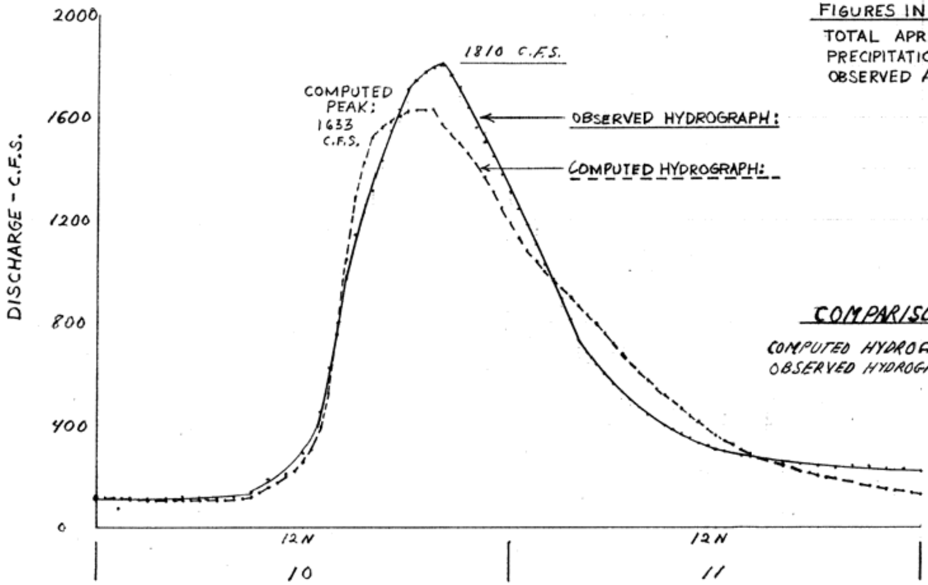
REPRODUCTION OF THE APRIL 1980 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A6



HYETOGRAPH



APRIL 1983 HYDROGRAPH

THIESSEN NETWORK
APRIL 1983

POLYGON BOUNDARIES AND STATION SYMBOLS:

——— TOTAL STORM STATION •

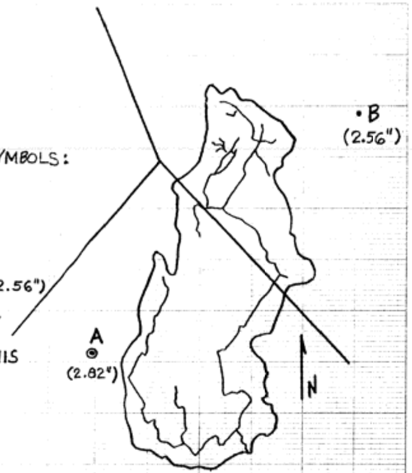
– – – – RECORDING STATION ⊙

⊙ A SCARSDALE (2.82")

• B WESTCHESTER CO. AIRPORT (2.56")

NO RECORDING STATION POLYGON BOUNDARIES APPEAR BECAUSE SCARSDALE (A) IS THE ONLY RECORDING STATION FOR THIS EVENT.

FIGURES IN PARENTHESES:
TOTAL APRIL 1983 STORM PRECIPITATION, IN INCHES, OBSERVED AT THE STATION



MAMARONECK RIVER BASIN

SCALE:

COMPARISON

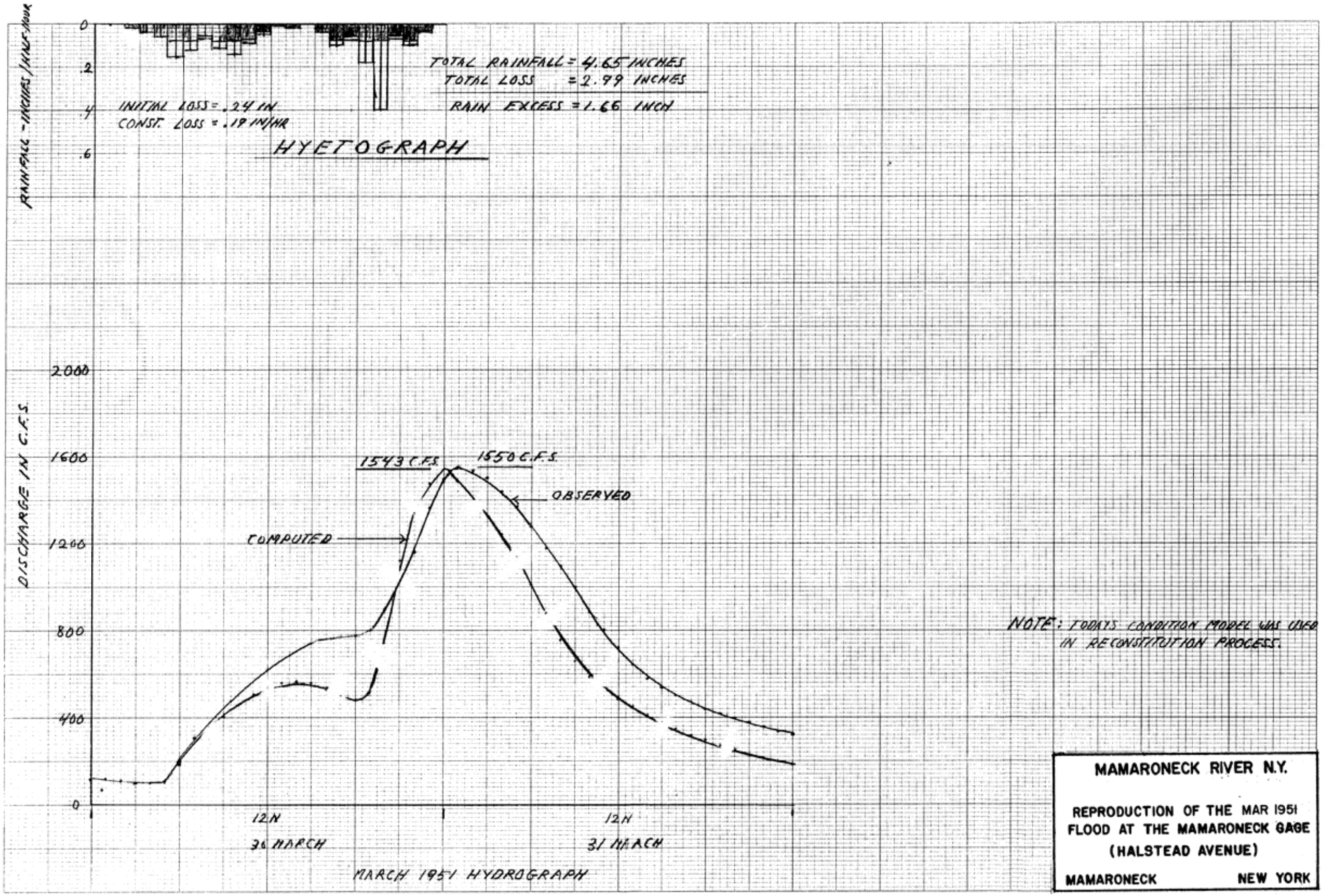
	VOLUME (IN)	TIME OF PEAK (HRS.)
COMPUTED HYDROGRAPH	1.88	19
OBSERVED HYDROGRAPH	1.92	20

MAMARONECK RIVER N.Y.

REPRODUCTION OF THE APRIL 1983 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A7

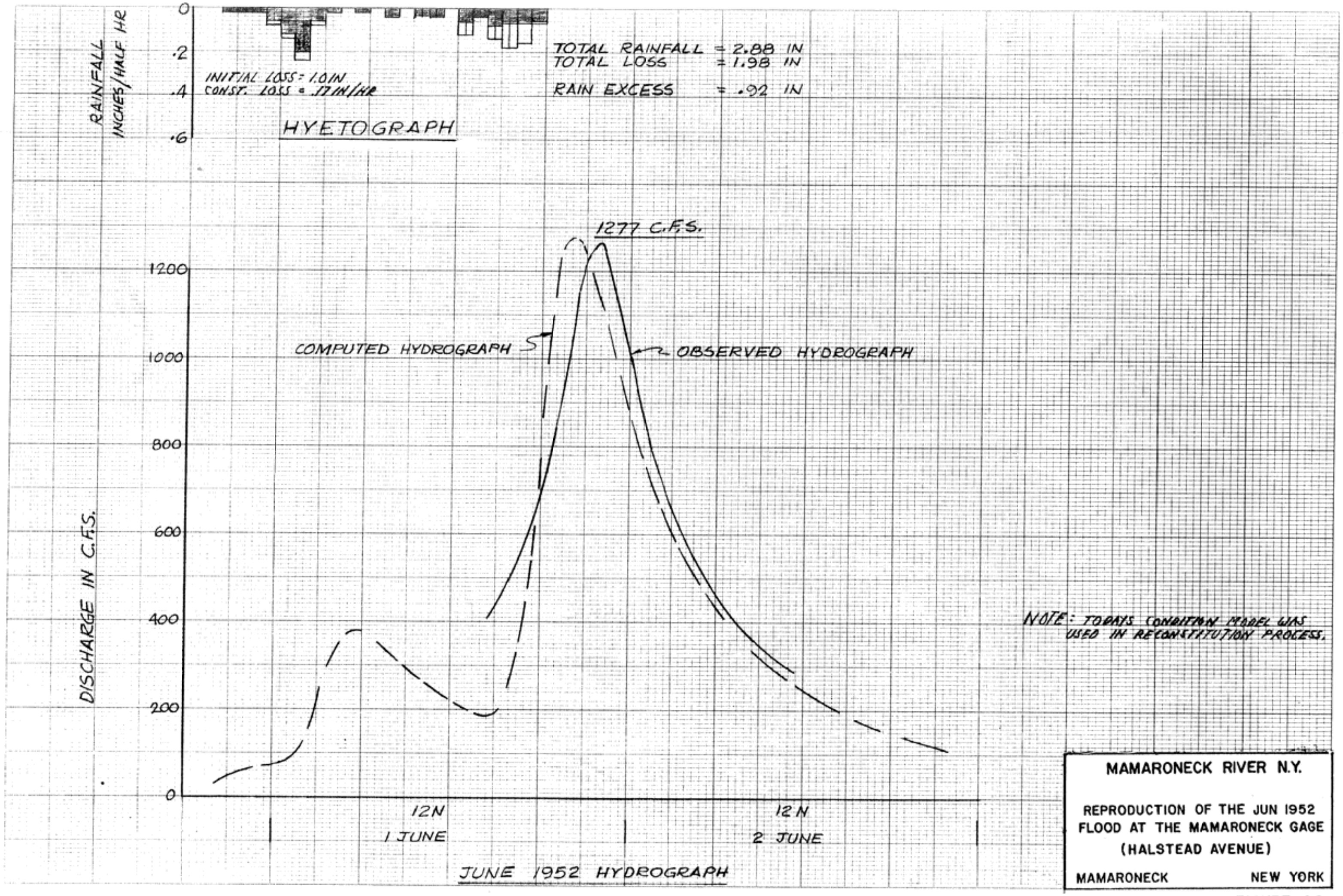


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE MAR 1951 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A8

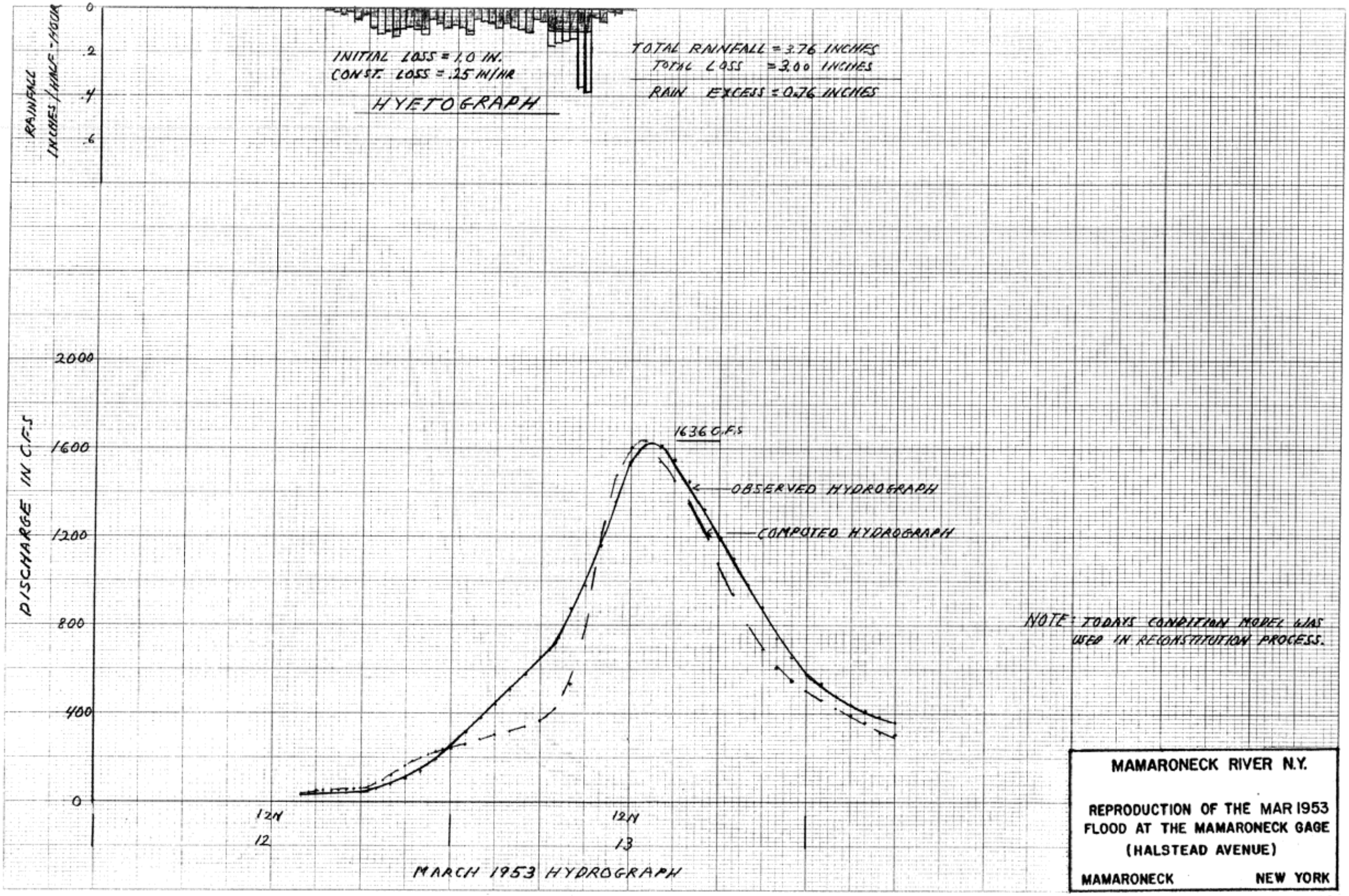


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE JUN 1952 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A9

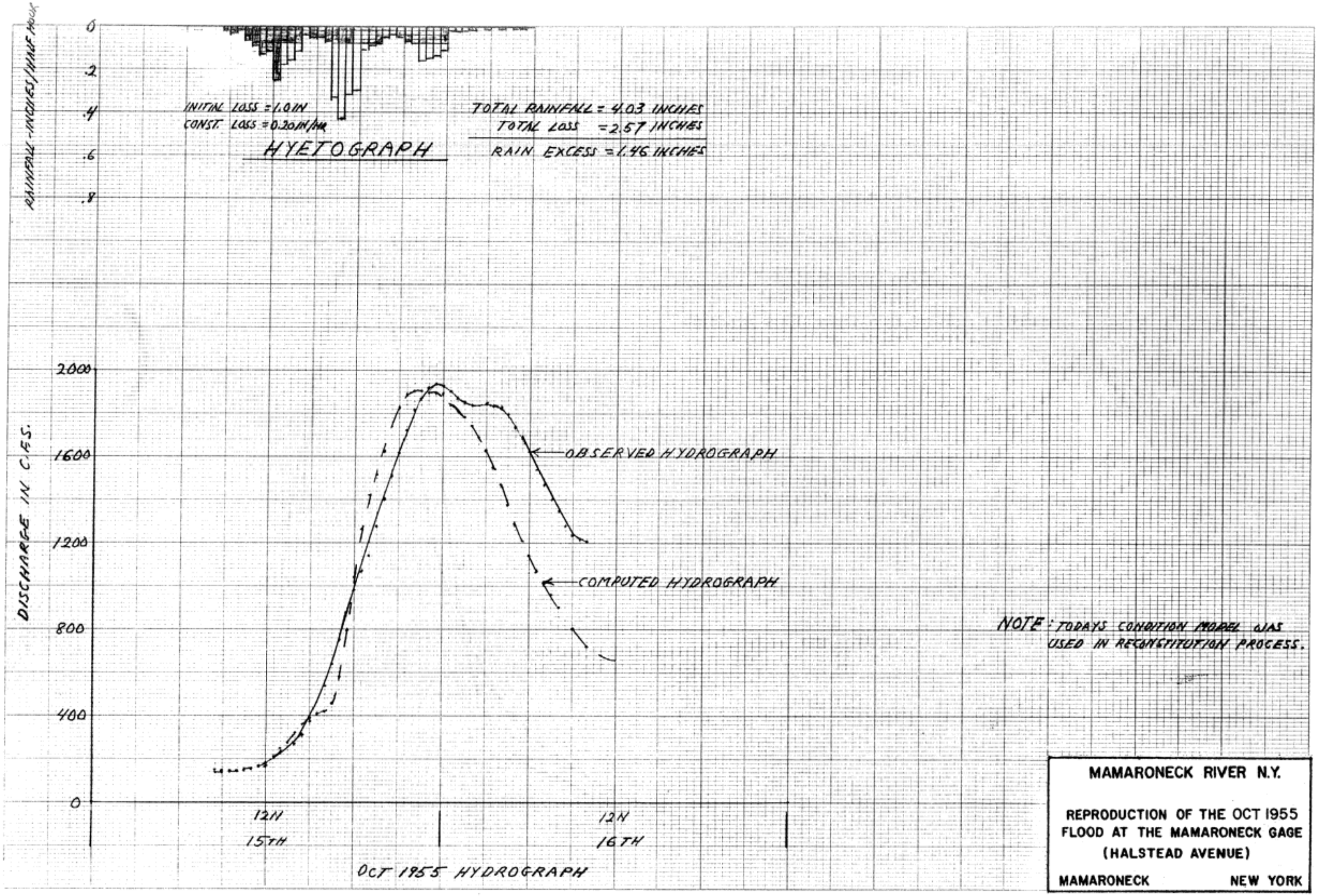


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE MAR 1953 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A10

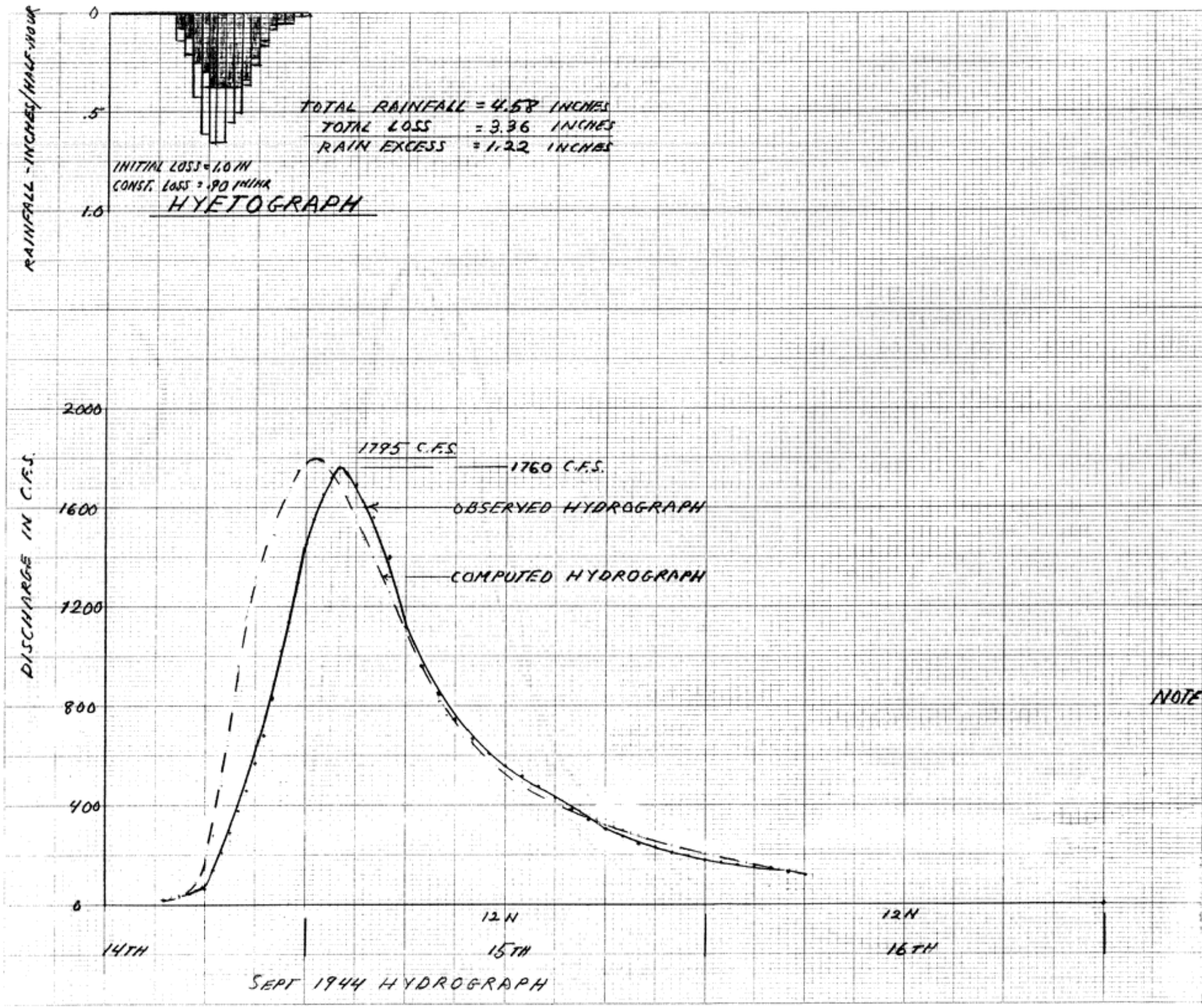


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE OCT 1955 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A11

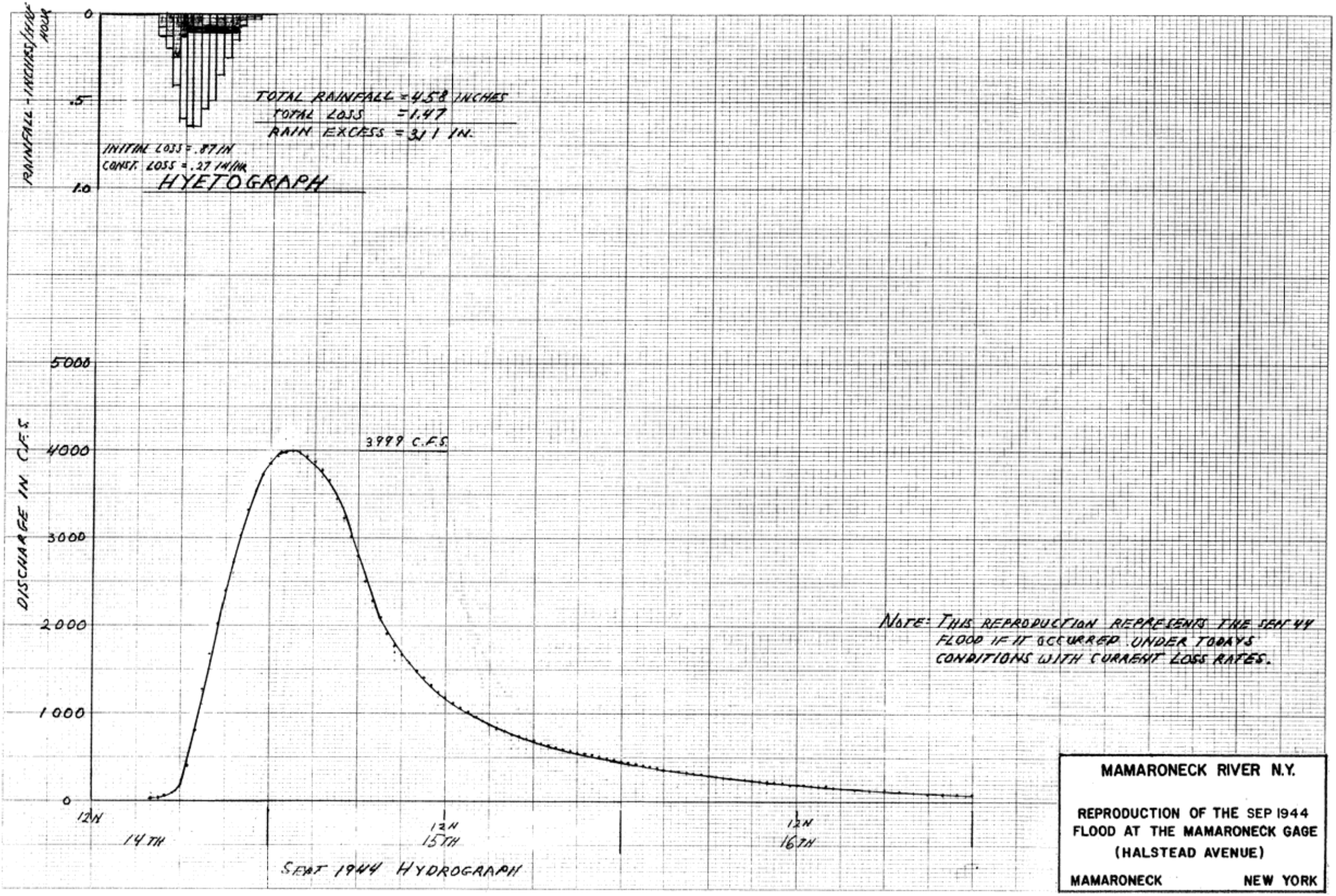


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE SEP 1944
FLOOD AT THE MAMARONECK GAGE
(HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A12

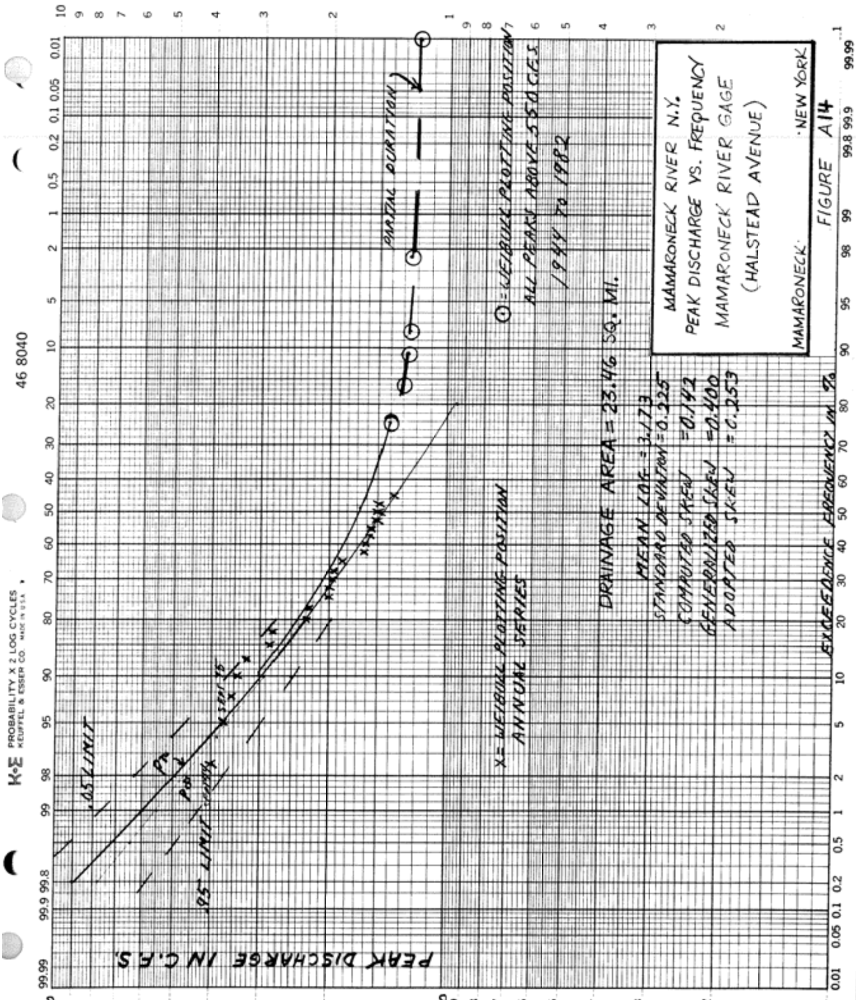


MAMARONECK RIVER N.Y.

REPRODUCTION OF THE SEP 1944 FLOOD AT THE MAMARONECK GAGE (HALSTEAD AVENUE)

MAMARONECK NEW YORK

FIGURE A13



K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
MAMARONECK RIVER N.Y.
PEAK DISCHARGE VS. FREQUENCY
MAMARONECK RIVER GAGE
(HALSTEAD AVENUE)
MAMARONECK NEW YORK
FIGURE A14
PEAK DISCHARGE IN C.F.S.
EXCEEDENCE FREQUENCY IN %
PARTIAL DURATION
⊙ = WEIBULL PLOTTING POSITION
ALL PEAKS ABOVE 550 CFS
1944 TO 1982
X = WEIBULL PLOTTING POSITION
ANNUAL SERIES
DRAINAGE AREA = 23.46 SQ. MI.
MEAN LOG = 3.173
STANDARD DEVIATION = 0.225
COMPUTED SKEW = 0.142
GENERALIZED SKEW = 0.400
ADOPTED SKEW = 0.253
.05 LIMIT
.95 LIMIT
10,000
1000

FIGURE A 14

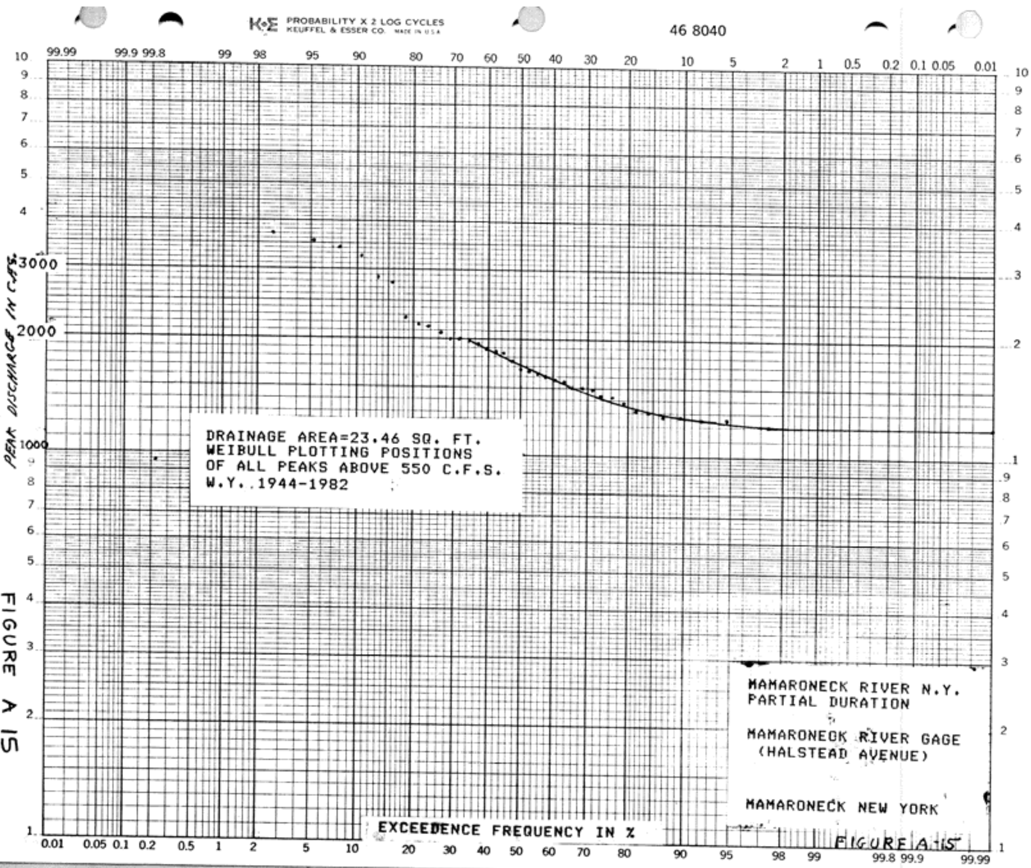
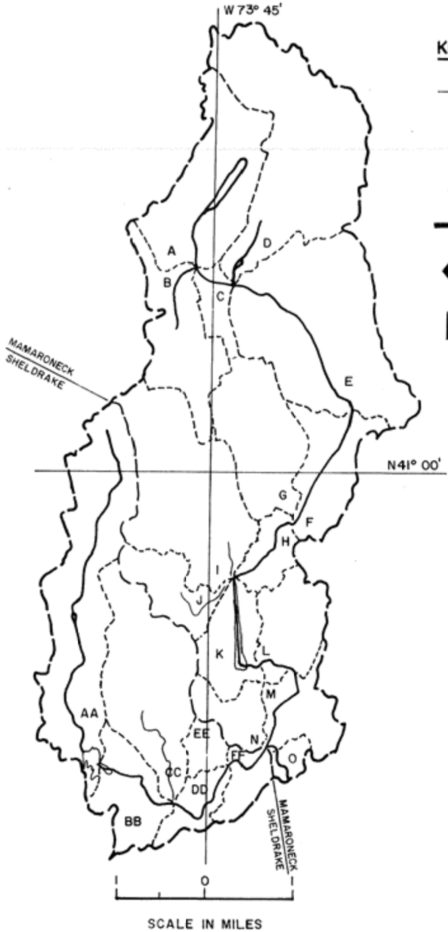
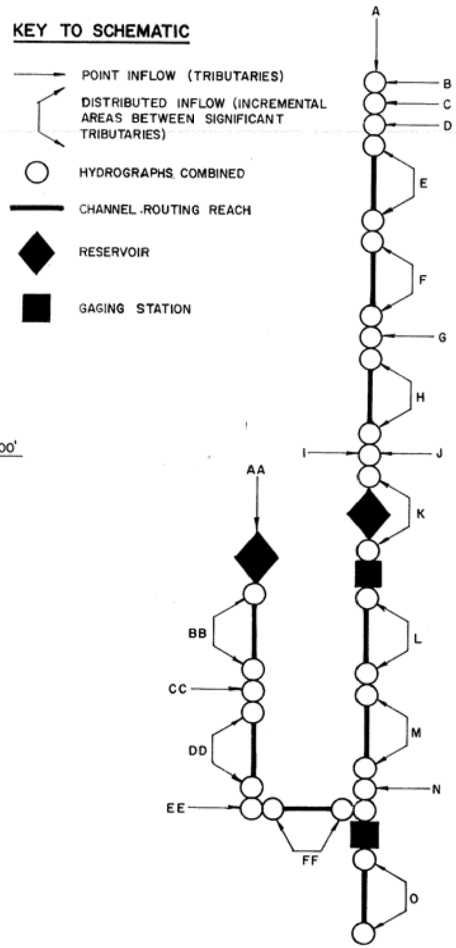

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
PEAK DISCHARGE IN C.F.S.
3000
2000
1000
DRAINAGE AREA=23.46 SQ. FT.
WEIBULL PLOTTING POSITIONS
OF ALL PEAKS ABOVE 550 C.F.S.
W.Y. 1944-1982
MAMARONECK RIVER N.Y.
PARTIAL DURATION
MAMARONECK RIVER GAGE
(HALSTEAD AVENUE)
MAMARONECK NEW YORK
EXCEEDENCE FREQUENCY IN %
FIGURE A-15

FIGURE A 15



SUBAREA MAP



SCHEMATIC

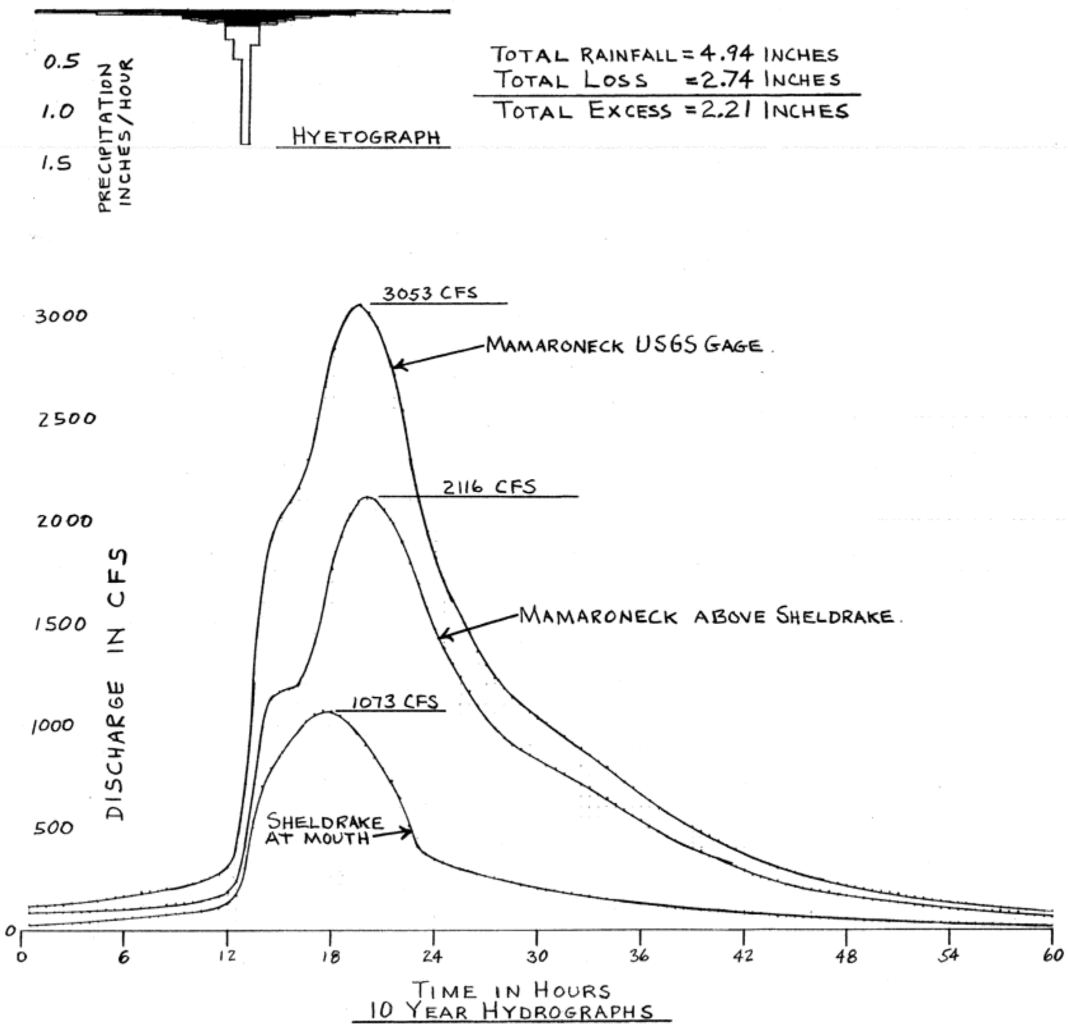
SUBAREA	DRAINAGE AREA (SQ. MILES)	CLARK'S "TC"	CLARK'S "R"	REMARKS
A	1.74	3.14	3.14	
B	1.18	2.02	2.02	
C	0.32	0.68	0.68	
D	2.95	5.58	5.58	
E	2.94	2.44	2.44	
F	0.92	1.52	1.52	
G	1.32	2.16	2.16	
H	0.43	1.10	1.10	
I	2.24	2.52	2.52	WEST BRANCH MAMARONECK
J	0.63	2.70	2.70	
K	0.68	3.20	3.20	
L	0.83	1.08	1.08	
M	0.68	2.26	2.26	
N	0.44	1.14	1.14	
AA	2.63	3.18	3.18	FIRST SHELDRAKE SUBAREA
BB	0.73	1.62	1.62	
CC	1.84	2.82	2.82	EAST BRANCH SHELDRAKE
DD	0.37	1.54	1.54	
EE	0.24	0.56	0.56	
FF	0.35	1.74	1.74	LAST SHELDRAKE SUBAREA
O	0.17	0.70	0.70	LAST MAMARONECK SUBAREA

MAMARONECK RIVER N.Y.

BASIN MAP, NODAL NETWORK AND UNIT GRAPH PARAMETERS

MAMARONECK NEW YORK

FIGURE A16

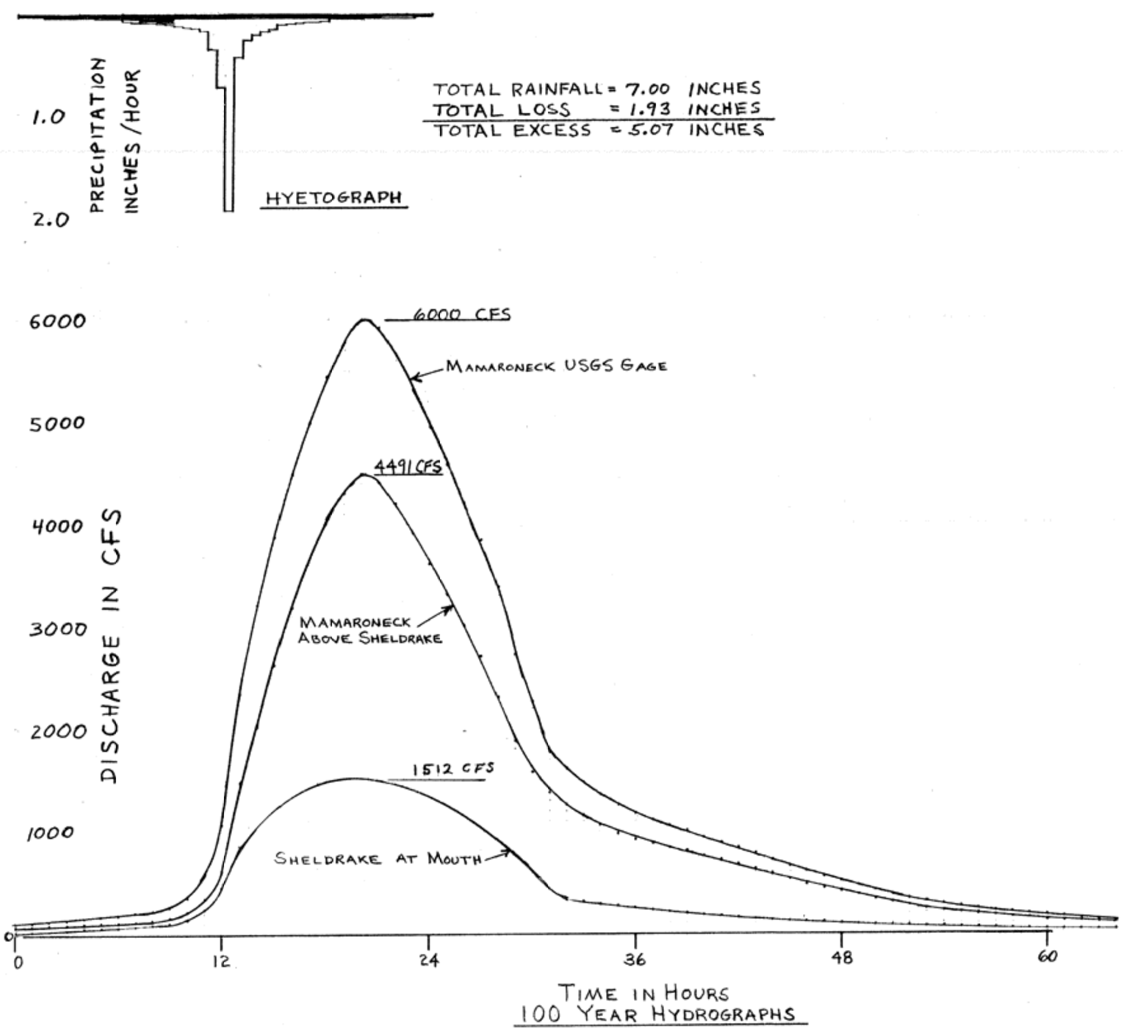


MAMARONECK RIVER N.Y.

10-YEAR FLOOD HYDROGRAPHS

MAMARONECK NEW YORK

FIGURE A17

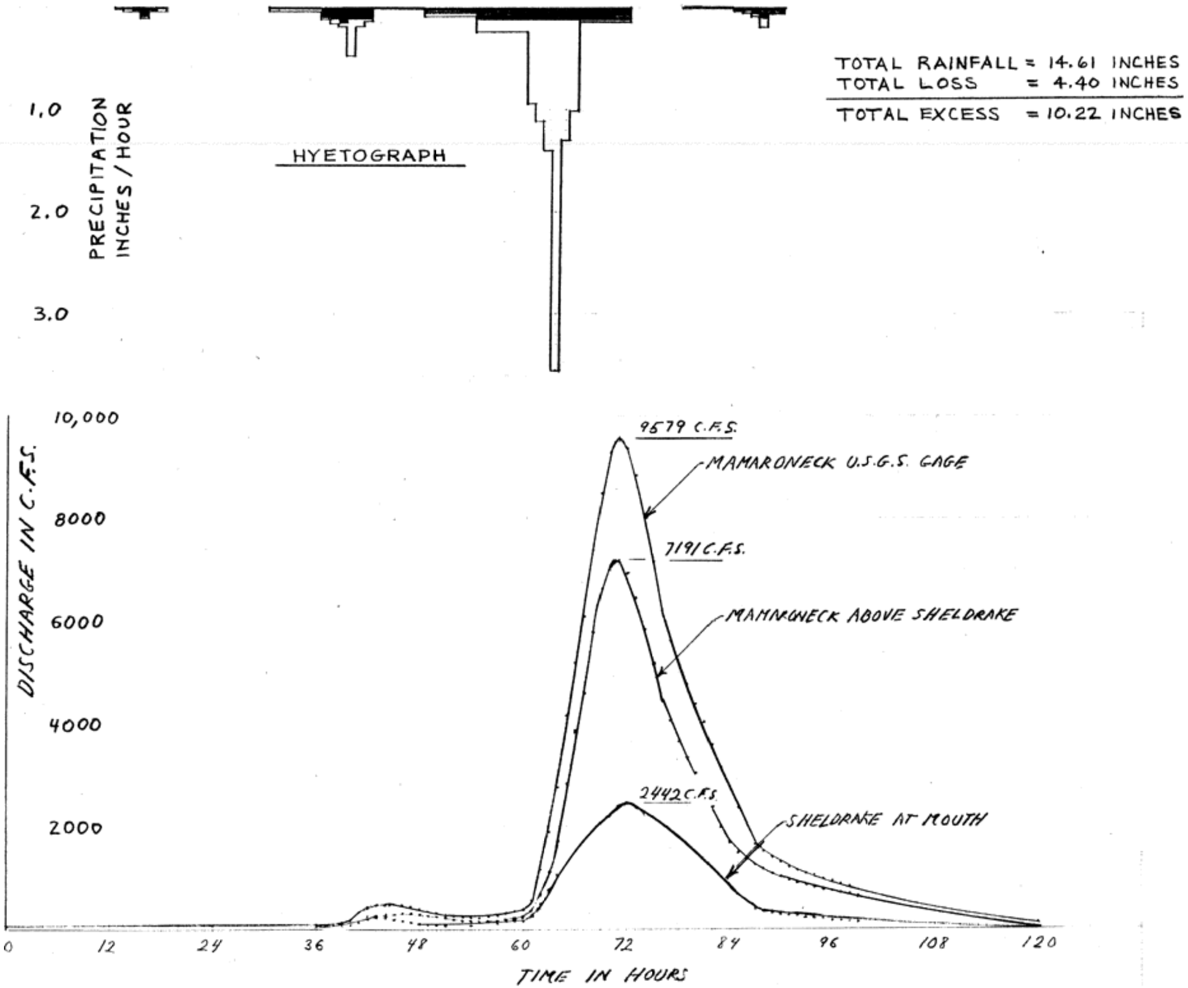


MAMARONECK RIVER N.Y.

100-YEAR FLOOD HYDROGRAPHS

MAMARONECK NEW YORK

FIGURE A18



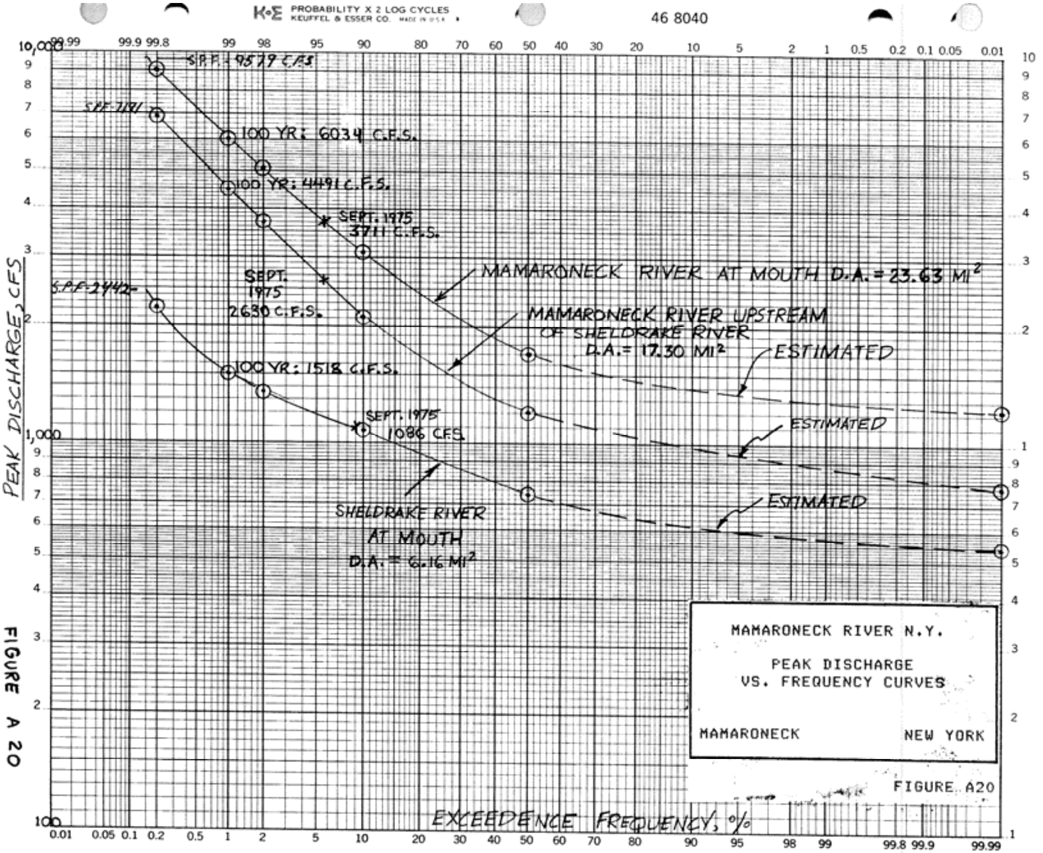
STANDARD PROJECT FLOOD HYDROGRAPHS

MAMARONECK RIVER N.Y.

STANDARD PROJECT FLOOD HYDROGRAPHS

MAMARONECK NEW YORK

FIGURE A19



K•E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
S.P.F. - 9579 C.F.S.
S.P.F.-7191
S.P.F.-2442-
100 YR: 6034 C.F.S.
100 YR: 4491 C.F.S.
100 YR: 1518 C.F.S.
SEPT. 1975 3711 C.F.S.
SEPT. 1975 2630 C.F.S.
SEPT. 1975 1086 C.F.S.
MAMARONECK RIVER AT MOUTH D.A. = 23.63 MI²
MAMARONECK RIVER UPSTREAM OF SHELDRAKE RIVER D.A. = 17.30 MI²
SHELDRAKE RIVER AT MOUTH D.A. = 6.16 MI²
ESTIMATED
ESTIMATED
ESTIMATED
PEAK DISCHARGE, CFS
EXCEEDENCE FREQUENCY, %
MAMARONECK RIVER N.Y.
PEAK DISCHARGE
VS. FREQUENCY CURVES
MAMARONECK
NEW YORK
FIGURE A20

FIGURE A 20

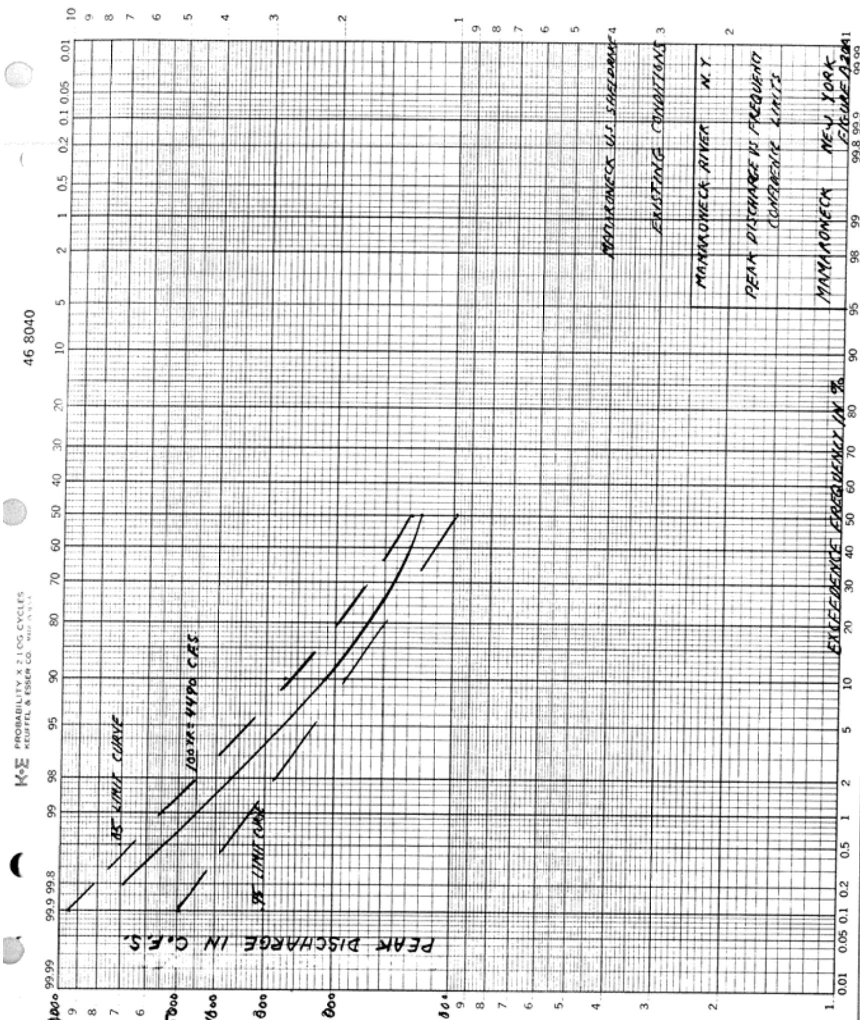
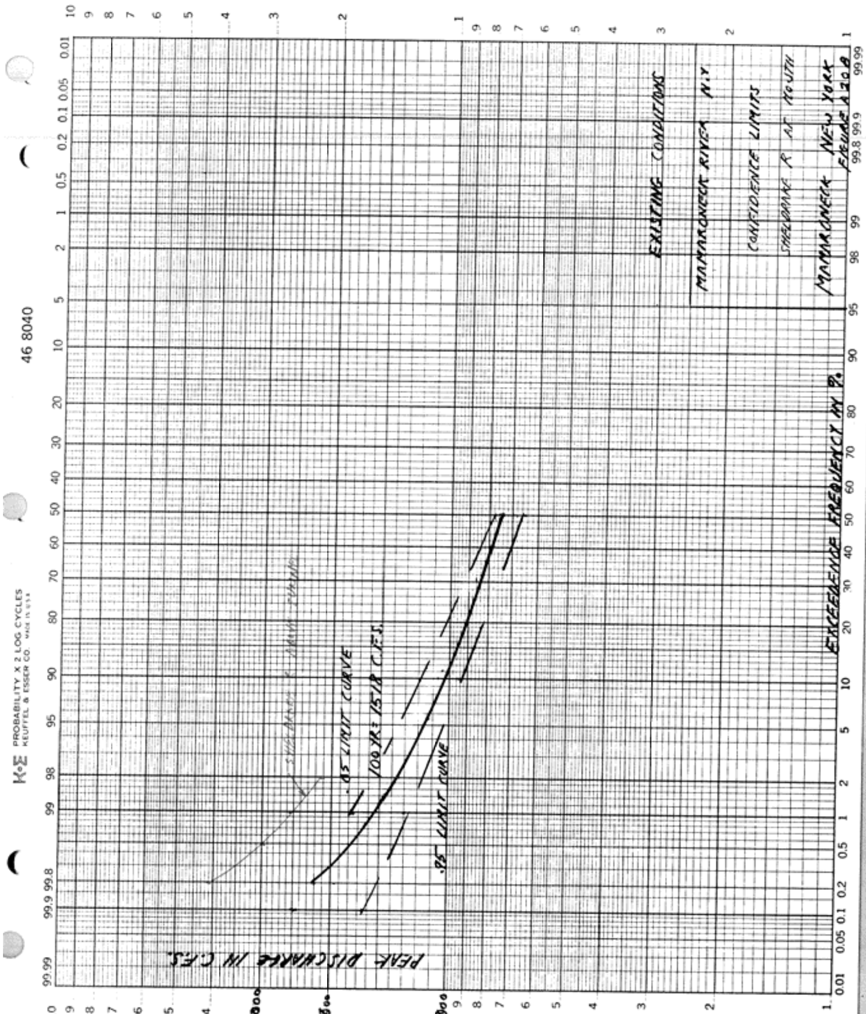
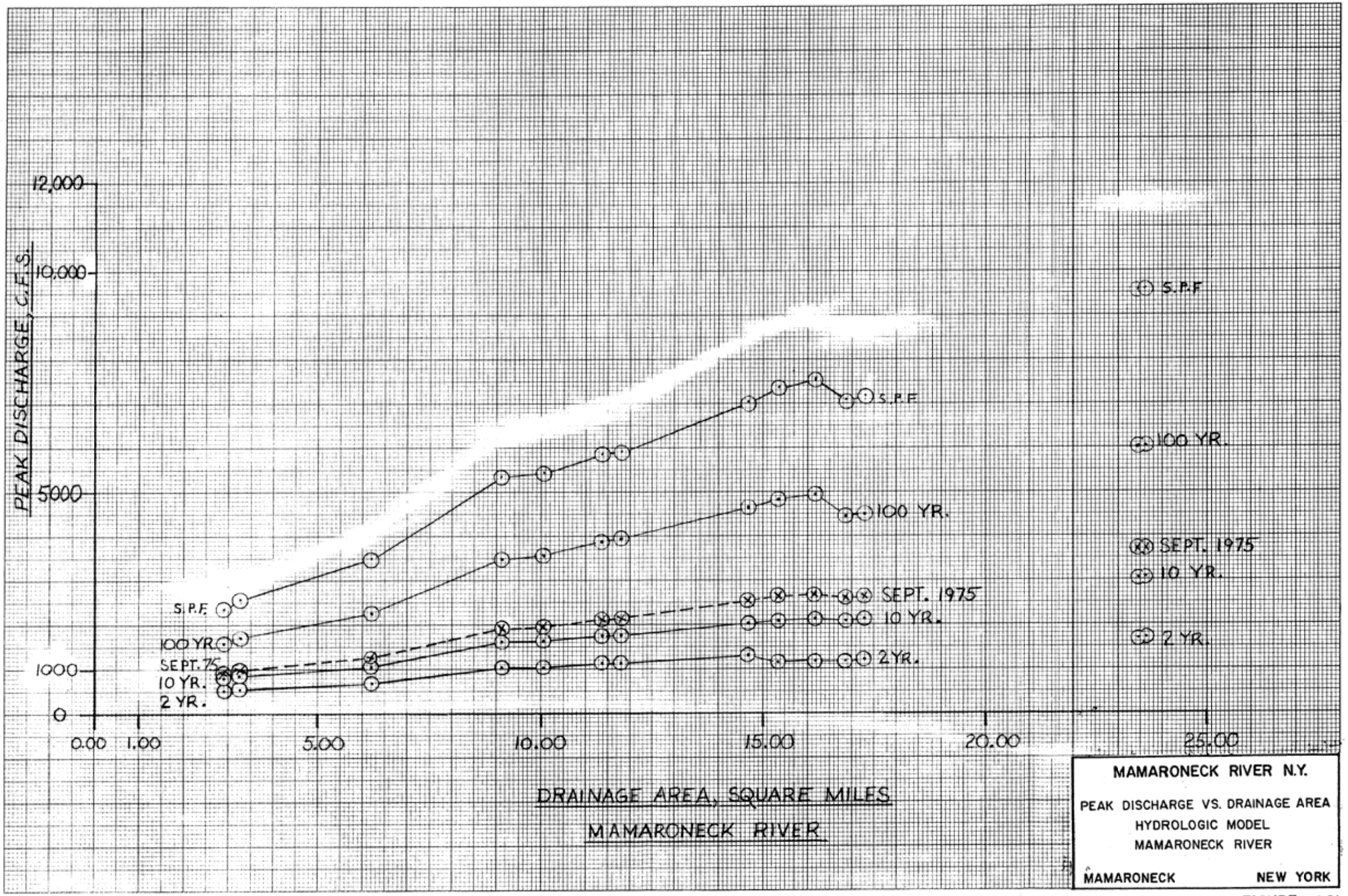

46 8040
K&E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO.
MAMARONECK U.S. SHELDRAKE
EXISTING CONDITIONS
MAMARONECK RIVER N.Y.
PEAK DISCHARGE VS FREQUENCY
CONFIDENCE LIMITS
MAMARONECK NEW YORK
FIGURE A20A
EXCEEDENCE FREQUENCY IN %
PEAK DISCHARGE IN C.F.S.
100 YR = 4490 CFS
05 LIMIT CURVE
95 LIMIT CURVE

FIGURE A 20 A



EXISTING CONDITIONS
MAMARONECK RIVER N.Y.
CONFIDENCE LIMITS
SHELDRAKE R AT MOUTH
MAMARONECK NEW YORK
FIGURE A20B
EXCEEDENCE FREQUENCY IN %
PEAK DISCHARGE IN C.F.S.
.05 LIMIT CURVE
100 YR = 1518 C.F.S.
.95 LIMIT CURVE
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO.
46 8040

FIGURE A 20 B

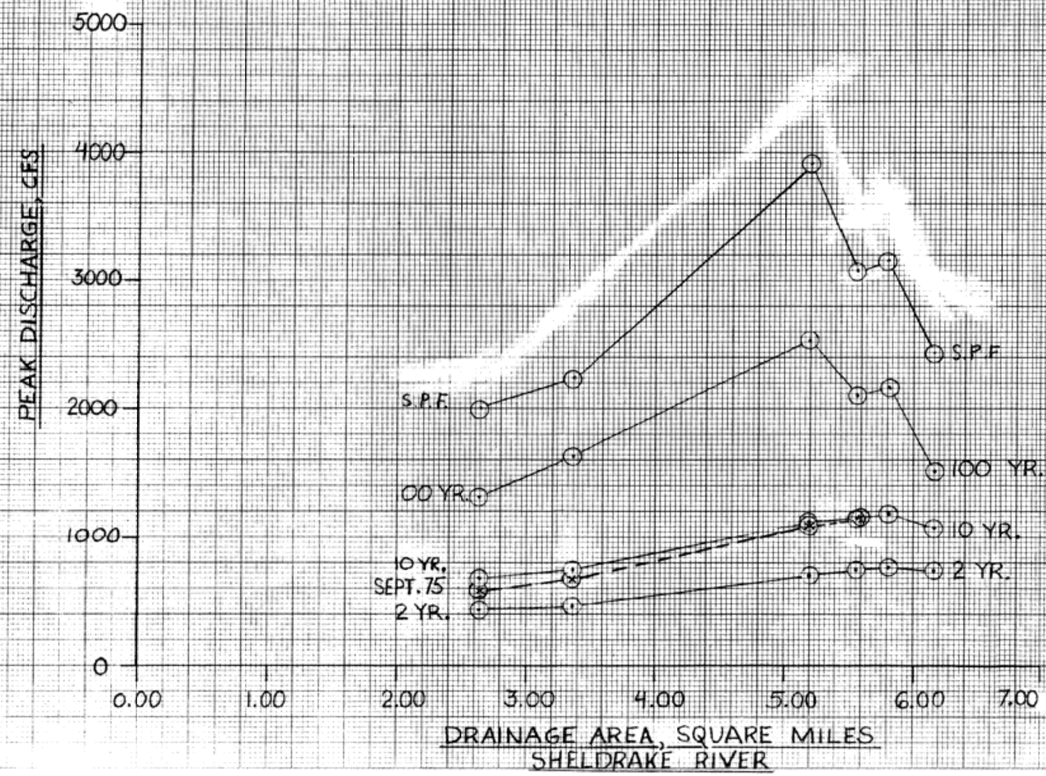


MAMARONECK RIVER N.Y.

PEAK DISCHARGE VS. DRAINAGE AREA
HYDROLOGIC MODEL
MAMARONECK RIVER

MAMARONECK NEW YORK

FIGURE A21



MAMARONECK RIVER N.Y.

PEAK DISCHARGE VS. DRAINAGE AREA
HYDROLOGIC MODEL
SHELDRAKE RIVER

MAMARONECK NEW YORK

FIGURE A22

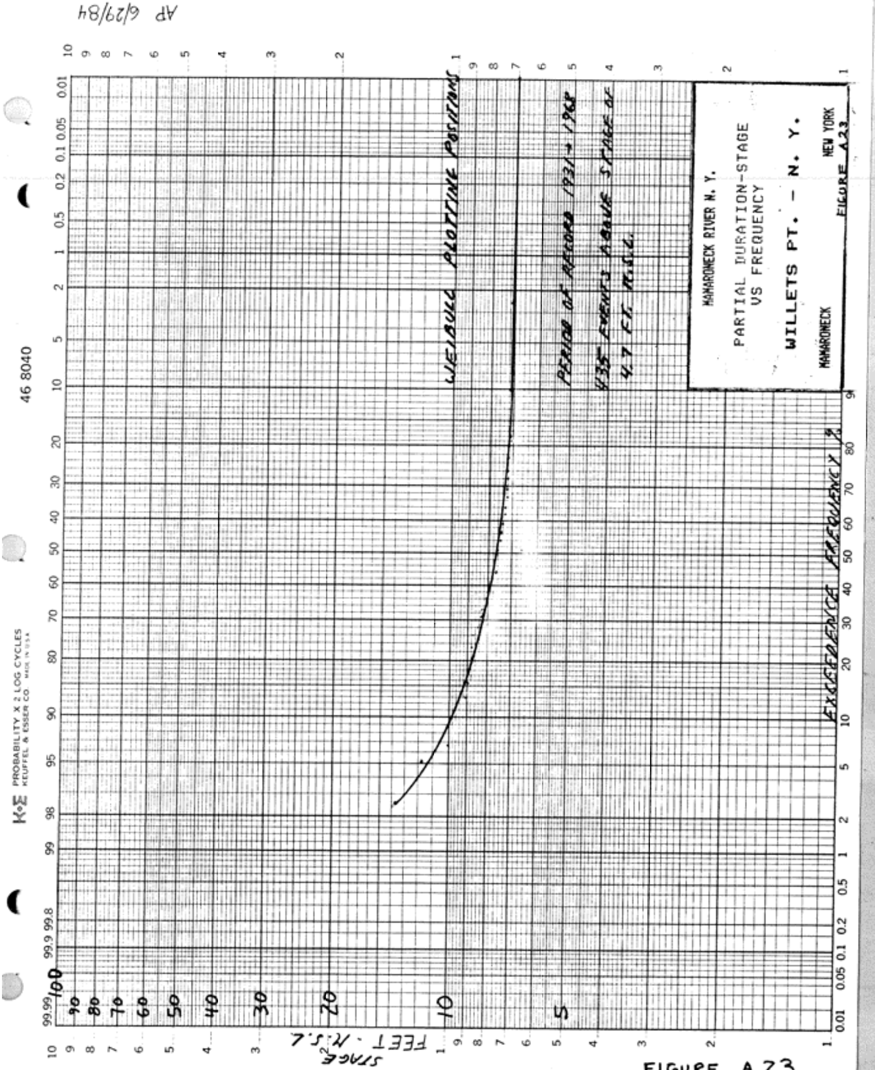

AP 6/29/84
46 8040
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
MAMARONECK RIVER N. Y.
PARTIAL DURATION-STAGE
VS FREQUENCY
WILLETS PT. - N. Y.
MAMARONECK
NEW YORK
FIGURE A23
WEIBULL PLOTTING POSITIONS
PERIOD OF RECORD 1931 - 1968
435 EVENTS ABOVE STAGE OF
4.7 FT. M.S.L.
EXCEEDENCE FREQUENCY %
STAGE
FEET - M.S.L.

FIGURE A 23

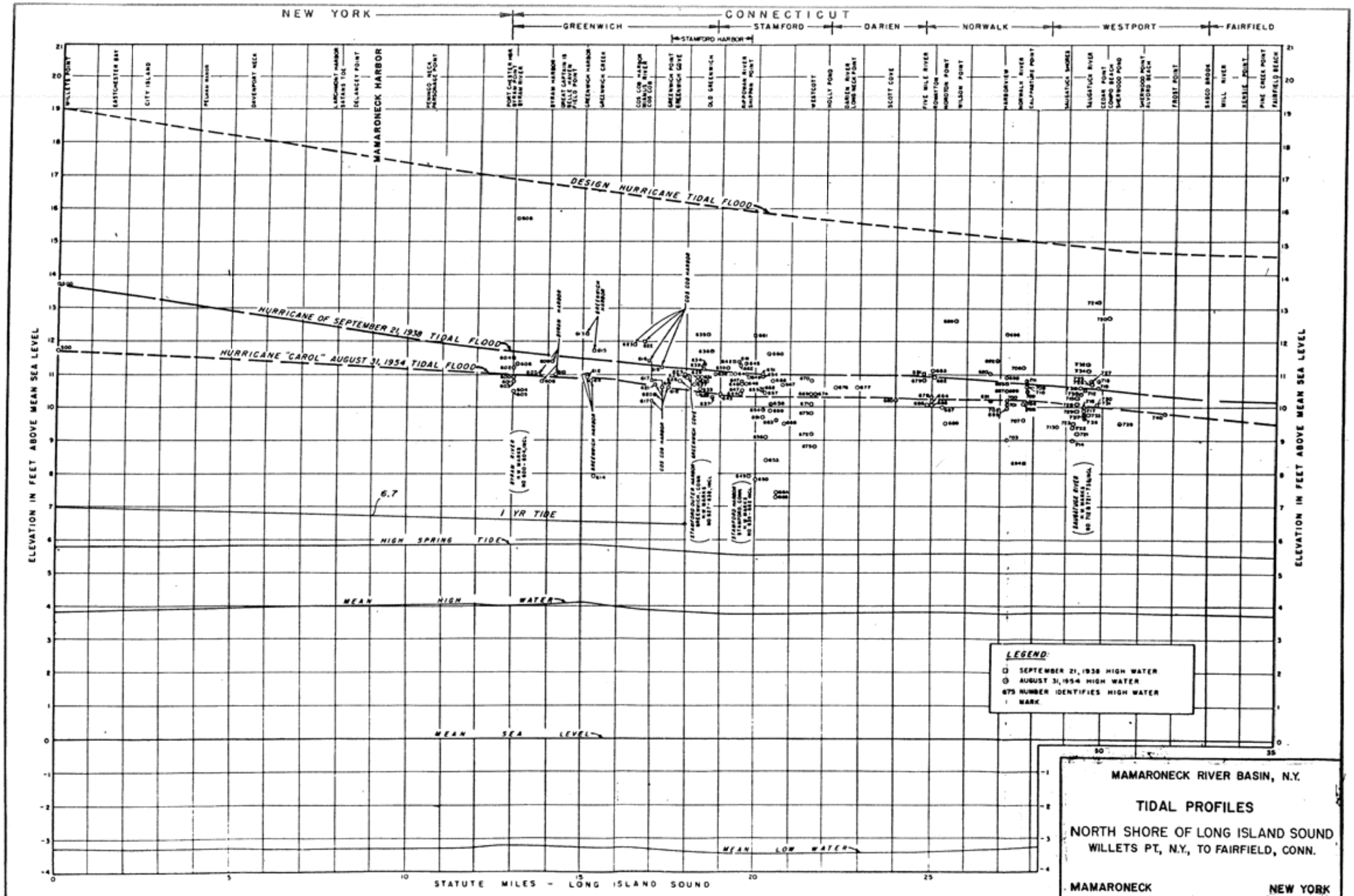

NEW YORK
CONNECTICUT
GREENWICH
STAMFORD
STAMFORD HARBOR
DARIEN
NORWALK
WESTPORT
FAIRFIELD
MAMARONECK HARBOR
DESIGN HURRICANE TIDAL FLOOD
HURRICANE OF SEPTEMBER 21, 1938 TIDAL FLOOD
HURRICANE "CAROL" AUGUST 31, 1954 TIDAL FLOOD
6.7
1 YR TIDE
HIGH SPRING TIDE
MEAN HIGH WATER
MEAN SEA LEVEL
MEAN LOW WATER
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
STATUTE MILES - LONG ISLAND SOUND
LEGEND:
SEPTEMBER 21, 1938 HIGH WATER
AUGUST 31, 1954 HIGH WATER
675 NUMBER IDENTIFIES HIGH WATER MARK
MAMARONECK RIVER BASIN, N.Y.
TIDAL PROFILES
NORTH SHORE OF LONG ISLAND SOUND
WILLETS PT, N.Y., TO FAIRFIELD, CONN.
MAMARONECK
NEW YORK

FIGURE A24

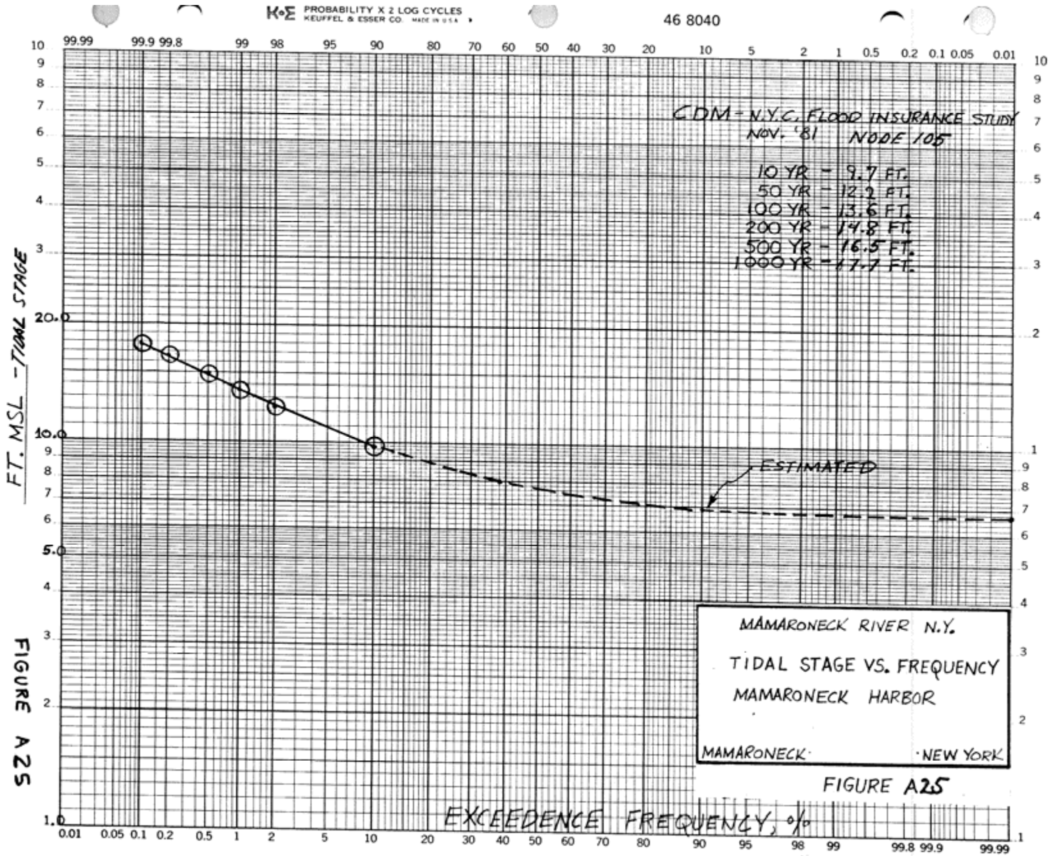
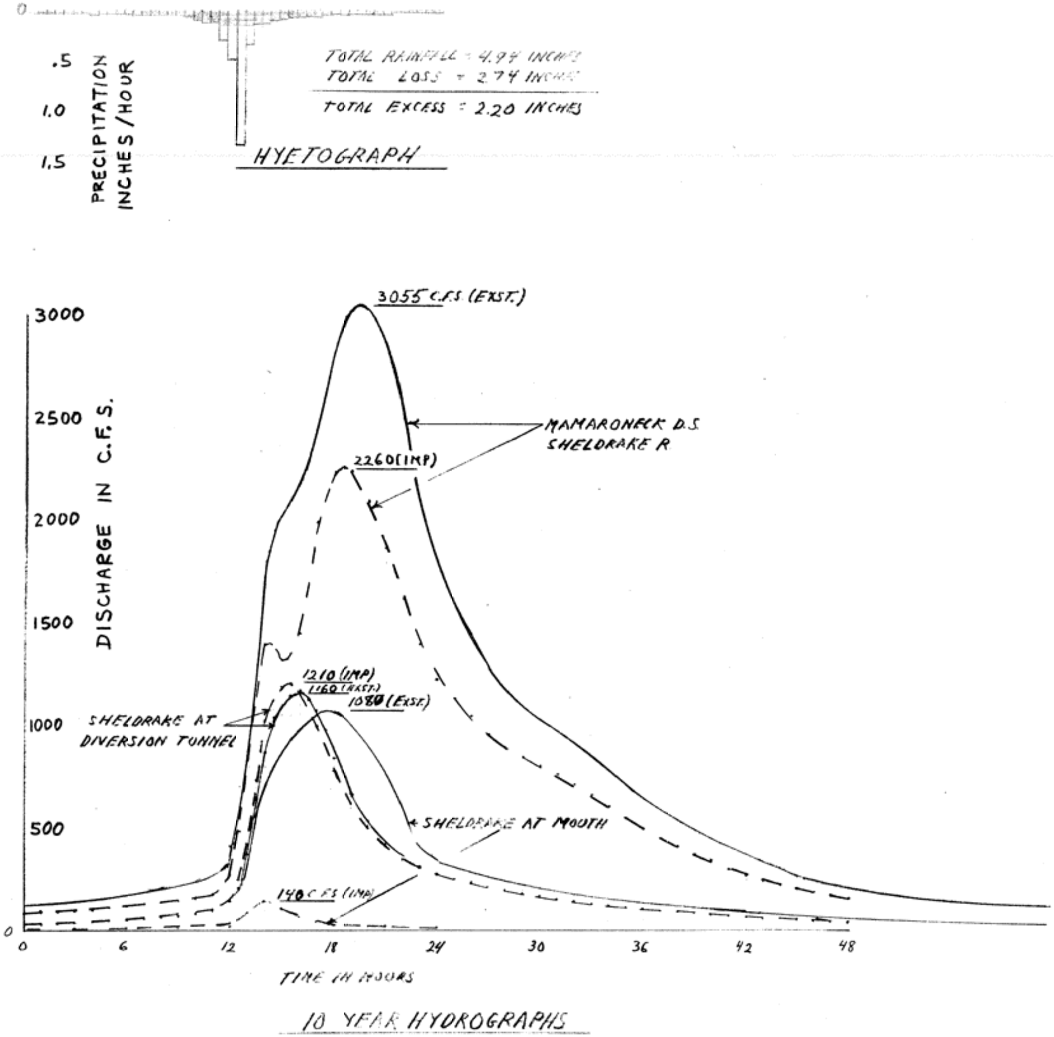

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
CDM - N.Y.C. FLOOD INSURANCE STUDY
NOV. '81 NODE 105
10 YR - 9.7 FT.
50 YR - 12.2 FT.
100 YR - 13.6 FT.
200 YR - 14.8 FT.
500 YR - 16.5 FT.
1000 YR - 17.7 FT.
ESTIMATED
FT. MSL - TIDAL STAGE
EXCEEDENCE FREQUENCY, %
MAMARONECK RIVER N.Y.
TIDAL STAGE VS. FREQUENCY
MAMARONECK HARBOR
MAMARONECK
NEW YORK
FIGURE A25

FIGURE A25



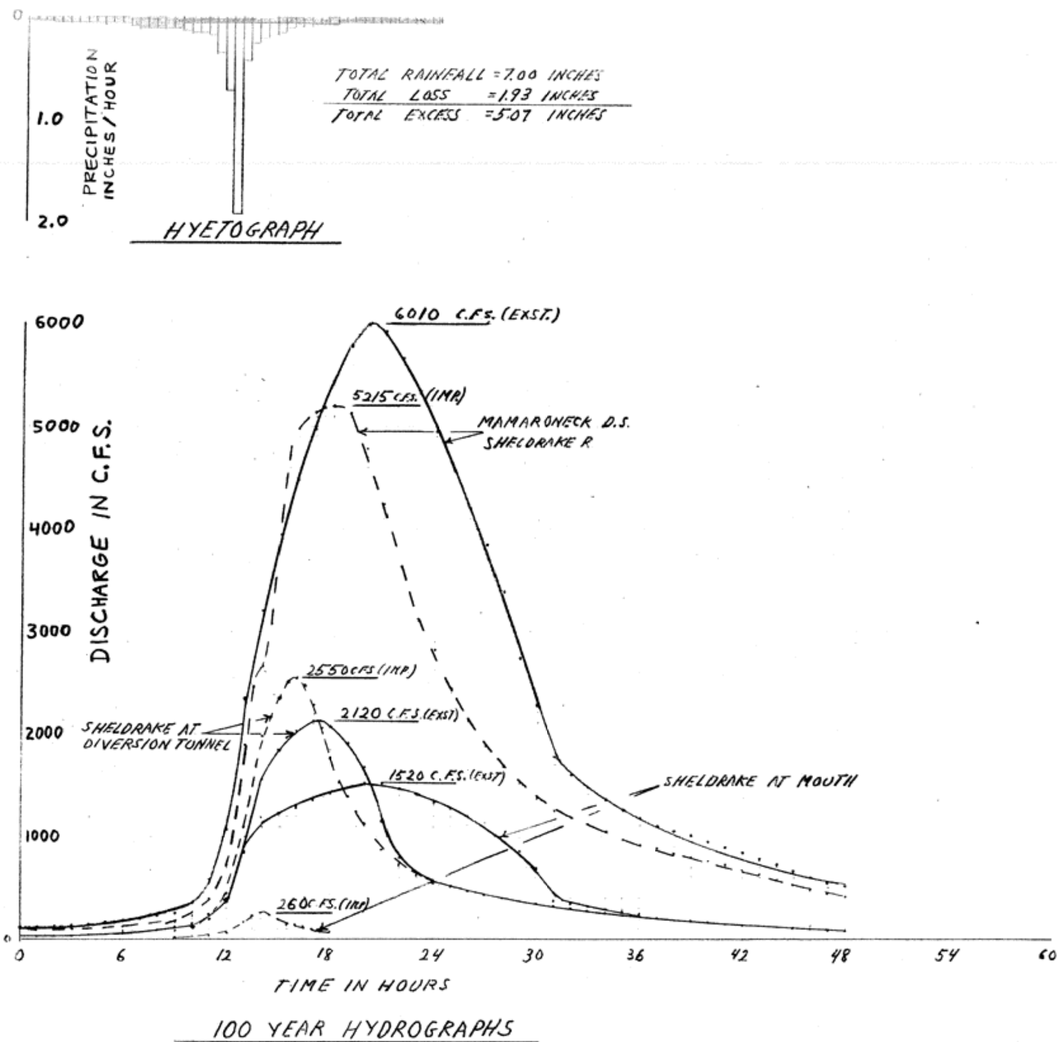
NOTE: IMPROVED CONDITIONS CONSIST OF THE SHELDRAKE DIVERSION TUNNEL AND SHELDRAKE AND MAMARONECK RIVER CHANNEL IMPROVEMENTS

MAMARONECK RIVER N.Y.

10 YEAR FLOOD HYDROGRAPH EXISTING AND IMPROVED CONDITIONS

MAMARONECK NEW YORK

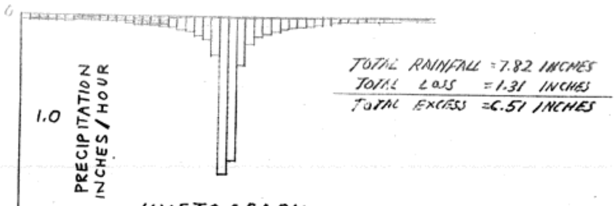
FIGURE A26



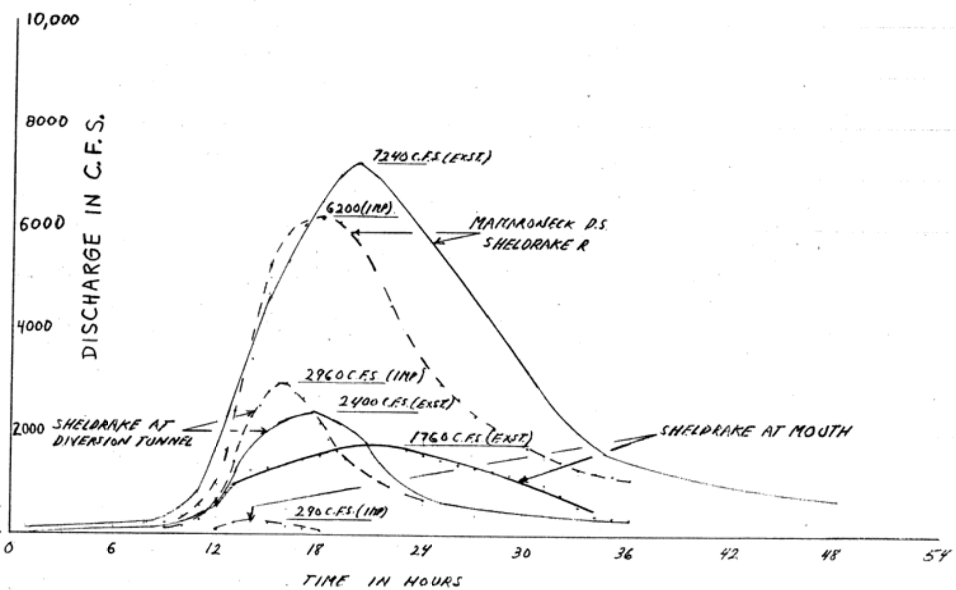
NOTE: IMPROVED CONDITIONS CONSIST OF THE SHELDRAKE DIVERSION TUNNEL AND SHELDRAKE AND MAMARONECK RIVER CHANNEL IMPROVEMENTS

MAMARONECK RIVER N. Y.
100 YEAR FLOOD HYDROGRAPH EXISTING AND IMPROVED CONDITIONS
MAMARONECK NEW YORK

FIGURE A27



HYETOGRAPH



200 YEAR HYDROGRAPHS

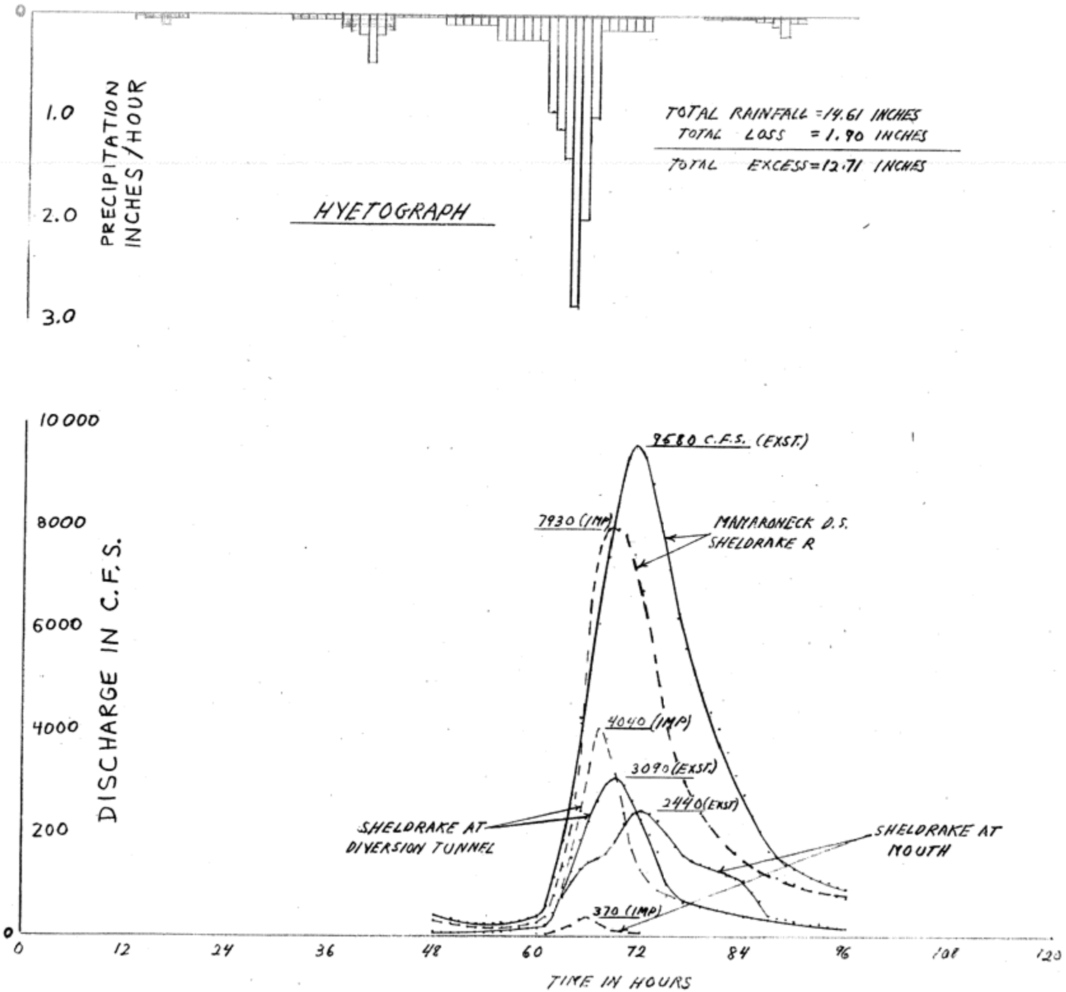
NOTE: IMPROVED CONDITIONS CONSIST OF THE SHELDRAKE DIVERSION TUNNEL AND SHELDRAKE AND MAMARONECK RIVER CHANNEL IMPROVEMENTS

MAMARONECK RIVER N.Y.

200 YEAR FLOOD HYDROGRAPH EXISTING AND IMPROVED CONDITIONS

MAMARONECK NEW YORK

FIGURE A28



STANDARD PROJECT FLOOD HYDROGRAPHS

NOTE: IMPROVED CONDITIONS CONSIST OF THE SHELDRAKE DIVERSION TUNNEL AND SHELDRAKE AND MAMARONECK RIVER CHANNEL IMPROVEMENTS.

MAMARONECK RIVER N.Y.

STANDARD PROJECT FLOOD HYDROGRAPH

EXISTING AND IMPROVED CONDITIONS

MAMARONECK NEW YORK

FIGURE A29

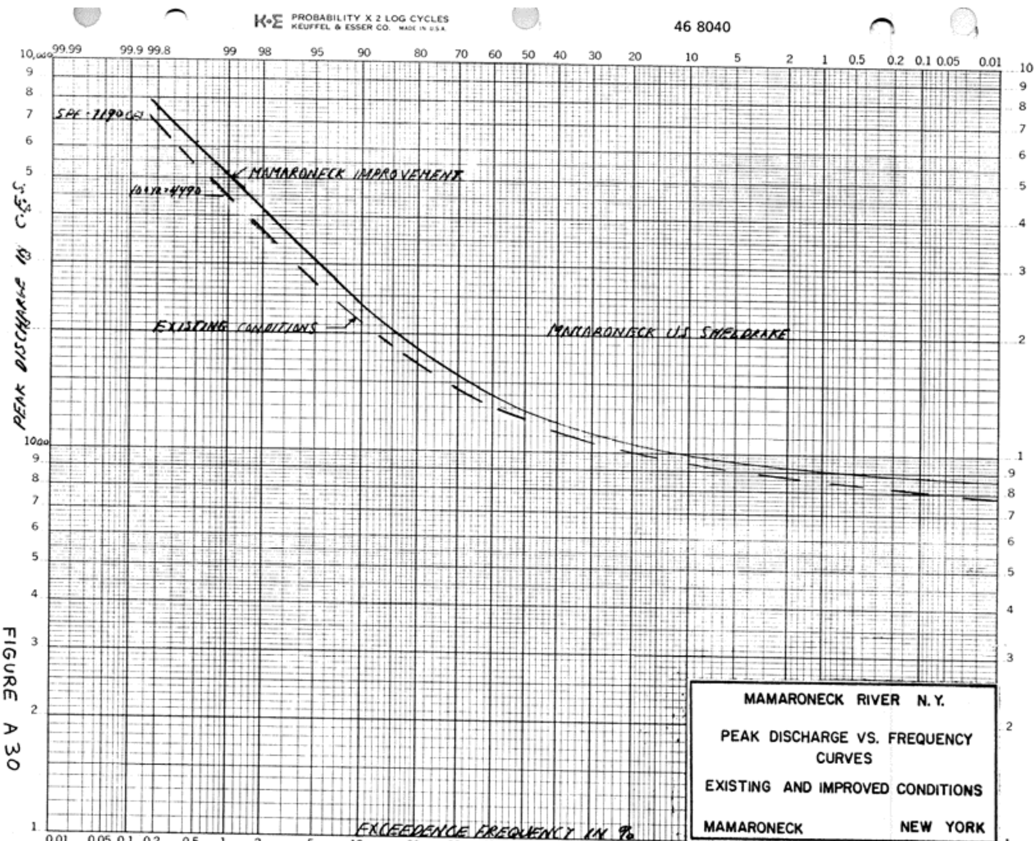


FIGURE A30

FIGURE A 30

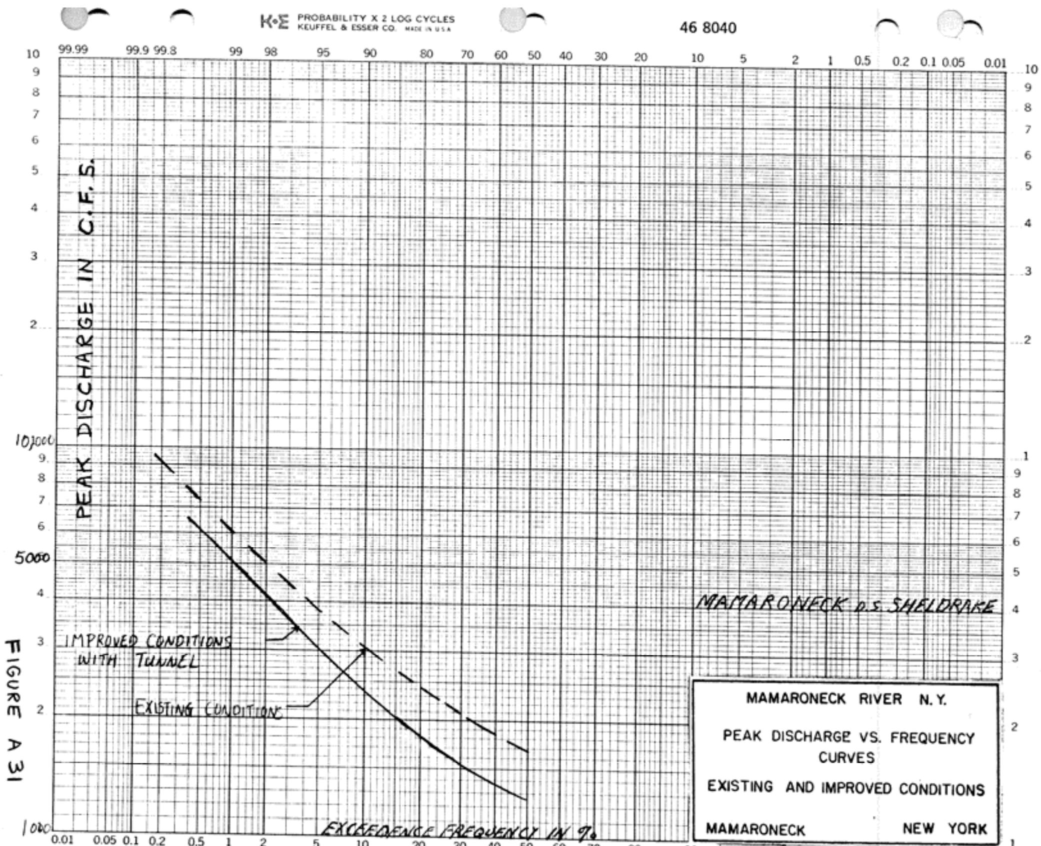


FIGURE A 31

FIGURE A 31

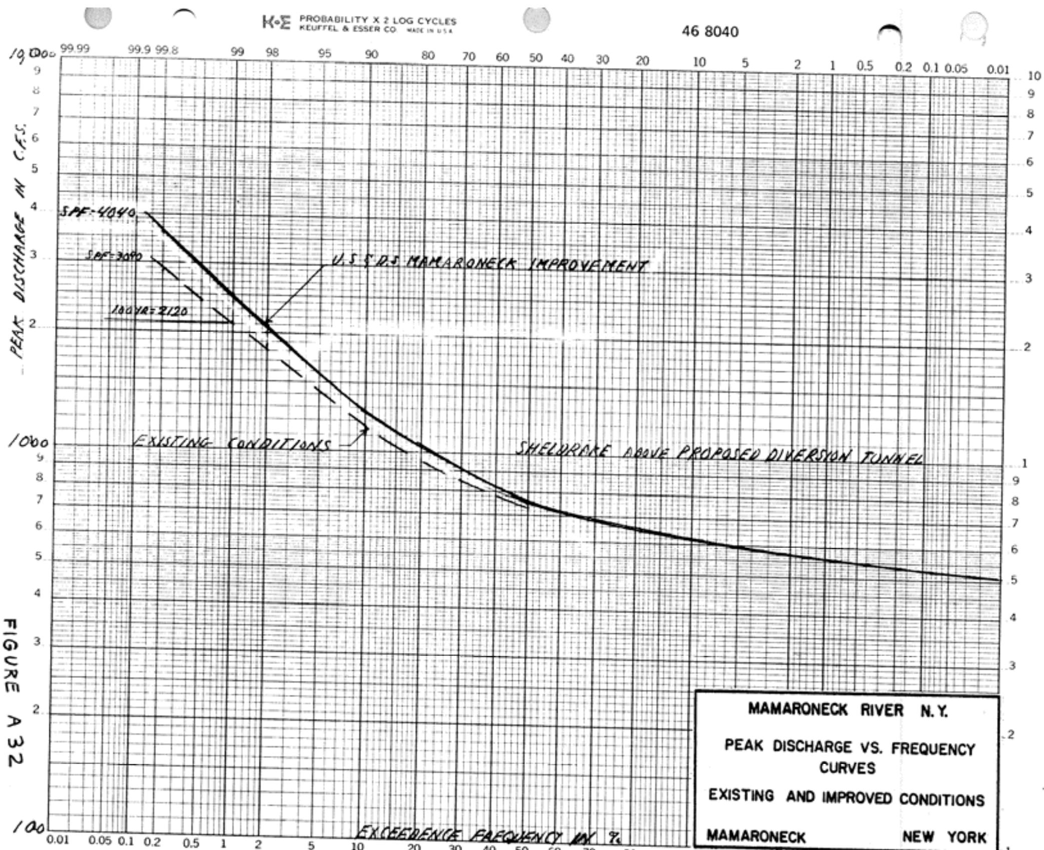


FIGURE A32

FIGURE A 32

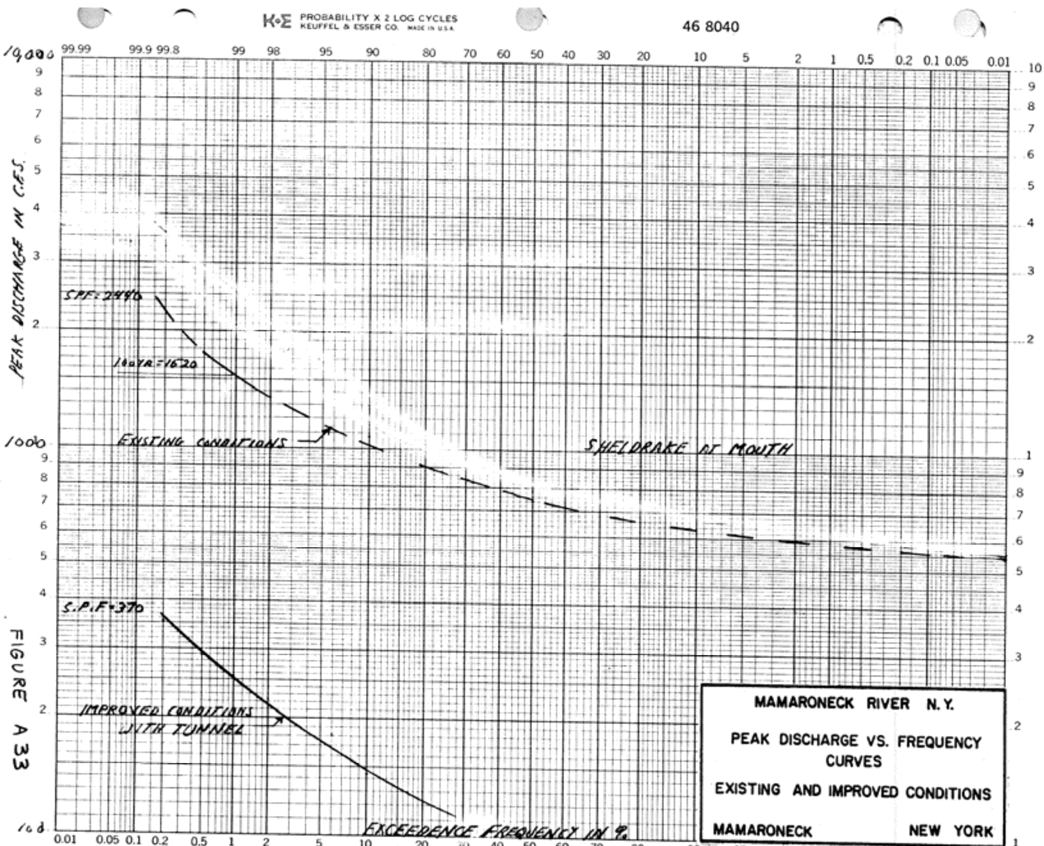


FIGURE A 33

FIGURE A33

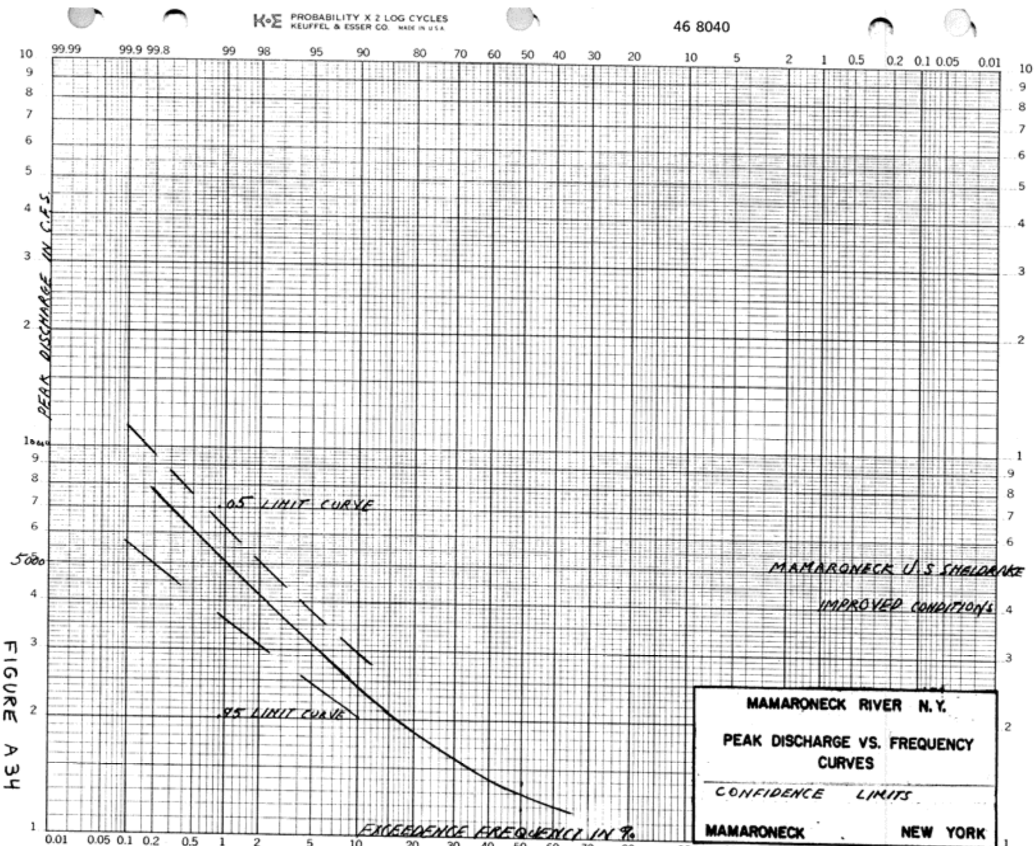


FIGURE A34

FIGURE A34

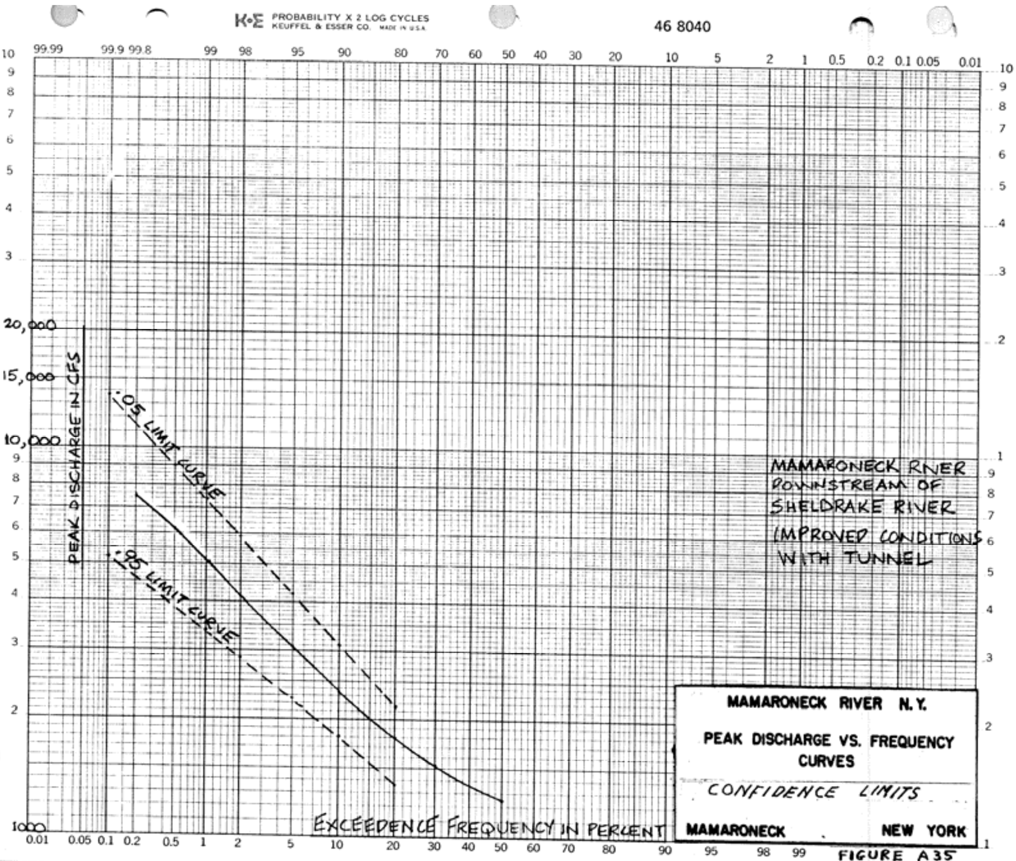

K•E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
PEAK DISCHARGE IN CFS
20,000
15,000
10,000
1000
EXCEEDENCE FREQUENCY IN PERCENT
.05 LIMIT CURVE
.95 LIMIT CURVE
MAMARONECK RIVER
DOWNSTREAM OF
SHELDRAKE RIVER
IMPROVED CONDITIONS
WITH TUNNEL
MAMARONECK RIVER N.Y.
PEAK DISCHARGE VS. FREQUENCY
CURVES
CONFIDENCE LIMITS
MAMARONECK
NEW YORK
FIGURE A35

FIGURE A 35

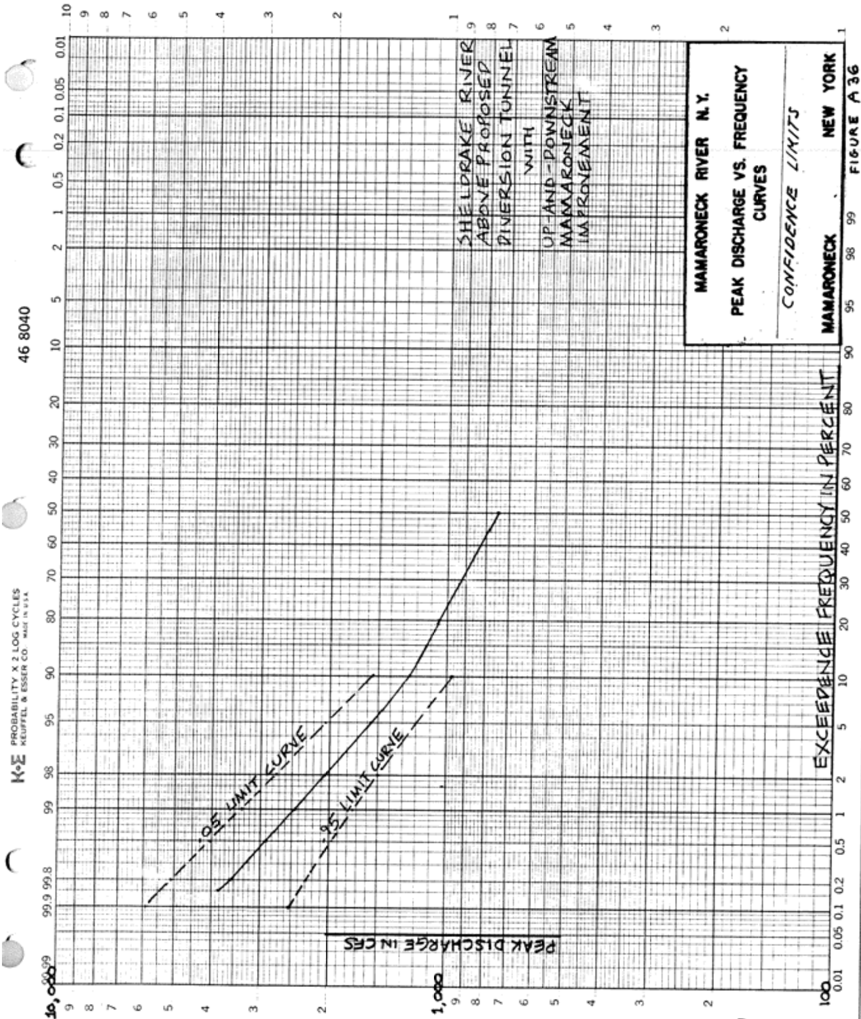

SHELDRAKE RIVER
ABOVE PROPOSED
DIVERSION TUNNEL
WITH
UP-AND-DOWNSTREAM
MAMARONECK
IMPROVEMENT
MAMARONECK RIVER N.Y.
PEAK DISCHARGE VS. FREQUENCY CURVES
CONFIDENCE LIMITS
MAMARONECK NEW YORK
FIGURE A 36
.05 LIMIT CURVE
.95 LIMIT CURVE
EXCEEDENCE FREQUENCY IN PERCENT
PEAK DISCHARGE IN CFS
10,000
1,000
100
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040

FIGURE A 36

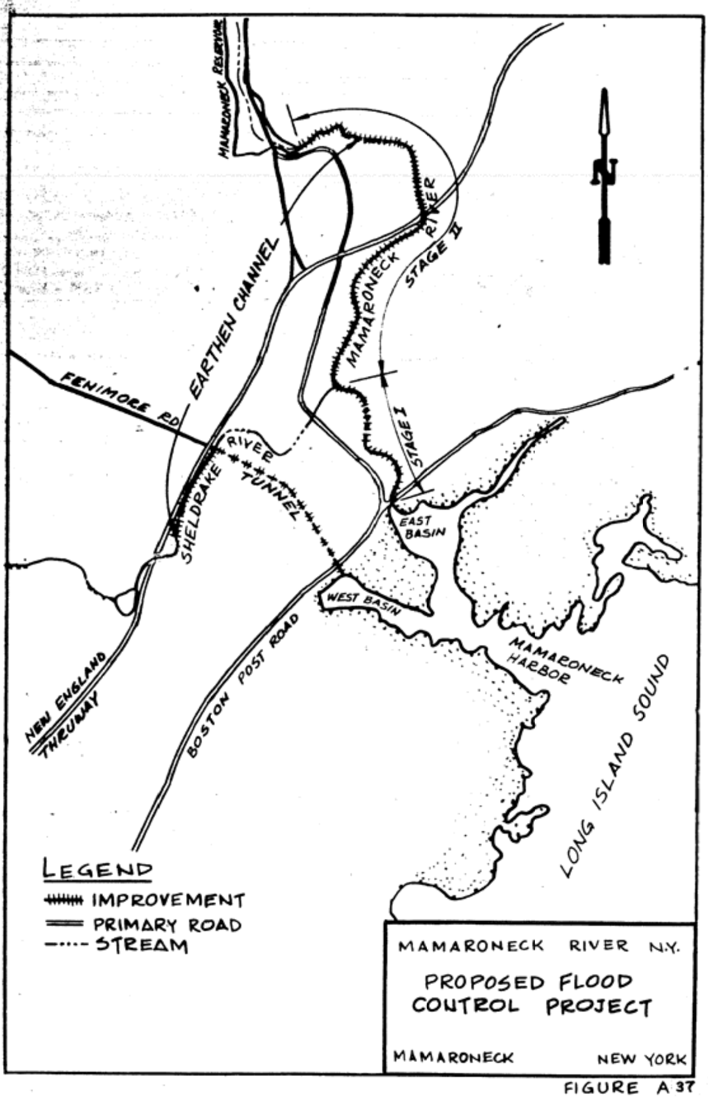

MAMARONECK RESERVOIR
MAMARONECK RIVER
STAGE II
STAGE I
EARTHEN CHANNEL
FENIMORE RD
SHELDRAKE RIVER
TUNNEL
EAST BASIN
WEST BASIN
NEW ENGLAND THRUWAY
BOSTON POST ROAD
MAMARONECK HARBOR
LONG ISLAND SOUND
N
LEGEND
IMPROVEMENT
PRIMARY ROAD
STREAM
MAMARONECK RIVER N.Y.
PROPOSED FLOOD CONTROL PROJECT
MAMARONECK NEW YORK

FIGURE A 37

ANNEX A1

RESIDUAL FLOODING

ANNEX A1
MAMARONECK RIVER AT MAMARONECK
RESIDUAL FLOODING

GENERAL

The Proposed Mamaroneck channel consists of a 45 to 60 ft. bottom width channel from its mouth upstream to the Joint Water Works Dam, a distance of 2.25 miles. The improvement reach along the Sheldrake extends from its confluence with the Mamaroneck River upstream for a distance of 1 mile to the Rockland Avenue Bridge. The upper Sheldrake River channel improvement will be deepened and widened to a bottom width of 40 feet. The Sheldrake River diversion consists of a tunnel system running beneath Fenimore Road from the Sheldrake River to the West Basin of Mamaroneck Harbor. The lower Sheldrake River (downstream of the Fenimore Road Bridge) will be filled, creating a 10 foot trapezoidal channel. A likely location for overtopping to begin is at station 64+80, D.S. of First Street on the Mamaroneck River, U.S. of the Sheldrake River Confluence shown on Figure A1-1. The design discharge in this reach of stream has a approximate 0.5 percent exceedance frequency and a 200 year level of protection.

River stage hydrographs for the September 1975 (largest flood of record), 200 year and Standard Project Flood at Station 64 +80 on the Mamaroneck River U.S. of the Sheldrake River are shown on Figures A1-2, 3, and 4. These figures also show peak flood elevation, the warning times of impending inundation, duration of inundation, and river rate of rise at the beginning of inundation. A typical cross section at this location, showing the existing and improved channel is shown on Figure A1-5. Table A1-1 contains residual flood information for these floods, along with depth and velocity of inundation.

With the 200 year design flood maximum elevation of 23.4 ft. M.S.L., there would be some residual flooding on the left bank above elevation 21.5 ft. M.S.L., shown on Figure A1-5. This would occur about 15 hours after the start of runoff, as indicated on the 200 year stage hydrograph, shown on Figure A1-3. This would allow sufficient warning time for possible evacuation. The maximum rate of rise of the river at this time is about 3 feet per hour. The duration of flooding above stage 21.5 feet M.S.L. would be approximately 6.5 hours. The residual flood damage in this area would be minimal because the damageable property is above el. 25 ft. M.S.L. on the left bank. On the right bank, there would be some local flooding of streets and some disruption of traffic on Howard Ave. The velocities in the overbank for all floods up to the 200 year flood is 0 and is less than 0.2 ft/sec for the Standard Project Flood. Therefore, the 200 year flood, with a maximum elevation of 23.4 ft. M.S.L. would not cause any significant damage in this area.

A reoccurence of the September 1975 flood, shown on Figure A1-2, with a maximum elevation of 18.25 ft. M.S.L. would not reach the top of the low bank and would not cause any residual flooding.

If the standard project flood occurred as shown on Figure A1-4, the maximum elevation would be 26.1 feet above M.S.L. or 4.6 feet above the low bank. The maximum rate of rise at the time of overtopping would be 4 ft per hr. and the duration of flooding above elevation 21.5 ft M.S.L. would be approximately 9 hours. Therefore this rare event could cause significant flooding.

Another possible location for overtopping to begin is at Station 55+18, upstream of Rockland Avenue on the Sheldrake River. This reach is immediately upstream of the Sheldrake River improvement. River stage hydrographs for the September 1975 (largest flood of record), 200 year and standard project flood are shown on Figures A1-6,7, and 8. With the 200 year design flood maximum elevation of 28.7 ft. M.S.L., as shown on Figure A1-7, there would be some overtopping on the right bank, as can be seen on the cross section shown on Figure A1-9. This flooding would occur about 13 hours after the start of runoff, shown on Figure A1-7. This would allow sufficient warning time for possible evacuation. The maximum rate of rise of the river at this time is about 2 feet per hour. The duration of flooding above stage 25 feet M.S.L. would be approximately 7 hours. The average depth of flooding on the right bank during the maximum elevation associated with the 200 year flood was about 3 feet. The velocity in the left overbank was about 1.9 ft. per second and in the right overbank about 2.3 ft. per second. With the 200 year flood under improved conditions, there would be some remaining flooding to about 3 houses on the right bank. The left bank in this area, which is bounded by the New England Thruway, is undeveloped.

A reoccurence of the September 1975, shown on Figure A1-6 with a maximum elevation of 24.2 ft. M.S.L. would not reach the top of the low bank and would not cause any residual flooding.

An occurence of the Standard Project Flood, as shown on Figure A1-8, with a maximum elevation of 30.9 ft. M.S.L. would cause significant flooding throughout the area with inundation depths of approximately 6 feet and velocities of 1.8 ft per second in the left bank and 2.1 ft per second in the right bank.

It is noted that the inundation maps under existing and improved conditions for the 100-year flood and the Standard Project Flood are shown on Figures B31 through B44 of the Hydraulics Appendix.

Risk. There is a chance or risk that the Mamaroneck project could have its design level exceeded. The level of protection for this channel plan is 200 year. An inundation risk analysis for any 10, 30, 50, or 100 year period was developed is summarized in Table A1-2.

GENERAL DESIGN MEMO MAMARONECK AND SHELDRAKE RIVERS, N.Y.
RESIDUAL FLOODING INUNDATION DATA
TABLE A1-1

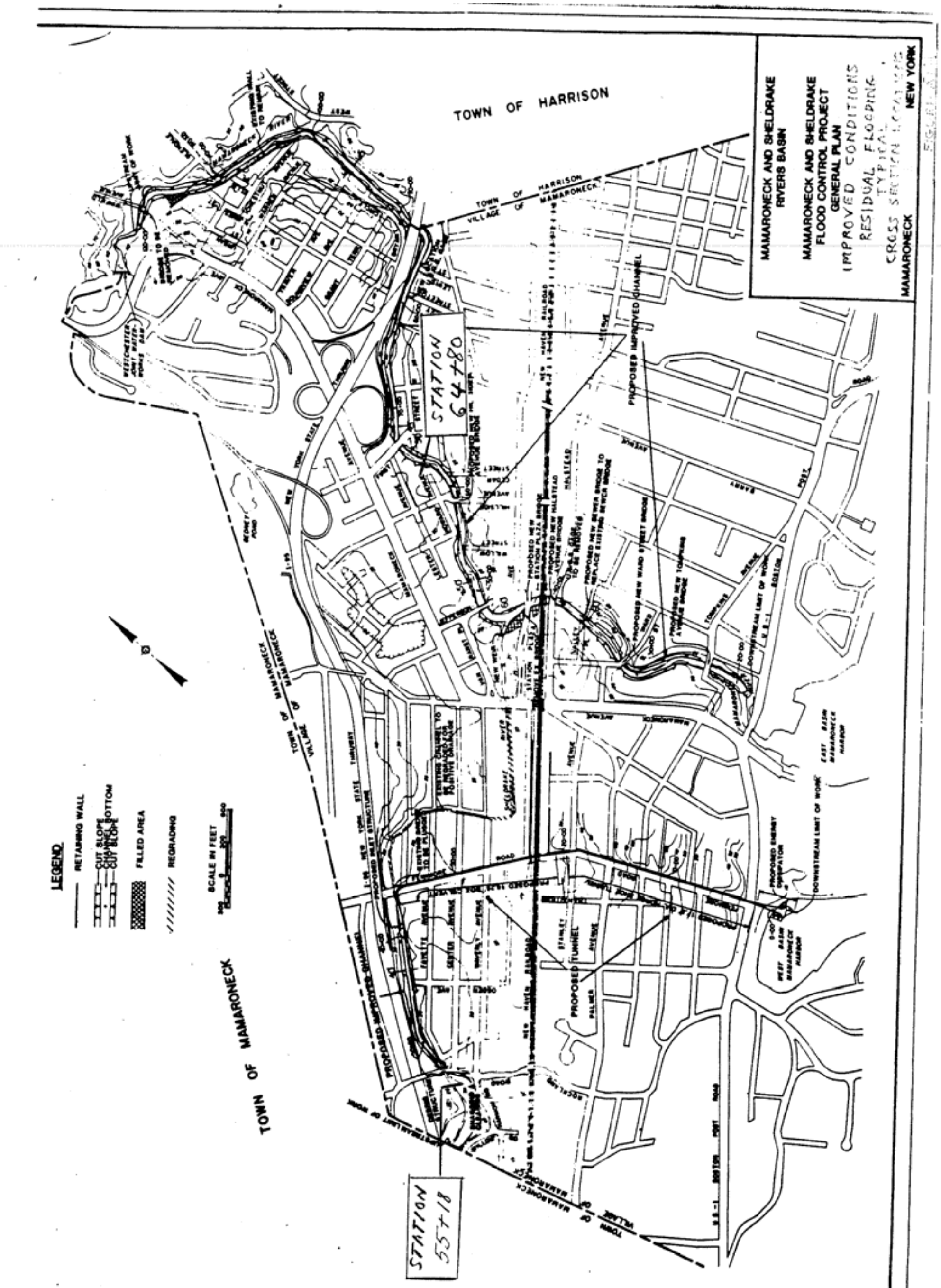
	SEPT 1975	200 YEAR	STANDARD
LOCATION-STA.64+80 MAMARONECK RIVER D.S. FIRST STREET			
PEAK DISCHARGE (C.F.S.)	2630	6150	7820
PEAK RIVER EL. (FT. M.S.L.)	18.3	23.4	26.1
WARNING TIME (HRS.)	12.5	15	18
MAXIMUM RATE OF RISE FT/HR	1.5	3	4
INUNDATION DURATION (HRS)	——	6.5	9.0
APPROX. DEPTH (FT.)	——	2	4.5
OVERBANK VELOCITY (FT/SEC)	——		
LEFT BANK		0	.15
RIGHT BANK		0	.10
SHELDRAKE RIVER U.S. ROCKLAND AVE STA. 55+18			
PEAK DISCHARGE (C.F.S.)	1150	2960	4040
PEAK RIVER EL (FT. M.S.L.)	24.7	28.7	30.9
WARNING TIME (HRS.)	9.5	13	16
MAXIMUM RATE OF RISE FT/HR	2.0	2	1.5
INUNDATION DURATION (HRS)	——	7	8.5
APPROX. DEPTH FT.	——	3	5.4
OVERBANK VELOCITY (FT/SEC)	——		
LEFT BANK		1.9	1.8
RIGHT BANK		2.3	2.1

MAMARONECK RIVER AT MAMARONECK
INUNDATION RISK ANALYSIS
TABLE A1-2

200 YEAR DESIGN FLOOD

$M = 200$ $\quad P = \frac{1}{200} = .005$ $\quad q = 1-.005 = .995$

PERIOD IN YEARS (N) EVENT	10	30	50	100
NO DESIGN FLOOD OCCURS $P_0 = q^n$	0.95	0.86	0.78	0.61
EXACTLY ONE DESIGN FLOOD OCCURS $P_1 = N(P)(q)^{N-1}$	0.05	0.13	0.20	0.30
TWO OR MORE DESIGN FLOODS OCCUR $P_{2+} = 1 - P_0 - P_1$	0.001	0.01	0.02	0.09



pg A1-5

FIGURE A1-1

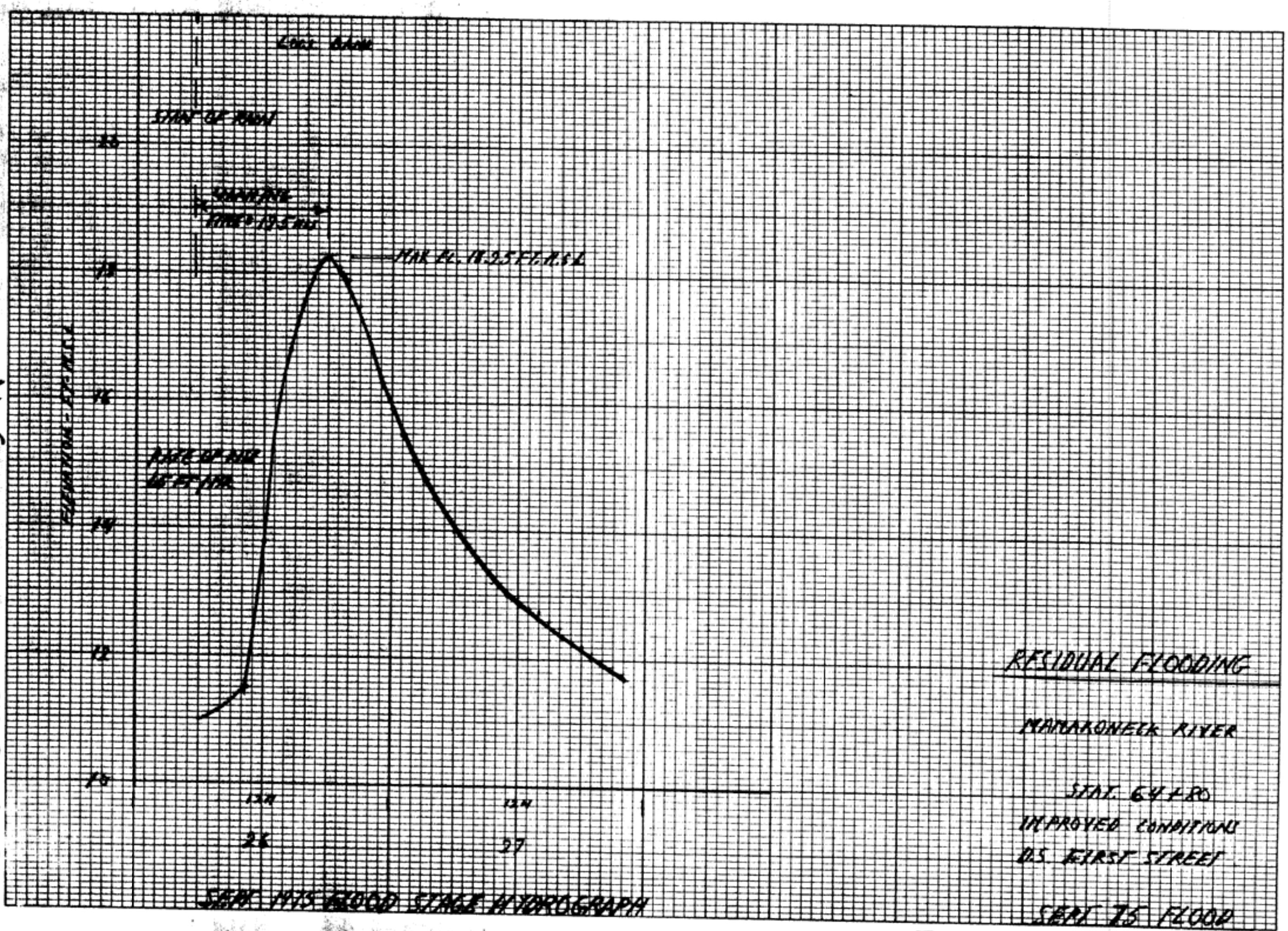


FIGURE A1-2

FIGURE A1-2

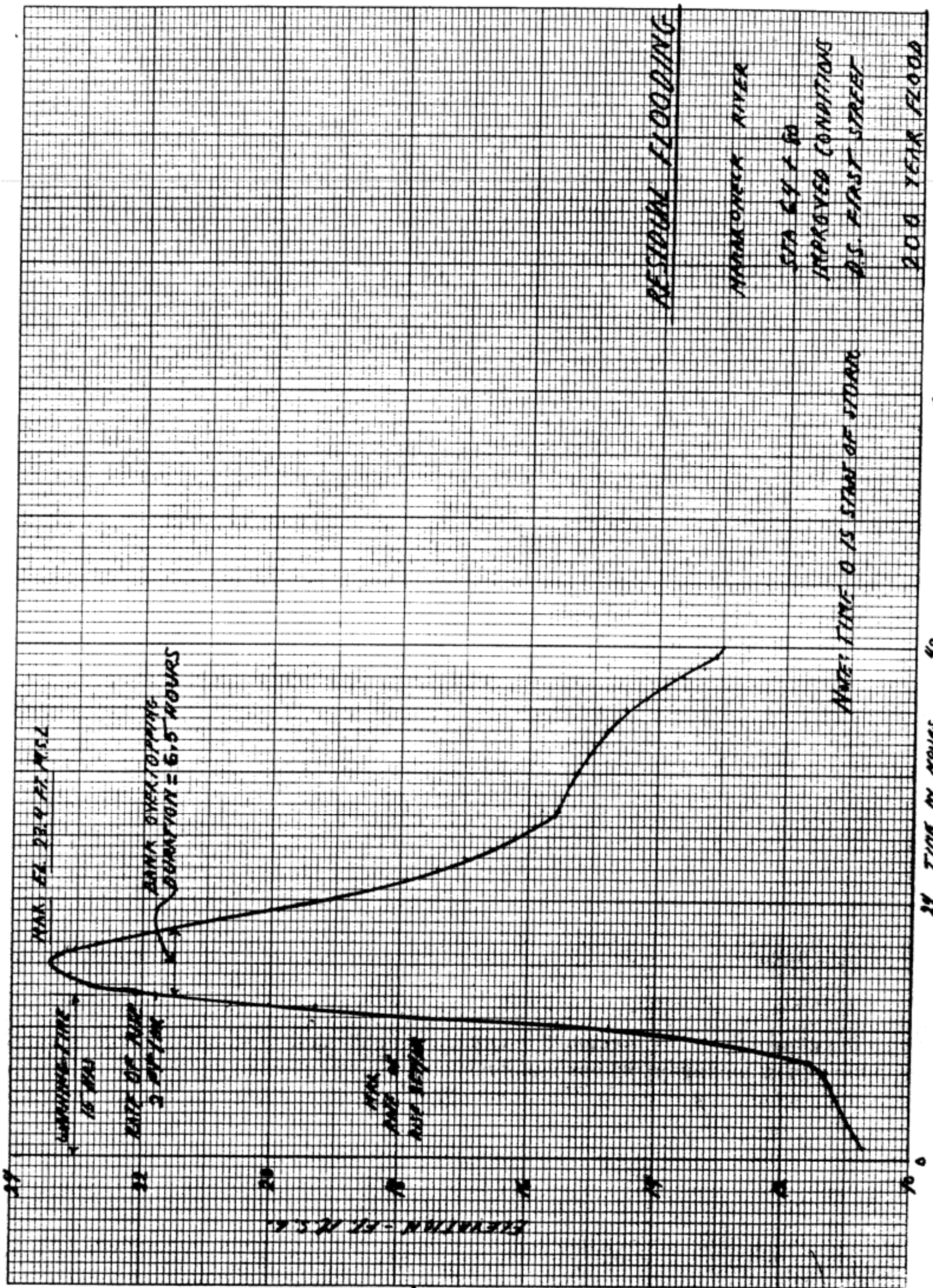
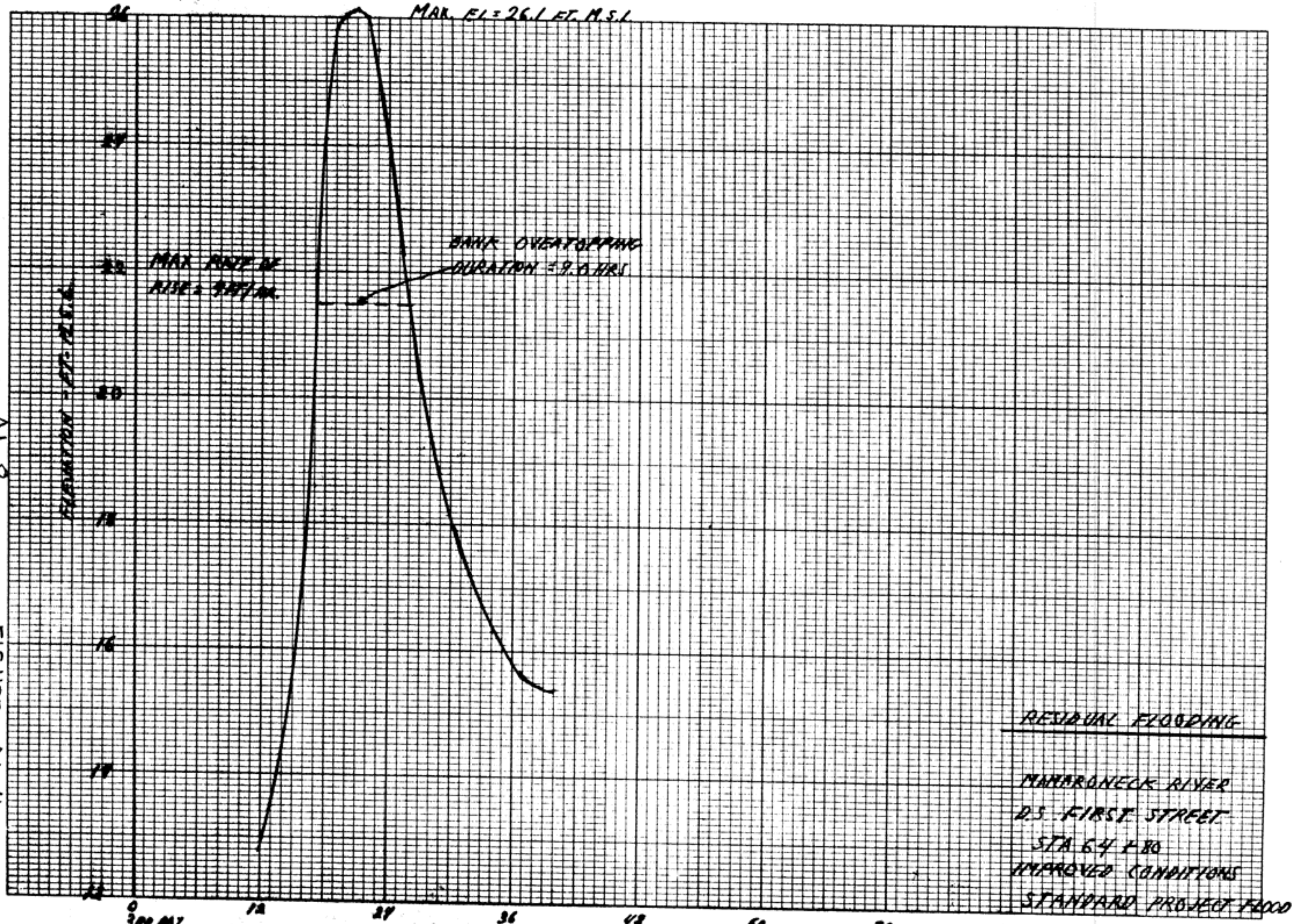


FIGURE A1-3

FIGURE A1-3

K+E 12 X 20 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A. 46 1970



STANDARD PROJECT FLOOD STAGE HYDROGRAPH

FIGURE A1-4

pg A1-8

FIGURE A1-4

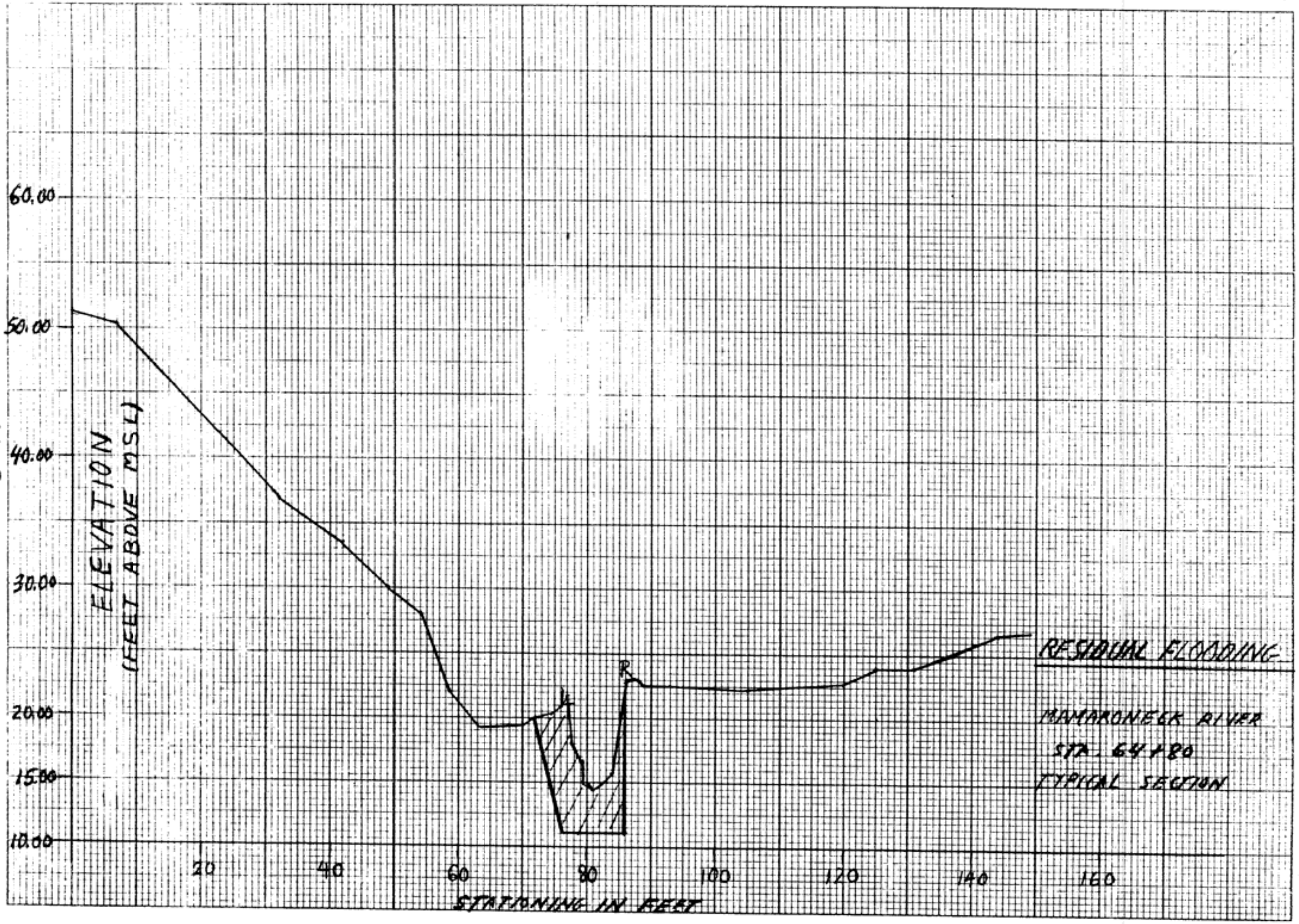
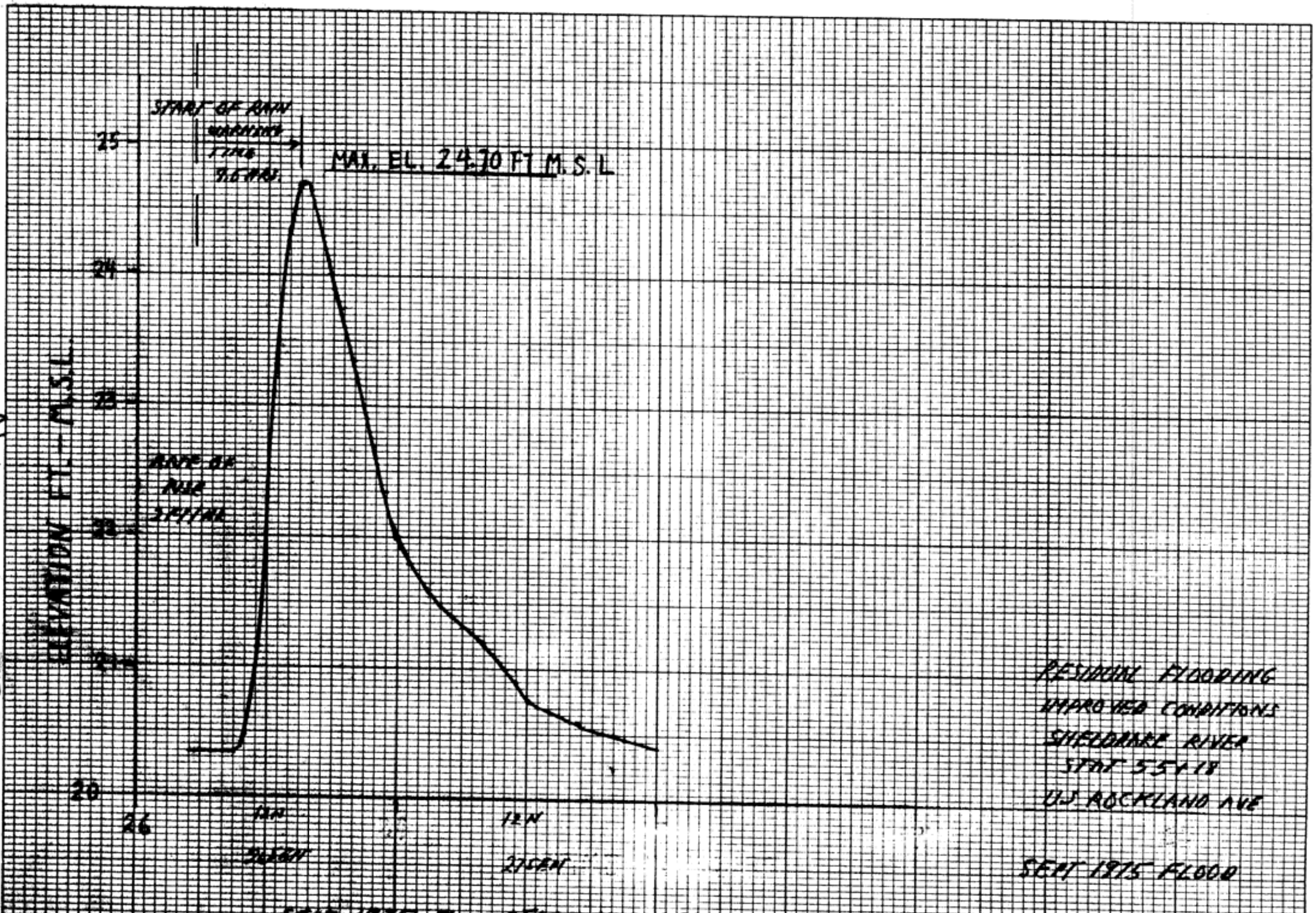


FIGURE A1-5

FIGURE A1-5

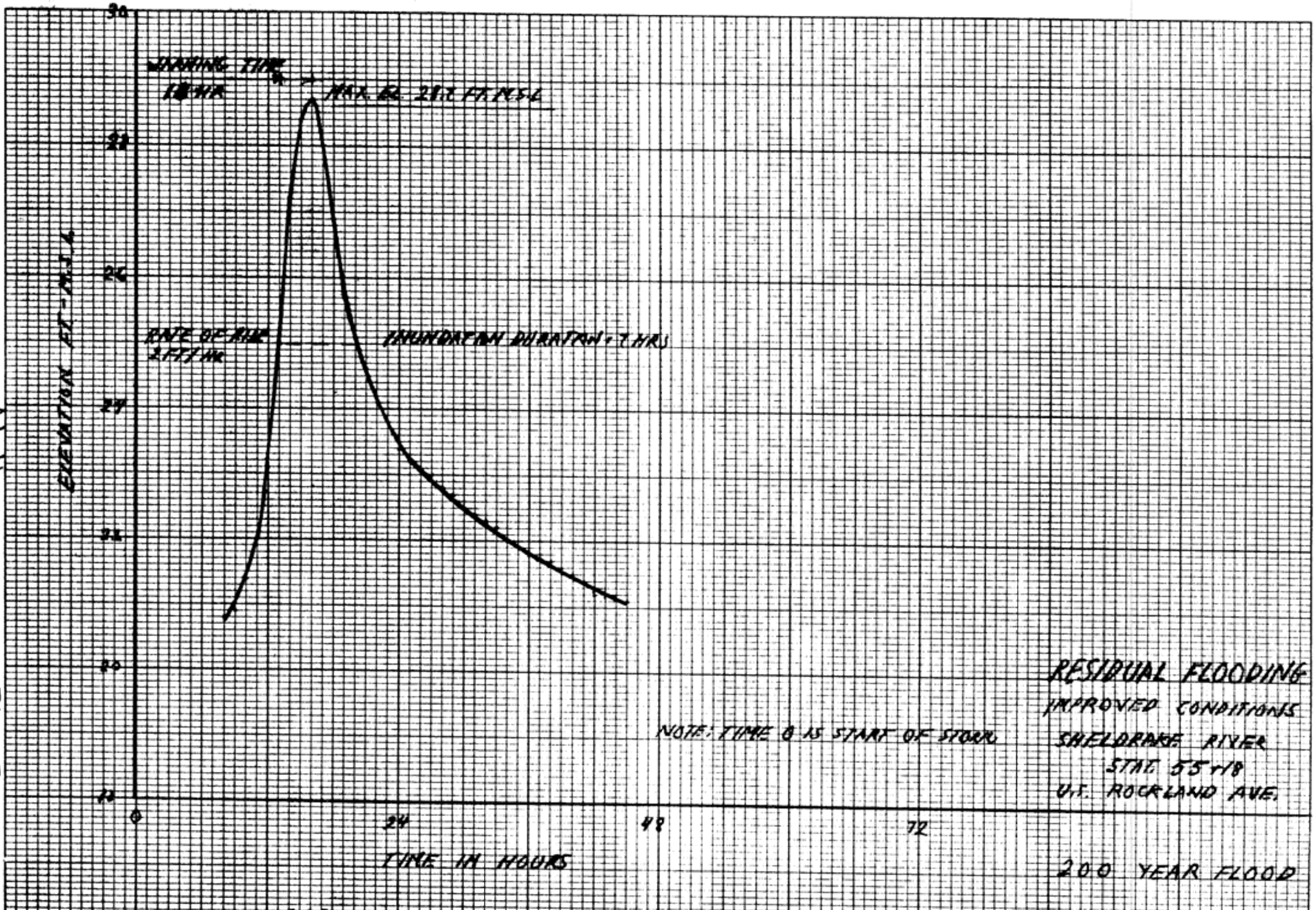
Pg A1-10



SEPT 1975 FLOOD STAGE HYDROGRAPH

FIGURE A1-6

FIGURE A1-6

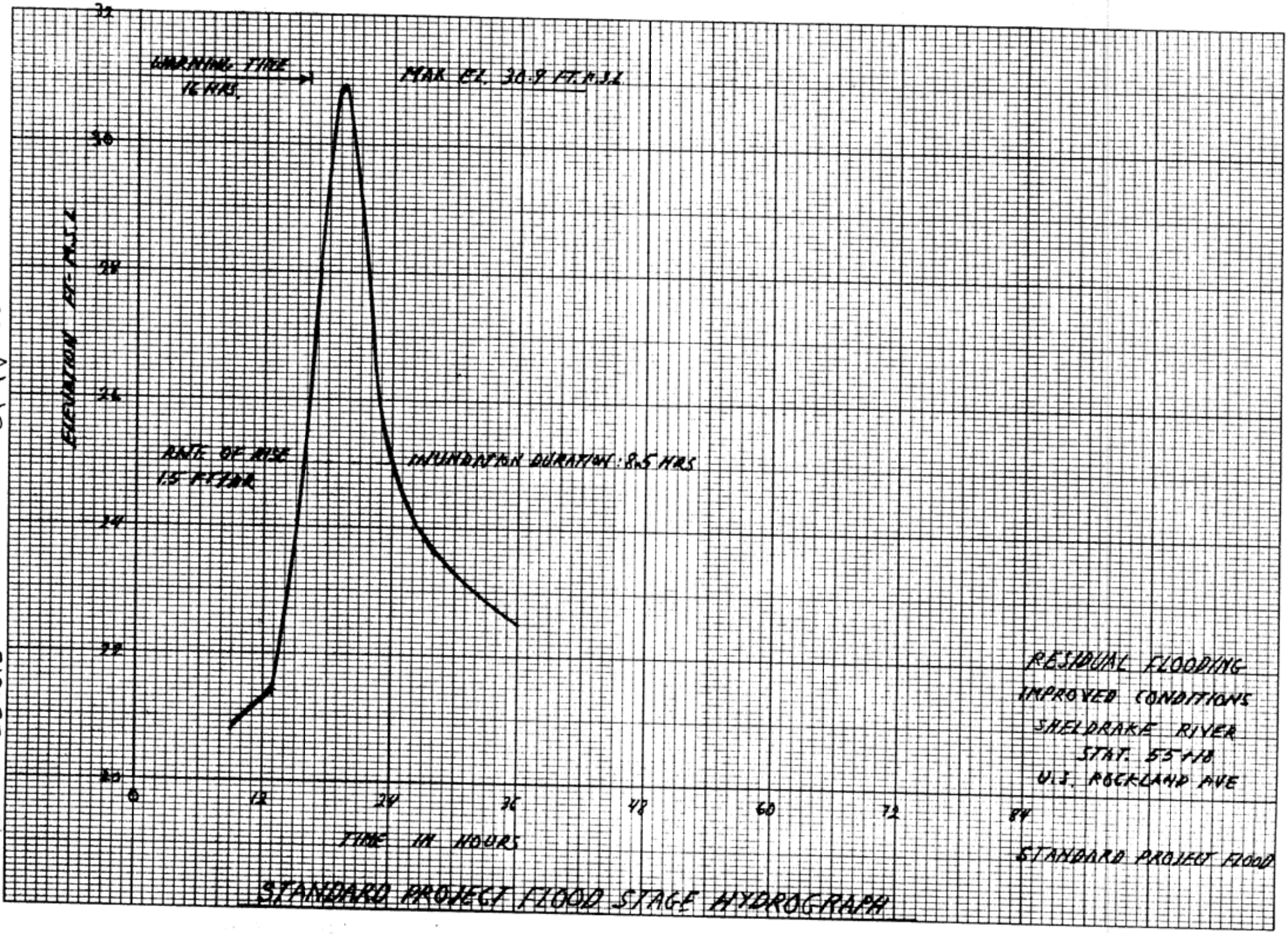


200 YEAR STAGE HYDROGRAPH

FIGURE A1-7

pg A1-11

FIGURE A1-7



STANDARD PROJECT FLOOD STAGE HYDROGRAPH

FIGURE A1-8

FIGURE A1-8

RESIDUAL FLOODING

SHELDRAKE RIVER
STA 55+18
TYPICAL SECTION

STATION 55+18 (UPSTREAM OF ROCKLAND AVENUE)
SHELDRAKE RIVER

GENERAL DESIGN MEMO
MAMARONECK & SHELDRAKE RIVERS
RESIDUAL FLOODING
FIGURE A1-9

FIGURE A1-9

WESTCHESTER COUNTY STREAMS
MAMARONECK AND SHELDRAKE RIVERS
FLOOD CONTROL PROJECT
VILLAGE OF MAMARONECK

GENERAL DESIGN MEMORANDUM

APPENDIX B

HYDRAULICS

JANUARY 1989

MAMARONECK AND SHELDRAKE RIVERS
MAMARONECK, NEW YORK

GENERAL DESIGN MEMORANDUM

HYDRAULIC APPENDIX

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MAMARONECK AND SHELDRAKE RIVERS
MAMARONECK, NEW YORK

GENERAL DESIGN MEMORANDUM

HYDRAULIC APPENDIX

I. GENERAL

B1. Project Area

The Mamaroneck and Sheldrake River project areas are within the Village and the Town of Mamaroneck in the County of Westchester, New York. This study extends from the mouth of the Mamaroneck River upstream to the Westchester County Joint Water Works Dam, a distance of 2.25 miles. The study also extends along the Sheldrake River from its' confluence with the Mamaroneck River upstream for a distance of 1.0 mile to the Rockland Ave. bridge. Maps of these drainage basins are shown in the Hydrology Appendix.

B2. Objective

The objective of the study is to develop and evaluate the hydraulic design details of the improvements which will prevent recurrent flooding in the project areas. These improvements include channel enlargement, stream realignment, bridge replacement and diversion. The following paragraphs contain a description of the hydraulic studies which were conducted.

B3. Prior Report

A "Feasibility Report for Flood Control, Mamaroneck and Sheldrake River Basin and Byram River Basin" (Ref. 1) was completed in October 1977. The Feasibility Report recommended a combination of channel widening and deepening, retaining walls, stream realignment, bridge replacement and enlargement, levees, and a diversion tunnel. The recommended plan was economically favorable, and the project was recommended for further development.

B4. Existing Channel

The project reach on the Mamaroneck River is 2.25 miles in length and contains several major bends. The channel side slopes are moderate and vary from 5 to 15 feet in height. The channel bottom is on a moderate slope, and varies in width from 20 feet at the upstream end to 55 feet at the mouth. The mouth of the Mamaroneck river is a short steep reach which is subject to some tidal inundation. The upstream reaches are subject to fluvial flooding only. The Sheldrake River is a major tributary of the Mamaroneck River and with its' confiuence at station 46+00 (See Figure B1). The Sheldrake also has some major bends and a moderate bottom slope; it is however narrower than the Mamaroneck River and it has slightly steeper side slopes. The Sheldrake River is not subject to tidal flooding because it empties into the Mamaroneck River upstream of the tidal reach. The length of the Sheldrake River in the study area is about 1.0 mile. Figures B1, B2 & B3 are profiles of the existing conditions flowlines for the Mamaroneck River. Figure B4 is a profile of the existing conditions flowlines for the Sheldrake River.

B5. Present Flooding Problems

Flooding along the Mamaroneck River is caused by low channel capacity, two 90 degree bends forming an "S" turn at the Station Plaza Bridge, and a constriction at the Tompkins Ave bridge. Flooding along the Sheldrake River is due to a low channel capacity, and backwater from the Mamaroneck River. Table B1 indicates existing (and improved) bridge capacities along the Mamaroneck and Sheldrake Rivers developed with backwater computations.

B6. Plan of Improvement

The following is a brief description of the proposed improvements to the Mamaroneck and Sheldrake Rivers. A more detailed description can be found in Paragraph B17 "Detailed Description of the Improvements".

The Mamaroneck River channel improvements include channel enlargement, stream realignment and bridge replacement. The channel will be widened and deepened to an improved bottom width which varies from 45 to 60 feet (See Figures B5 and B6). The improved bridge information is shown in Table B1.

The Lower Sheldrake River (downstream of the Fenimore Road bridge) will be regraded using a small amount of fill to create a 10 foot trapezoidal channel which will maintain positive drainage during low flow conditions. Figure B7 is a profile of the Lower Sheldrake River improvements.

The Upper Sheldrake River channel improvements include channel enlargement, stream alignment, diversion, clearing and snagging. The Sheldrake River will be deepened and widened to a bottom width of 40 feet (Figure B8). None of the bridges along the Upper Sheldrake require replacement.

TABLE B1

BRIDGE INFORMATION

MAMARONECK RIVER

BRIDGE NAME	ELEVATION OF LOW CHORD (FT. NGVD)	APPROXIMATE CAPACITY (CFS)	IMPROVED FREEBOARD ABOVE DESIGN FLOW (FT)	INVERT (FT. NGVD)	EXISTING BRIDGE WIDTH (FT)	REMARKS
Boston Post Rd.	17.0	7970/7970	9.3	8.3	50	Remains as Existing
Tompkins Ave.	16.6/13.8	7970/7970	4.0	1.3/1.0	33	Replaced with 60′ Span
Ward Ave.	24.2/21.8	7500/7970	6.4	7.6/4.8	35	Replaced with 60′ Span
Valley Place	18.5/19.0	2000/2050	3.5	8.0/5.9	53	Replaced with 60′ Span
Halstead Ave.	23.2/23.4	3600/7930	5.3	9.5/6.6	40	Replaced with 60′ Span
New Haven RR	30.0	5600/7930	13.3	11.6/6.7	48	Remains with Channel Deepening
Station Plaza	24.7/23.7	3700/7930	5.2	10.0/7.2	70	Replaced with 60′ Span
Jefferson Ave.	21.0	1600/7400	2.3	11.3/8.2	70	Remains with Channel Deepening
Hillside Ave.	25.0/24.5	2600/7720	4.1	13.4/10.3	32	Replaced with 45′ Span
North Barry Ave.	27.6	2800/7610	4.0	14.5/11.9	54	Remains with Channel Deepening
New England Thru.	28.7	1900/6000	0.2	18.0/16.0	55	Remains with Channel Deepening
Winfield Ave.	33.3/---	850/---	---	26.3/24.6	25	Removed - Channel widened to 45′
			SHELDRAKE RIVER			
Park Walk Bridge 1	20.0	800/300	0	11.0/11.0	20	Remains with Channel Regrading
Park Walk Bridge 2	19.8	800/300	0	11.9/11.9	21	Remains with Channel Regrading
Mamaroneck Ave.	23.0	900/450	3	13.7/13.7	23	Remains with Channel Regrading
Waverly Ave.	20.3	500/290	0	14.4/15.4	27	Remains with Channel Regrading
Center Ave.	21.5	600/170	1	14.5/15.5	15	Remains with Channel Regrading
Fenimore Road	25.8	1100/---	5	16.5/---	36	To be Filled
Rockland Ave.	28.0	1400/---	0.5	19.4/---	36	Remains as Existing
New England Thru.	31.0	1900/4040	1.5	22.0	36	Remains as Existing

NOTES:
1) DUAL LISTINGS SUCH AS 16.6/18.3 REPRESENT EXISTING/IMPROVED CONDITIONS
2) SINGLE LISTINGS APPLY TO BOTH EXISTING AND IMPROVED CONDITIONS
3) EXISTING BRIDGE WIDTHS REPRESENT THE TOTAL OF THE CLEAR OPENINGS
4) DASHES (---) INDICATE NOT APPLICABLE
5) CAPACITIES ARE BASED ON BACKWATER COMPUTATIONS

The Sheldrake River diversion consists of a tunnel system running beneath Fenimore Road from the Sheldrake River to the West Basin of Mamaroneck Harbor. The tunnel profile is shown in Figure B9. Figures B10 and B11 contain tunnel inlet and outlet details.

In addition to the above improvements, mitigation measures requested by the U.S. Fish and Wildlife Service will be incorporated into the Mamaroneck and Sheldrake channels.

II. HYDRAULIC BASIS OF DESIGN

B7. General

As discussed previously, the objective for the development of improvements is to prevent recurrent flooding in the project areas, along the Mamaroneck and Sheldrake Rivers. The following paragraphs present the hydraulic basis for the development of such plans.

B8 Plan Formulation

Plan formulation and optimization analyse were performed in the Feasibility Study (Ref. 1) to evaluate the various alternative solutions to the flooding problems of the Mamaroneck and Sheldrake Rivers. The alternatives evaluated included channel work, levees and floodwalls, reservoirs, diversion, non-structural plans and combinations of the above. As a result of the formulation analysis the recommended plan of improvement consists of modifications to the Mamaroneck and Upper Sheldrake Rivers and the construction of a Sheldrake River diversion tunnel.

B9. Calibration

Floodmarks from 3 events (June 1972, Sept. 1975, April 1983) were available for use in calibrating the HEC-2 model. The June 1972 floodmarks were used for calibration in the 1977 Feasibility Report. The Sept. 1975 floodmarks represent the largest flood of record. The April 1983 floodmarks were obtained by District personnel on the day following the storm. Photographs were taken of the peak flood levels by a local consulting firm.

The April 1983 event was used for calibration because: the storm represents the most recent event; it occurred only four months after the topographic survey was acquired; the floodmarks were highly reliable because they were observed during or shortly after the storm by qualified personnel; and it contained the largest set of floodmarks. The 1975 floodmarks were then used for verification because with a return period of 20 years they reflect the largest flood of record. The 1972 floodmarks were then used for comparison to the feasibility report.

Existing conditions flow lines were computed using the HEC-2 backwater program. Manning's "n" values were adjusted until the computed water surface elevations were reasonably close to the observed April 1983 levels. Further computations were then made and Manning's "n" values were adjusted to reproduce the 1975 and 1972 floodmarks and to retain the calibration of the April 1983 event.

Table B2 below lists the floodmarks for the 3 storms in terms of location, flood levels and the water surface elevations developed by the HEC-2 program. Figures B1 through B4 are profiles of the existing conditions for the project.

TABLE B2a
FLOODMARKS - APRIL 1983 STORM

RIVER	STATION	OBSERVED WATER SURFACE ELEVATION Ft. N.G.V.D.	COMPUTED WATER SURFACE ELEVATION Ft. N.G.V.D.
Mamaroneck	22+50	8.4	8.7
	25+70	10.0	10.2
	37+60	17.0	16.9
	38+00	17.4	17.0
	40+00	17.0	17.0
	41+10	17.9	17.7
	42+05	18.5	18.2
	42+70	18.7	18.2
	45+90	19.3	18.9
	46+25	19.3	18.9
	49+40	19.2	18.9
	66+70	22.2	22.4
	121+60	33.7	33.3
	126+80	35.5	35.1
	129+40	45.1	44.9
Sheldrake	1+00	19.3	18.9
	4+20	19.5	19.1

TABLE B2b
FLOODMARKS - SEPTEMBER 1975 STORM *

RIVER	STATION	OBSERVED WATER SURFACE ELEVATION Ft. N.G.V.D.	COMPUTED WATER SURFACE ELEVATION Ft. N.G.V.D.
Mamaroneck	22+50	11.9	11.6
	34+00	20.6	20.0
	36+25	20.4	20.2
	39+50	20.8	21.7
	50+00	25.0	24.9
	51+00	25.0	24.9
	52+50	25.1	24.9
	57+00	24.9	24.9
	65+50	27.2	26.3
	79+20	29.0	28.8
	98+50	31.0	31.4

* No floodmarks available for the Sheldrake River.

TABLE B2c
FLOODMARKS - JUNE 1972 STORM

Mamaroneck	39+00	21.2	21.6
	65+40	25.3	26.1
	121+60	35.9	35.6
Sheldrake	4+40	24.6	24.8
	8+40	24.3	24.8
	17+90	24.9	25.2
	19+20	24.9	25.2
	35+40	25.4	25.7

Because Mannings' "n" values are sensitive to changes in water depths a relative roughness (or "K" value) analysis was conducted. The "K" values were computed from the existing conditions calibration "n" values using the equation n=[R**(1/6)] /[21.9 log(12.2R/K)]. Once the "K" values were computed, different "n" values were determined for the design channel and depth, i.e. "K" values were held constant and "n" values were computed for varying depths. The computed "n" values were the same as the "n" values determined by the calibration runs, which is to be expected with the range of hydraulic radii encountered in these channels.

B10 Hydraulic Losses

a. Channel Work

The values of the Manning's roughness coefficients used are indicated in Table B3 below.

TABLE B3
MANNING'S ROUGHNESS COEFFICIENTS

Material	Manning's "n" value
Concrete	0.014
Rip-rap	0.035
Improved Channel	0.035
Exist. Channel	
cobbles & moderate vegetation	0.035
large cobbles & heavier vegetation	0.050
Overbanks, lawns	0.035
Overbanks, medium brush	0.060
Overbanks, dense brush	0.100

For natural channel cross sections the values of "0.1" and "0.3" were used as the contraction and expansion coefficients respectively. In the vicinity of bridges, the coefficients were increased to "0.3" and "0.5", as recommended in the HEC-2 Manual (Ref. 2) and "The Handbook of Hydraulics" by Brater and King (Ref. 3). In the area of the Station Plaza Bridge, the coefficients were further increased to "0.5" and "0.7" for existing conditions only, due to the extremely poor channel alignment.

b. Diversion Tunnel

Loss coefficients for the Sheldrake River diversion tunnel were selected based on a review of U.S.B.R. studies of prototype performance of similar structures (Ref. 4). Rugosity values for the concrete tunnel were determined to be 0.002 feet for capacity design and 0.00005 feet for velocity design. For use in the physical model these roughness heights translate into Manning's "n" value of 0.013 and 0.010 respectively. For a discussion of the capacity and velocity design considerations see paragraph B13 "Velocity and Capacity Design Considerations".

B11. Coincidental Tidal Stages and Fluvial Flows

The Mamaroneck River flows into the East Basin of Mamaroneck Harbor about 400 feet below the downstream limit of the proposed improvements. The Sheldrake River presently discharges into the Mamaroneck River just upstream of the Station Plaza Bridge. Once the diversion tunnel is in place the Sheldrake River will discharge directly into the West Basin of Mamaroneck Harbor. Tide levels inside the two basins correspond to levels in Long Island Sound, as derived from NOAA tide information.

An analysis was developed to determine the effect of tailwater conditions in the harbor on flood flows. Seventeen events were used from among the 24 highest annual peak discharges on the Mamaroneck River at the Halstead Avenue gaging station. Seven of the 24 events were initially eliminated from further consideration because there was no coincidental tidal gage or astronomical data available to conduct the analysis. The remaining seventeen historical flood events were used to identify time periods during which tidal elevations and surges were examined.

Gage tide elevations for the storm periods were obtained from NOAA for the Willet's Point, New York, station and astronomical tide elevations for the same station were determined from Tide Tables published yearly by NOAA, U.S. Department of Commerce (Ref. 5). Data from Willet's Point was applied directly to the harbor of Mamaroneck without adjustment because the correction factors were less than 2%.

Gage tides elevations were obtained at the time of peak flow and at +/- 1 hour from the time of peak flow. The tide elevation at the peak and the highest of the tide elevations at +/- 1 hour from the peak on the Mamaroneck River are plotted against peak flow in Fig. B12. No obvious correlation between flow and tide elevation was evident except that most of the tide levels appear to be higher than mean sea level. Tidal surges were calculated for the time of peak flow and +/- 1 hour from peak flow time on the Mamaroneck and Sheldrake Rivers. The peak flow on the Sheldrake was determined to occur two hours prior to the peak on the Mamaroneck River. In order to find representative surges for the various types of meteorological events, the storms were grouped into Hurricanes and Northeasters. Tidal surge data was then developed for the West Basin since under improved conditions the Sheldrake River Tunnel will discharge into it. Tables B4 and B5 present the maximum surges for the two hour period around peak flow and were found to be 2.24 ft. for hurricanes and 2.14 ft. northeasters.

A surge value of 2.2 feet corresponds to the average maximum surges experienced around the peak times of fluvial flooding. This average maximum surge was added to mean high water to yield a tidal tailwater elevation of 6.6 feet N.G.V.D. Since the astronomical tide is independent of atmospheric events, it is possible and likely for mean high water to occur during fluvial floods. The combination of an average maximum experienced surge with a high tide is considered a reasonable approach. Flowline computations for the Mamaroneck River and the diverted Sheldrake River were started using a 1 year tidal tailwater of 6.7 feet N.G.V.D. This elevation is equivalent to an ocean stage with a one year recurrence interval as developed in the New York City Flood Insurance Study, Total Stillwater Frequency Elevations Implementation and Results by CDM (Ref. 6).

TABLE B4
TIDAL SURGE COMPARISON OF MAMARONECK HARBOR FOR HURRICANES

EVENT DATE	SURGE AT -1 HR (ft)	SURGE AT PEAK FLOW (ft)	SURGE AT +1 HR (ft)	MAX of 2 HR PEAK PERIOD (ft)
19 Aug 60	1.95	1.35	1.07	1.95
29 May 68	2.20	1.90	1.99	2.20
28 Aug 71	2.64	3.70	1.37	3.70
19 Jun 72	1.32	0.93	0.43	1.32
26 Sept 75	0.38	0.94	0.34	0.94
10 Aug 76	-0.26	2.61	1.96	2.61
8 Nov 77	2.61	2.31	2.98	2.98
Average	1.55	1.96	1.45	2.24

TABLE B5
TIDAL SURGE COMPARISON OF MAMARONECK HARBOR FOR NORTHEASTERS

EVENT DATE	SURGE AT -1 HR (ft)	SURGE AT PEAK FLOW (ft)	SURGE AT +1 HR (ft)	MAX of 2 HR PEAK PERIOD (ft)
13 Mar 53	0.63	0.13	-0.47	0.63
15 Oct 55	3.83	4.36	3.93	4.36
16 Apr 61	1.87	1.42	1.42	1.87
25 Mar 69	0.58	0.60	0.49	0.60
10 Feb 70	1.89	0.89	1.35	1.89
21 Dec 73	0.12	2.83	1.07	2.83
21 Jan 79	1.20	2.46	2.51	2.51
4 Jan 82	2.21	2.42	2.12	2.42
Average	1.54	1.89	1.55	2.14

B12. Flowline Computation

A. Open Channels

Flowline computations were made to develop the hydraulic gradient of the streams in their existing condition and in the improved condition for the purpose of determining their hydraulic characteristics and establishing the extent of protection required. The computations are based on starting at a point of known energy (Mamaroneck Harbor) and determining the changes in the hydraulic gradient by the application of the laws of continuity and conservation of energy as described in EM 1110-2-1409, "Backwater Curves in River Channels" (Ref. 7). Flowline computations were accomplished with the use of the HEC-2 "Water Surface Profiles" computer program.

The hydraulic profiles for the existing and improved conditions of the Mamaroneck and Sheldrake Rivers are shown in Figures B1 - B8, a comparison is made in Table B6, and rating curves are provided in Figures B13 - B19. These figures represent the current degree of development in the basin, i.e. present urbanization. Since the watershed is highly developed and little if any future development is expected, these figures also represent the future urbanized conditions.

Flow levels for the improved Mamaroneck River converge with existing levels at the Westchester Joint Water Works Dam (approximately 500 feet upstream of the project) and will have no effect on the headwaters. Although the hydraulic computations of the Upper Sheldrake River end at the New York State Thruway bridge, the flowlines will converge at the Larchmont Lake Dam, approximately 800 feet upstream of the bridge. Lowering tailwaters downstream of the dam have no impact on this reach and it was not neccesary to compute the flowlines for the reach.

B. Diversion Tunnel

The Sheldrake River Diversion Tunnel was tested at the United States Army Corps of Engineers Waterways Experiment Station in Vicksburg, Ms. utilizing a 1:25 scale physical model. The model reproduces about 400 feet of approach channel, the ogee drop structure and converging approach of the tunnel inlet, the 3550 ft. tunnel, the stilling basin at the downstream end of the tunnel, and a portion of the West Basin of Mamaroneck Harbor. The model study was conducted using using two different Manning's "n" values, fresh water, and several stationary tide levels. Several investigations indicated that the presence of fluctuating brackish water in the harbor would not effect the flow conditions used to design the model. Other details of the model, tests, results and conclusions are contained in the WES report "Sheldrake

River Tunnel, Mamaroneck, NY, Hydraulic Model Investigation" (Ref. 8).

TABLE B6
MAMARONECK AND SHELDRAKE RIVERS
FLOWLINE COMPARISON IN FEET N.G.V.D.

LOCATION	SPF		100 YEAR		50 YEAR		2 YEAR	
	Exist	Imp	Exist	Imp	Exist	Imp	Exist	Imp
Mamaroneck at Mouth	6.7	6.7	6.7	6.7	6.7	6.7	6.7	6.7
Mamaroneck U.S. Tompkins Ave.	22.7	12.8	18.0	9.7	15.7	8.9	9.8	6.9
Mamaroneck at Sheldrake Conf.	37.7	22.0	32.1	18.3	29.6	16.9	19.0	11.5
Mamaroneck U.S. Thruway	40.6	33.5	35.8	28.9	34.5	27.3	26.9	21.9
Mamaroneck U.S. Winfield Ave.	42.2	35.7	40.2	32.6	39.7	31.5	34.8	26.8
Mamaroneck U.S. Waterworks Dam	50.1	50.1	48.8	48.8	48.3	48.3	47.0	45.7
Sheldrake at Mouth	37.7	22.0	32.1	18.3	29.6	16.9	19.0	11.5
Sheldrake U.S. Waverly Ave.	37.9	22.3	32.3	19.5	29.7	19.1	22.3	17.8
Sheldrake U.S. Tunnel	38.0	21.9	32.4	19.4	29.9	19.0	24.3	17.4
Sheldrake D.S. Thruway	39.0	31.2	33.6	29.0	30.9	28.6	26.6	26.0

B13 Velocity and Capacity Design Considerations

A. General

The design of the selected plan design was based on both capacity and velocity considerations.

For capacity design a rough "n" value was used to impede flow and therefore limit the hydraulic capacity of the structure. The hydraulic capacity is limited to insure that the design of a structure is sufficiently large to convey the design flow under adverse or deteriorated conditions.

For velocity design a smooth "n" value was used to minimize the energy losses. The energy losses are minimized to insure that the structure is designed to withstand the maximum expected energy.

B. Channels

The size, and shape of the channel was selected based on the capacity design. The rip-rap was also designed based on the capacity flowlines, however the rip-rap design was checked by decreasing the "n" values by 0.005. Decreasing the "n" values only had a minimal effect on the rip-rap design and it was then determined that a velocity design wasn't necessary and a single set of "capacity" flowlines would be used. See paragraph B19 "Channel Protection" for more details.

C. Tunnel

Selection of the tunnel system components used rough "n" values for sizing, shape, transitions, and wall heights. Smooth "n" values were used for development of the inlet drop structure and outlet stilling basin.

The capacity design was developed using the SPF flow of 4039 cfs combined with a tailwater of 6.7 feet NGVD in the West Basin. Backwater levels at the crest of the inlet drop structure were limited to 23.0 feet NGVD, which is the maximum energy level which will not cause damages in the reachs upstream. The velocity design was developed using the SPF flow of 4039 cfs combined with a tailwater of -2.4 feet NGVD (MLW) in the West Basin. Therefore energy dissipation during a minimum tailwater was assured.

The WES physical model study gives a complete description of the systems' performance. Tests confirmed the original design and recommended various improvements, largely in the tunnel transition and stilling basin dimensions.

B14 Sedimentation and Channel Stability

A study was conducted for the Mamaroneck and Sheldrake Rivers, the Diversion Tunnel, and Mamaroneck Harbor to identify any sediment related problems and project impacts on the sedimention process. The study also evaluated lateral and vertical stability. The study involved field investigations, field measurements and analysis, a historic topographic survey investigation and comparison, and sediment transport computations. Details of the sedimentation study are contained in the "Sediment Transport Analysis" report (Ref. 9). A summary of the procedures and conclusions follows.

A field reconnaissance was conducted on May 6, 1986 to examine both rivers from the upstream dams to Mamaroneck Harbor for vegetation, debris, watershed and/or streambank erosion, sedimentation, self-armoring, and other sediment transport indicators. Little evidence of bank erosion was found; trees and shrubberies near the top of banks were straight and well established. A few locations of minor sedimentation and scour were found. These locations (shown on Figures B20 thru B23) were noted and later used to confirm the validity of the historic comparisons and the sediment transport computations. Significant quantities of silt were found in the Mamaroneck Reservoir and the Larchmont Garden Lake but no sediment was found immediately downstream from the dams indicating that the dams serve as primary sediment sinks.

An investigation of past topographic mapping, cross-sections, and profiles from 1939 to 1982 indicated that the top of bank locations had changed little through natural means over the years but many changes were caused by bridge construction and channel improvements. To determine the vertical stability of the existing channels, a comparison was made between the measured 1975 and 1982 thalwegs, see Figures B20 thru B23. The use of a comparison between any other available surveys would have reflected too many man-made changes. The comparison indicated a few sections of minor scour and erosion which were in agreement with the areas noted in the field study.

The Laursen formula was selected as the most applicable formula for quantification of the sediment transport in both rivers under existing and improved conditions. It is applicable to streams with bed material ranging from silt to very fine gravel. Because the Laursen formula is based on empirical data from various streams over a large range of flows it is used with an average daily flow. Large flows are included in the empirical derivation of the formula but have a small effect on longterm sediment transport.

Each river was divided into several segments of approximately homogeneous flow, slope, and channel size, see Figures B20 thru B24. Tables B7 and B8 present the hydraulic characteristics of each segment of the rivers under both existing and improved conditions. In addition to the hydraulic data presented in these tables, data on the grain size distribution of the bed material was also used in the computation of the sediment discharge. The grain size distribution data was obtained from seven bed samples taken in 1984.

Using the Laursen formula, bed load sediment transport capacities under existing conditions were computed. The results are presented in Tables B9 and B10. The computed capacities do not reflect the actual sediment transport values, they only reflect the reach's potential to carry sediment. The capacities of the segments were compared to each other to identify potential areas of scour and deposition. When the transport capacity of a reach is less than the capacity of the next upstream reach, it is likely that deposition will occur. If the capacity of a reach is greater than the capacity of the next upstream reach, then scour is likely to occur. As shown in Table B9, Mamaroneck River Segments 2, 4, and 7 are likely to scour, and Segments 3, 5, and 6 are likely to shoal. This is consistent with the thalweg comparison and field investigation. Table B10 (Sheldrake River) shows Segments 2, 4 and 6 are likely to scour, and Segments 3 and 5 are likely to shoal. Once again this was verified by the thalweg comparison and field investigation, lending substantial credence to the analytical procedures used.

Potential sediment discharge capacities for improved conditions were computed using the improved hydraulic characteristics with the analytical procedures used for existing conditions. The results are shown in Tables B9 and B10. Once again the computed values do not correspond to the actual or expected sediment transport values but only the reach's potential to carry sediment and may be used for comparison purposes only. The improved potential sediment discharge capacities are greater than the existing capacities due to the improved hydraulic efficiency of the new channel. An increase in the sediment transport capacities implies that sedimentation will not be the problem, but that scour is likely to occur. Because the proposed project entails lining the channel bottom and side slopes with riprap where needed, scour will be prevented. Therefore both sedimentation and scour are not expected to occur in the new channel.

TABLE B7

SEGMENTATION OF THE MAMARONECK RIVER

SEGMENT	STATIONS	LENGTH (FT)	EXISTING CONDITIONS WIDTH (FT)	AVERAGE DEPTHa (FT)	SLOPE	IMPROVED CONDITIONS WIDTH (FT)	AVERAGE DEPTHa (FT)	SLOPE
1	109+55 to 116+11	656	36.0	1.80	0.00381	45.0	0.48	0.0020
2	77+11 to 109+55	3244	31.7	1.29	0.00247	45.0	0.48	0.0020
3	55+33 to 77+11	2178	33.7	1.41	0.00091	45.0	0.48	0.0020
4	47+44 to 55+33	789	30.2	0.95	0.00342	45.0	0.48	0.0020
5	37+00 to 47+44	1044	38.7	1.04	0.00268	45.0	0.48	0.0020
6	28+44 to 37+00	856	63.9	2.53	0.00058	60.0	0.41	0.0020
7	19+11 to 28+44	933	36.4	1.35	0.00322	60.0	0.41	0.0020
8	0+00 to 28+44	1911	37.0	b	0.00950	60.0	b	0.0020

a-Depth under average flow conditions. b-Fluctuates with tidal stages.

TABLE B 8

SEGMENTATION OF THE SHELDRAKE RIVER

SEGMENT	STATIONS	LENGTH (FT)	EXISTING CONDITIONS WIDTH (FT)	AVERAGE DEPTHa (FT)	SLOPE	IMPROVED CONDITIONS WIDTH (FT)	AVERAGE DEPTHa (FT)	SLOPE
1	69+33 to 76+44	711	25.0	0.44	0.00563	25.0	0.44	0.00563
2	62+89 to 69+33	644	25.0	0.39	0.00854	25.0	0.39	0.00854
3	34+22 to 62+89	2867	22.0	0.68	0.00163	40.0	0.28	0.00200
4	19+33 to 34+22	1489	18.0	0.64	0.00141	18.0b	0.64b	0.00141b
5	6+66 to 19+33	1267	17.7	0.88	0.00071	17.7b	0.88b	0.00071b
6	0+00 to 6+66	666	23.0	0.63	0.00390	23.0b	0.63b	0.00390b

a-Depth under average flow conditions. b-Diverted through new diversion tunnel.

TABLE B9

COMPARISON OF SEDIMENT DISCHARGES (a)

MAMARONECK RIVER

SEGMENT	STATION	SEDIMENT DISCHARGE (lb/sec/ft) EXISTING CONDITION	SEDIMENT DISCHARGE (lb/sec/ft) IMPROVED CONDITION	BED SHEAR VELOCITY (ft/sec) EXISTING CONDITION	BED SHEAR VELOCITY (ft/sec) IMPROVED CONDITION
1	109+55 to 116+11	0.0019	0.1659	0.470	0.178
2	77+11 to 109+55	0.0126	0.1659	0.320	0.178
3	55+33 to 77+11	0.0062	0.1750	0.203	0.178
4	47+44 to 55+33	0.0542	0.1622	0.323	0.178
5	37+00 to 47+44	0.0150	0.1622	0.300	0.178
6	28+44 to 37+00	0.0001	0.1204	0.217	0.162
7	19+11 to 28+44	0.0258	0.1204	0.374	0.162
8b	0+00 to 19+11	---	---	--	--

a-Computed using the Laursen formula. b-Not computed because reach is tidally influenced.

TABLE B10

COMPARISON OF SEDIMENT DISCHARGESa

SHELDRAKE RIVER

SEGMENT	STATION	SEDIMENT DISCHARGE (lb/sec/ft) EXISTING CONDITION	SEDIMENT DISCHARGE (lb/sec/ft) IMPROVED CONDITION	BED SHEAR VELOCITY (ft/sec) EXISTING CONDITION	BED SHEAR VELOCITY (ft/sec) IMPROVED CONDITION
1	69+33 to 76+44	0.0587	0.0587	0.282	0.282
2	62+89 to 69+33	0.0965	0.0965	0.327	0.327
3	34+22 to 62+89	0.0146	0.0590	0.189	0.134
4	19+33 to 34+22	0.0370	b	0.170	--
5	6+66 to 19+33	0.0103	b	0.142	--
6	0+00 to 6+66	0.0186	b	0.281	--

a-Computed using the Laursen formula. b-To be diverted through the new tunnel to the west basin.

The sedimentation potential of Mamaroneck Harbor has also been evaluated. In the "Sediment Transport Analysis" report (Ref. 9) a review of the historical dredging records from 1933 to 1981 indicate a siltation rate in the East Basin of about 2.3 in./yr. and a rate of about 0.8 in./yr. in the West Basin. Because the West Basin is almost entirely bulkheaded, and isn't fed by any tributaries, it was concluded that the siltation in the basin was due to tidal influences and not shoreline erosion or fluvial sediment sources. Therefore, the rate of siltation in the West Basin is a tidal rate and the higher rate of siltation in the East Basin can be attributed to river runoff. Computing the amount of sediment due to the combined flows of the two rivers is accomplished by subtracting the tidal rate of 0.8 in./yr. from the total rate of 2.3 in./yr. to get a rate of 1.5 in./yr. Assuming that the contribution of sediment is the same as the ratio of the flow rates, the siltation rates contributed by the Mamaroneck and Sheldrake rivers are 1.1 and 0.4 in./yr. respectively.

Since, under improved conditions, the Sheldrake River will be diverted into the West Basin, the siltation rate in the East Basin can be expected to decrease by 0.4 in./yr. while the rate in the West basin can be expected to increase by the same amount. Therefore, the post-project siltation rate in East Basin will decrease by about 17% to 1.9 in./yr. and the rate in the West Basin will increase by about 50% to a rate of 1.2 in./yr.

During the studies conducted in Reference 9, which was summarized above, a simultaneous analysis was conducted at the Waterway Experiment Station in order to get an independent, second evaluation of the sediment transport potential in the West Basin (Ref. 10). The volume of sediment delivered to the harbor by the river system was computed using regional river sediment yield information. The regional sediment yeild was found to be the same as the volume of deposition removed from both basins of the harbor through dredging. Because the volume of sediment delivered by the rivers was the same as the volume dredged it was assumed that there is no tidal transport of sediment in from Long Island Sound. It was also assumed that sediment delivered to the East Basin by the river systems ends up distributed between the two basins. A 14% percent transfer of sediment between the two basins in either direction was then computed. The increase in sediment deposition in the West Basin was then computed using the following formula:

$$X = S\,w\,(1 - p) + S\,(1 - w)\,p - S\,p$$

Increase in W. Basin	From the Tunnel	From the E. Basin	Previous Condition

where:

"X" is the increase in sediment deposition in the W. Basin
"S" is the total sediment supply
"w" is the percentage of sediment delivered to the W. Basin through the diversion tunnel
"p" is the percentage of sediment delivered to one basin that deposits in the other (14%).

The above method indicates that the shoaling rate in the West Basin may increase to a rate of 3100 C.Y./yr. which is about 2.3 in./yr. This rate and the rate obtained by the other method are of an order of magnitude that should not cause dredging problems.

Because the invert of the Sheldrake diversion tunnel ranges from 2.2 feet above mean sea level to 10.0 feet below mean sea level it is subject to tidal inundation. Therefore the sedimentation process in the tunnel will be similar to that in the West Basin. However the tunnel will be periodically subjected to high velocities caused from river flows. A 1 year flood in the Sheldrake River at low tide will result in velocities of about 5 to 8 fps. The bed shear stress caused by these flows range from about 5 to 12 lb/sq.ft which is far greater than the critical shear stress of 0.2 to 0.4 lb/sq.ft for the Sheldrake River sediment. It is concluded that any sedimentation in the tunnel will be flushed out into the West Basin at least once every year.

B15 Water Surface Profile Sensitivity

A. General

In conjunction with the sedimentation study, a sensitivity analysis was conducted to evaluate the effects of possible changes in channel roughness and geometry from the design conditions. Three different parameters were analyzed to determine their effects on the flowlines: "n" values, sedimentation and scour.

B. Manning's "n" value

For this analysis, "n" values were varied by factors of 1.25 and 0.75. As can be seen in Table B11, the computed water surface flow lines were not found to vary substantially. The relationship between the "n" values and changes in depth was also evaluated. As noted in paragraph B9 "Calibration" the "n" values are not significantly affected by the range of flows encountered in this project.

C. Sedimentation

The improved conditions runs were modified by raising the channel invert 1 and 2 ft. to simulate sedimentation along the entire channel bottom. For an analysis of the sedimentation processes, see paragraph B14, "Sedimentation and Channel Stability". The results of the analysis, contained in Table B11, indicate that the flow levels can be expected to rise approximately 1 ft. for every foot of sediment spread uniformly along the channel bed.

D. Scour

The improved conditions runs were modified by lowering the channel invert 1 and 2 ft. to simulate erosion along the entire channel bottom. For an analysis of the scour processes, see paragraph B14, "Sedimentation and Channel Stability". The results of the analysis, contained in Table B11, indicate that the flow levels can be expected to drop approximately 1 ft. for every foot of scour uniformly eroded along the entire channel bed.

B16 Freeboard

A. General Channel Freeboard

Freeboard for the Mamaroneck & Sheldrake channels is a minimum of 2.0 feet, which is in accordance with the freeboard selected in the feasibility report. Justification for the 2.0 feet of freeboard is based upon review of the conditions noted in EM 1110-2-1601, Reference 14. These conditions are:

1) Erratic hydrologic phenomena 2) Future development 3) Unforeseen embankment settlement 4) Accumulation of silt 5) Trash and debris 6) Aquatic or other growth 7) Variations in resistance 8) Consequences of damage

A description of the behavior of the Mamaroneck and Sheldrake Rivers to the above listed conditions follows:

TABLE B11
SENSITIVITY ANALYSIS OF FLOWLINES

STATION	FLOOD FREQUENCY	DISCHARGE (CFS)	IMPROVEMENTS -2 FT	-1 FT	BASE	+1 FT	+2 FT	1.25*N	0.75*N
					MAMARONECK RIVER				
31+80	2 YR	1254	7.24	7.70	8.41	9.29	10.09	8.43	8.40
	200 YR	6220	11.42	11.96	12.89	13.67	14.50	13.08	12.85
	SPF	7974	12.84	13.12	14.30	15.05	16.24	14.48	14.19
43+60	2 YR	1245	9.12	10.04	10.95	11.88	12.93	11.06	10.86
	200 YR	6200	16.76	17.77	18.51	19.09	20.22	18.64	18.42
	SPF	7930	18.92	19.93	20.68	21.08	22.24	20.80	20.56
58+45	2 YR	1208	11.87	12.86	13.78	14.85	15.84	13.79	13.78
	200 YR	6010	18.38	19.38	20.28	21.29	22.30	20.34	20.23
	SPF	7723	20.63	21.88	22.96	23.50	24.38	23.03	22.90
104+00	2 YR	1185	21.82	22.79	24.36	25.28	25.73	24.36	24.36
	200 YR	5860	28.63	29.47	30.74	31.17	31.95	30.79	30.66
	SPF	7549	30.44	31.48	32.41	33.31	34.60	32.49	32.30
					SHELDRAKE RIVER				
38+99	2 YR	752	17.52	17.52	17.52	17.46	17.29	17.52	17.52
	200 YR	2960	20.98	20.98	20.98	20.77	20.34	20.98	20.98
	SPF	4039	22.25	22.25	22.25	21.99	21.48	22.25	22.25
49+70	2 YR	752	18.79	19.60	20.38	21.57	22.57	20.38	20.38
	200 YR	2960	23.83	23.64	24.44	25.57	26.55	24.44	24.44
	SPF	4039	24.39	25.08	25.84	26.98	28.15	25.84	25.83
60+50	2 YR	752	20.94	21.92	22.66	23.83	24.75	22.67	22.67
	200 YR	2960	25.84	26.74	27.34	28.47	29.36	27.37	27.34
	SPF	4039	27.64	28.50	29.11	30.26	31.22	29.15	29.11

1) There are no erratic hydrologic phenomena associated with these rivers since the flow rates increase with increases in drainage area, and these streams have not demonstrated any large variations in flood potential when compared with other streams in the area. The development of the flows for these rivers is normal.

2) Future urban development is expected to be minimal and no adjustment to the flows was made. If development were to occur for some unforeseen reason, an increase in flows corresponding only to an increase of 0.25 feet in stage would be possible.

3) Levees are not specified for this project so settelment of fill or embankment material isn't an issue.

4) Accumulation of silt is expected to be minimal as stated in paragraph B14 "Sedimentation and Channel Stability".

5) There is some indication of trash and debris accumulation in the existing channels. Realignment is expected to reduce any adverse effects from that source.

6) Because the calibration and verification events occurred in April, September, and October a fair range of seasonal variations in vegetation were accounted for in the simulation.

7) The Mannings' "n" values and expansion / contraction coefficients used in determining the flowlines are very reliable due to the number of floodmarks obtained and the number of storms used.

8) There are no significant drainage divides in the overbanks, so overtopping the specified channel will not result in an abrupt change in the damage levels as experienced with dam and levee designs. In addition the level of protection for this project is very high (200 yr. and SPF) and exceeding the design level will not cause a sudden increase in the damages because such a large storm will have already exceeded the capacity of the local drainage systems. Also the floodwaters on both rivers rise slowly providing ample flood warning.

In summary, a review of the revelant factors indicates that 2 feet of freeboard is adequate for this design.

B. Bridges

Freeboard at the bridges was made on an individual basis and varies with the existing roadway geometry. In general, for a reconstructed bridge, freeboard is a minimum of 3.0 feet as stated in the feasibility report. For bridges which remain in place under improved conditions, freeboard may be less than 3.0 feet because it is not economically feasible to reconstruct those bridges. (Table B1 contains freeboard information).

C. Special Freeboard Conditions

The tunnel inlet and outlet structures will have a minimum of 5 feet of freeboard to account for possible wave patterns, unpredictable eddying, and the substantial consequences should the capacity be exceeded.

III. HYDRAULIC DESIGN OF IMPROVEMENTS

B17 Detailed Description of the Selected Plan

A. Mamaroneck River

The Mamaroneck River channel improvements include channel enlargement, stream realignment, bridge replacement, and mitigation measures for fish and wildlife. Downstream of Tompkins Avenue the channel will have a bottom width of 45 feet for a length of about 200 feet. Further enlargement would threaten the residential development along the banks. The 3,000 foot length between Tompkins and Jefferson Avenues will have a bottom width of 60 feet. At Jefferson Avenue the bottom will be widened to 71 feet to utilize the existing bridge opening. Upstream of Jefferson Avenue the remaining 7400 feet of channel will vary between 45 and 50 feet in bottom width.

The channel will involve excavation depths of 2 to 5 ft., a uniform improved slope of 0.2 % , and a trapezoidal cross section with 1:2.5 side slopes. Where limited by structural development 1:2 side slopes will be used. Vertical retaining walls will be specified at the following locations:

1. Through all bridges
2. Along the right bank above the Tompkins Avenue Bridge
3. Along the left bank above the Ward Avenue Bridge
4. Along both banks from below the Valley Place Bridge to the Station Plaza Bridge
5. Along both banks at the Hillside Avenue Bridge

6. Along the right bank below the North Barry Avenue Bridge

7. Along the left bank above the North Barry Ave. Bridge

8. Along the right bank below the New England Thruway Bridge.

The channel will be realigned at the "S" turn between the railroad bridge and the Jefferson Avenue bridge. It will also undergo realignment between the New England Thruway bridge and the Winfield Avenue bridge.

The Fish and Wildlife mitigation measures that have been incorporated consist of a 0.5 foot high log dam, 2 boulder fields, 2 rock dams which are also 0.5 feet high and 7315 feet of sub-channel. The sub-channel is a "V" shaped channel 1.5 feet deep with 1 on 3 side slopes that meanders within the bottom of the proposed main channel. These structures will be submerged under normal conditions and do not detract from the capacity of the system.

The U.S.G.S. gaging station which is presently located just downstream of the Halstead Ave. bridge will be removed and replaced at Bradley St. about 700 feet downstream of the New England Thruway bridge.

Table B1 lists the proposed bridge improvements on the Mamaroneck and Sheldrake Rivers. Figures B5 & B6 are profiles of the Mamaroneck River improvements.

B. Upper Sheldrake

The Upper Sheldrake River channel improvements extend from the Fenimore Road bridge to the Rockland Ave. bridge for a length of about 1,500 feet. They include channel enlargement, stream realignment and diversion, and mitigation measures for fish and wildlife.

The main channel will be deepened an average of 1 foot, widened to a bottom width of 40 feet and constructed at a uniform slope of 0.2 %. The channel will be trapezoidal in cross section with 1:2.5 side slopes. Where limited by structural development, 1:2 sideslopes will be used with a vertical retaining wall specified along the right bank downstream of the Rockland Avenue bridge.

None of the bridges on this river require replacement. The existing channel downstream of the Fenimore Road bridge will be filled and regraded to facilitate local runoff. Figures B7 & B8 are profiles of the channel improvements.

The mitigation measures include a 0.5 foot high log dam and a "V" shaped sub-channel which is 1.5 feet deep with 1 on

3 side slopes meanders within the bottom of the proposed main channel. These measures will not impact the channel capacity.

In addition clearing and snagging and a debris structure will also be incorporated into the project. The clearing and snagging will extend from the Rockland Ave. bridge to the Larchmont Gardens Lake Dam. The debris structure will be placed on the upstream face of the Rockland Ave. bridge.

C. Sheldrake River Diversion

The river diversion consists of a tunnel system running beneath Fenimore Road from the Sheldrake River to the West Basin of Mamaroneck Harbor. This system, which is approximately 4010 feet in length, is comprised of an inlet structure, the tunnel works and an outlet structure. Figure B9 is a tunnel profile.

1. The inlet structure (Figure B10) consists of the following:

   a. An inlet weir with a crest width of 60 feet and crest height 2.5 feet above the approach channel.

   b. A vertical invert drop of 12.2 feet from the weir crest to toe.

   c. A 175 foot long transition from a bottom width of 60 feet to a bottom width of 16.25 feet at the tunnel entrance.

2. The tunnel section (Figure B9) is 3550 feet long and consists of the following:

   a. A 1625 feet long section of tunnel that is square in cross-section, 16.25 feet high, and 16.25 feet wide.

   b. A 1817 feet long section of tunnel that has a U.S.B.R. horse-shoe shape and a 17.5 foot radius.

   c. A 108 foot long transition section between the square and horseshoe shapes.

3. The outlet structure (Figure B11) is 212 feet long and consists of the following:

   a. An outlet section 75 feet long from the 17.5 foot horseshoe conduit to the 17.5 foot rectangular section.

   b. A 90 foot long transition and drop from the 17.5 foot width to the 45 foot wide stilling basin.

c. A stilling basin and end sill 47 feet long and 45 feet wide.

Cut and cover methods will be used to construct the square shape. The horseshoe will be constructed using drill and blast methods. See the tunnel profile, Figure B9 for more information. An 84 degree horizontal bend will be at the upstream end and two smaller horizontal bends occur as the tunnel follows the alignment of Fenimore Road. The tunnel discharges into a stilling basin before emptying into the West Basin of Mamaroneck Harbor.

D. Lower Sheldrake River

The Lower Sheldrake River improvements extend from Mamaroneck Ave. to Fenimore Road for a length of 2660 feet. In this reach the existing channel will be filled about 1.5 feet to form a trapezoidal shape with a 10 foot bottom width, 1 on 3 side slopes, and a uniform bottom slope of 0.2%. The fill will be entirely within the existing channel and will not extend to the top of the existing bank. The channel will convey flow from the drainage area below the tunnel diversion along with the flow of a small tributary that passes under the New England Thruway.

A small concrete weir about 6 feet in height and 38 feet in length will be placed 20 feet downstream of the downstream footbridge. The weir will provide a small natural pool in the Station Plaza Park as requested by The Fish and Wildlife Service.

B18 Level of Protection

The design frequency for the Mamaroneck and Sheldrake Rivers is the 200 year flood, and the Standard Project Flood for the Sheldrake River Diversion Tunnel. Table B12 lists the fluvial levels of protection for various project conditions. Figures B13 through B19 are the rating curves of several stations throughout the project.

Figures B17 and B17a are a rating curve and a stage-frequency curve, respectively, for the Sheldrake River at station 8+79. Because station 8+79 is located downstream of the proposed tunnel diversion point and it is subjected to much less flow under improved conditions, therefore that section of the river has been filled and regraded to facilitate the small flows expected. Because of the fill, the flow capacity of the section has been reduced as seen in Figure B17, but as a result of the diversion the level of protection has been greatly increases as seen in Figure B17a.

TABLE B12
FLUVIAL LEVELS OF PROTECTION

RIVER	STATION	IMPROVED COND. LEVEL OF PROTECTION	FLOW (cfs)
Mamaroneck	19+00	200 YR.	6220
	36+00	200 YR.	6220
	58+00	200 YR.	6010
	67+00	200 YR.	6010
	104+00	200 YR.	5860
Sheldrake	1+00	200 YR.	290
	10+00	200 YR.	140
	52+00	SPF	4039
	60+00	200 YR.	4039

B19. Channel Protection

The criteria for the protection of the banks and bottom of the earth channel against erosion are based on guidelines contained in: ETL 1110-2-60, "Criteria for Riprap Channel Protection", 13 June 1968 (Ref. 11); ETL 1110-2-120, "Additional Guidance for Riprap Channel Protection", 14 May 1971 (Ref. 12): Miscellaneous Paper H-78-7, "Practical Riprap Design", June 1978 (Ref. 13); and EM 1110-2-1601, "Hydraulic Design of Flood Control Channels", 1 July 1970 (Ref. 14). The riprap was designed using maximum side slopes, and minimum bend radii. Several methods were used to calculate the layer thicknesses and the largest reasonable thickness was selected. The riprap "n" values were decreased by a value of 0.005 to determine if the riprap design would be affected by these changes. The average channel velocities increased by less than 0.3 fps which would change the size of the riprap layer by no more than 3 inches. As a practical matter a change of 3" is considered too small to require a revised riprap analysis. The location and size of the required riprap is indicated on the profile and plan sheets. Table B13 is a list of channel velocities for existing and improved conditions. Note that most drainage pipes and ditches will receive a 12" layer of riprap at the point where they meet an improved side slope. Tables B14 and B15 specify the riprap requirements.

TABLE B13
CHANNEL VELOCITIES IN F.P.S.

RIVER	STATION	IMPROVED CONDITIONS	EXISTING CONDITIONS
Mamaroneck	18+00	9.9	9.0
	28+00	9.2	7.6
	36+00	6.8	7.3
	38+00	10.2	5.3
	40+00	10.1	7.8
	42+00	9.3	4.6
	44+00	6.8	1.6
	46+00	5.5	1.6
	50+00	7.5	2.0
	60+00	9.0	5.0
	71+00	8.9	3.8
	83+00	5.7	7.6
	110+00	7.0	3.7
	120+00	9.0	4.3
	123+00	4.0	3.3
	133+00	2.5	2.3
Sheldrake	40+00	7.2	1.8
	45+00	8.9	0.6
	50+00	7.5	1.3
	55+00	7.0	1.3

TABLE B14
RIPRAP REQUIREMENTS *

RIVER	STATION FROM	STATION TO	RIPRAP THICKNESS
Mamaroneck	17+00	18+50	18"
	18+50	20+00	12"
	20+00	21+40	36"
	21+40	23+80	24"
	28+60	29+00	18"
	29+00	31+70	30"
	31+70	33+20	24"
	36+80	40+80	24"
	42+30	43+55	12"
	56+80	59+60	18"
	60+60	60+80	12"
	60+80	62+70	30"
	62+70	62+90	12"
	114+60	119+00	12"
	119+00	121+20	18"
Lower Sheldrake	1+65	1+90	12"
Upper Sheldrake	39+00	39+65	12"
	44+30	49+90	24"
	49+90	52+65	18"
	52+65	53+15	36"
	53+95	54 25	36"
	54+25	54+50	24"
	54+50	55+75	12"
	61+00	61+45	24"

* Notes: A 12" layer of riprap is provided at most of the drainage out falls.

TABLE B15
RIPRAP SPECIFICATIONS

Layer Thickness	D100 (lbs.) max	D100 (lbs.) min	D50 (lbs.) max	D50 (lbs.) min	D15 (lbs.) max	D15 (lbs.) min
12"	86	35	26	17	13	5
18"	292	117	86	58	43	18
24"	691	276	205	138	102	43
30"	1350	540	400	270	200	84
36"	2333	933	691	467	346	146

B20 Downstream Effects

A) General

Both rivers will be discharging directly into Mamaroneck Harbor. Because both basins of Mamaroneck Harbor are tidally controlled, the project will not increase the flood levels of the harbor. The impacts of the project on sedimentation been reviewed in paragraph B10 and discussed in detail in References 9 and 10. Reference 10 also examines the scour potential in the harbor and the impacts on navigation. No impacts to the East Basin are expected but a summary of the impacts to the West Basin is written below.

B) Erosion

The scour potential for the West Basin was defined by combining sediment characteristics with the hydraulic information obtained from the WES physical model. Characteristics such as grain size and critical shear stresses were obtained from the analysis of several grab samples. The model current velocities were converted to shear stresses along the bed and then compared to the critical shear stress for the bed material. The maximum computed shear stress was 0.159 psf and occurred near the tunnel outlet, using a peak surface velocity during mean low water (-2.7 feet NGVD) with the Standard Project Flow of 4039 cfs. The computed shear stresses were used to conservatively compute the erosion over the discharge hydrograph of the Standard Project Flood during a constant mean low tide although the tide will remain at low water levels only a short time. The resulting depth of erosion was 3.5 inches, which is a minimal amount of erosion and eliminated the need for any further analysis.

The scour potential for the East Basin is expected to decrease or remain the same because less flow reaches the basin as a result of the diversion tunnel.

C) Sedimentation

The sedimentation potential has been addressed in paragraph B14 "Sedimentation and Channel Stability"

D) Navigation

The potential impacts to navigation have also been evaluated. The problems addressed were navigation into a strong current, navigation with a strong current, and vessels docked & moored in a strong current.

To evaluate the impact of navigating into a strong current the critical speed of several crafts was calculated. The critical speed of a craft is the speed at which the craft will not be able to make headway under its' own power for a given current condition. The analysis also involves the use of a blockage factor which is a way of defining the extent to which the current is confined by a boat. In general the lower the blockage factor the higher the critical speed will be.

Using the extreme case of a -2.7 foot tide with the SPF flow, a maximum current speed of 2.6 fps was observed at the confined throat of the basin. The throat of the basin is approximately 300 feet wide and about 7.5 feet deep. The computed craft size that would experience critical speed is 29 feet wide with a 7 foot draft. A vessel of that size would impede flow in the confined throat to a point that it would effectively raise the current velocities in the area. Fortunatly vessels of that size do not frequent these waters. Therefore vessels in the basin will not experience problems navigating against the current.

Evaluating the impact of navigating with a strong current involves relative rudder speeds, individual boat designs, and individual seamanship. All the vessels in the basin can navigate at cruising speeds in open waters with a 2.6 fps current; however it will be difficult to operate at docking speeds when a current is pushing the vessel. It is believed that the skipper will maneuver the boat into the current before attempting to dock, which will involve passing the dock or mooring at moderate speed, proceeding to an open area, and then maneuvering the boat to approach the dock from down current. Facing a boat into the wind or current before docking is common practice and does not represent a problem in the design. It is unlikely that any vessels will be operating during an SPF event.

The docking and mooring arrangement near the outlet to the tunnel will have to be changed. The floating docks and mooring anchors can easily be rearranged to orient the boats in a position and direction that will not cause excessive motion.

B21 Storm Drainage

An inventory was conducted of all culverts, drainage ditches, gutters and tributaries within the project area. The drainage facilities that presently discharge into the Mamaroneck and Sheldrake Rivers by gravity will continue to do so. The lengths of these drainage facilities will be adjusted and new headwalls will be constructed as required. Where the drainage facility discharges onto an unprotected improved sideslope, a 12" layer of riprap will be provided. A "V" shaped drainage ditch, 0.5 ft. deep with 1 on 3 sideslopes and a 3.0 ft. topwidth will be placed behind all retaining walls to convey runoff to 2' x 3' drop inlets. These drop inlets will prevent the retaining walls, normally 6" above grade, from being overtopped repeatedly. These drainage facilities are shown on the 1"=30' plan sheets (Plates).

B22 Sanitary Pipe Crossings

Within the study area there are 11 existing sanitary pipe crossings on the Mamaroneck River and none on the Sheldrake River. They range in size from 6" to 48". Table B16 contains the description of the crossings together with a brief description of the proposed improvements.

B23 Pre and Post Project Comparisons

The improvements on the Mamaroneck River and the Lower Sheldrake River provide a 200 year level of protection. The upper Sheldrake River provides protection for the SPF event. For these design flows, the waters are essentially confined to the channel banks as shown on the inundation maps (Figures B24 thru B44). However under existing conditions only the 2 year event is confined to the banks.

TABLE B16
SANITARY CROSSINGS

MAMARONECK RIVER

STATION	LOCATION	SIZE	REMARKS
15+40	E. Boston Post Rd.	6"	To remain under existing bridge
23+20	Tompkins Ave	8"	To remain under new bridge
32+25	Ward Ave	6"	Crossing to be eliminated and to be reconnected to 66" line under Ward Ave.
37+60	Valley Place	48"	Sewer bridge to be replaced
40+60	D/S Halstead Ave	20"	Replace with a new siphon
48+40	Jefferson Ave	21"	Replace with a new siphon
54+20	Willow St	8"	Replace with a lift station
59+05	Hillside Ave	8"	Crossing to be eliminated and to be reconnect to 66" line under Hillside Ave.
56+10 to 62+50		66"	Realign trunk line away from new bank slope
64+10	Howard Ave	12"	Replace with a new siphon
71+70	N. Barry Ave.	2 x 6"	Remove siphon and relocate line to River St.
77+90	River St	2 x 8"	Replace with new a siphon
80+80	Bradley Ave	8"	Crossing to be eliminated
84+80	D/S NYS Thruway	10"	Replace with new a siphon
101+00	Right Bank	10"	Realign line away from new bank slope
109+90	Ellis Place	10"	Replace existing siphon with a lift station

* There are no changes to the crossings on the Sheldrake River.

SHELDRAKE RIVER

STATION	LOCATION	SIZE	REMARKS
2+22	Boston Post Rd.	8"	New pipe in sleeve in tunnel ceiling
19+55	Stanley Ave.	8"	Rerouted to connect to an existing 36" pipe
19+55 27+52	Fenimore Rd.	10"	Rerouted to connect to an to existing 36" pipe
27+52	Waverly Ave.	10"	New pipe in sleeve in tunnel ceiling
32+82	Fayette Ave.	10"	Replace existing pipe
32+82 35+38	Fenimore Rd.	10"	Relocate parallel to Fenimore Rd.
35+38	Northrup Ave.	10"	Relocate parallel to the tunnel
36+15	Sheldrake River	10"	Relocate under Fenimore Rd. Bridge

B24 Operation and Maintenance

The main features of the recommended project that include routine operation and maintenance are as follows:

1. 16,900 feet of channelization that may require shoal and debris removal and replacement of lost riprap.

2. Concrete retaining walls that require periodic inspection for repair of any cracking and toe scour.

3. 4010 feet of tunnel system which is self cleaning (see paragraph B14 "Sedimentation and Channel Stability") will require periodic inspection for cracking, wear, and other possible maintenance needs.

4. The two dams upstream of the project area on the Sheldrake River and the small weir at the entrance to the tunnel must be periodically dredged to remove trapped sediment. These three structures trap sediment which could possibly deposit in the tunnel.

5. The Debris Deflector on the upstream face of the Rockland Ave. bridge must be periodically inspected for debris removal and possible repair.

B25 Care of Water During Construction

To minimize potential environmental impacts, it is recommended that the following measures be taken during construction:

Construction activities in the existing channel should be scheduled during periods of low flow as much as practical.

Sediment control devices (such as silt screens, staggered channel obstructions, siltation basins, and alternate side construction) should be employed during construction to prevent erosion into the river.

New York State water quality standards must be maintained during construction.

The contractor must minimize the removal of vegetation associated with the construction and maintain of the project facilities, including related transmission lines.

The contracting agency should work closely with the construction contractor, the Corps of Engineers, and local authorities for approval of mutually agreeable erosion control measures and an Environmental Protection Plan prior to initiation of construction.

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11. U.S. Army Corps of Engineers, ETL 1110-2-60, Criteria for Riprap Channel Protection, June 1968

12. U.S. Army Corps of Engineers, ETL 1110-2-120, Additional Criteria for Riprap Channel Protection, May 1971

13. U.S. Army Corps of Engineers, WES, MP-H-78-7, Practical Riprap Channel Design, June 1978

14. U.S. Army Corps of Engineers, EM 1110-2-1601, Hydraulic Design of Flood Control Channels, July 1970

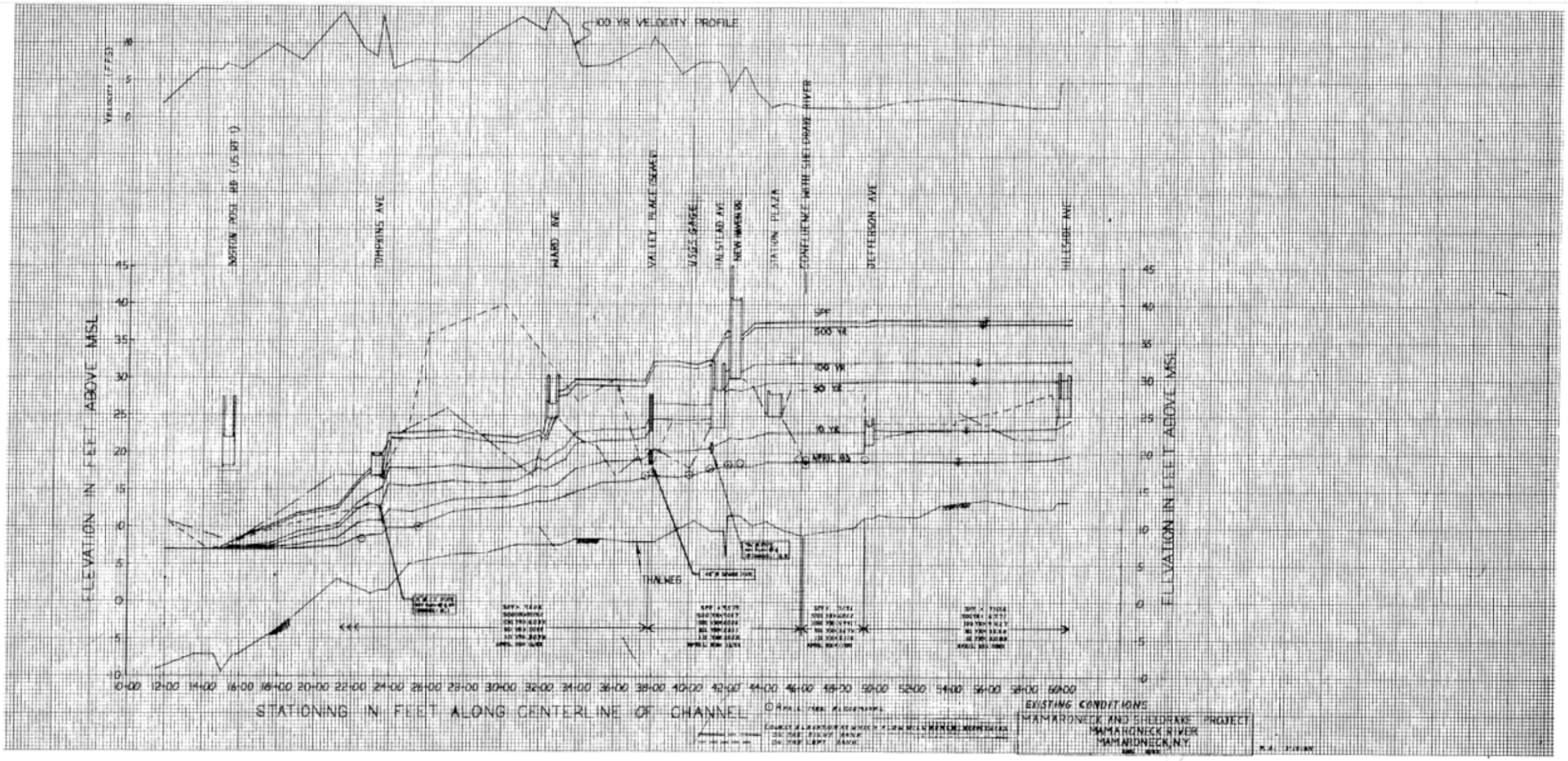


FIGURE B1

FIGURE B1

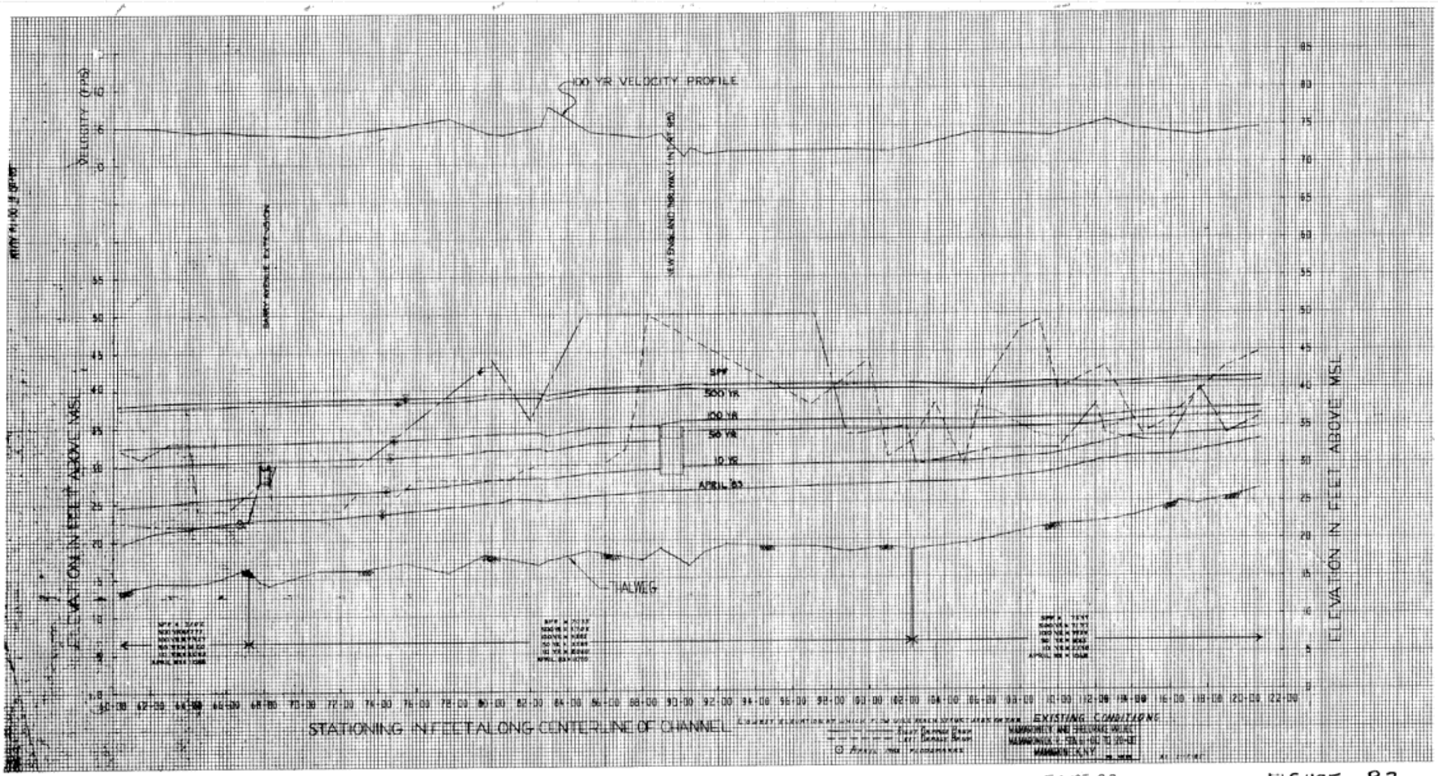


FIGURE B2

FIGURE B2

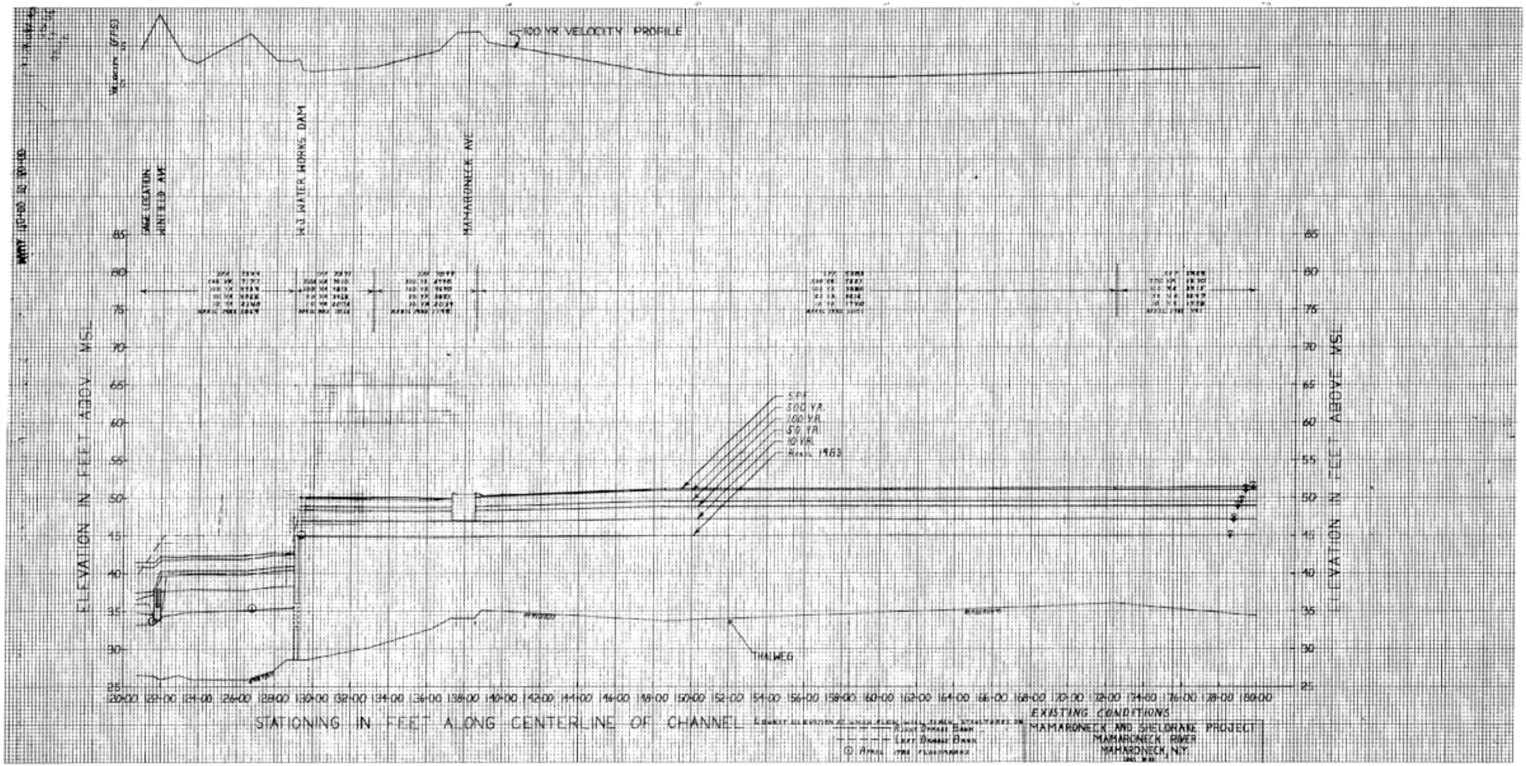


FIGURE B3

FIGURE B3

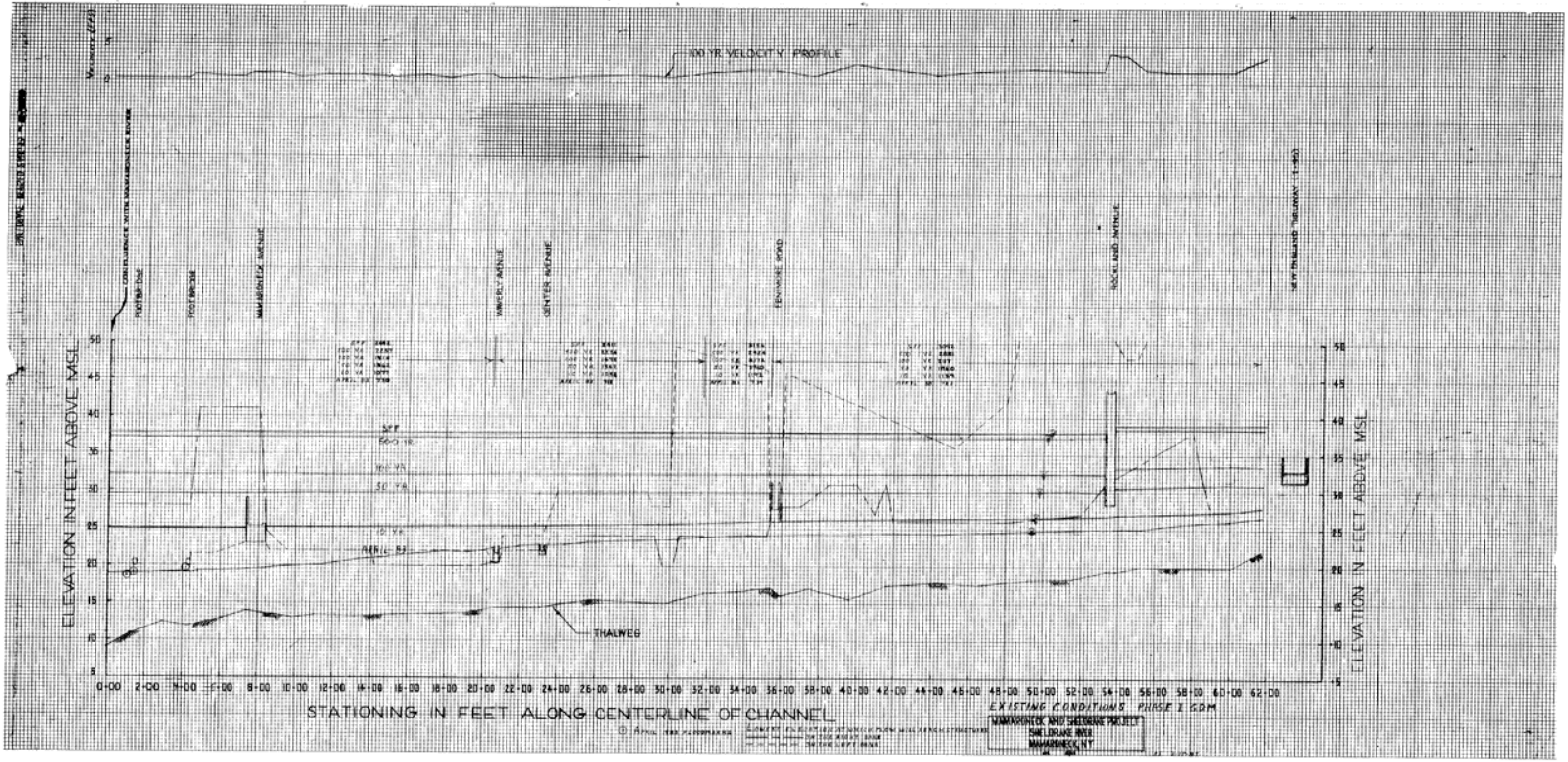


FIGURE B4

FIGURE B4

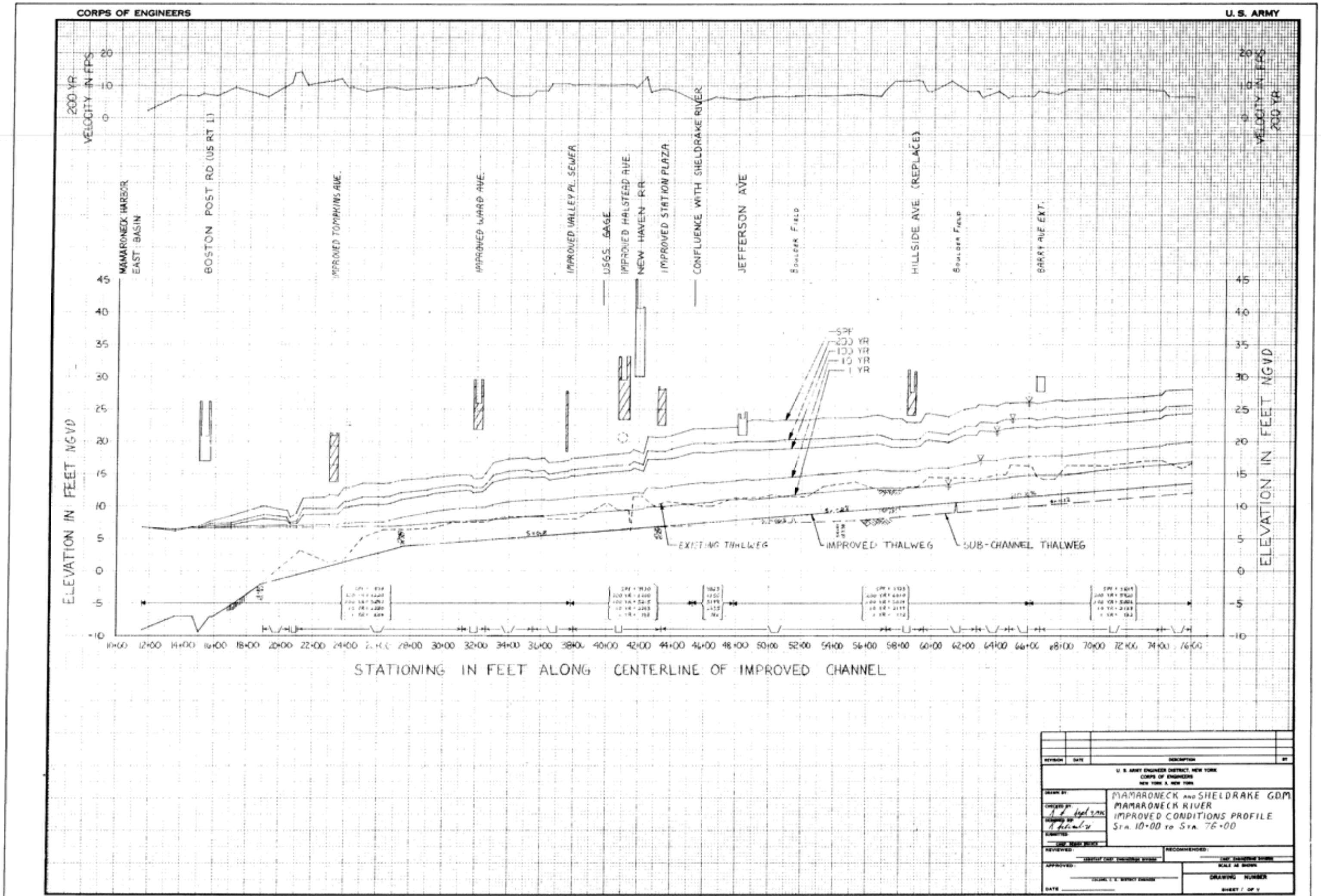
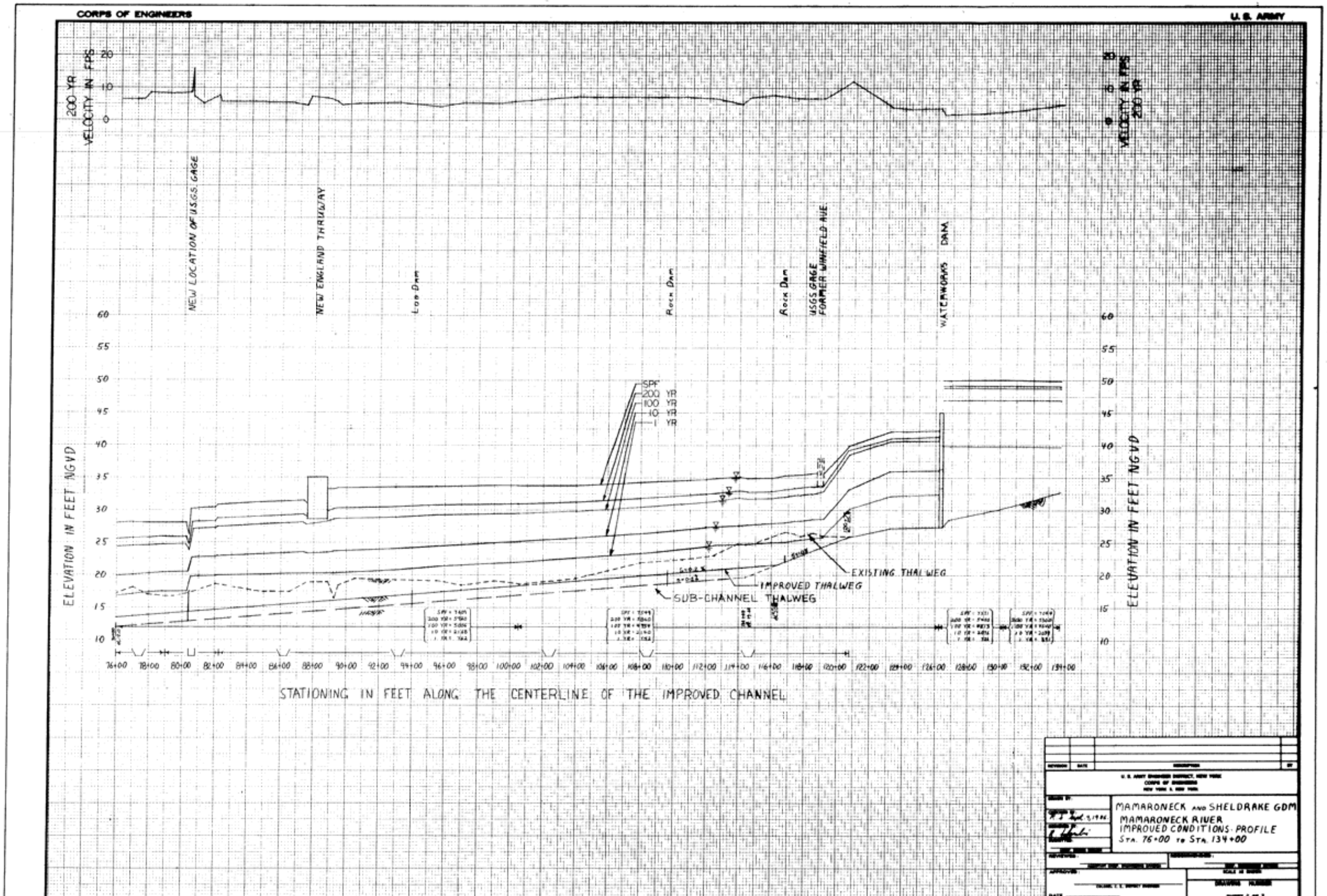

CORPS OF ENGINEERS
U. S. ARMY
200 YR VELOCITY IN FPS
VELOCITY IN FPS 200 YR
MAMARONECK HARBOR EAST BASIN
BOSTON POST RD (US RT 1)
IMPROVED TOMPKINS AVE.
IMPROVED WARD AVE.
IMPROVED VALLEY PL. SEWER
USGS GAGE
IMPROVED HALSTEAD AVE.
NEW HAVEN R.R.
IMPROVED STATION PLAZA
CONFLUENCE WITH SHELDRAKE RIVER
JEFFERSON AVE
BOULDER FIELD
HILLSIDE AVE. (REPLACE)
BOULDER FIELD
BARRY AVE EXT.
SPF
200 YR
100 YR
10 YR
1 YR
EXISTING THALWEG
IMPROVED THALWEG
SUB-CHANNEL THALWEG
ELEVATION IN FEET NGVD
STATIONING IN FEET ALONG CENTERLINE OF IMPROVED CHANNEL
U. S. ARMY ENGINEER DISTRICT, NEW YORK
CORPS OF ENGINEERS
NEW YORK, N. NEW YORK
MAMARONECK AND SHELDRAKE GDM
MAMARONECK RIVER
IMPROVED CONDITIONS PROFILE
STA 10+00 TO STA 76+00
DRAWING NUMBER

FIGURE B5



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200 YR VELOCITY IN FPS
ELEVATION IN FEET NGVD
NEW LOCATION OF U.S.G.S. GAGE
NEW ENGLAND THRUWAY
Log Dam
Rock Dam
Rock Dam
USGS GAGE
FORMER WINFIELD AVE.
WATERWORKS DAM
SPF
200 YR
100 YR
10 YR
1 YR
EXISTING THALWEG
IMPROVED THALWEG
SUB-CHANNEL THALWEG
STATIONING IN FEET ALONG THE CENTERLINE OF THE IMPROVED CHANNEL
U.S. ARMY ENGINEER DISTRICT, NEW YORK
CORPS OF ENGINEERS
NEW YORK, N. NEW YORK
MAMARONECK AND SHELDRAKE GDM
MAMARONECK RIVER
IMPROVED CONDITIONS PROFILE
STA. 76+00 TO STA. 134+00

FIGURE B6

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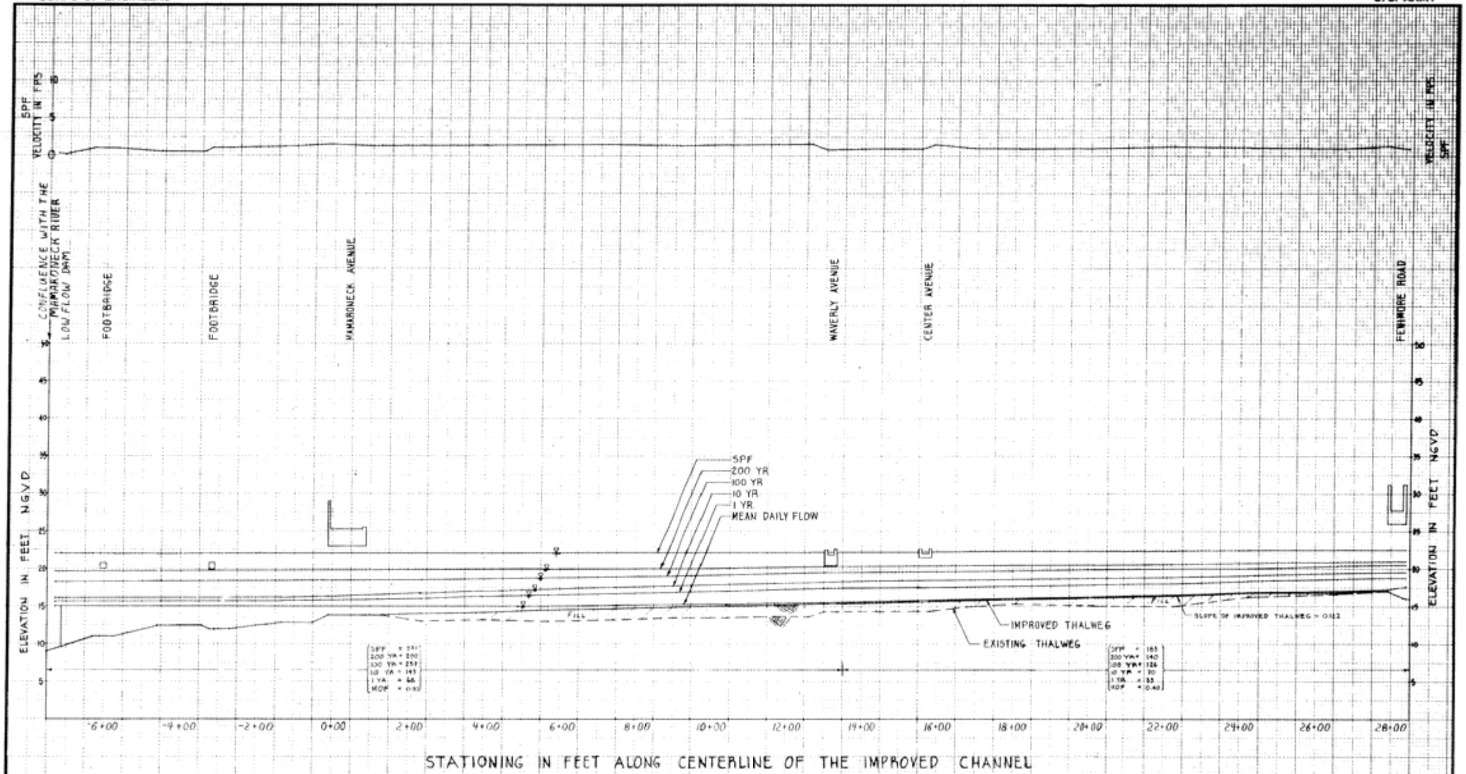
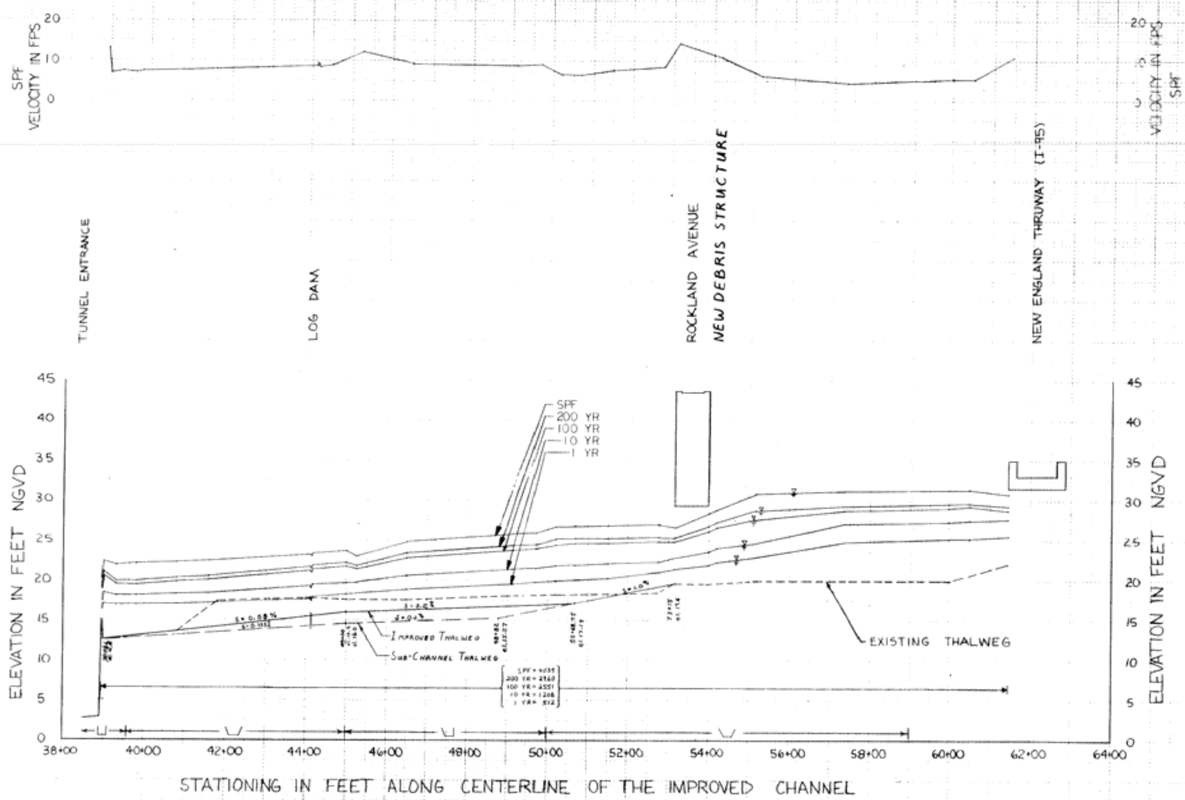


U. S. ARMY ENGINEER DISTRICT, NEW YORK
CORPS OF ENGINEERS
NEW YORK, N. NEW YORK

MAMARONECK AND SHELDRAKE GDM
LOWER SHELDRAKE RIVER
IMPROVED CONDITIONS PROFILE
STA. -7+50 TO STA. 28+50

FIGURE B7



REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, NEW YORK
CORPS OF ENGINEERS
NEW YORK 3, NEW YORK

DRAWN BY:

CHECKED BY: R. A. Sept 10, 1986

DESIGNED BY: R. Schoenberg

SUBMITTED:
CHIEF, DESIGN BRANCH

MAMARONECK AND SHELDRAKE GDM
UPPER SHELDRAKE RIVER
IMPROVED CONDITIONS PROFILE
STA. 38+00 TO STA. 64+00

REVIEWED:
ASSISTANT CHIEF, ENGINEERING DIVISION

RECOMMENDED:
CHIEF, ENGINEERING DIVISION

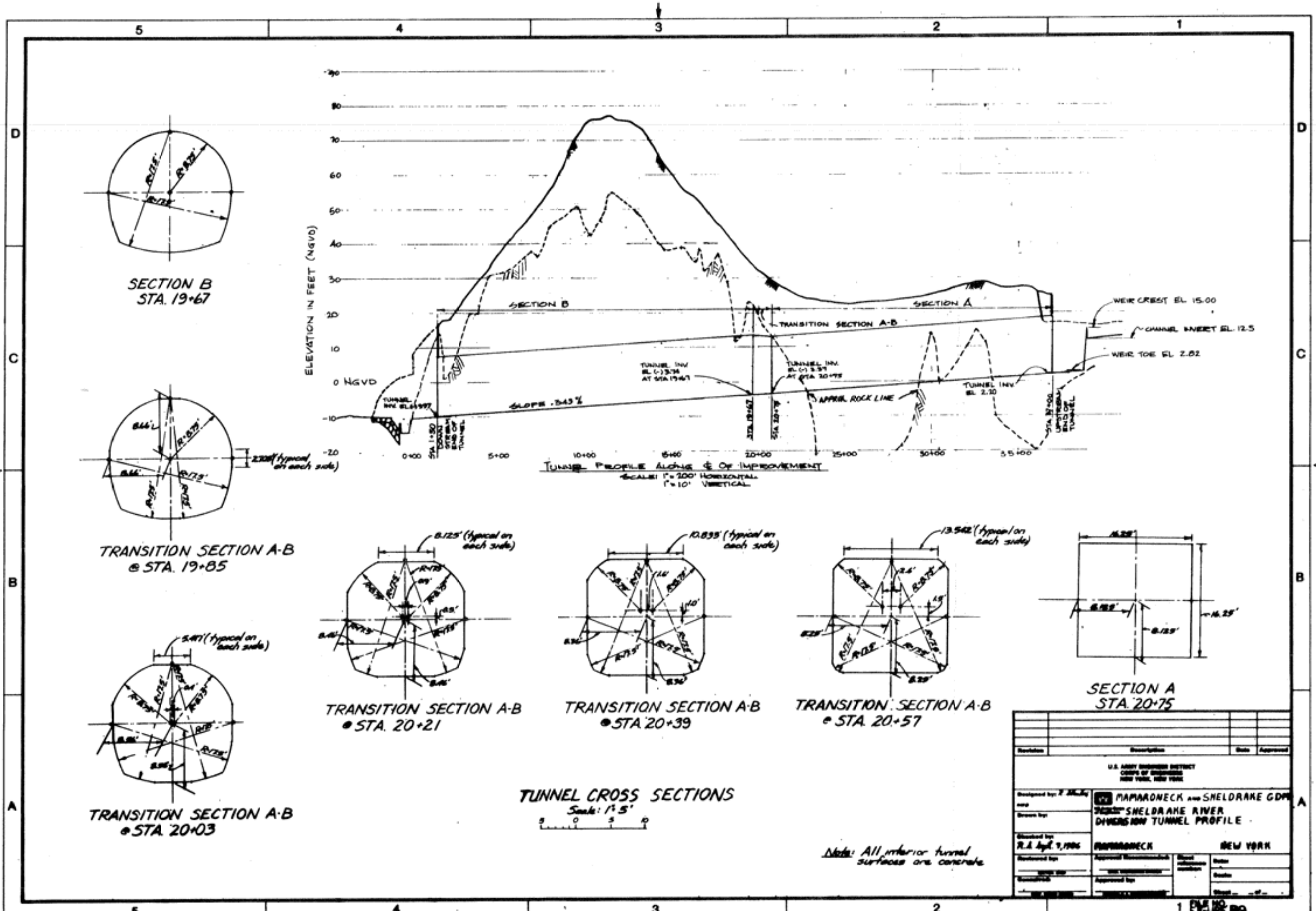
APPROVED:
COLONEL C. E., DISTRICT ENGINEER

DATE

SCALE AS SHOWN

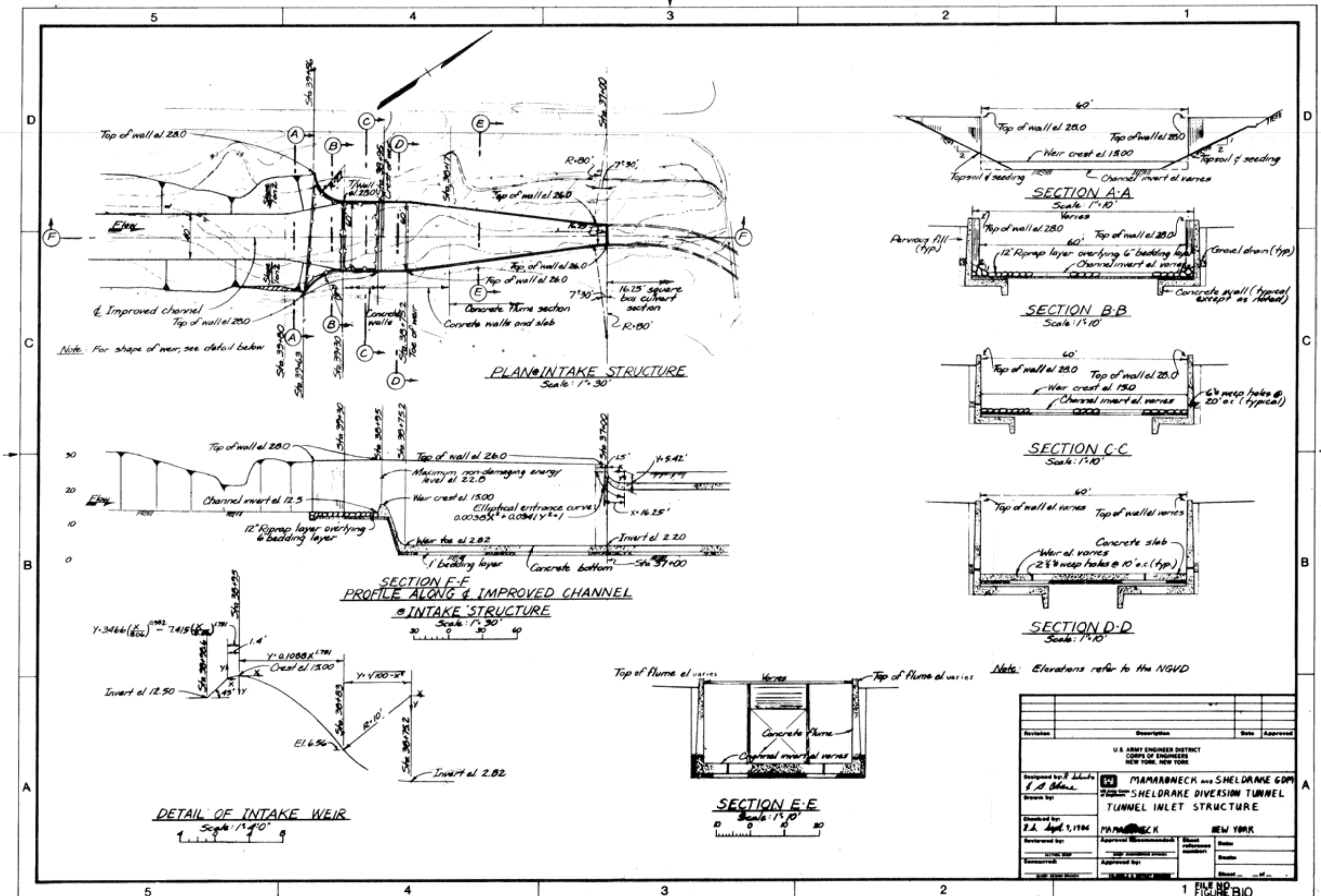
DRAWING NUMBER

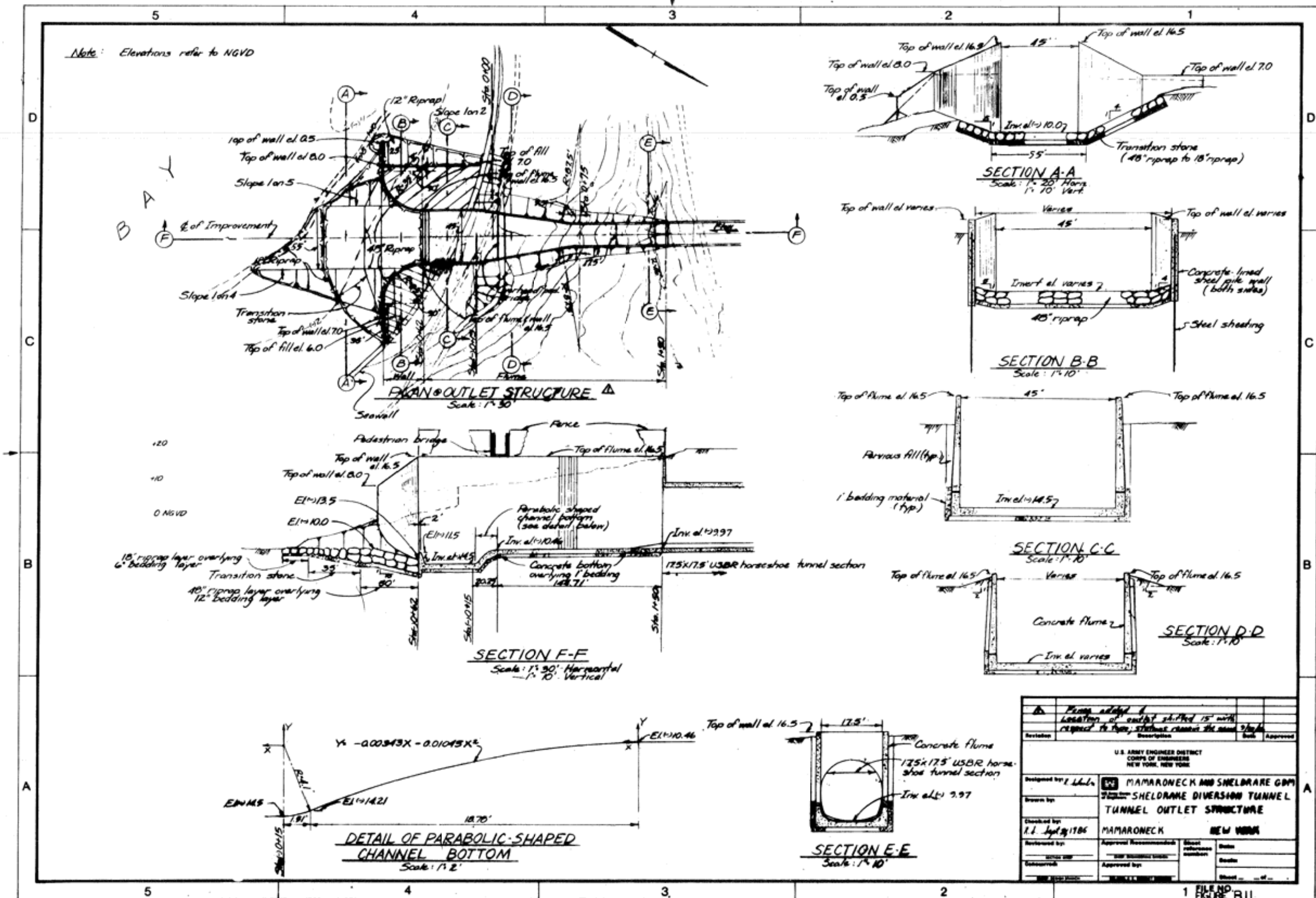
SHEET - OF -



SECTION B
STA. 19+67
TRANSITION SECTION A-B
@ STA. 19+85
TRANSITION SECTION A-B
@ STA. 20+03
TUNNEL PROFILE ALONG ℄ OF IMPROVEMENT
SCALE: 1" = 200' HORIZONTAL
1" = 10' VERTICAL
ELEVATION IN FEET (NGVD)
0 NGVD
SECTION B
SECTION A
TRANSITION SECTION A-B
SLOPE .343%
APPROX. ROCK LINE
WEIR CREST EL 15.00
CHANNEL INVERT EL 12.5
WEIR TOE EL 2.82
TUNNEL INV EL 2.20
TRANSITION SECTION A-B
@ STA. 20+21
TRANSITION SECTION A-B
@ STA 20+39
TRANSITION SECTION A-B
@ STA. 20+57
SECTION A
STA. 20+75
TUNNEL CROSS SECTIONS
Scale: 1"= 5'
Note: All interior tunnel surfaces are concrete
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK AND SHELDRAKE GDM
SHELDRAKE RIVER
DIVERSION TUNNEL PROFILE
MAMARONECK
NEW YORK

FIGURE B9


PLAN @ INTAKE STRUCTURE
Scale: 1" = 30'
Top of wall el. 28.0
Top of wall el. 26.0
℄ Improved channel
Note: For shape of weir, see detail below
Concrete flume section
Concrete walls and slab
16.25' square box culvert section
R = 80'
7°30'
SECTION F-F
PROFILE ALONG ℄ IMPROVED CHANNEL
@ INTAKE STRUCTURE
Scale: 1" = 30'
Maximum non-damaging energy level el. 22.8
Weir crest el. 15.00
Channel invert el. 12.5
Elliptical entrance curve
12" Riprap layer overlying 6" bedding layer
Weir toe el. 2.82
1' bedding layer
Concrete bottom
Invert el. 2.20
Y = 5.42'
X = 16.25'
DETAIL OF INTAKE WEIR
Scale: 1" = 4'0"
Crest el. 15.00
Invert el. 12.50
Y = 0.1088X^1.791
Y = √(100 - X²)
R = 10'
El. 6.56
Invert el. 2.82
SECTION A-A
Scale: 1" = 10'
Top of wall el. 28.0
Weir crest el. 15.00
Topsoil & seeding
Channel invert el. varies
SECTION B-B
Scale: 1" = 10'
Pervious fill (typ.)
12" Riprap layer overlying 6" bedding layer
Channel invert el. varies
Gravel drain (typ.)
Concrete wall (typical except as noted)
SECTION C-C
Scale: 1" = 10'
Weir crest el. 15.0
6" weep holes @ 20' o.c. (typical)
SECTION D-D
Scale: 1" = 10'
Top of wall el. varies
Concrete slab
Weir el. varies
2½" weep holes @ 10' o.c. (typ.)
SECTION E-E
Scale: 1" = 10'
Top of flume el. varies
Concrete flume
Channel invert el. varies
Note: Elevations refer to the NGVD
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK and SHELDRAKE GDM
SHELDRAKE DIVERSION TUNNEL
TUNNEL INLET STRUCTURE
MAMARONECK NEW YORK
FILE NO.
FIGURE B10


Note: Elevations refer to NGVD
BAY
PLAN @ OUTLET STRUCTURE
Scale: 1"=30'
12" Riprap
Slope 1 on 2
Top of wall el. 0.5
Top of wall el. 8.0
Slope 1 on 5
℄ of Improvement
48" Riprap
18" Riprap
Slope 1 on 4
Transition stone
Top of wall el. 7.0
Top of fill el. 6.0
Seawall
Wall
Flume
SECTION A-A
Scale: 1"=20' Horiz. 1"=10' Vert.
Top of wall el. 16.5
Top of wall el. 8.0
Top of wall el. 0.5
Top of wall el. 7.0
Inv. el. (+) 10.07
Transition stone (48" riprap to 18" riprap)
SECTION B-B
Scale: 1"=10'
Top of wall el. varies
Invert el. varies
48" riprap
Concrete-lined steel pile wall (both sides)
Steel sheeting
SECTION C-C
Scale: 1"=10'
Top of flume el. 16.5
Pervious fill (typ.)
1' bedding material (typ.)
Inv. el. (+) 14.5
SECTION D-D
Scale: 1"=10'
Concrete flume
Inv. el. varies
SECTION F-F
Scale: 1"=30' Horizontal, 1"=10' Vertical
Pedestrian bridge
Fence
Top of flume el. 16.5
Top of wall el. 16.5
Top of wall el. 8.0
El. (+) 13.5
El. (+) 10.0
El. (+) 11.5
Parabolic shaped channel bottom (see detail below)
Inv. el. (+) 10.46
Inv. el. (+) 9.97
Concrete bottom overlying 1' bedding
18" riprap layer overlying 6" bedding layer
Transition stone
48" riprap layer overlying 12" bedding layer
17.5'x17.5' USBR horseshoe tunnel section
DETAIL OF PARABOLIC-SHAPED CHANNEL BOTTOM
Scale: 1"=2'
Y= -0.00343X - 0.01043X²
El. (+) 10.46
El. (+) 14.21
18.70'
SECTION E-E
Scale: 1"=10'
Top of wall el. 16.5
Concrete flume
17.5'x17.5' USBR horseshoe tunnel section
Inv. el. (+) 9.97
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK AND SHELDRAKE GDM
SHELDRAKE DIVERSION TUNNEL
TUNNEL OUTLET STRUCTURE
MAMARONECK NEW YORK
Revision
Description
Date
Approved
Designed by:
Drawn by:
Checked by:
Reviewed by:
Approval Recommended:
Approved by:
FILE NO.
FIGURE B11

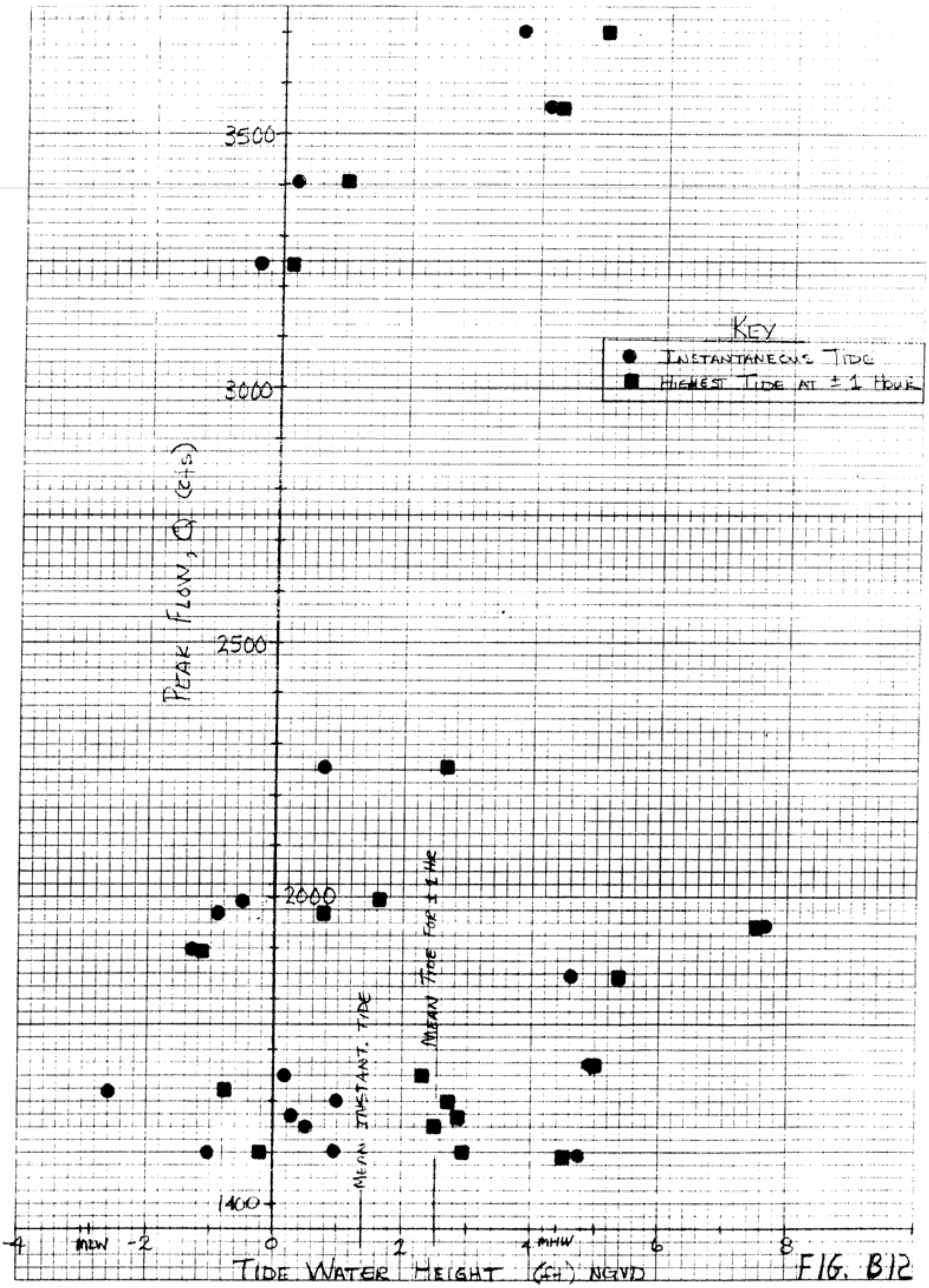


Fig. B12 - PEAK FLOW VS. TIDE HEIGHT AT MAMARONECK RIVER

7/3/84 scc

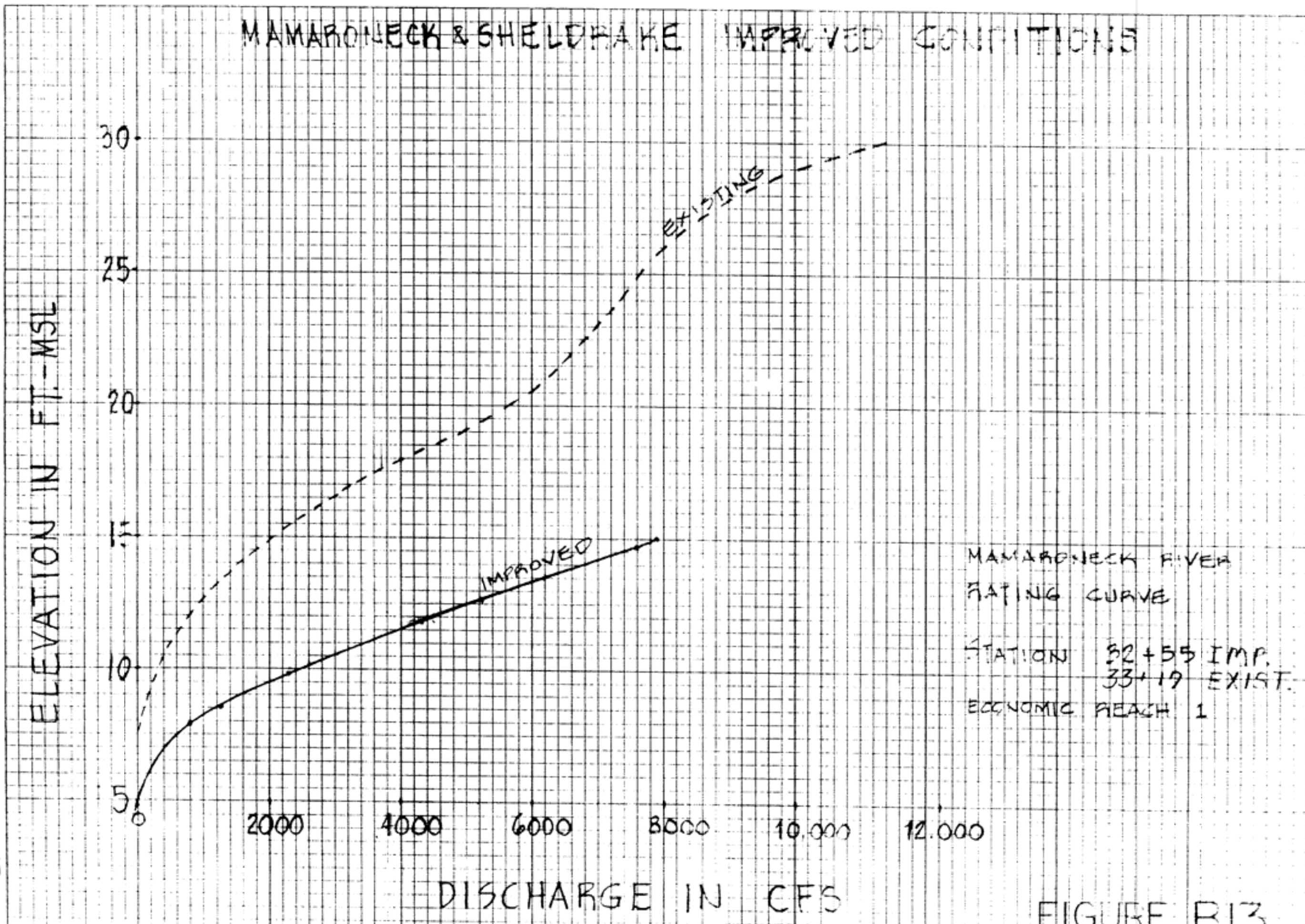

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
ELEVATION IN FT-MSL
30
25
20
15
10
5
0
2000
4000
6000
8000
10,000
12,000
DISCHARGE IN CFS
EXISTING
IMPROVED
MAMARONECK RIVER
RATING CURVE
STATION 32+55 IMP.
33+19 EXIST.
ECONOMIC REACH 1
FIGURE B13

FIGURE B13

46 0703

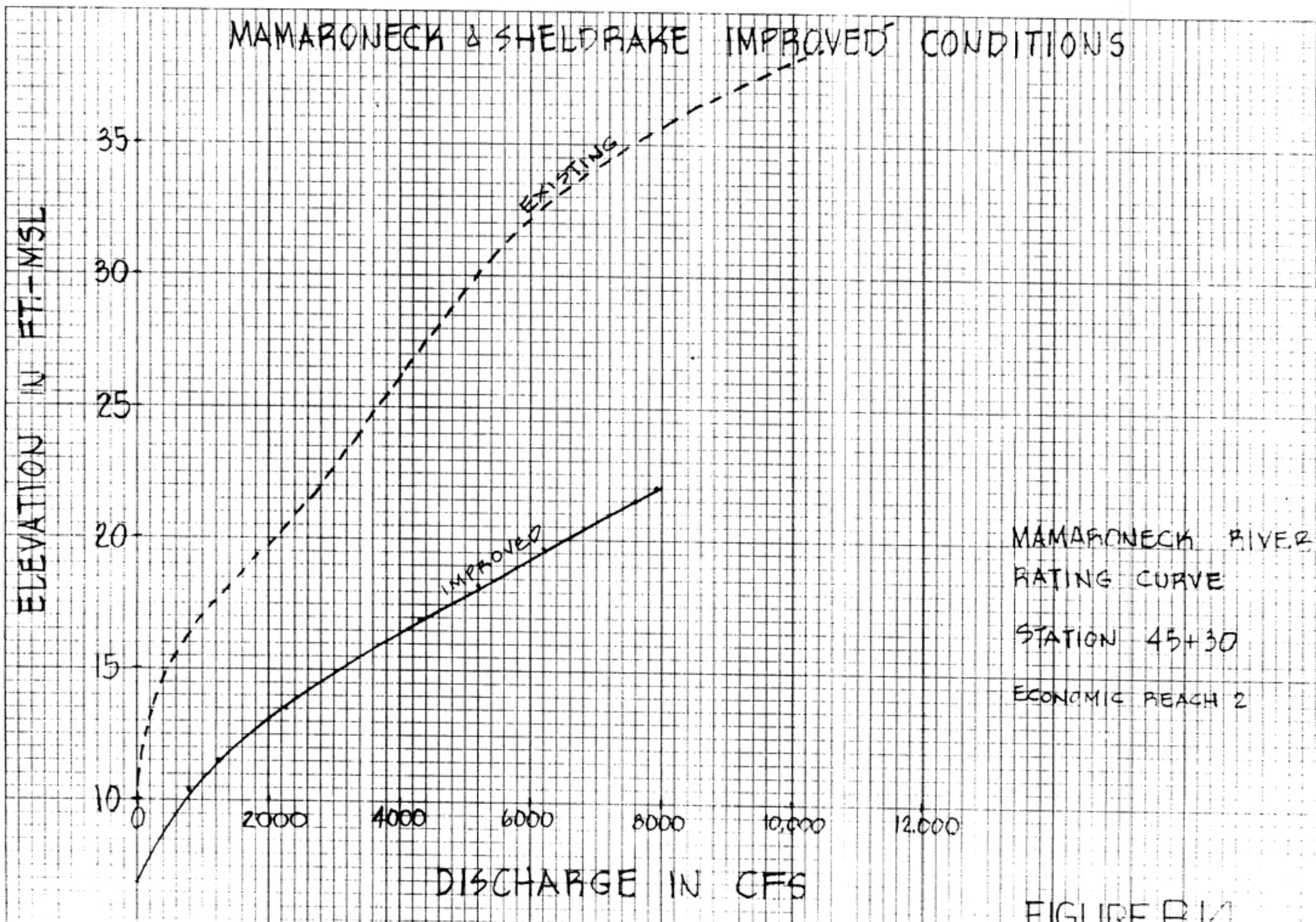


FIGURE B 14

FIGURE B 14

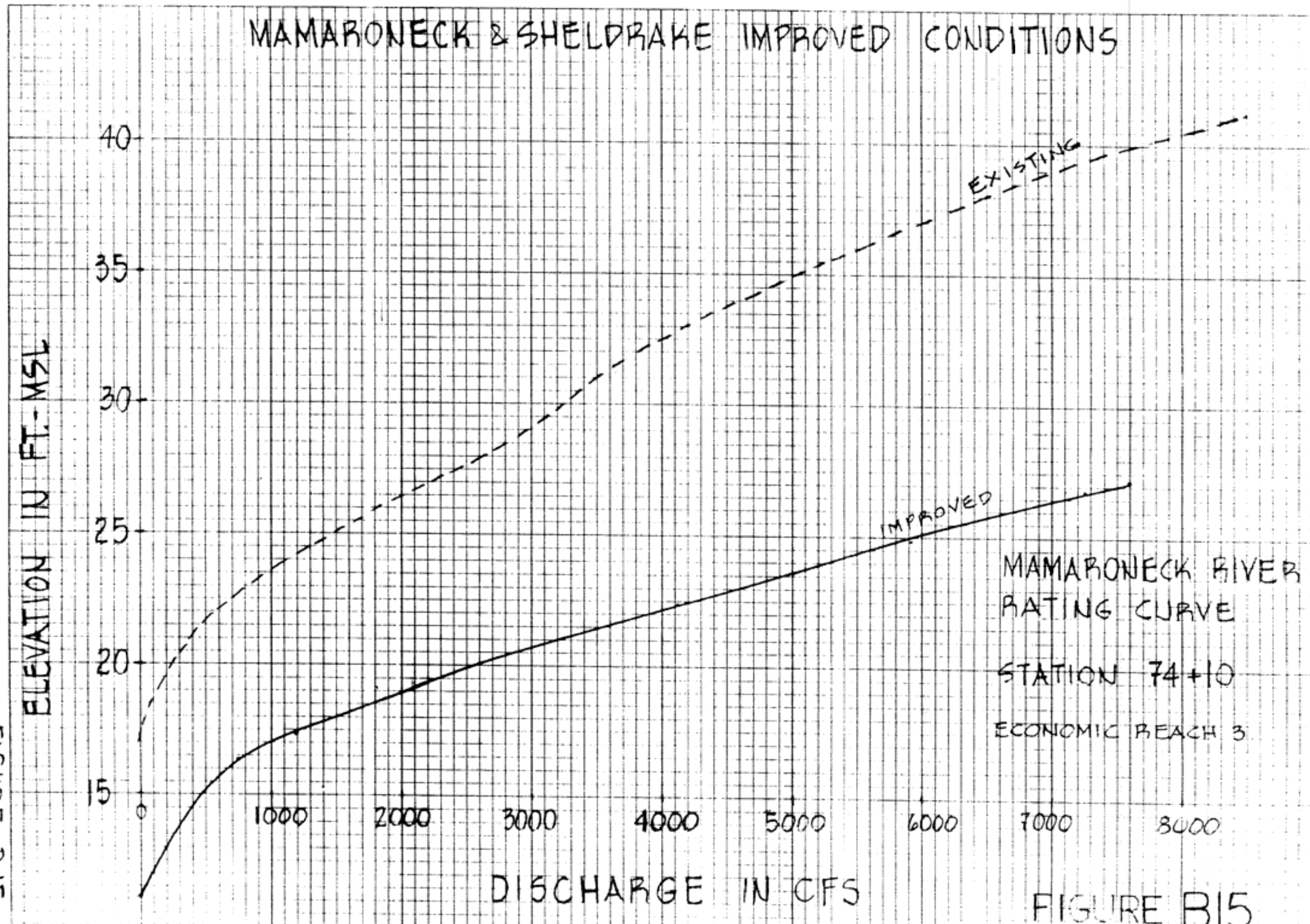

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
EXISTING
IMPROVED
MAMARONECK RIVER
RATING CURVE
STATION 74+10
ECONOMIC REACH 3
ELEVATION IN FT.-MSL
15
20
25
30
35
40
0
1000
2000
3000
4000
5000
6000
7000
8000
DISCHARGE IN CFS
FIGURE B15

FIGURE B15

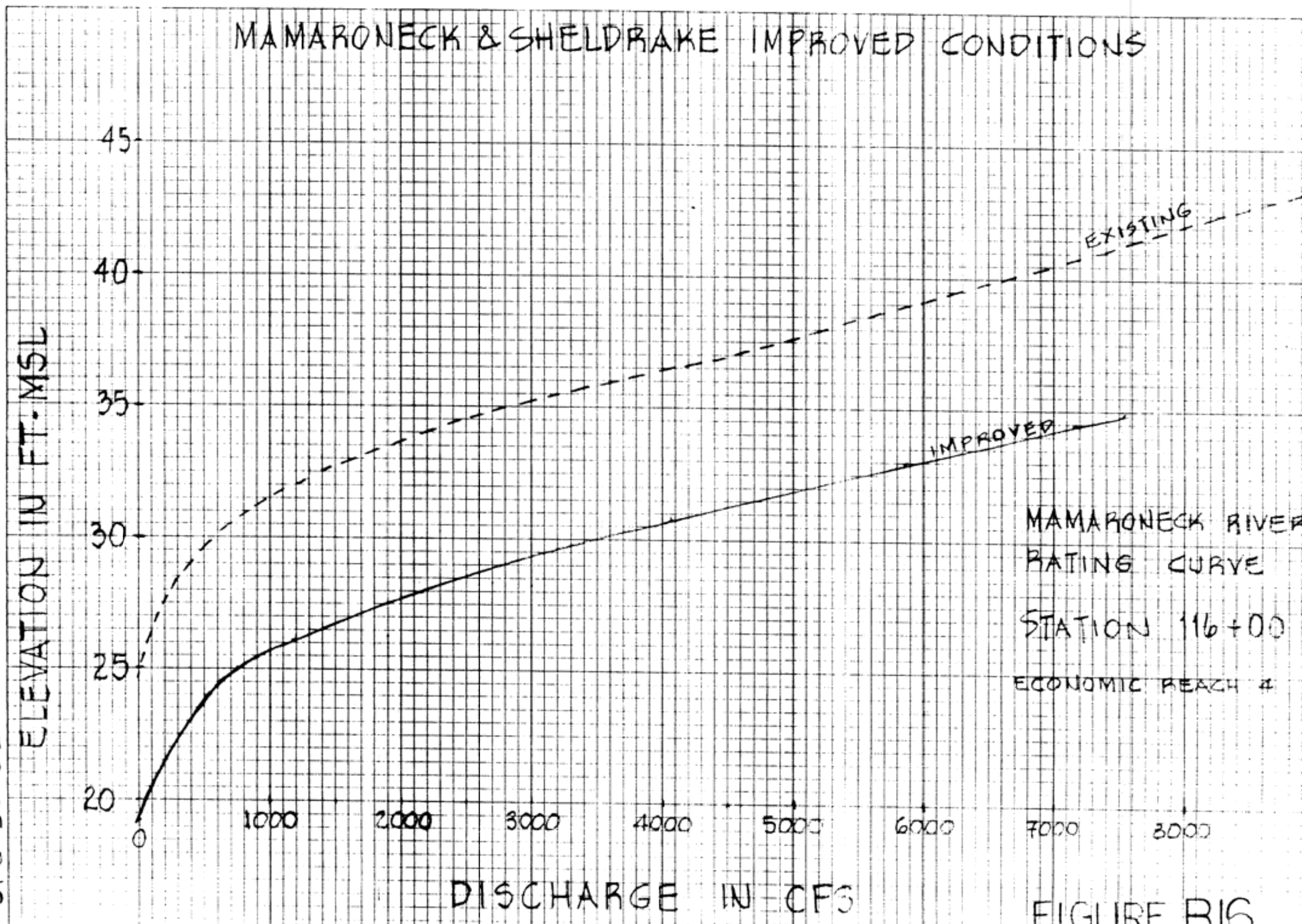

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
45
40
35
30
25
20
ELEVATION IN FT. MSL
0
1000
2000
3000
4000
5000
6000
7000
8000
DISCHARGE IN CFS
EXISTING
IMPROVED
MAMARONECK RIVER
RATING CURVE
STATION 116+00
ECONOMIC REACH 4
FIGURE B16

FIGURE B16

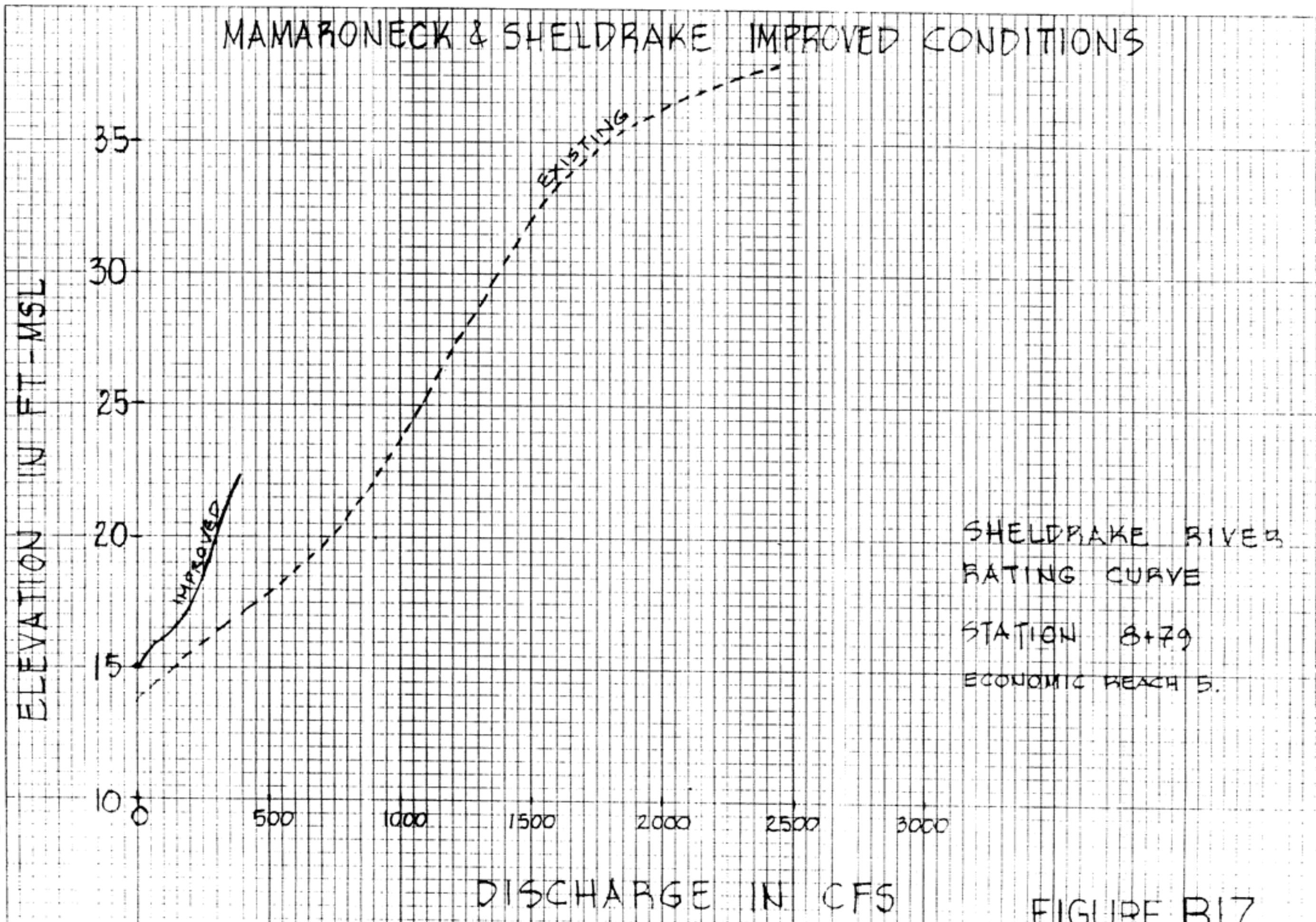
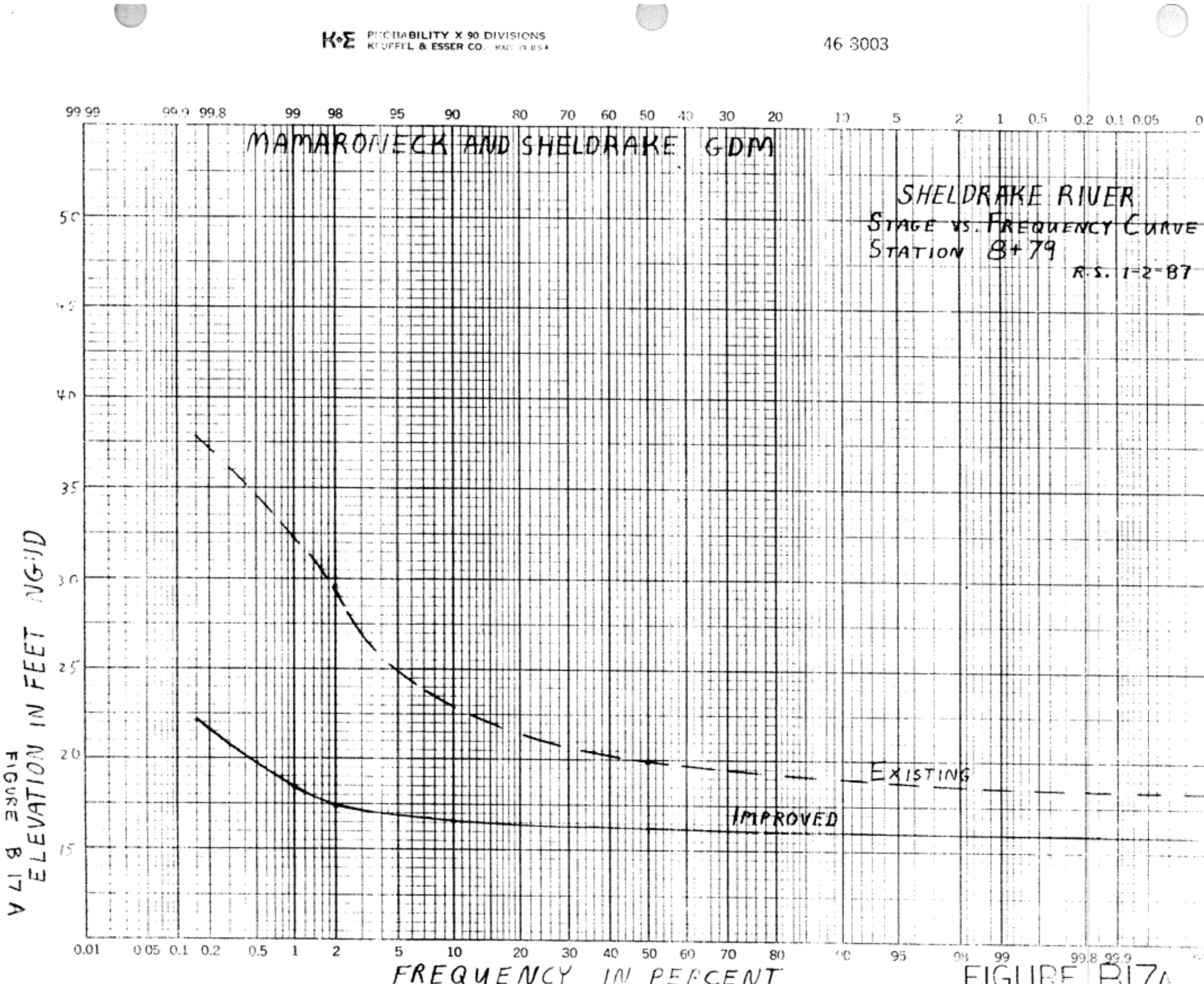

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
ELEVATION IN FT-MSL
35
30
25
20
15
10
0
500
1000
1500
2000
2500
3000
DISCHARGE IN CFS
IMPROVED
EXISTING
SHELDRAKE RIVER
RATING CURVE
STATION 8+79
ECONOMIC REACH 5.
FIGURE B17

FIGURE B17



MAMARONECK AND SHELDRAKE GDM
SHELDRAKE RIVER
STAGE VS. FREQUENCY CURVE
STATION 8+79
R.S. 1-2-87
EXISTING
IMPROVED
ELEVATION IN FEET NGVD
FREQUENCY IN PERCENT

FIGURE B17A

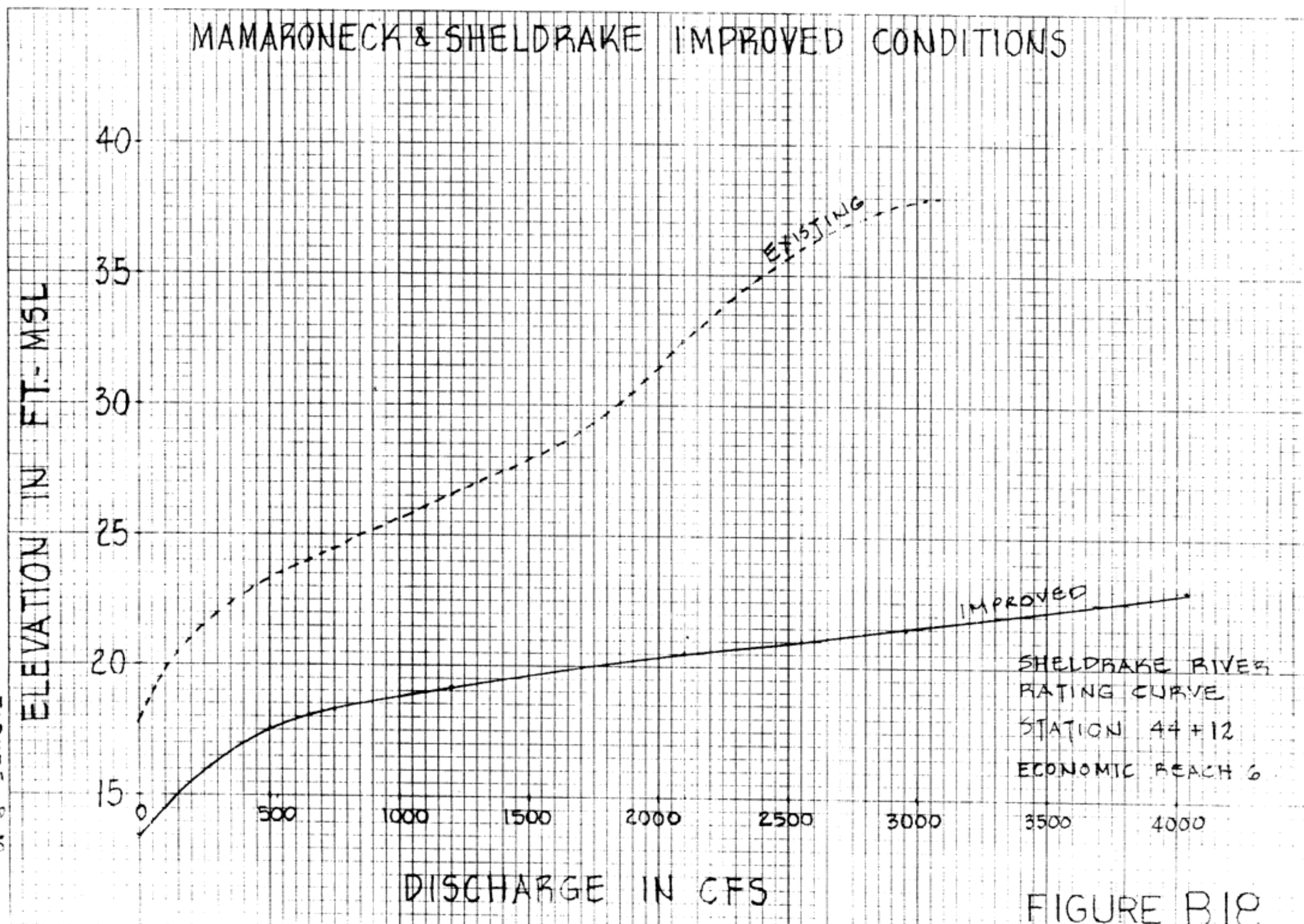

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
ELEVATION IN FT.-MSL
40
35
30
25
20
15
0
500
1000
1500
2000
2500
3000
3500
4000
DISCHARGE IN CFS
EXISTING
IMPROVED
SHELDRAKE RIVER
RATING CURVE
STATION 44 + 12
ECONOMIC REACH 6
FIGURE B18

FIGURE B 18

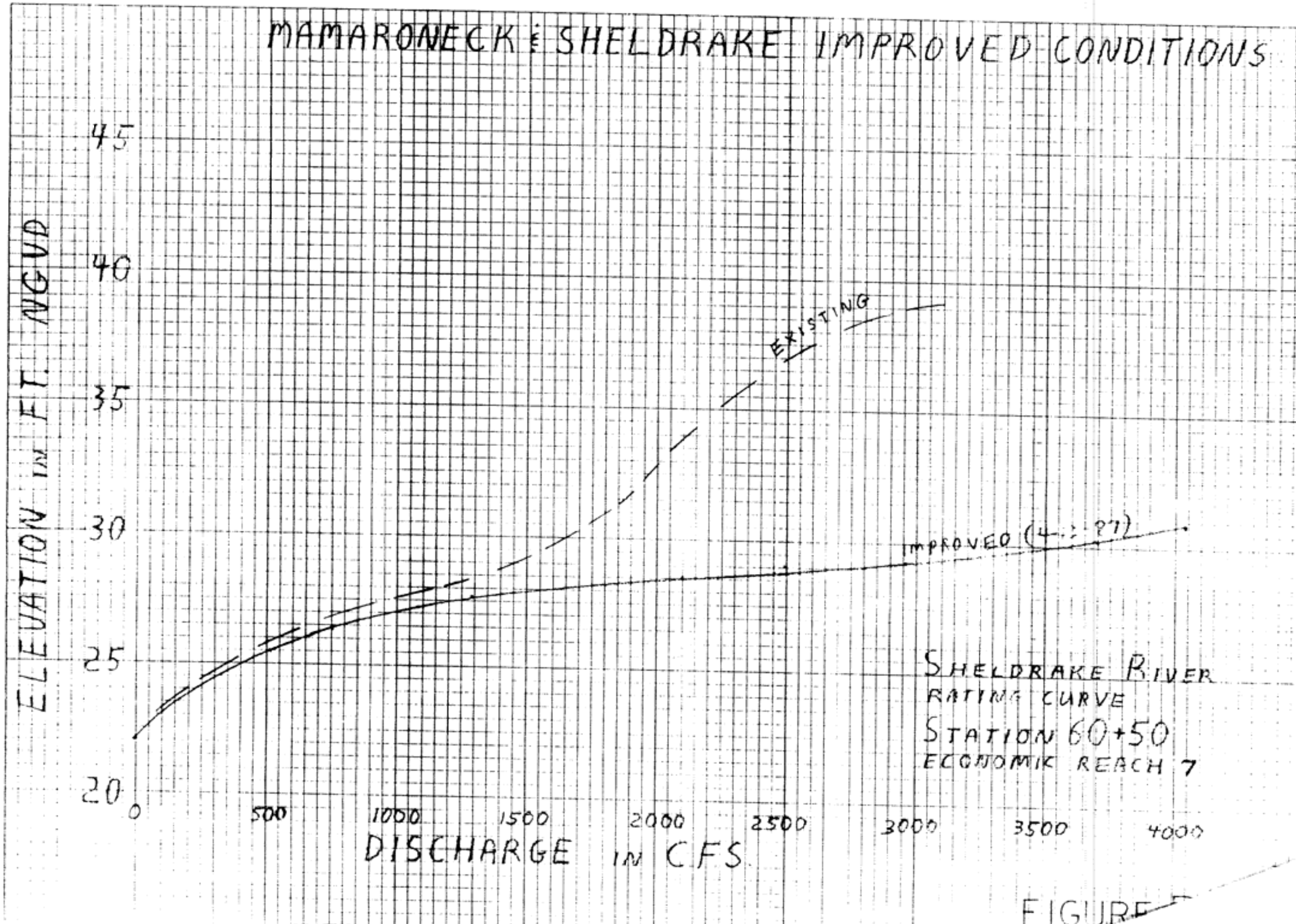

MAMARONECK & SHELDRAKE IMPROVED CONDITIONS
ELEVATION IN FT. NGVD
45
40
35
30
25
20
0
500
1000
1500
2000
2500
3000
3500
4000
DISCHARGE IN CFS
EXISTING
IMPROVED
SHELDRAKE RIVER
RATING CURVE
STATION 60+50
ECONOMIK REACH 7
FIGURE

FIGURE B19

COMPARISON OF 1975 AND 1982 THALWEGS

Mamaroneck River

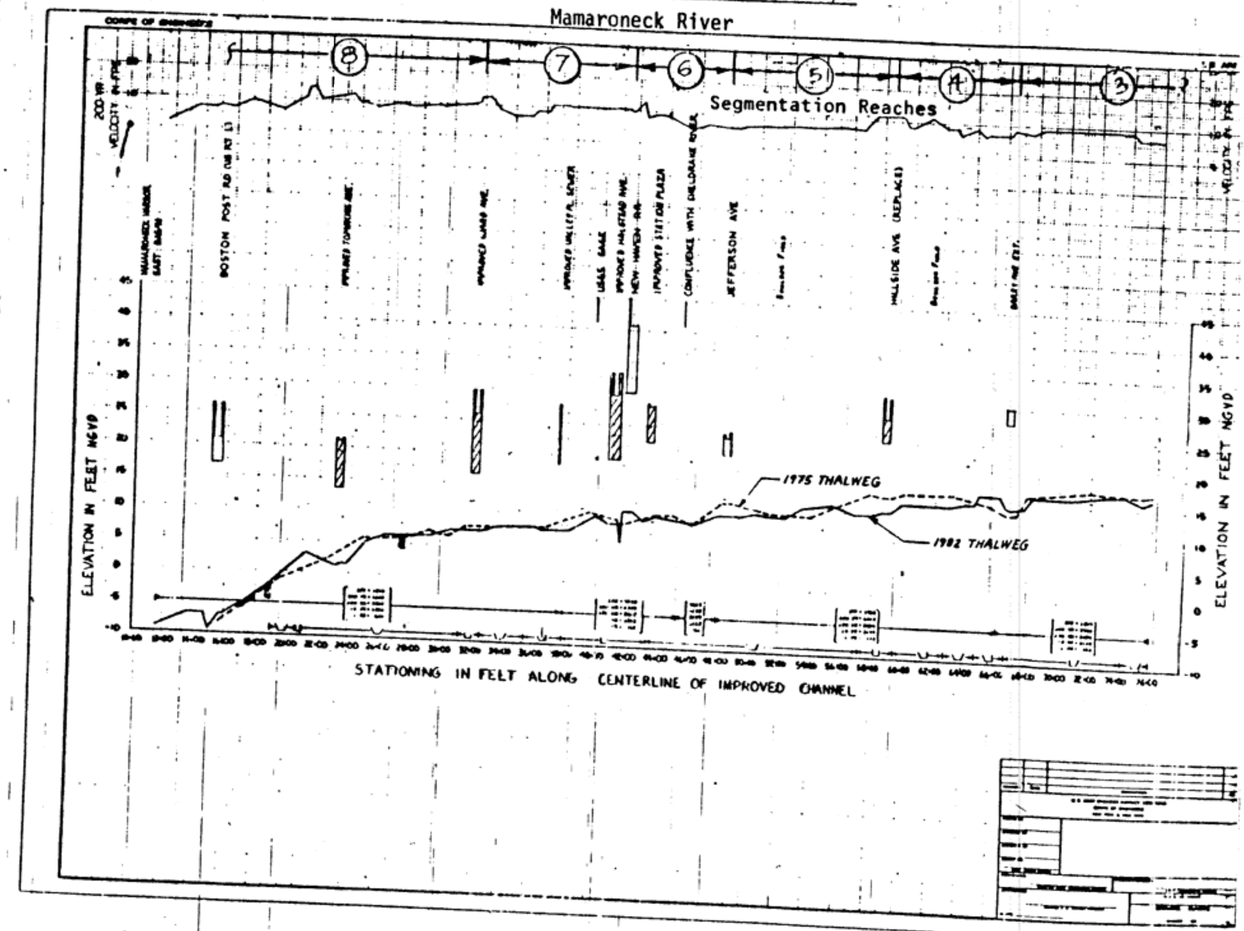


FIGURE B20

FIGURE B20

COMPARISON OF 1975 AND 1982 THALWEGS

Mamaroneck River

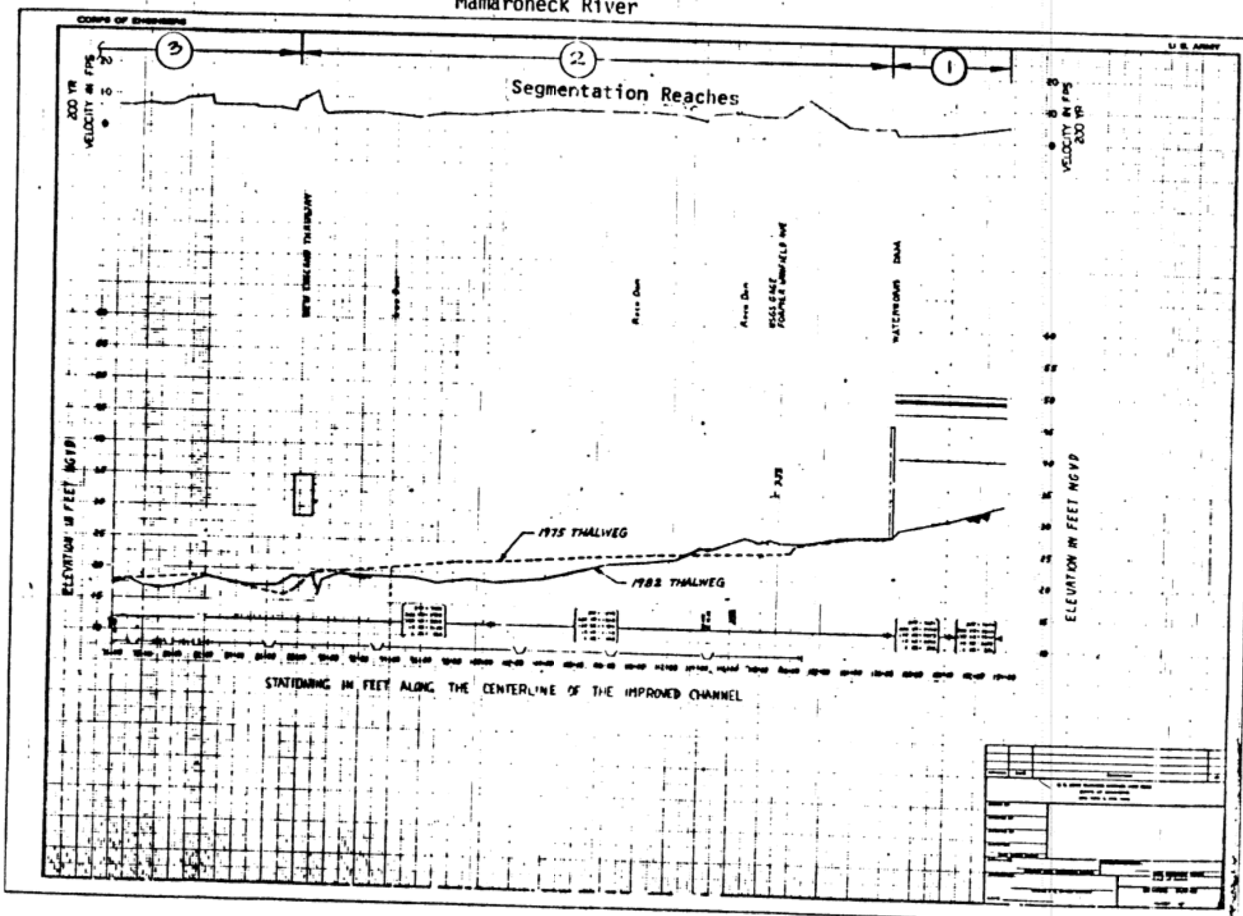


FIGURE B 21

FIGURE B 21

COMPARISON OF 1975 AND 1982 THALWEGS

Sheldrake River

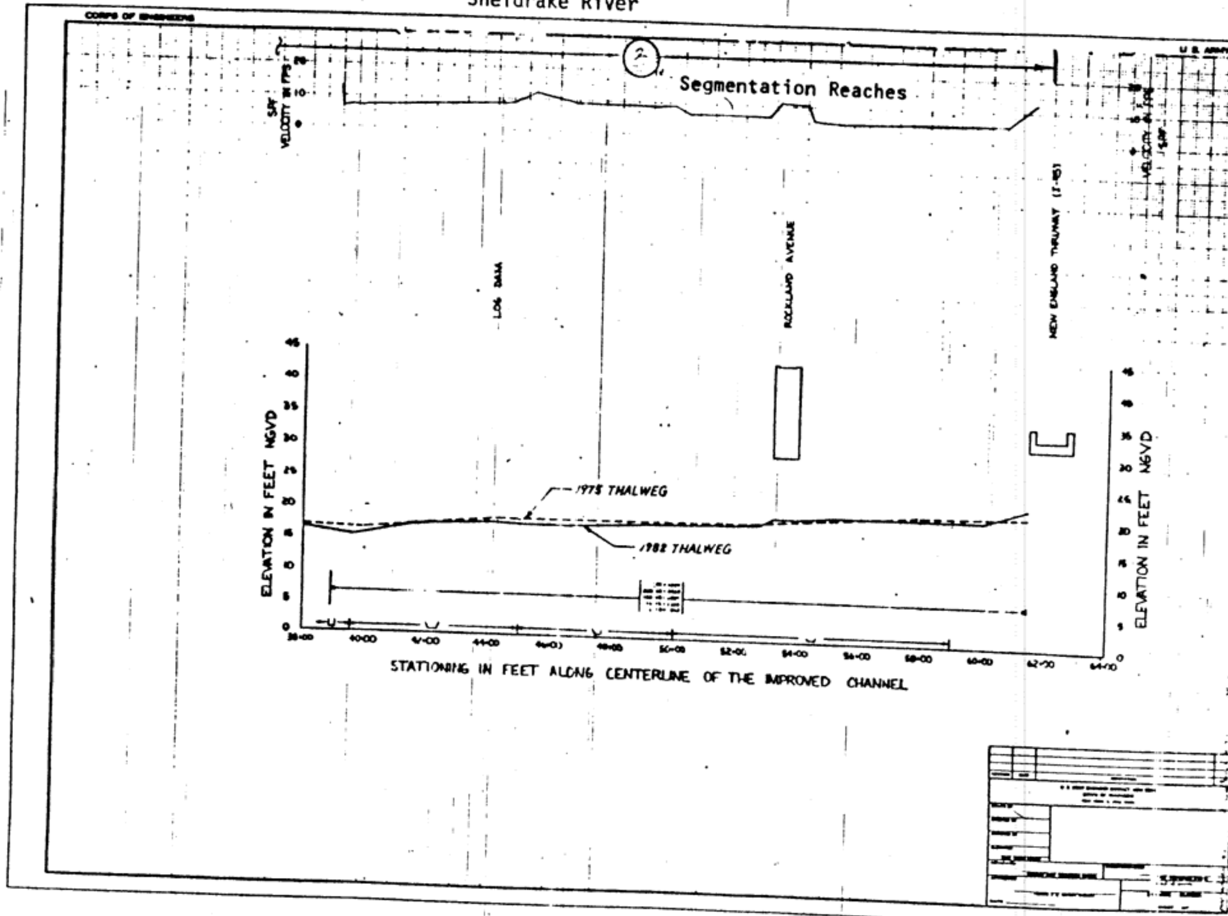


FIGURE B22

FIGURE B22

COMPARISON OF 1975 AND 1982 THALWEGS

Sheldrake River

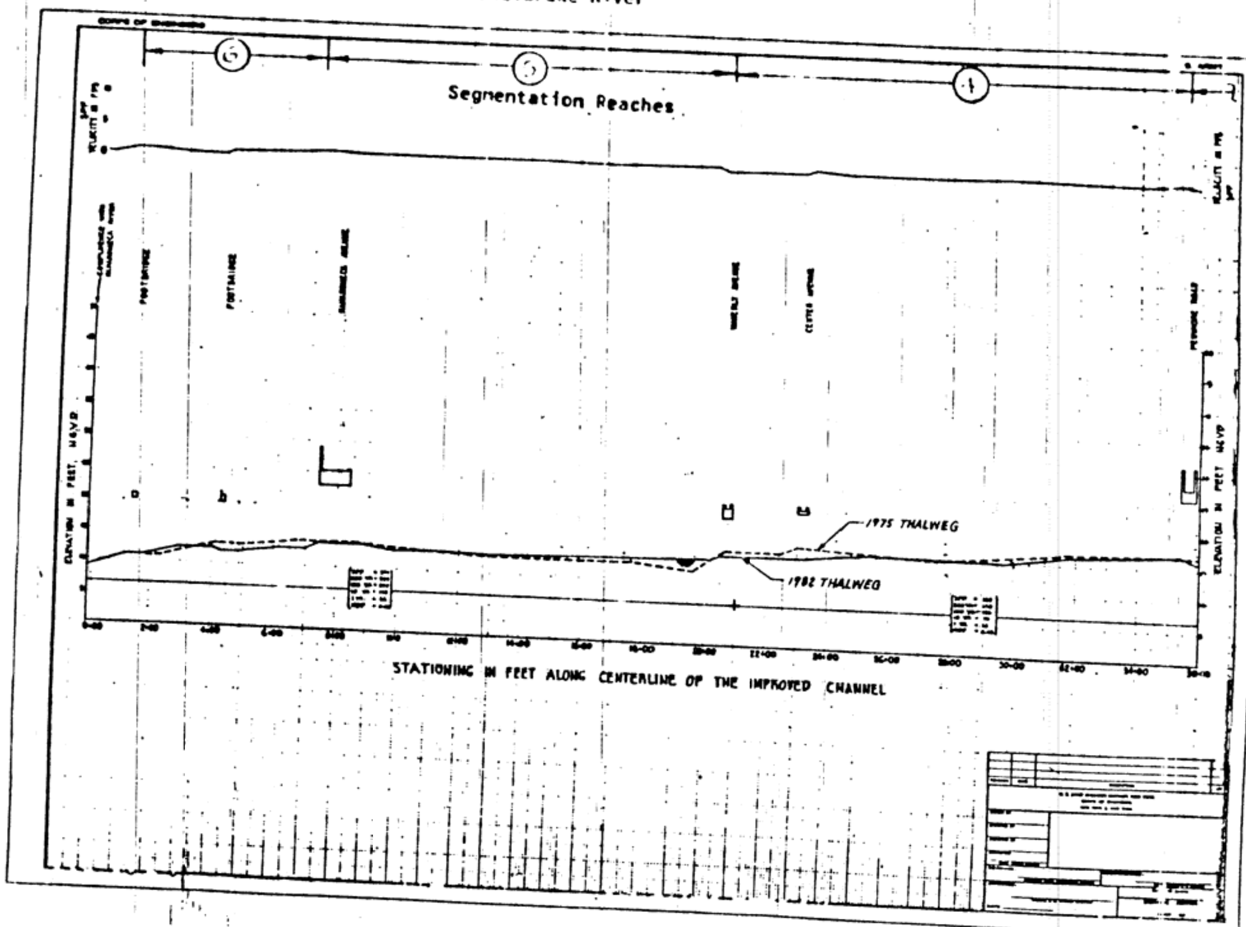
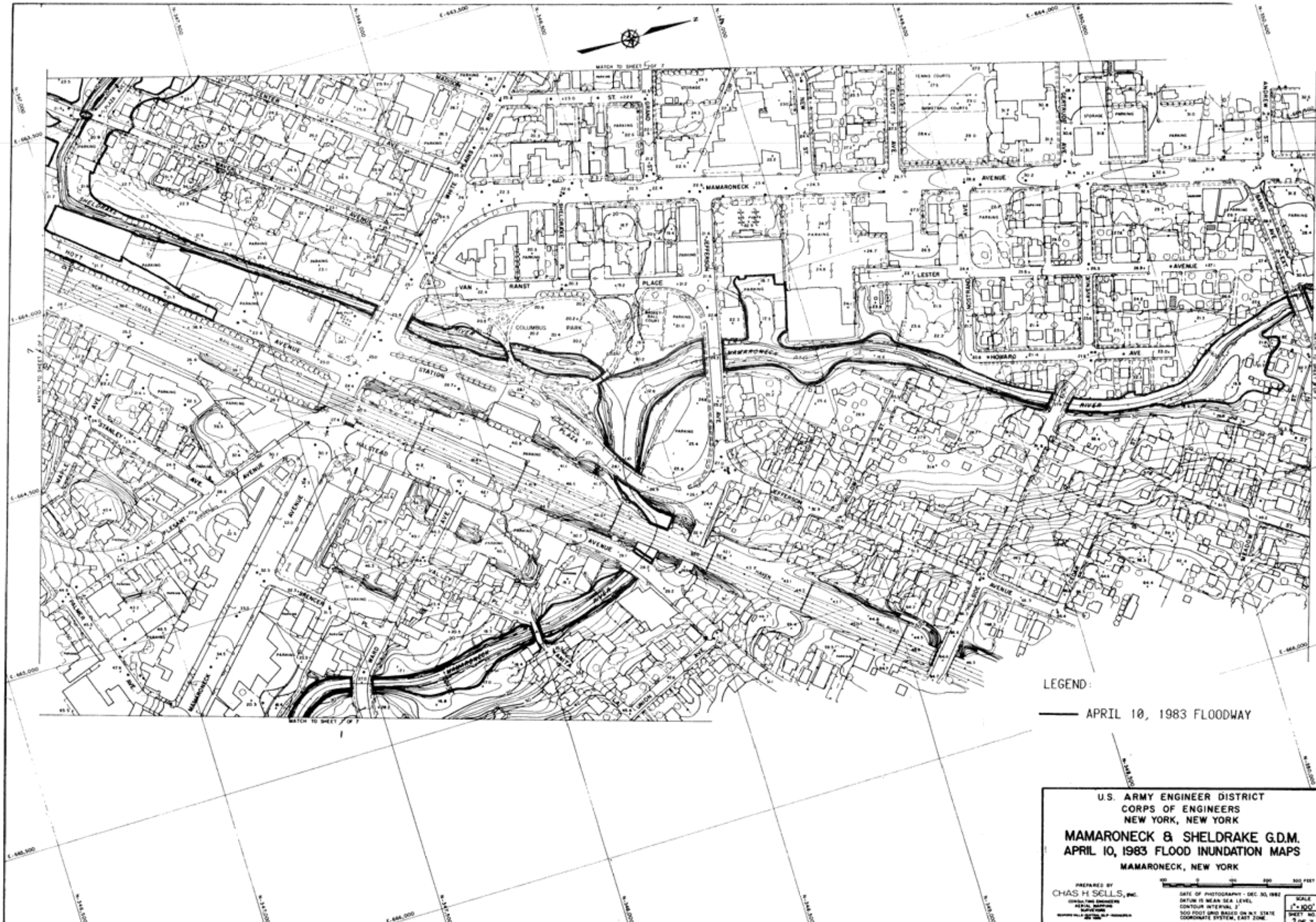


FIGURE B-23

MATCH TO SHEET 2 OF 7
N
JOHNSON ST.
WEST BOSTON POST RD.
MAMARONECK AVE.
EAST BOSTON POST ROAD
MAMARONECK RIVER
HARBOR ISLAND PARK
MAMARONECK HARBOR
BOAT STORAGE
PARKING
TOMPKINS AVE.
UNION AVENUE
LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
CONSULTING ENGINEERS
AERIAL MAPPING
SURVEYORS
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE
SCALE
1" = 100'
SHEET NO.
1 OF 7

FIGURE B24



U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

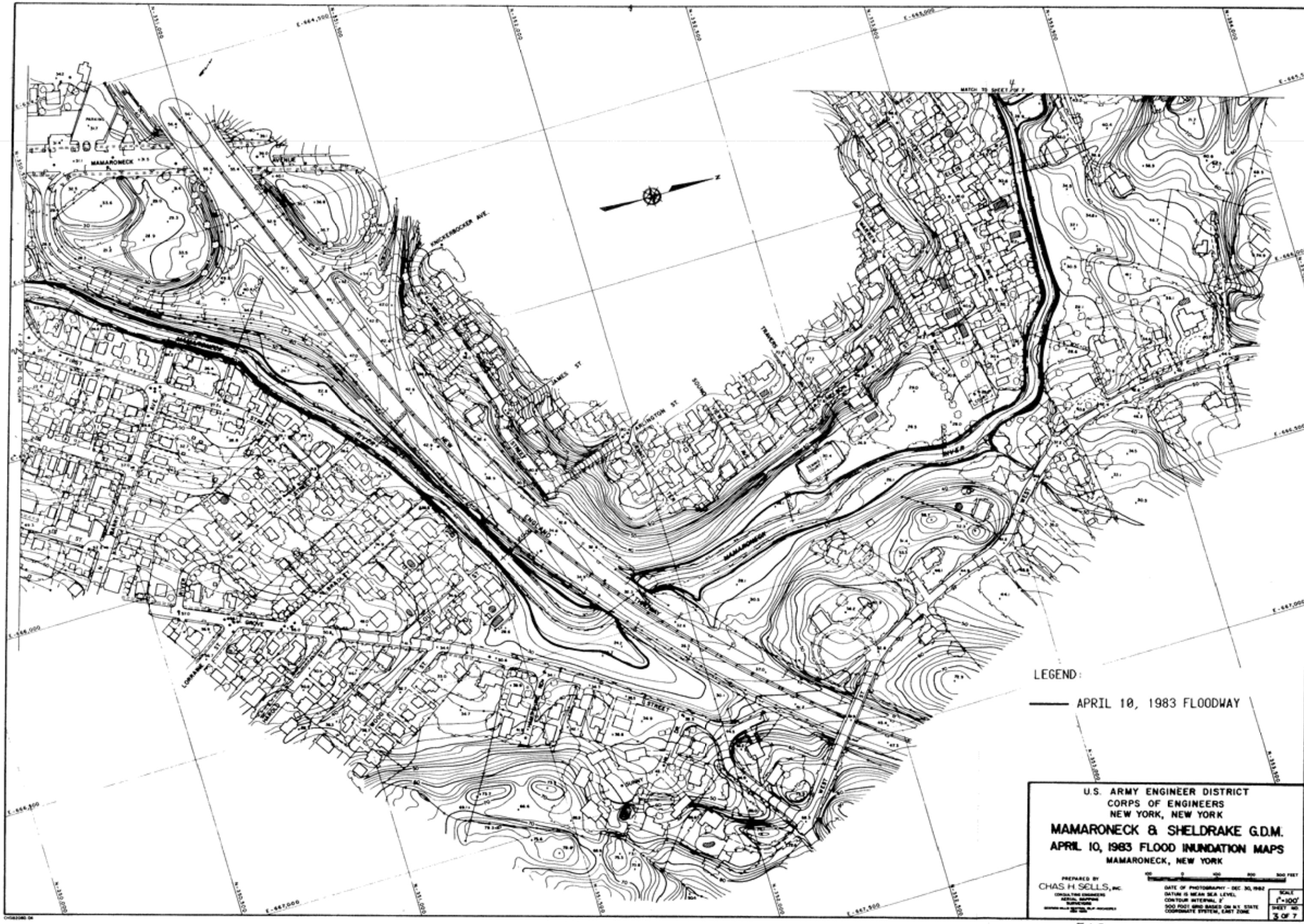
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK

PREPARED BY CHAS H SELLS, INC.

DATE OF PHOTOGRAPHY - DEC 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE

SCALE 1"=100'
SHEET NO. 2 OF 7

FIGURE B.25



MAMARONECK
KNICKERBOCKER AVE.
JAMES ST
ARLINGTON ST.
MAMARONECK
GROVE
STREET
LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE
SCALE 1"=100'
SHEET NO. 3 OF 7

FIGURE B.26

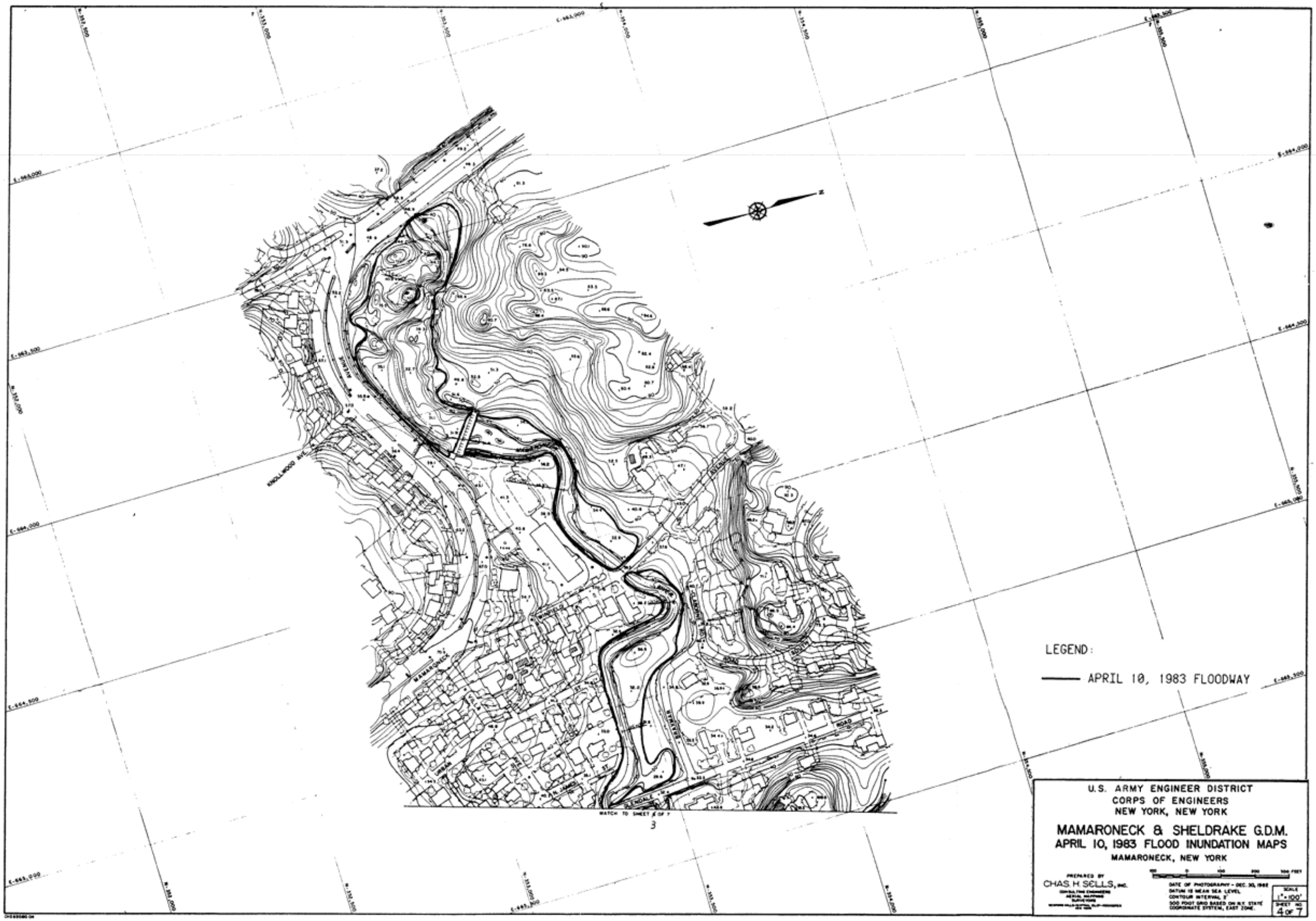
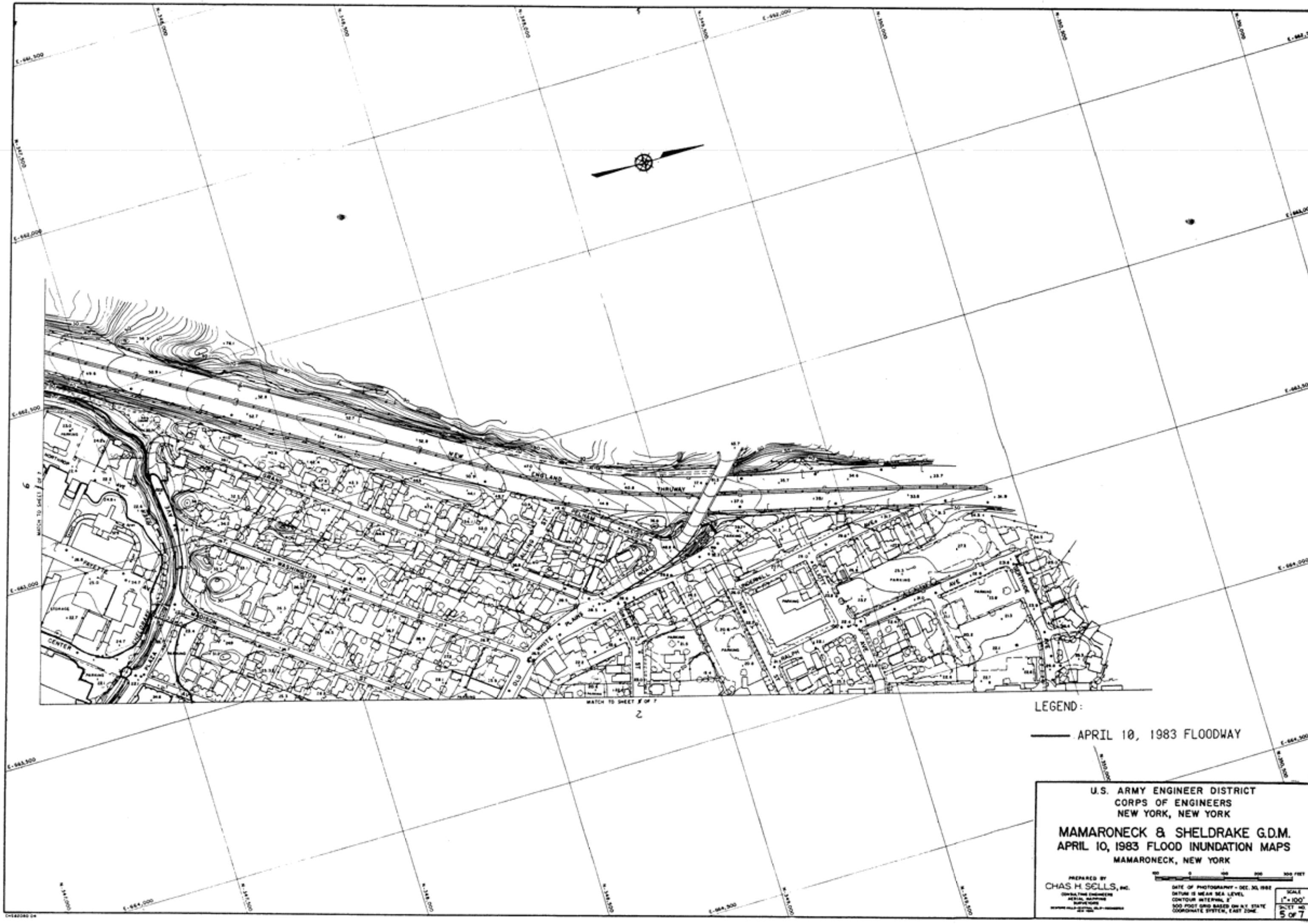

LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
CHAS. H. SELLS, INC.
4 OF 7

FIGURE B 27



NEW ENGLAND THRUWAY
WASHINGTON
MADISON
CENTER
PLAZA
WHITE PLAINS
OLD
UNDERHILL
RALPH
AVE
PARKING
STORAGE
MATCH TO SHEET 5 OF 7
MATCH TO SHEET 6 OF 7
LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
CONSULTING ENGINEERS
AERIAL MAPPING
SURVEYORS
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE.
SCALE 1" = 100'
SHEET NO. 5 OF 7

FIGURE B-[illegible]

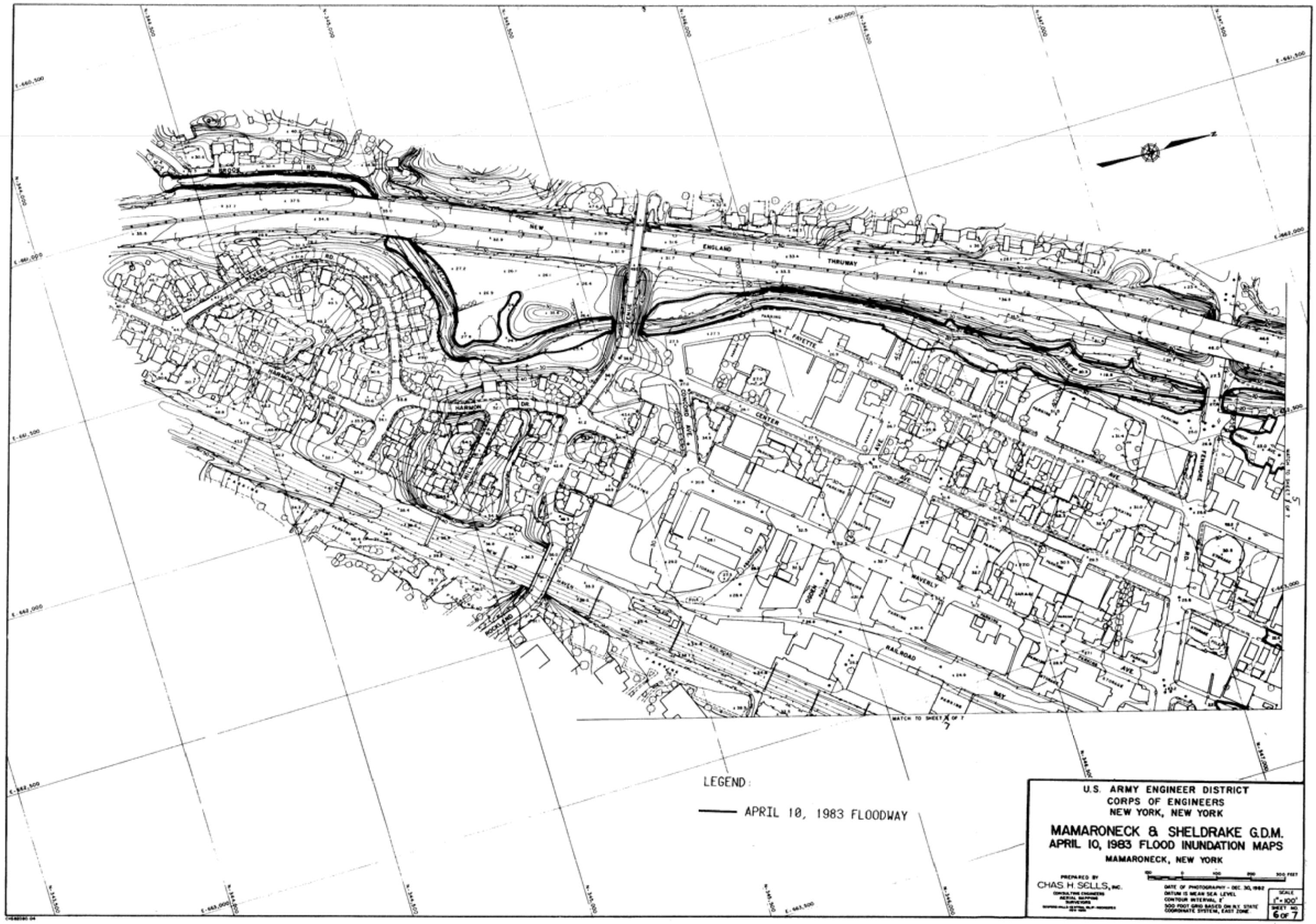
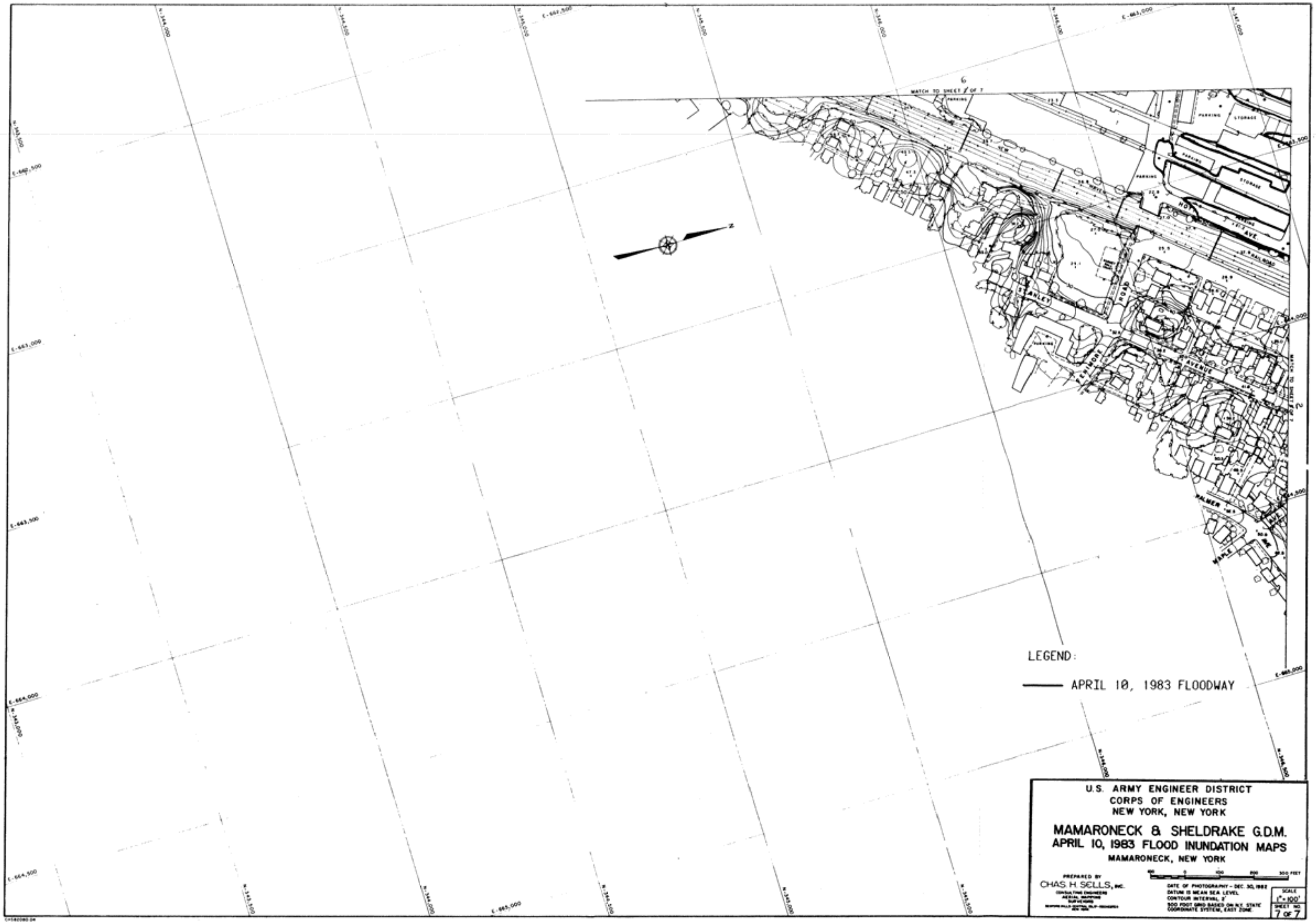

NEW ENGLAND THRUWAY
LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS H. SELLS, INC.
SHEET NO. 6 OF 7

FIGURE B.29



MATCH TO SHEET 6 OF 7
PARKING
STORAGE
PARKING
STORAGE
PARKING
NEW
HOYT
AVE
STANLEY
ROAD
AVENUE
PALMER
MAPLE
LEGEND:
APRIL 10, 1983 FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
APRIL 10, 1983 FLOOD INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
CONSULTING ENGINEERS
AERIAL MAPPING
SURVEYORS
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE.
SCALE
1" = 100'
SHEET NO.
7 OF 7

FIGURE B.30

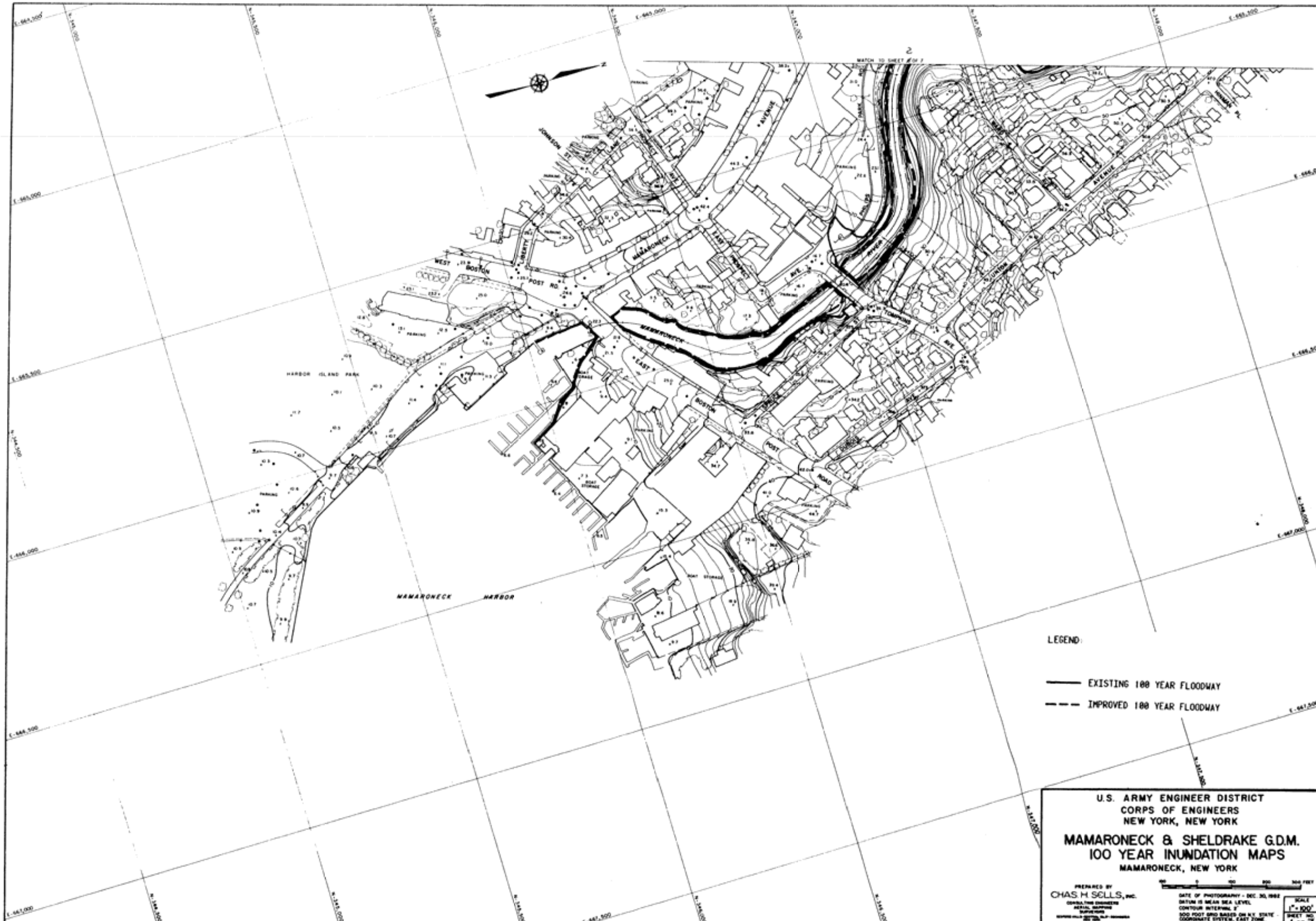

MATCH TO SHEET 2 OF 7
MAMARONECK HARBOR
HARBOR ISLAND PARK
LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
SCALE 1" = 100'
SHEET NO. 1 OF 7

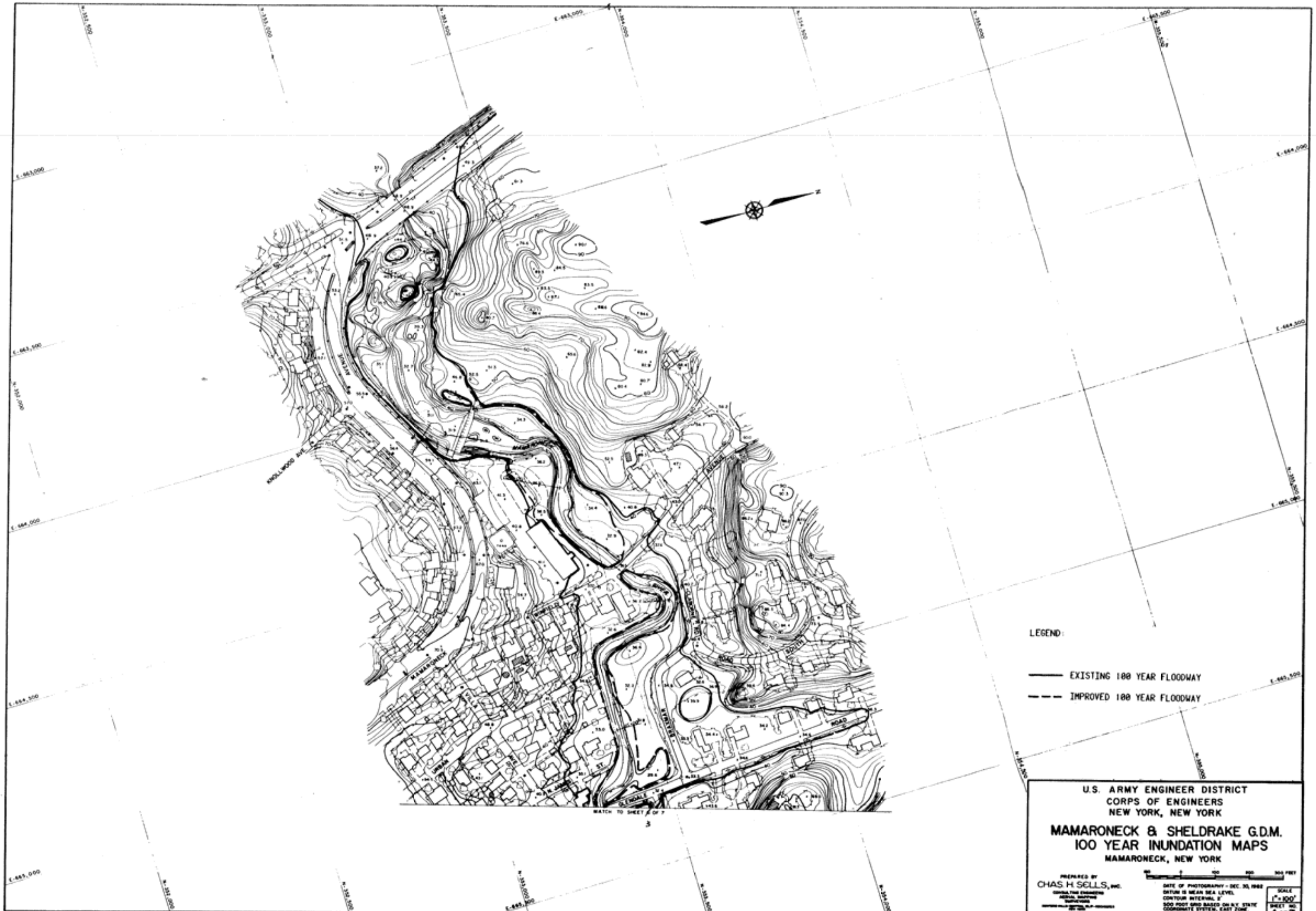
FIGURE B.31


LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
SCALE
1"=100'
SHEET NO.
2 OF 7
MAMARONECK AVENUE
SHELDRAKE AVENUE
HOYT AVE
COLUMBUS PARK
STATION PLAZA
MAMARONECK RIVER
HALSTEAD AVENUE
VAN RANST PLACE
LESTER PL
HOWARD AVE

FIGURE B.32


MAMARONECK
KNICKERBOCKER AVE.
JAMES ST
ARLINGTON ST
RIVER
MAMARONECK
LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS H SELLS, INC.
SCALE
1" = 100'
SHEET NO.
3 OF 7

FIGURE B33



LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
MATCH TO SHEET 3 OF 7
KNOLLWOOD AVE
MAMARONECK
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1962
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE.
SCALE 1" = 100'
SHEET NO. 4 OF 7

FIGURE B 34

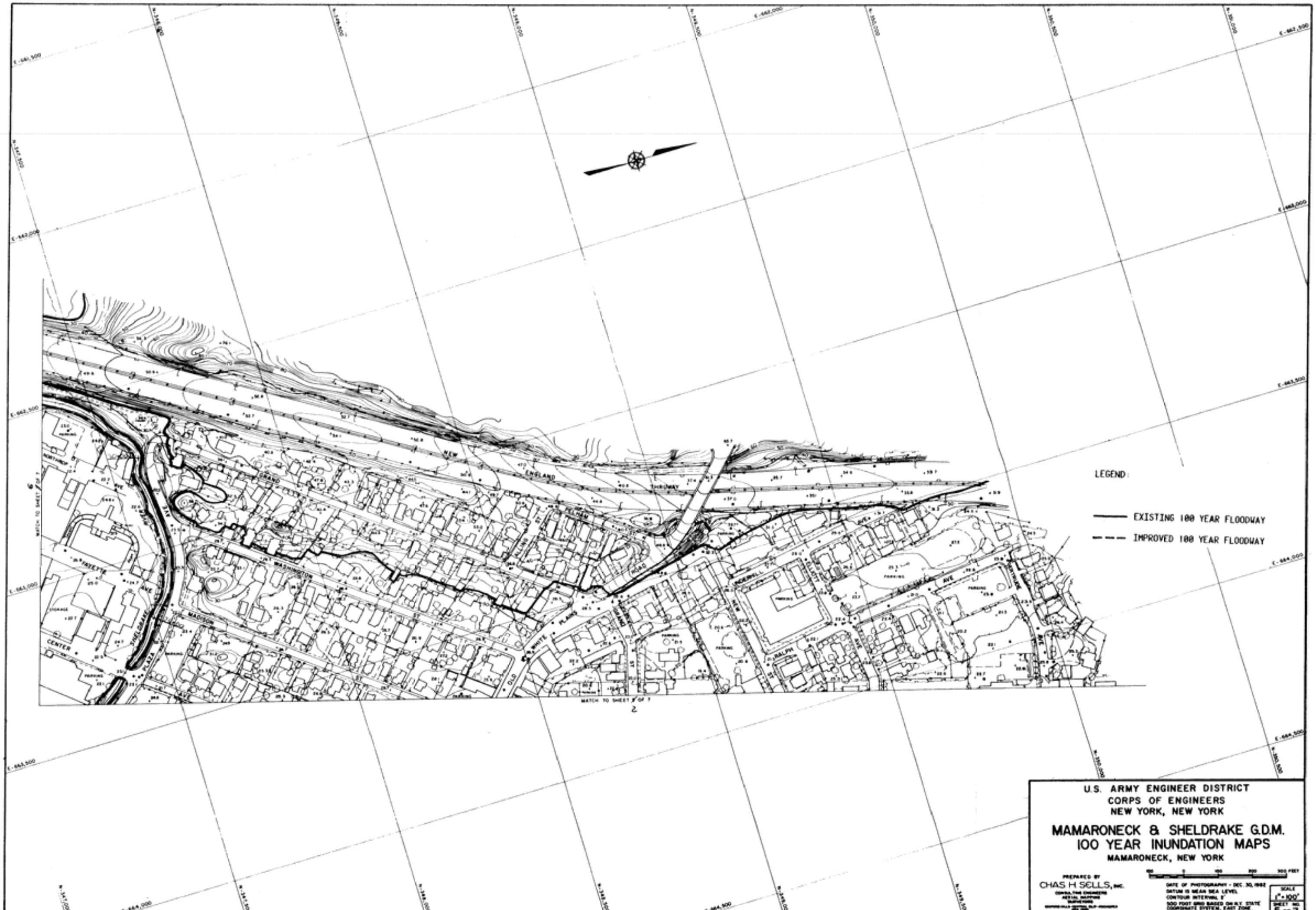
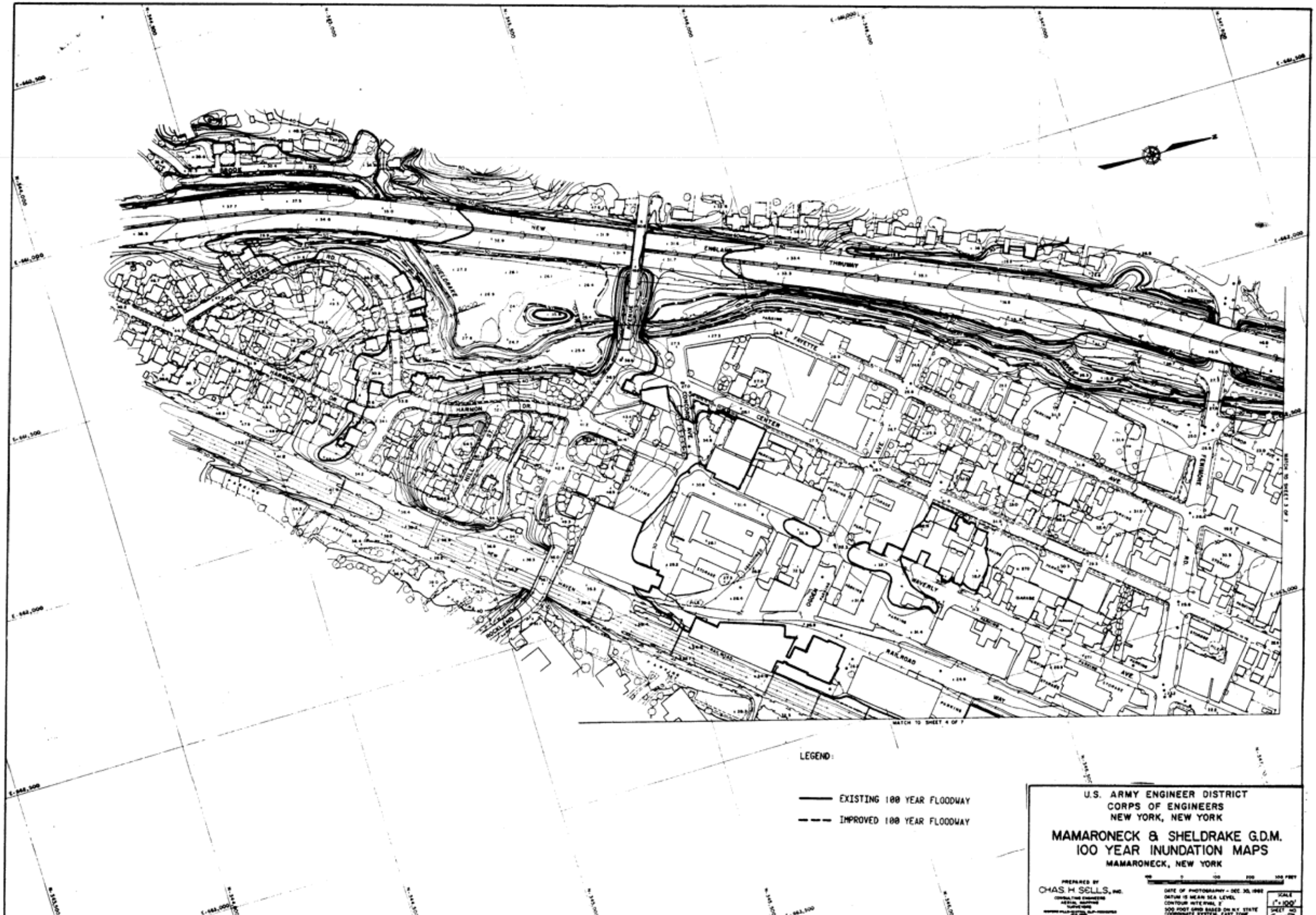
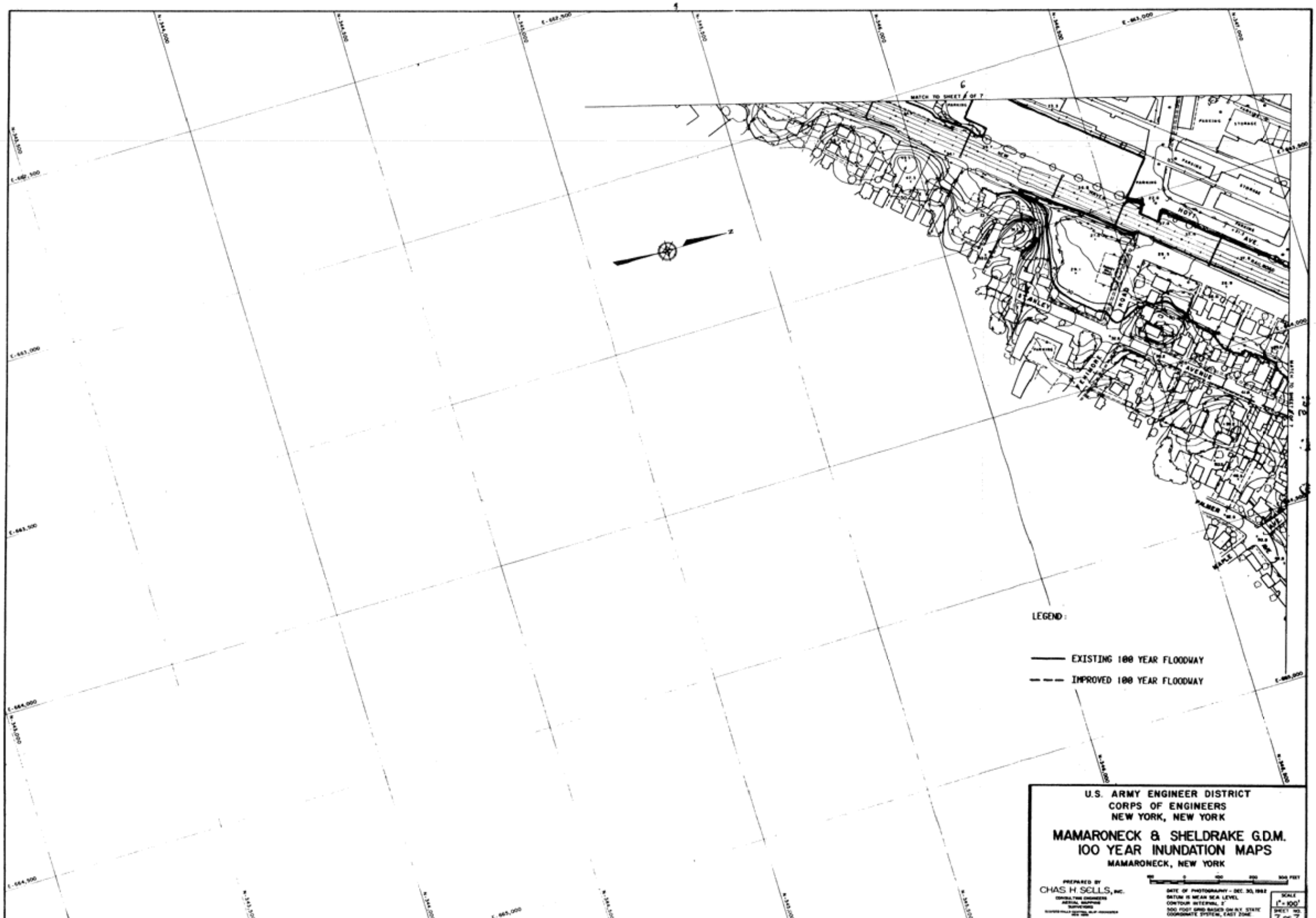

LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
MATCH TO SHEET 2 OF 7
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE.
SCALE
1" = 100'
SHEET NO.
5 OF 7

FIGURE B35



NEW ENGLAND THRUWAY
SHELDRAKE
FAYETTE
CENTER AVE
HARMON DR.
WAVERLY
RAILROAD WAY
FENIMORE RD.
ROCKLAND
MATCH TO SHEET 4 OF 7
LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE
COORDINATE SYSTEM, EAST ZONE
SCALE
1" = 100'
SHEET NO
6 OF 7

FIGURE B36



MATCH TO SHEET 6 OF 7
NEW
HOYT
AVE
STANLEY
ROAD
AVENUE
PARKING
STORAGE
LEGEND:
EXISTING 100 YEAR FLOODWAY
IMPROVED 100 YEAR FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
100 YEAR INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE
SCALE 1" = 100'
SHEET NO. 7 OF 7

FIGURE B 37

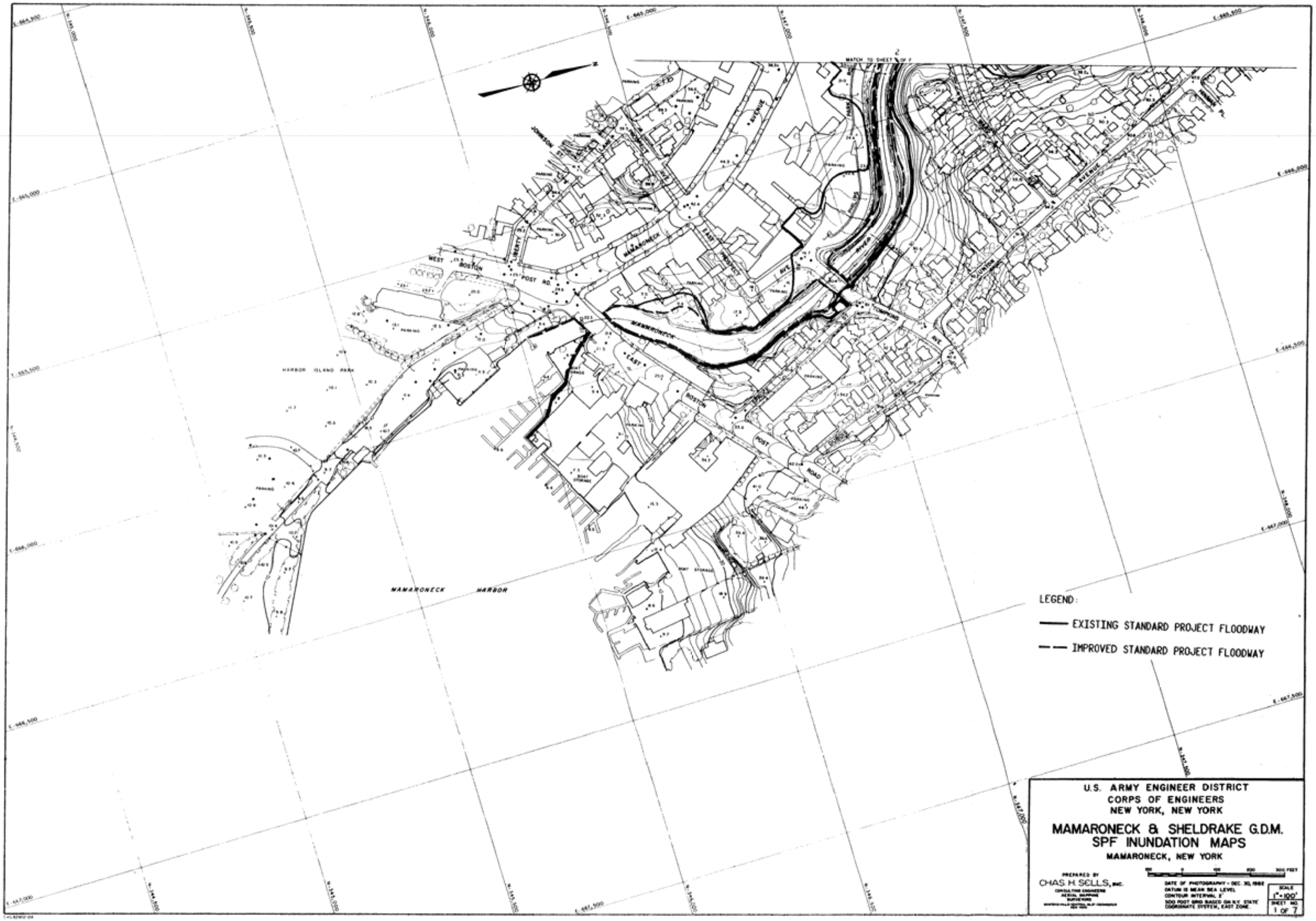
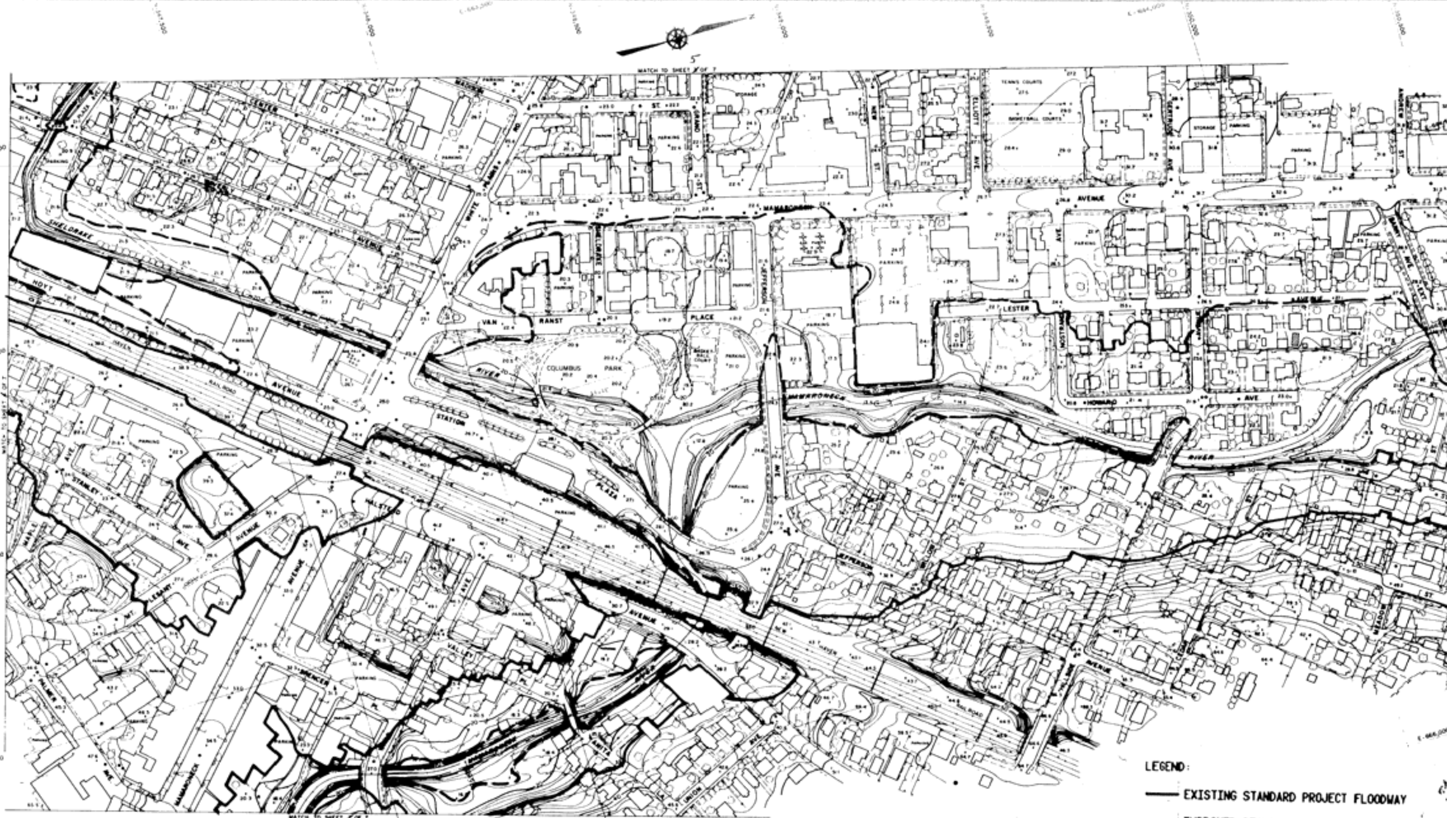

MATCH TO SHEET 2 OF 7
WEST BOSTON POST RD
MAMARONECK AVE
EAST BOSTON POST ROAD
MAMARONECK RIVER
TOMPKINS AVE
UNION AVENUE
HARBOR ISLAND PARK
MAMARONECK HARBOR
BOAT STORAGE
PARKING
LEGEND:
EXISTING STANDARD PROJECT FLOODWAY
IMPROVED STANDARD PROJECT FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
CONSULTING ENGINEERS
AERIAL MAPPING
SURVEYORS
DATE OF PHOTOGRAPHY - DEC. 30, 1982
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE.
SCALE
1" = 100'
SHEET NO.
1 OF 7

FIGURE B 38



LEGEND:

EXISTING STANDARD PROJECT FLOODWAY

IMPROVED STANDARD PROJECT FLOODWAY

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

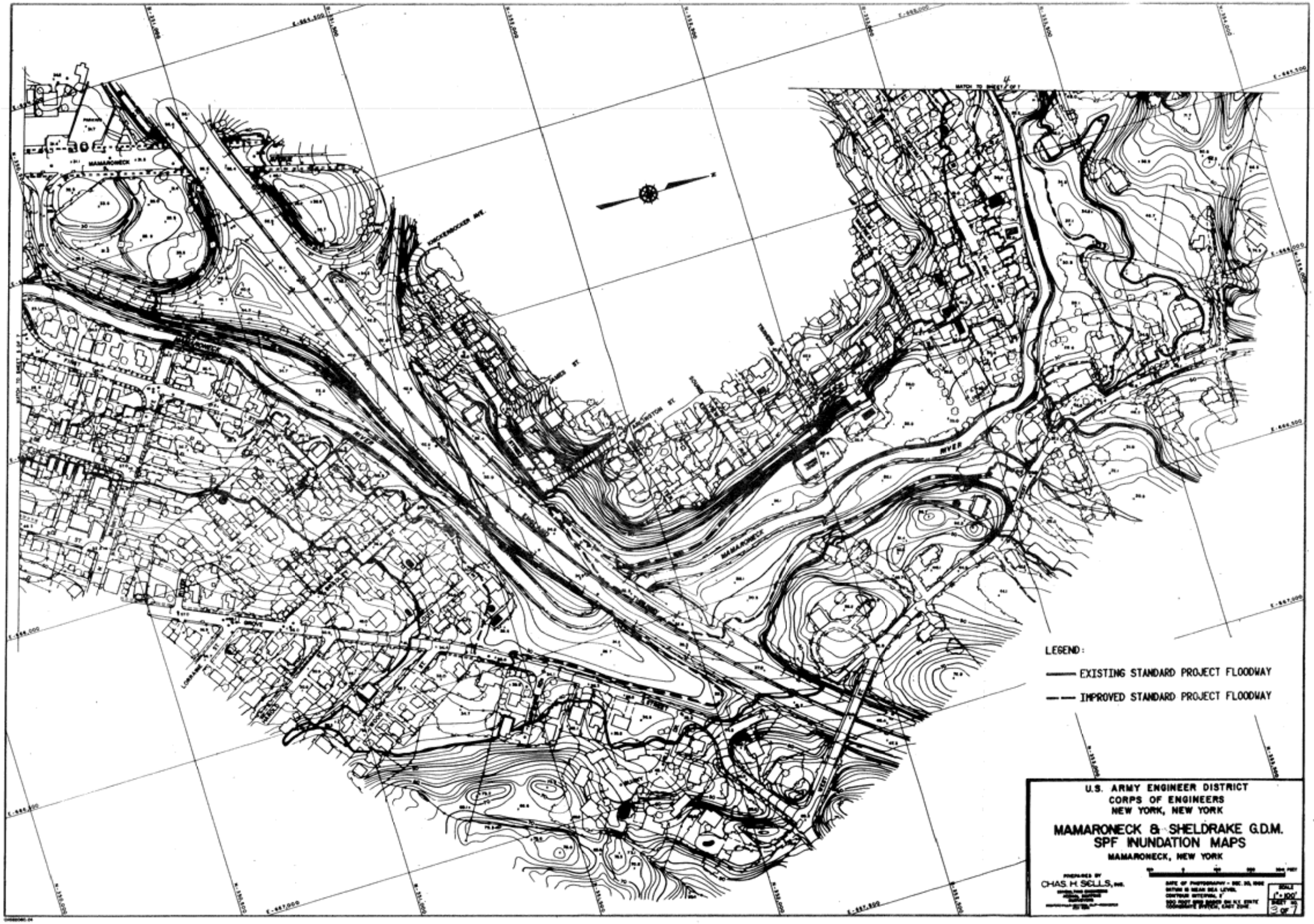
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK

PREPARED BY CHAS H SELLS, INC.

SCALE 1" = 100'

SHEET NO. 2 OF 7

FIGURE B 39



MATCH TO SHEET 4 OF 7
KNICKERBOCKER AVE.
RIVER
MAMARONECK
LEGEND:
EXISTING STANDARD PROJECT FLOODWAY
IMPROVED STANDARD PROJECT FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.

FIGURE B40


LEGEND:
EXISTING STANDARD PROJECT FLOODWAY
IMPROVED STANDARD PROJECT FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK
CHAS. H. SELLS, INC.

FIGURE B41

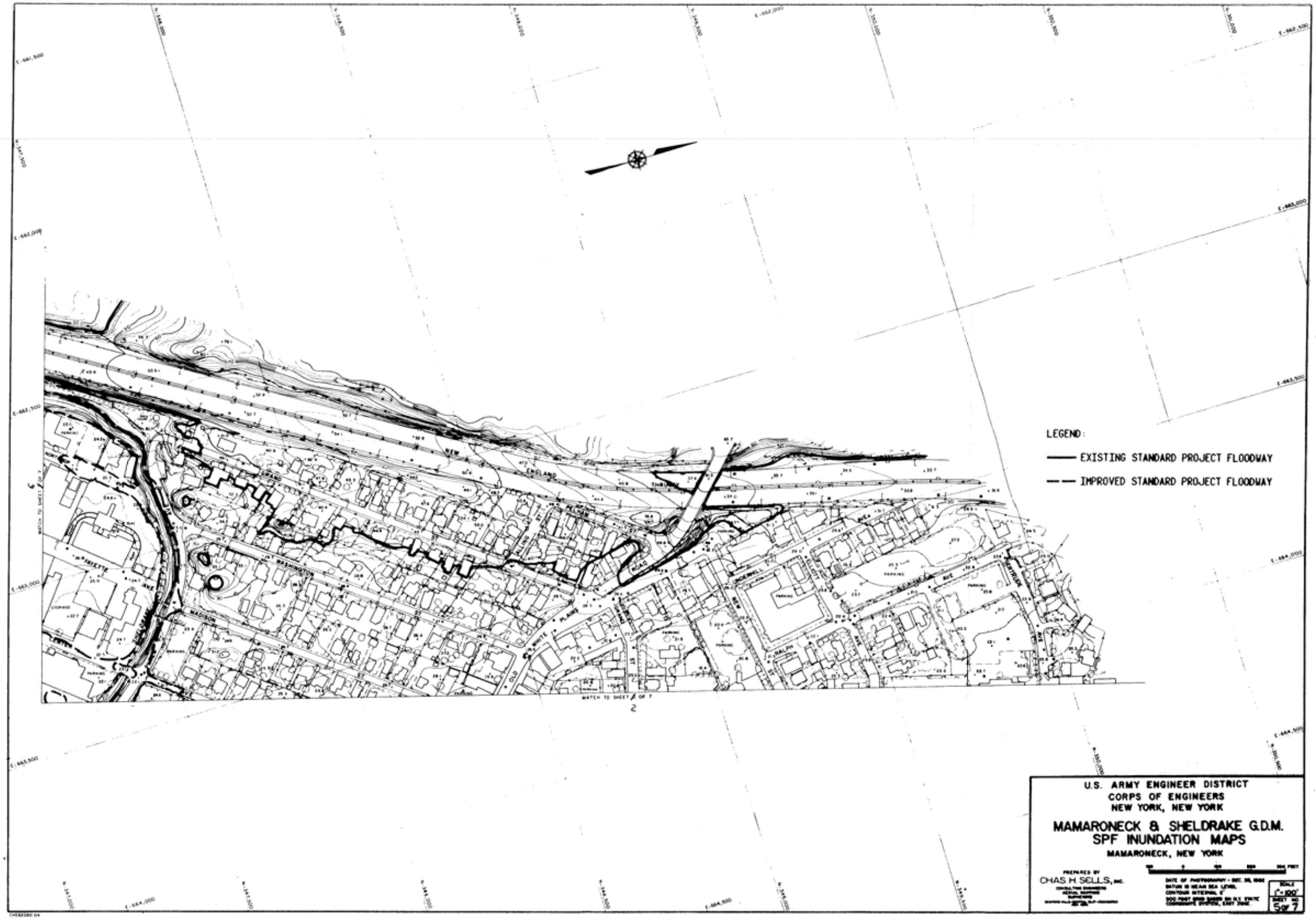

LEGEND:
EXISTING STANDARD PROJECT FLOODWAY
IMPROVED STANDARD PROJECT FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
SCALE
1" = 100'
SHEET NO.
5 OF 7

FIGURE B 42

LEGEND:

—— EXISTING STANDARD PROJECT FLOODWAY

— — IMPROVED STANDARD PROJECT FLOODWAY

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

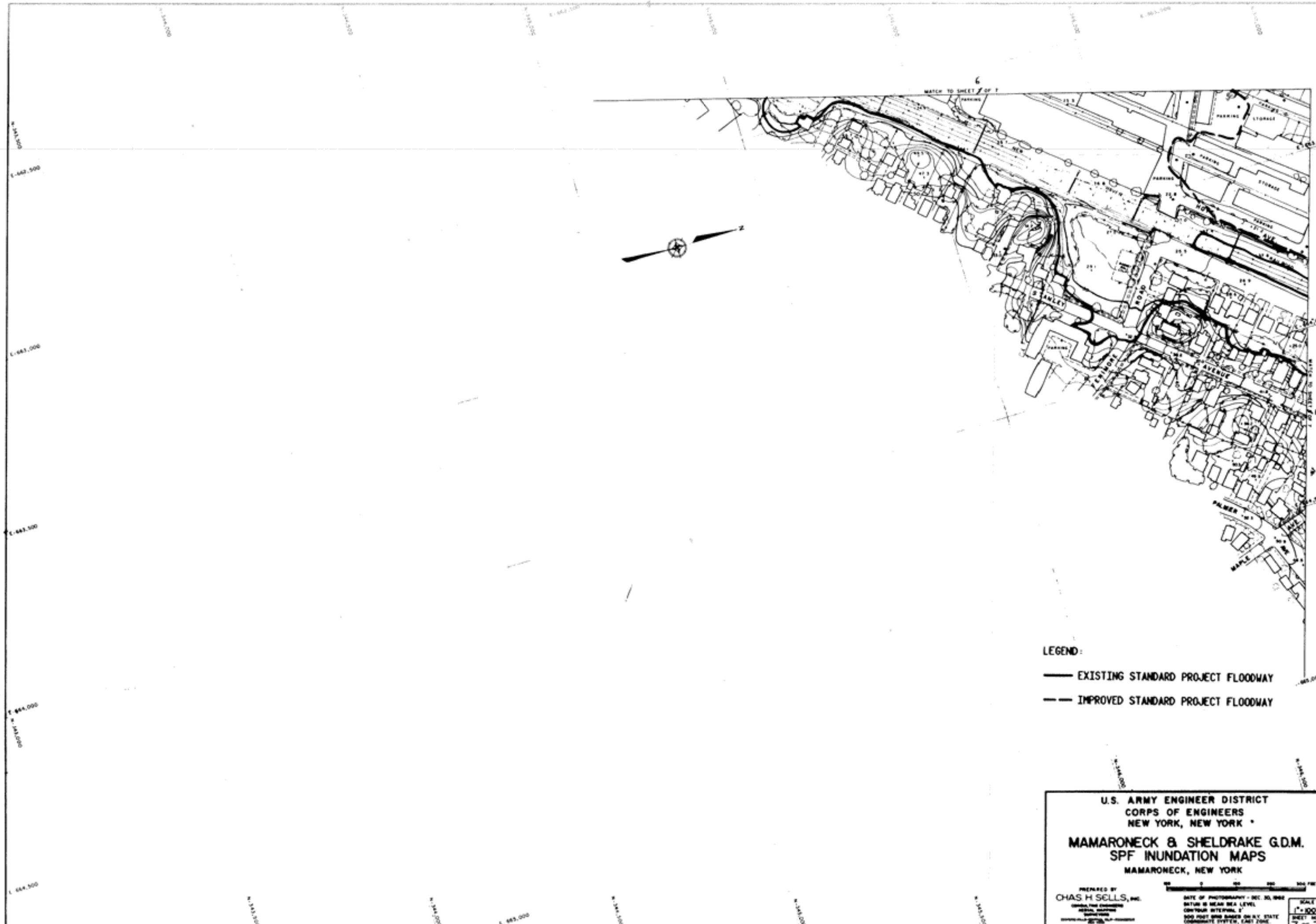
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK

PREPARED BY
CHAS H SELLS, INC.
CONSULTING ENGINEERS
AERIAL MAPPING
SURVEYORS

DATE OF PHOTOGRAPHY - DEC. 30, 1962
DATUM IS MEAN SEA LEVEL
CONTOUR INTERVAL 2'
500 FOOT GRID BASED ON N.Y. STATE COORDINATE SYSTEM, EAST ZONE

SCALE 1" = 100'
SHEET NO. 6 OF 7

FIGURE B43



STANLEY
AVENUE
PALMER
MAPLE
PARKING
STORAGE
LEGEND:
EXISTING STANDARD PROJECT FLOODWAY
IMPROVED STANDARD PROJECT FLOODWAY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK
MAMARONECK & SHELDRAKE G.D.M.
SPF INUNDATION MAPS
MAMARONECK, NEW YORK
PREPARED BY
CHAS. H. SELLS, INC.
SCALE
1" = 100'
SHEET NO.
7 OF 7

FIGURE B 44

ATTACHMENT A

SAMPLE RIPRAP CALCULATIONS

COMPUTATION SHEET		Page 1 of 3
Subject RIPRAP COMPUTATIONS		Project Mamaroneck [illegible]
Computed by R.A.	Date 4/12/88	Checked by Ra — Date

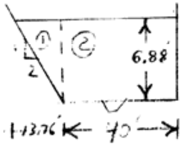
STATION 45+25 Sheldrake River

WSEL = 22.92
INV = 16.04
Q_T = 4039. (200 yr)

'n' = 0.032

A = 322.5

$\overline{V}_{AVE}$ = 12.52
ρ_w = 62.26



ALPHA METHOD - GEOMETRIC COMPUTATIONS

SUB-SECTION	①	②	③	④	⑤	⑥
X_n	13.76	40.0				
Y_n	3.44	6.88				
A	47.33	275.2				
P	15.38	46.88				
R	3.08	5.87				
C	56.0	62.37				
$CR^{1/2}$	[illegible]	151.1				
$CR^{1/2}A$	4,650.0	41,587.	Σ 46,236			
$CR^{3/2}A$	14,307.0	244,129	Σ 258,436			
Q	406.2	3632.8				
V	8.58	13.20				

$$Q = \frac{Q_T \, CR^{1/2}A}{\Sigma CR^{1/2}A}$$

$$CR^{1/2}_{MEAN} = \frac{\Sigma CR^{1/2}A}{A} = 143.37$$

$$\overline{R} = \frac{\Sigma CR^{3/2}A}{\Sigma CR^{1/2}A} = 5.589$$

$$S = \frac{\overline{V}^2}{(CR^{1/2}_{MEAN})^2} = 0.00763$$

NAVY FORM 229 Apr 79

ATTACHMENT A

COMPUTATION SHEET

Subject RIPRAP COMPUTATIONS	Project Mamaroneck & Sheldrake
Computed by R L · Date 4/12/88	Checked by RA · Date

METHOD I LOCAL BOUNDARY SHEAR METHOD

$V = 8.58 \quad Y = 3.44 \quad$ ASSUME $D_{50} = 0.85$

$$\tau_0 = \frac{\gamma V^2}{\left(32.6 \log\left(\frac{12.2\,Y}{D_{50\,MIN}}\right)\right)^2} = \frac{\gamma V^2}{\left(32.6 \log\left(\frac{12.2\,Y}{1.195\,D_{50\,MIN}}\right)\right)^2} = 1.62$$

ALLOWANCE FOR NON-UNIFORMITY OF FLOW (τ_{NUF})

$\tau_0 = 1.5\,\tau_0 = 2.43$

CORRECTION FOR BEND

BEND R/W => PLATE 34 => $\tau_b/\tau_0 = 1.0$

CORRECTION FOR SIDESLOPE

$Z = 2.0$ => PLATE 36 => $K = 0.72$

RIPRAP DESIGN SHEAR

$\tau = .09(\gamma_s - \gamma_w) D_{50\,MIN} K \Rightarrow D_{50\,MIN} = \frac{\tau}{.09(\gamma_s - \gamma_w) K} = 0.$

SUBSECTION	1	2	3	4	5
V	8.58	13.2			
Y	3.44	6.88			
ASSUME D50 MIN	0.85	1.15			
τ_0	1.62	3.14			
τ_0 NUF	2.43	4.71			
BEND R/W	9.13	9.13			
τ_b/τ_0	1.0	1.0			
τ_b	2.43	4.71			
Z	2.0	0			
K	0.72	1.0			
D50 MIN	0.82	1.15			

ATTACHMENT A

COMPUTATION SHEET		Page 3 of 3	
Subject RIPRAP COMPUTATIONS		Project	
Computed by R A	Date	Checked by Ra	Date

METHOD 2 AVERAGE BOUNDARY SHEAR METHOD

$\bar{R} = 5.589 \quad S = 0.00763$

$\tau_0 = \gamma \bar{R} S = 62.4(5.589)(0.00763) = 2.66$

$\tau = 1.5 (\text{NUFF})(2.66) = 2.25$

CORRECTION FOR BEND $\tau = \tau_b / \tau_0 \cdot \tau = 2.25$

CORRECTION FOR SIDESLOPE $Z = 2.0$, $K = 0.72$

RIPRAP DESIGN SHEAR

$\tau = .04(\gamma_s - \gamma_w) D_{50}$

$D_{50}\,(\text{BOTTOM}) = \dfrac{2.25}{(.04)(102.6)} = 0.55$

$D_{50}(\text{SIDESLOPE}) = 0.55/.72 = 0.76$

Selection of Layer Thickness

$D_{50_{min}} = 1.15 \Rightarrow W_{50_{min}} = \dfrac{\pi (D_{50min})^3}{6} \gamma_s = \dfrac{\pi (1.15)^3 165}{6} = 131.4^{\#}$

from ETL 1110-2-120 $W_{50_{min}} = 131.4$ lbs $\Rightarrow$ Layer $= 24''$

use a continuous layer; bottom and sides

NAVY FORM 228 Apr 79

ATTACHMENT A