MAMARONECK & SHELDRAKE RIVERS
NEW YORK
FLOOD RISK MANAGEMENT
GENERAL REEVALUATION REPORT
FOR
THE VILLAGE OF MAMARONECK

Appendix C1: Hydrology

US Army Corps of Engineers®
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EXECUTIVE SUMMARY

This is an executive summary of the documentation of the existing and improved conditions hydrology analysis and modeling that was completed for the current flood risk management study of the Mamaroneck and Sheldrake Rivers in Westchester County, New York. It is both a continuation and updating of the hydrology of the 1989 GDM. The documentation consists of text, tables and figures.

The hydrologic modeling procedure of the current effort is both a continuation and an update of the hydrologic modeling that was completed for the 1989 GDM. The storms of water years 1990 through 2010 were added to the analysis. These most recent storms are of special importance, because they occurred after the USGS gage on the Mamaroneck River downstream of Halstead Avenue, the major calibration point of the hydrology of this study, was discontinued.

The seven most significant storms of these water years, those of May 17 1990, April 16 1996, October 19 1996, September 15-16 1999 (Tropical Storm Floyd), September 8 2004, October 7-16 2005, and the "Tax Day" nor'easter storm of April 15-16 2007, are described in separate paragraphs. This most recent significant storm resulted in the largest historic flood to date in the Mamaroneck and Sheldrake Rivers Basin.

The update included a conversion of the HEC-1 model used for the GDM to the current hydrologic computer program HEC-HMS. The percent impervious area values of the hydrologic model sub-basins were updated using the latest current available GIS data and Google Earth aerial photography, to account for land use changes that occurred since the GDM. The storage-discharge data of the hydrologic model was updated using storage-discharge data from the current HEC-RAS water surface profile hydraulic computer model. This hydraulic model was calibrated to high water marks for the April 15-16 2007 event, which is the largest flood of record.

The quality and accuracy of the hydrologic model from the GDM was verified by using it to reproduce two historic floods that had occurred since the GDM; those of July 1984 and May 1989. The April 15-16 2007 flood was also reproduced and confirmed in conjunction with the hydraulic model's calibration.

The flood frequency analysis of the USGS gage on the Mamaroneck River downstream of Halstead Avenue had to be updated from the GDM. The first step was to reconsider the historic urbanization adjustment of flood peak history documented in the GDM.

The only historic urbanization adjustment done in the GDM was to update the peak flow of the September 15 1944 flood event. In the current analysis, the flood peaks of water years 1945 through 1973 were also updated to present conditions, to more completely account for the historic urbanization that occurred in the Mamaroneck and Sheldrake Rivers basin in that time period.

The next step was to account for the flood peak history that had occurred since the completion of the GDM hydrology, that of water years 1983 through 2010.

Gaged peak flow data was available for water years 1983 through 1989, 1999, 2009 and 2010, and was gathered.

A function of total runoff vs. total rain was estimated from the hydrology of the GDM and used to estimate the gaged peaks of floods for which claims were made to FEMA's National Flood Insurance Program: May 1990, April 1996, October 1996 and September 2004. The flood peaks of July 24 1938, October 2005 and April 2007 were also estimated.

The remaining missing flood peak history at the Mamaroneck gage at Halstead Avenue was estimated by correlation with peak flow data of two adjacent gaged basins : Saw Mill River at Yonkers NY (water years 1993-1995, 2008) and Norwalk River at South Wilton, Connecticut (water years 1991-92, 1998, 2000-03, and 2005).

The result was a homogeneous sample of annual peak discharges, representing current hydrologic conditions, from which an accurate and valid existing conditions peak discharge vs. frequency calculation was made using accepted statistical procedures.

The existing conditions peak discharge vs. frequency curve for water years 1938, and 1944 through 2010, for the long term USGS gage Mamaroneck River at Mamaroneck at Halsted Avenue, was computed. The hydrologic model of the basin was calibrated to this peak discharge vs. frequency curve by using the specific-frequency hypothetical rainfall and adjusting the rainfall infiltration loss parameters until the computed peak discharges matched the gage peak discharge vs. frequency curve. This was done to compute existing conditions specific-frequency hypothetical peak discharges throughout the basin, including the locations at which flood damage reduction measures are proposed.

Floods modeled included the historic April 2007 flood, and the hypothetical 1, 2, 5, 10, 25, 50, 100, 200, 250 and 500 year floods. The Standard Project Flood was also modeled.

A comparison was made between the current existing conditions discharge vs. frequency curve at the Mamaroneck gage and the frequency curve in the GDM. The differences between the two curves are documented and explained in this appendix, along with the reasons why the current frequency curve is more accurate than the frequency curve in the GDM.

Westchester County planning officials, and representatives of Mamaroneck River basin communities, were consulted about possible future development of currently undeveloped land areas within the basin, to provide input to hydrologic modeling of future unimproved conditions.

Future values of sub-basin percent impervious area were estimated from this data and input to the hydrologic model of the basin. The model was then run to compute future unimproved conditions peak discharges throughout the basin. Increases from existing conditions peak discharges were small, ranging from zero to 2.3 %.

Hydrologic parameters of risk and uncertainty for both existing and future unimproved conditions peak discharge vs. frequency were determined and documented.

MAMARONECK & SHELDRAKE RIVER BASINS
WESTCHESTER COUNTY, NEW YORK
GENERAL RE-EVALUATION REPORT (GRR)
FLOOD RISK MANAGEMENT STUDY

1.0 OBJECTIVE OF STUDY

This appendix documents the existing hydrologic analysis conditions, including modeling, that was conducted for the current flood risk management study of the Mamaroneck and Sheldrake Rivers basin in Westchester County, New York. It is both a continuation and update of the hydrology documented in the 1989 GDM.

1.1 Basin Description

The Mamaroneck and Sheldrake Rivers watershed is 23.63 square miles in area, and is located along the northwestern coast of Long Island Sound within the New York City metropolitan area. The Mamaroneck and Sheldrake Rivers Basin lies entirely within Westchester County, New York and contains portions of the Village and Town of Mamaroneck, the City of White Plains, the Village and Town of Harrison, Village of Larchmont, the City of Rye, North Castle, and the Village of Scarsdale. The multi-stem watershed is roughly elliptical in shape, with a maximum length of 9 miles in a north south direction, and an east-west width of 2 to 3 miles. The terrain is gently rolling and lightly wooded in its upstream end and generally cleared in its lower valley. The ridges extend generally in a north-south direction. The watershed is suburban in nature, and drains into Long Island Sound. The watershed is shown on Figure 1.

1.2 Project Area

Specifically, the study area is defined by the flood damage areas located in the Village of Mamaroneck and the Town of Harrison (Figure 3). On the Mamaroneck River, the damage area extends from below Tompkins Avenue upstream to the Westchester County Joint Water Works Dam. On the Sheldrake River damages occur from the confluence with the Mamaroneck River upstream to the Village line at the New England Thruway (I-95) Bridge.

1.3 Climate

The study area has a moderate climate, with an average temperature of 51 degrees Fahrenheit. The extreme temperatures observed, based on the available data for all stations, are 18 degrees Fahrenheit below zero and 105 degrees Fahrenheit above zero at Scarsdale, New York. The average growing season is 184 days. The relative humidity averages about 67 percent. The prevailing winds are from the northwest with an average velocity of 14 miles per hour. The average annual precipitation is approximately 44.2 inches.

1.4 Peak Discharge Records

There are, at present, two active USGS recording stream gages recording peak discharges in the study area, in the Village of Mamaroneck. The short-term, upstream gage is located on the Mamaroneck River, at the Winfield Avenue bridge, a few hundred feet downstream of the Westchester Joint Waterworks (Mamaroneck Reservoir) Dam, and several miles upstream of the confluence with the Sheldrake River. Its USGS i.d. number is 01300800.

The downstream, long-term gage is the calibration point of the current hydrologic analysis. It is located on the Mamaroneck River, 113 ft. downstream from the Halstead Avenue bridge, and 700 ft. downstream of the confluence with the Sheldrake River. It is referred to hereinafter as “the Mamaroneck gage”. The drainage area at this gage is 23.46 square miles. Its USGS i.d. number is 01301000. Data from it is given in Tables 1 through 3.

1.5 Average Discharge

The 43-year average annual discharge at the Mamaroneck gage for water years 1945-52, 1955-89, is 35.9 cfs, which is equivalent to about 20.8 inches of runoff or about 46.7 percent of the estimated average annual basin rainfall of 44.2 inches.

2.0 STORM TYPES

The greatest floods have been caused by (1) intense rain of transcontinental type storms (2) localized thunderstorms (3) tropical storms of West Indian origin or (4) less intense rains of long

duration falling on snow-covered, frozen, or saturated ground. As such, flood-producing storms can, and do, occur at any time of the year.

2.1 Storms of Water Years 1990-2010

The storms for water years 1990-2010 are of concern, because these storms occurred after the gage at Halstead Avenue (the Mamaroneck gage) was discontinued at the end of water year 1989, ending a continuous long-term record seven of the most significant storms of this period are described below. Four of these storms caused flooding within the study area, for which flood damage claims were filed with FEMA's National Flood Insurance Program (NFIP).

2.1.1 May 17, 1990 storm and flood:

The flood in the study area that resulted from this storm was one for which flood damage claims were filed with FEMA's National Flood Insurance Program (NFIP). Total basin rainfall was estimated at 2.71 inches using rain data from the White Plains Maple Moor and Westchester County Airport rain gages. It was estimated to have caused a water year 1990 annual peak flow of 2130 cfs at the Mamaroneck gage.

2.1.2 April 16, 1996 storm and flood:

The flood in the study area that resulted from this storm was one for which flood damage claims were filed with FEMA's National Flood Insurance Program (NFIP). Total basin rainfall was estimated at 3.14 inches using rain data from the Pleasantville, Dobbs Ferry-Ardsley and Westchester County Airport rain gages. It was estimated to have caused a water year 1996 annual peak flow of 2250 cfs at the Mamaroneck gage.

2.1.3 October 19-22, 1996 nor'easter storm and flood:

The storm that caused this flood developed when a low-pressure area approaching from the Midwest was blocked by a dip in the jet stream. The low slid off the New Jersey coast and reformed the night of Friday, October 18, 1996, funneling much rain onshore. High pressure to the north also blocked the storm, causing it to drift slowly and drop rain all the way. Winds of 50 mph off the ocean were produced by the pressure difference between the high and low systems.

Meanwhile, Hurricane Lili, out over the Atlantic Ocean, about 1000 miles to the southeast, fed even more moisture into the storm.

The flood in the study area that resulted from this storm was one for which flood damage claims were filed with FEMA's National Flood Insurance Program. Total basin rainfall was estimated at 4.40 inches using rain data from the Dobbs Ferry-Ardsley, Pleasantville, and Westchester County Airport rain gages. It was estimated to have caused a water year 1997 annual peak flow of 3060 cfs at the Mamaroneck gage.

2.1.4 Storm of September 15-16, 1999 (Tropical Storm Floyd):

Floyd made landfall on September 16, 1999 near Cape Fear, North Carolina, with Category Two winds of 105 mph. After crossing eastern North Carolina and Virginia, Floyd weakened to a tropical storm. Its center moved offshore along the coasts of the Delmarva Peninsula and New Jersey. On September 17th Floyd moved over Long Island NY, making landfall again roughly at the Queens-Nassau counties border, and New England, where it became extratropical.

Rainfall totals from Floyd were as high as 12 to 16 inches over portions of New Jersey, 4 to 8 inches over southeastern New York, and up to 11 inches over portions of New England. The inland flooding from Floyd was a disaster of immense proportions in the Eastern United States, particularly in North Carolina. The 56 USA direct deaths due to Floyd is the largest hurricane death toll since Agnes caused the deaths of 122 people in 1972. Estimates of total USA damages range from three to over six billion dollars.

Floyd resulted in new flood peaks of record at at least sixty stream gages within the portions of New Jersey and New York within New York District's civil works boundaries.

However, Floyd caused only moderate flooding at the Mamaroneck gage, at which a peak flow of 2230 cfs was recorded, only 4.2 % greater than the current two year peak flow of 2140 cfs computed at that gage (see section 1.17.8 below). Total rainfall over the study basin was estimated at 7.18 inches, from the Dobbs Ferry-Ardsley and Westchester County Airport rain gages, but which resulted in a total estimated runoff of only 2.27 inches due to the drought condition that preceded Floyd.

2.1.5 Storm of September 8, 2004:

The flood in the study area that resulted from this storm was one for which flood damage claims were filed with FEMA's National Flood Insurance Program (NFIP). Total basin rainfall was estimated at 5.55 inches using rain data from the Dobbs Ferry-Ardsley and Westchester County Airport rain gages. It was estimated to have caused a water year 2004 annual peak flow of 4620 cfs at the Mamaroneck gage.

The annual peak flow of water year 2004 at the adjacent USGS-gaged basins Saw Mill River at Yonkers NY and Norwalk River at South Wilton CT occurred from another storm ten days later on September 18 2004. This later storm was investigated as it occurred over the Mamaroneck and Sheldrake Rivers basin and was found to have a total basin rainfall of 2.65 inches. It was estimated to have caused a peak flow of 2460 cfs at the Mamaroneck gage.

The water year 2004 annual peak flow at the Mamaroneck gage was therefore taken as the 4620 cfs peak resulting from the September 8, 2004 storm as described above.

2.1.6 Storms of October 7-16, 2005:

The floods in the study area that resulted from these storms as they occurred over the Mamaroneck and Sheldrake River basin were not floods for which flood damage claims were filed with FEMA's National Flood Insurance Program, but they were investigated with a hydrologic model because these storms were known to be prolonged and intense.

Total basin rainfall for the October 7-8, 2005 storm was estimated at 5.31 inches using rain data from the Dobbs Ferry-Ardsley and Westchester County Airport rain gages. It was estimated to have caused a water year 2006 annual peak flow of 2830 cfs on October 9 2005 at the Mamaroneck gage.

Total basin rainfall for the October 12-16 2005 storms was estimated at 7.29 inches using rain data from the Dobbs Ferry-Ardsley and Westchester County Airport rain gages. It was estimated to have caused a peak flow of 2560 cfs at the Mamaroneck gage, less than the estimated October 9, 2005 peak flow of 2830 cfs.

The water year 2006 annual peak flow at the Mamaroneck gage was therefore taken as the estimated 2830 cfs peak resulting from the October 7-8 2005 storm as described above.

2.1.7 Nor'easter storm of April 15-16, 2007:

The April 15-16, 2007 nor-easter storm dropped about three to ten inches of rain on the watersheds within the New York District's civil works boundaries between the early morning of Sunday April 15, 2007 and the early afternoon of Monday April 16, 2007, resulting in new flood peaks of record at ten USGS gages in New Jersey, and, in New York, at the USGS gages Saw Mill River at Yonkers and, most importantly for the present study, Mamaroneck River at Mamaroneck (Halstead Avenue) New York.

The approximate time distribution of the total rainfall of the April 15-16, 2007 nor-easter over the watersheds of the New York District was an average of 7 to 7 ½ inches between about 2 a.m. on Sunday April 15th to 2 p.m. on Monday April 16th 2007, with most rain within the 24 hours beginning at 2 a.m. on Sunday the 15th. Greatest hourly amounts were from 0.6 to 0.8 inches at approximately 2 p.m. on Sunday April 15th 2007.

Unlike Tropical Storm Floyd, which broke the summer 1999 drought and fell on dry ground, the April 2007 nor'easter caused significant flooding because it was preceded by the smaller March 1-2 and April 12-13, 2007 storms, and, as such, and for other reasons of antecedent soil moisture conditions, fell on saturated ground.

The March 1-2, 2007 storm also caused flooding within the Mamaroneck River basin, albeit far less than the flooding resulting from the April 15-16, 2007 storm. Total basin rainfall for this storm was estimated at 3.18 inches using rain data from the Dobbs Ferry-Ardsley and Westchester County Airport rain gages. It was estimated to have caused a peak flow of 2680 cfs at the Mamaroneck gage.

The April 15-16, 2007 nor'easter had a drop in central pressure of 28 millibars in 24 hours, which qualified it as a meteorological bomb, a drop in central pressure of at least 24 millibars in

24 hours. The lowest central pressure of about 966 millibars is near the border of the pressure defined Categories 2 and 3 once used on the Saffir-Simpson Hurricane Scale.

A peak flow of 5330 cfs on April 15 2007 was estimated at the Mamaroneck gage by hydrologic modeling, and subsequently confirmed by hydraulic calibration to recorded high water marks on the Mamaroneck and Sheldrake Rivers. This is a 100 year flood peak on the current existing conditions peak discharge vs. frequency curve at the Halstead Avenue gage. See Section 3 below.

Total rainfall over the study basin was estimated at 7.90 inches using data from the Westchester County Airport rain gage, and local observations of total storm rainfall at East White Plains and New Rochelle, New York.

3.0 HYDROLOGIC MODELING PROCEDURE

The HEC-HMS computer program (Hydrologic Modeling System, U.S. Army Corps of Engineers) was used to hydrologically model the Mamaroneck and Sheldrake Rivers watershed to the USGS gage # 01301000, Mamaroneck River at Mamaroneck. This gage at Halstead Avenue served as the downstream boundary condition and calibration point of the hydrologic model.

The earlier HEC-1 (Flood Hydrograph Package) model input files used in the hydrology of the 1989 GDM were successfully imported into, and used, in HEC-HMS.

The HEC-HMS model structure used for the present GRR is the same as the HEC-1 model structure used in the hydrology of the 1989 GDM, and is described below. A diagram of it is shown on Figure 2.

The watershed was divided into twenty-one (21) sub-basins, fifteen (15) for the Mamaroneck River and six (6) for the Sheldrake River, with eleven (11) routing reaches and thirty (30) combining points defined for the purpose of the hydrologic analysis and calibration.

The following conventions were used for the names of the HEC-HMS model nodes:

- "S" denotes a sub-basin hydrograph computation, with input rainfall data, a drainage area, an infiltration loss function, a percent impervious area, and a unit hydrograph.
- "C" denotes nine of the upstream-most hydrograph combinations.
- "RT" denotes a hydrograph routing. Special names were assigned to routings through reservoirs as given below :
- RESFLO : Route through Mamaroneck Reservoir
- RESOUT : Route through Sheldrake Lake

The 15 Mamaroneck River sub-basins were denoted by the single letters A through O, and the 6 Sheldrake River sub-basins were denoted by the double letters AA through FF.

The adopted Clark unit hydrograph parameters of the sub-basins are shown on Figure 2.

They remain unchanged from their 1989 GDM values, because they were re-verified by post-GDM historic flood reproduction, as described in Section 3 below.

The HEC-HMS model nodes within the project area, with their descriptions and contributing drainage areas, starting at the upstream-most point, and working downstream to the mouth of the Mamaroneck River, are shown in Table 4 and are shown on Figure 3.

3.1 Existing Conditions percent impervious area

The values of sub-basin percent impervious area developed for the 1989 GDM were used as a basis for this report. Three significant changes were made to account for the following developments : 1) Changes in land use had occurred in the basin since the values of percent impervious area for the hydrologic model sub-basins had first been determined for the hydrology of the GDM in spring 1983. 2) Additional data such as average lot size of old and new residential areas and land use data from GIS sources. 3) Some of the values of sub-basin RTIMP used in the GDM appeared to have been underestimated.

The following land use types, and their corresponding values of percent impervious area, were identified within the basin as follows:

- Undeveloped open space: forest, lawn, parks, golf courses: assumed 2 % impervious to be conservative. Even totally undeveloped areas have some impervious surfaces such as exposed rock.

- Residential: varies from about 20 % to 55 % impervious. For residential areas, % impervious area was determined from average lot size, as determined from pilot areas on Google Earth aerial photography, using the relation from NRCS Publication TR-55, Urban Hydrology For Small Watersheds (Table 2-2a, pg. 2-5).

- For Lakes, Ponds, and Reservoirs, taken as 100 % impervious, because they are flat horizontal water surfaces that take in rainfall without any infiltration loss and instantaneously transform it into streamflow.

- The remaining land use types (i.e. Paved Roadways, Industrial, Parking Lots, etc.) and their values of percent impervious area were also taken from NRCS Publication TR-55 (Table 2-2a, pg. 2-5).

The percents by area of each of these land use types within each of the twenty-one hydrologic model sub-basins were calculated and measured, and then used to compute the area-averaged current existing conditions values of percent impervious area for each of the twenty-one hydrologic model sub-basins. The values are given in Table 5 and were input to the HEC-HMS models of the Mamaroneck and Sheldrake Rivers basin.

3.2 Existing Conditions storage-discharge data

A HEC-RAS computer model of the Mamaroneck and Sheldrake Rivers was developed for the current GRR using 2004 arial survey data and updated (2010) cross-section data. The model was calibrated to high water marks of the April 15-16, 2007 flood, which is the current flood of record. Please see Appendix C2 Hydraulics for further details.

The HEC-RAS model was run with a full range of hypothetical peak flows ranging from the smallest (1 year flood) to the largest (Standard Project Flood) to generate storage-discharge data for proposed project reaches of the Mamaroneck and Sheldrake Rivers. This data was then input

to the appropriate routing reaches of the HEC-HMS models of the Mamaroneck and Sheldrake River basin. This provided revised flows which changed by only a few percent.

3.3 Post-GDM Historic Flood Calibration

The storms and floods of July 5-7, 1984 and 16 May 16, 1989 were selected for reproduction to re-verify the HEC-1 model updated from the GDM. The July 5-7, 1984 flood was selected for reproduction, because it had the largest annual peak flow of the six post-GDM water years for which full discharge hydrographs were available, before the gage at Halstead Avenue was discontinued. The latest and most recent historic flood reproduction shown in the 1989 GDM was that of the April 10-11, 1983 flood. The May 16, 1989 flood was also selected for reproduction, because it was a smaller flood, slightly smaller than a one year peak flow at the Mamaroneck gage.

The two flood reproductions re-verify the quality of the HEC-HMS model as updated from the HEC-1 model from the GDM. They are shown on Figures 6 and 7.

3.4 Flood Frequency Analysis – Existing Conditions

The one long-term USGS-stream gaged location in the Mamaroneck and Sheldrake Rivers basin is the Mamaroneck River at Mamaroneck NY (Halsted Avenue), also known as “the Mamaroneck gage”. Its existing conditions peak discharge vs. frequency curve was revised and updated to include the twenty-eight years of flood peak history that had accumulated since the completion of the hydrology for the GDM: water years 1983 through 2010. This USGS-stream gaged location forms the downstream boundary condition, and hydrologic calibration point, of the study area.

3.4.1 Historic Urbanization Adjustment – 1989 GDM:

The only annual peak discharge recorded by the USGS-stream gage, Mamaroneck River at Mamaroneck NY (Halsted Avenue), that was updated to present conditions in the GDM was that of water year 1944, September 15, 1944. The updated peak flow value of 4,000 cfs was used in the current GRR flood frequency analysis. Further historic urbanization adjustments of gage-

recorded peak flow data from water years 1945 through 1973 was done as well, as described in subsequent sections.

3.4.2 Historic Urbanization Adjustment – Current GDM:

A plot of the annual peak flows recorded by the USGS-stream gage, Mamaroneck River at Mamaroneck NY (Halsted Avenue) from 1945 through 1973 shows a distinct upward trend that was probably caused by the urbanization that occurred in the Mamaroneck and Sheldrake Rivers basin in that time period. The plot is shown as Figure 9.

This is corroborated by the updating to present conditions of the September 15, 1944 flood peak that was documented in the GDM. The storms of September 14, 1944 and September 26-27, 1975 were similar in total rainfall, duration, and maximum hourly intensity, yet the earlier storm resulted in a recorded peak flow of only 1760 cfs at the Mamaroneck gage, whereas the later storm resulted in a peak flow of 3700 cfs, the flood of record, before the flood peaks of September 8, 2004 and April 15, 2007 surpassed it. In the hydrology of the GDM, once the September 14 1944 storm was run in HEC-1 with the same infiltration loss parameters as the September 1975 flood, a peak flow of 4000 cfs resulted, close to the 3700 cfs peak of the September 1975 flood.

This suggested a need to update to present conditions the rest of the flood peaks of water years 1945 through 1973, after which the urbanization of the Mamaroneck and Sheldrake Rivers basin seems to have reached its present value.

This urbanization consisted of a change in the basin from a pre-World War II, mostly rural condition, to a post-World War II, mostly urban and suburban residential condition. As this post-WW II “baby boom” urbanization occurred, pervious undeveloped areas of vegetated open land and open natural stream channels were replaced by manmade impervious surfaces of pavement and the roofs of houses, and storm sewers, respectively. This made the Mamaroneck and Sheldrake Rivers basin a more efficient flood peak producer, in that the increase in impervious surfaces and storm sewerage both increased the amount of runoff and enabled it to reach the main stream system more quickly.

An updating function was applied to the annual peak discharges of water years 1945 through 1973 to remove their upward trend, and generate for these water years, part of a homogeneous sample of annual peak discharges representing current hydrologic conditions, from which an accurate and valid existing conditions peak discharge vs. frequency calculation could be made using accepted statistical procedures.

The updating function is shown on Figure 10. The recorded annual peak discharges, and their updated values resulting from the updating function, are given in Table 1.

A plot of ALL the annual peak discharges (updated, recorded, and estimated as necessary, as described in subsequent sections) versus water years is shown on Figure 13. It shows NO upward trend, and shows it to be a homogeneous sample of annual peak discharges representing current hydrologic conditions, from which an accurate and valid existing conditions peak discharge vs. frequency calculation can be made using accepted statistical procedures.

The shape of Figure 10 is similar to that of analogous figures in the hydrology appendices of three earlier CENAD and HQ approved CENAN flood damage reduction studies in which adjustments were made to gaged annual peak discharge data to account for historic urbanization:

1) Robinson's Branch of the Rahway River, Rahway, NJ GDM (March 1986)

2) Elizabeth River at Hillside NJ, DPR (August 1987)

3) Lower Saddle River NJ, GDM (June 1996)

The shape of Figure 10 is a standard and long-accepted practice of CENAN for historic urbanization adjustment of gaged peak flow data. It is an exponentially decaying curve of updated to observed peak flow vs. time. It is continuous, and is considered a mathematically simple and straightforward way to remove and correct for the upward trend of annual peak flows with respect to time due to historic urbanization.

In each of these three earlier CENAN studies, gaged watershed percent impervious area, and other gaged watershed characteristics, such as unit hydrograph, were determined as functions of time over the period in which historic urbanization was known to have taken place**.** A correlation was always observed between increasing percent impervious area and increasingly peaked unit hydrograph. In each study, seven to ten older historic floods that had occurred during the period of historic urbanization were hydrologically modeled, reproduced, and updated to current conditions.

In each of these three past studies, the ratio of updated to observed peak flow as a function of time was best expressed mathematically by a concave upward, exponentially decaying curve, asymptotically approaching unity as the current degree of watershed urbanization was approached.

This is why such a curve was assumed and used for the historic urbanization adjustment of gaged data in the hydrology of the Mamaroneck Sheldrake GRR. For this analysis, no historic floods that had occurred during the period of historic urbanization were hydrologically modeled, reproduced, and updated to current conditions. Percent impervious area of the gaged basin as a function of time was also not determined. The earlier result from the hydrology of the 1989 Mamaroneck GDM, that of the September 15, 1944 flood peak updated from 1760 cfs to 4000 cfs, and the resulting ratio of 4000 cfs to 1760 cfs of 2.27, was used as an approximate guide to set the upper end of the historic urbanization adjustment curve in Figure 10.

Preliminary data on four other older floods that occurred in the Mamaroneck and Sheldrake Rivers Basin during the period of historic urbanization, those of March 1951, June 1952, March 1953 and October 1955, suggests that Figure 10 is accurate, and that it would show a correlation with increasing percent impervious area of the gaged basin during the period of historic urbanization.

3.4.3 Post-GDM gage data:

The USGS stream gage, Mamaroneck River at Mamaroneck NY (Halsted Avenue) continued to record peak flow data after the hydrology of the GDM was completed. The GDM hydrology included water year 1982, and a reproduction of the April 10-11, 1983 flood. The gage continued

through water years 1983 through 1989, until it was discontinued at the end of water year 1989, on September 30, 1989.

The flooding in Westchester County due to Tropical Storm Floyd (September 15-16, 1999) was so severe that USGS NY surveyed high water marks at several discontinued USGS stream gages, including Mamaroneck River at Mamaroneck NY (Halsted Avenue). These high water marks were used to determine the peak flow posted by USGS for Tropical Storm Floyd.

The gage was re-activated as a crest-stage gage for water years 2009 and 2010. Annual peak discharges for water years 1983 through 1989, 1999, 2009, and 2010 were therefore available at this gage and are presented in Table 1.

3.4.4 Estimation of missing gage data:

The annual peak flows at the Mamaroneck gage for water years 1990-98, and 2000-2008, 18 years total, were estimated.

The storm and flood history of these 18 water years was reviewed and examined. An estimated 40 possible events were identified as candidates for the 18 annual peak flows of these ungaged and missing years.

Some were estimated by correlation with adjacent gaged basins, Saw Mill River at Yonkers NY and Norwalk River at South Wilton Connecticut. The rest were estimated with a hydrologic rainfall-runoff model.

3.4.5 Floods for which FEMA NFIP claims were made:

These were the floods of Sept 8, 2004, Oct 19, 1996, April 16, 1996, and May 17, 1990.

3.4.6 Flood peak estimated and/or confirmed with hydrologic model:

Hydrologic computer simulation began with the following four floods for which there were no gage records on the Mamaroneck River: Sept 8, 2004, Oct 19, 1996, April 16, 1996, & May 17, 1990.

The peak flows estimated by hydrologic computer simulation for these storm-flood events were taken to be the annual peak flows for water years 2004, 1997, 1996 and 1990 respectively.

Total excess vs. total rain data from the hydrology appendix of the 1989 GDM was used to establish a function of total excess vs. total rain for hydrologic computer simulation / estimation of missing annual peak flows. The data appears in Table 6 and is plotted as Figure 4.

The unit hydrograph of "the Mamaroneck gage" at Halstead Avenue, was re-optimized from the two largest historic floods in the GDM : June 1972 and September 1975 (former flood of record) and averaged. An average correction factor was found between the ratio of total excess to total rain for the June 1972 and September 1975 floods as reproduced in the GDM by the HEC-1 model, and the ratio of total excess to total rain for these two floods as reproduced by the re-optimized unit hydrograph of the gaged basin. It is given in Table 6. The ratio was then used to "elevate" the function of total excess vs. total rain for hydrologic computer simulation from the GDM to estimate the peak flows for the following gaged and ungaged floods: WY 1938 : July 24, 1938 (update); WY 1990 : May 17, 1990; WY 1996 : April 16, 1996; WY 1997 : October 19, 1996; WY 2004 : Sept 8, 2004; WY 2006 : Oct 9, 2005; WY 2007 : April 15-16, 2007 (annual peak flow).

The July 1938 flood is included here because it is an important older flood that needed to be updated to current conditions. The October 9, 2005 flood is included here because its peak flow needed to be estimated. The April 15-16, 2007 flood is included here because it is the current flood of record, and has high water marks available for hydraulic calibration. The July 1938 flood update appears on Figure 5. The April 15-16, 2007 flood update appears on Figure 8.

The "elevated" function of total excess vs. total rain is shown on Figure 4 along with the original function estimated from data in the GDM. The peak flow of 5330 cfs estimated by this technique at the USGS gage Mamaroneck River at Mamaroneck (Halstead Avenue) for the April 15-16 2007 flood was subsequently confirmed by HEC-RAS water surface profile hydraulic computer model replication of high water marks of this flood using this peak flow of 5330 cfs.

Note that the "elevated" function of total excess vs. total rain derived as described above for this study is a conservative technique for estimating 7 out of 18 missing annual peak flows at the USGS gage Mamaroneck River at Mamaroneck (Halstead Avenue). It is not a hard and fast rule that all flood reproductions at this gage should be expected to follow.

The annual peak flows for water years 1938, 1990, 1996, 1997, 2004, 2006 and 2007 estimated at the USGS gage Mamaroneck River at Mamaroneck (Halstead Avenue) by hydrologic model are given in Table 1.

3.4.7 Flood Peaks estimated by correlation with adjacent gaged basins:

Saw Mill River at Yonkers, NY : Four annual peak flows, those of water years 1993-1995, and 2008, were estimated using a correlation between annual peak flows at the USGS gages Mamaroneck River at Mamaroneck, NY and the Saw Mill River at Yonkers, NY.

The annual peak flow data for the relation between Mamaroneck River at Mamaroneck, NY and the Saw Mill River at Yonkers, NY is given in Table 2 and plotted as Figure 11. The four annual peak flows estimated for Mamaroneck River at Mamaroneck, NY with this relation are given in Tables 1 and 2.

The annual peak flow data used for Mamaroneck River at Mamaroneck, NY in this correlation is the original data recorded by the gage, unadjusted for historic urbanization, because the Saw Mill River at Yonkers, NY annual peak flow data used in the correlation is also unadjusted for historic urbanization.

Norwalk River at South Wilton, Connecticut (Annual peak flows, water years 1991-1992, 1998, 2000-2003, 2005):

The annual peak flow data for the relation between Mamaroneck River at Mamaroneck, NY and the Norwalk River at South Wilton Connecticut is given in Table 3 and plotted as Figure 12. The eight annual peak flows estimated for Mamaroneck River at Mamaroneck, NY with this relation are given in Tables 1 and 2.

The annual peak flow data used for Mamaroneck River at Mamaroneck, NY in this correlation is the original data recorded by the gage, unadjusted for historic urbanization, because the Norwalk River at South Wilton, Connecticut annual peak flow data used in the correlation is also unadjusted for historic urbanization.

3.4.8 Current Existing Conditions flood frequency calculations at USGS Gage No. 01301000 :

Annual series :

The input to the flood frequency calculation, the annual peak flow data determined as described above, is summarized in Table 1. A plot of this data vs. time to demonstrate its homogeneity with respect to time is shown as Figure 13.

The annual series peak discharge vs. frequency relation was determined in accordance with Guidelines for Determining Flood Flow Frequency, Bulletin 17B. U.S. Water Resources Council, Washington, D.C., revised September 1981. The procedure uses the Log Pearson Type III distribution. It assumes that the common (base 10) logarithms of the annual peak discharges are normally distributed and that statistical procedures are applicable. This minimizes personal subjective judgment in plotting for economic studies.

A generalized skew of 0.5, and a mean-square error of this generalized skew of 0.185, was used for the USGS gage Mamaroneck River at Mamaroneck, NY (Halsted Avenue). The generalized skew was developed using the Regional Frequency Study : Upper Delaware and Hudson River Basins, New York District as developed by the Hydrologic Engineering Center in November 1974.

Statistical parameters of the computed curve are as follows: Mean logarithm (3.2749); Standard deviation (0.1950); Computed skew (- 0.1608); Regional skew (0.5000); Adopted skew (0.0000); 68 systematic events (water years 1938, 1944-2010).

The current annual series existing conditions peak discharge vs. frequency curve for the USGS gage No. 01301000 : Mamaroneck River at Mamaroneck, NY (Halstead Avenue) appears on Figure 14.

Current HEC-HMS model calibration was to the computed (P infinity) rather than the expected (P period of record) peak flow vs. frequency curves, to allow the correction to a finite period of record to be made by program HEC-FDA (Flood Damage Analysis) in accordance with procedures for risk and uncertainty as defined in EM 1110-2-1619, **Risk-Based Analysis For Flood Damage Reduction Studies** (USACOE, 1 August 1996).

The adopted skew coefficient of zero for the Mamaroneck River at Mamaroneck, NY (Halstead Avenue) is close in value to the adopted skew coefficients of gaged watersheds in the three earlier CENAN flood damage reduction studies in which adjustments were made to gaged annual peak discharge data to account for historic urbanization. They are referred to above in Section 3.0

1) Robinson's Branch of the Rahway River, Rahway, NJ GDM (March 1986): Regional skew of 0.4000 and Adopted skew of 0.1000

2) Elizabeth River at Hillside NJ DPR (August 1987): Regional skew of 0.5000 and Adopted skew of 0.0000

3) Lower Saddle River, NJ GDM (June 1996): Regional skew of 0.4000 and Adopted skew of 0.1000

CENAN hydrology experience has been that historic urbanization adjustment of gaged peak flow data tends to decrease the computed and adopted skew, because it increases older flood peaks and pushes them closer in value to the mean flood peak.

The regional skew coefficient of 0.50 was used in the current analysis because it was from a 1974 study peculiar to New York State, even though the value of 0.7 suggested by the appendix to Bulletin 17 B (1982) is more recent. Both of these values, however, should ideally be updated, with adjustments made for historic urbanization.

This would involve the updating and historic urbanization adjustment of the peak discharge vs. frequency curves of at least half a dozen adjacent gaged Westchester County, NY and Fairfield County, CT watersheds, most of which were discontinued at the end of water year 1989. The regional skew of 0.50 from the 1974 study peculiar to New York State was used for this study.

The adopted skew coefficient of zero was taken as evidence that the Mamaroneck and Sheldrake Rivers Basin annual peak flow data set analyzed was normally distributed, homogeneous and stationary with respect to time, and therefore suitable for Log Pearson Type III statistical analysis.

The adopted skew value of zero was also considered appropriate because it was seen as a practical lower limit. An adopted negative skew was considered inappropriate for the Mamaroneck and Sheldrake Rivers Basin because it would imply the effect of regulation or diversion on gaged peak flows, which is not known to exist, and which has no basis in physical fact.

Partial duration :

The partial duration portion of the existing conditions peak discharge vs. frequency curve for the USGS gage Mamaroneck River at Mamaroneck, NY (Halstead Avenue) was determined by applying the annual series to partial duration correction for the frequency curve determined at this gage in the 1989 GDM (Figure A-14) to the current (GRR) annual series peak discharge vs. frequency curve.

The current adopted existing conditions peak discharge vs. frequency curve for the USGS gage No. 01301000 : Mamaroneck River at Mamaroneck, NY (Halstead Avenue), "the Mamaroneck gage", appears on Figure 14.

3.5 Existing Conditions Specific-Frequency hypothetical floods calibration and computation

The driving input for all specific-frequency hypothetical floods for the HEC-HMS models of the Mamaroneck and Sheldrake Rivers basin was specific-frequency hypothetical rainfall data for the basin, based on point rainfall depths in inches taken from NOAA Technical Memorandum NWS Hydro-35, Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States, and Technical Paper No. 40, Rainfall Frequency Atlas Of The United States, U.S. Department of Commerce, Washington, D.C., 1961. Newer data from NOAA on-line Atlas 14 is still not available for New York State. The data is for return periods of 1, 2, 5, 10, 25, 50, 100,

200, 250 and 500 years, and durations of 5 and 15 minutes, and 1, 2, 3, 6, 12, and 24 hours. The point rainfall values are given in Table 7.

The drainage area of the USGS gage Mamaroneck River at Mamaroneck (Halstead Avenue) is 23.46 square miles. The point rainfall depths were modified to 23.46 square mile rainfall depths using procedures contained in the aforesaid Technical Paper No. 40, and in program HEC-HMS. A computation interval of 30 minutes, and a time base of 60 hours, were used in the HEC-HMS models of the hypothetical floods. The interval of 30 minutes was used because of the small drainage areas and times of concentration of the smallest HEC-HMS model sub-basins. The 60-hour time base was used to allow 36 hours after the end of the 24–hour hypothetical storms, to adequately compute the falling limbs of the slowest-reacting sub-basins in the model, the slowest of which is sub-basin D, the Spring Lake Branch, at the northern end of the basin, with a Clark unit hydrograph time of concentration, Tc, and storage coefficient, R, both equal to 5.58 hours.

A symmetrical triangular rainfall time distribution was assumed and used in program HEC-HMS for the hypothetical storms, with the greatest 30-minute intensity placed at the midpoint of the storm, 12 hours from its beginning, and the amounts for the durations of 1 to 24 hours centered symmetrically around it.

3.5.1 Standard Project Storm:

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 study watershed is located. The rainfall used to determine the Standard Project Storm for the Mamaroneck and Sheldrake River basin was obtained from the Civil Works Bulletin No. 52-8, dated March, 26 1952 and reprinted in June 1964 as EM 1110-2-1411 and revised March 1965, titled “Standard Project Flood Determinations”. The 200-square mile, 24 hour Standard Project Storm (SPS) index rainfall for the Mamaroneck and Sheldrake Rivers basin is 10.4 inches. The interpolated total Standard Project Storm rainfall for the Mamaroneck and Sheldrake Rivers basin with a total drainage area of 23.63 square miles is 14.62 inches. The Standard Project Storm rainfall time distribution on an hourly basis is shown on Figure 22 as both a hyetograph and a mass rainfall curve.

A high soil moisture content and ground water conditions due to preceding rains was assumed at the start of the Standard Project Storm. Therefore, infiltration loss of rainfall would be small. The initial loss of rainfall was conservatively assumed to be zero, and the constant loss rate of rainfall was conservatively assumed to be 0.10 inch per hour. The resulting rain excess appears as the hyetograph on Figure 22.

3.5.2 Standard Project Flood:

The Standard Project Flood (SPF) was computed by applying the Standard Project Storm to the HEC-HMS model of the basin.

The SPF is a practical flood risk management design goal, because it is developed from the most severe storms in the region. The SPF is also used as a basis for comparing alternative designs and levels of protection. The resulting existing conditions Standard Project Flood peak flows are shown in Table 9 and Standard Project Flood hydrographs at nodes MMUPSH, SHELMO and 301000 are shown on Figure 22.

For the 1 through 500 year storms and floods, the initial infiltration loss and the constant infiltration loss rate of the uniform loss rate option were adjusted in a trial and error process until the peak discharges of the existing conditions peak discharge vs. frequency curve at the USGS gage Mamaroneck River at Mamaroneck (Halstead Avenue), were reproduced, to the nearest 10 cfs. The final adopted existing conditions calibration values of HEC-HMS input variables initial loss and constant loss rate are given in Table 8. These variables are modified within the program by the variable sub-basin percent impervious area for each sub-basin runoff computation. The resulting current existing conditions peak discharges are summarized in Table 9.

Current existing conditions peak discharge vs. frequency curves at the Halstead Avenue gage, and at important ungaged locations within the Mamaroneck and Sheldrake River basin, are shown on Figures 15 through 18.

Specific-frequency hypothetical hydrographs at the Halstead Avenue gage, and at important ungaged locations within the Mamaroneck and Sheldrake Rivers basin, are shown on Figures 20

through 22. Peak discharge vs. drainage area relations for existing conditions appear as Figure 23 for the Mamaroneck River and as Figure 24 for the Sheldrake River.

3.6 Comparison with 1989 GDM peak discharge vs. frequency curve

The current SPF peak flow is 5.74 % higher than the SPF peak flow from the GDM, mostly because the current constant infiltration loss rate, 0.10 inch per hour, is lower than that used in the GDM, 0.15 inch per hour. A fraction of a percent of the increase is due to differences in computation methods internal to computer programs HEC-1, used in the GDM, and HEC-HMS, used in this GRR. The comparisons of the peak discharge vs. frequency curves for 1989 GDM to the updated curve in this report is shown in Table 10.

The current curve is higher than the GDM curve at the 1 through 25 year frequencies, and lower than the GDM curve at the 50 through 500 year frequencies. The curves cross, and are equal, at a 3 % or 33.33 year frequency. These differences can begin to be understood in terms of the differences in the statistical parameters of the annual series peak discharge vs. frequency curves in the 1989 GDM and this GRR, as shown in Table 11.

These three statistical parameters can perhaps be best and most readily understood in terms of their effect on the upper (10 to 500 year) parts of the peak discharge vs. frequency curves presented herein (Figures 14 through 19-E).

The mean logarithm moves the curve up and down the page. The standard deviation is its slope. The adopted skew coefficient is the curvature of its upper (10 to 500 year) part. The higher the adopted skew coefficient, the more sharply it curves upward from the 10 to the 500 year frequency. Zero adopted skew means no curvature (the frequency curve is a straight line on a log-probability plot).

The uppermost part of the curve, above the largest historic flood peak used in the analysis, is set by the standard deviation and adopted skew coefficient.

With this understanding in mind, we can now begin to explain the changes in the three statistical parameters from the 1989 GDM to this GRR:

The mean peak discharge has increased because, compared to the 1989 GDM, 1) twenty-nine additional annual peak discharges, those of water years 1945-1973, have been adjusted (increased) for historic urbanization, and 2) twenty-eight additional annual peak discharges, those of 1983 through 2010, have been added to the flood frequency calculation in this GRR. These include the two highest annual peak flows, those of September 8, 2004 (4620 cfs) and April 15, 2007 (5340 cfs). Both of these values surpass the highest annual peak flow used in the 1989 GDM frequency curve calculation, the 4000 cfs peak flow of the updated September 15, 1944 flood.

The standard deviation and adopted skew coefficient have decreased, because the upward historic urbanization adjustment of the annual peak discharges of water years 1945-1973 has moved them closer to the computed mean value of all sixty-eight of the annual peak discharges in the current flood frequency analysis.

The existing conditions peak discharge vs. frequency curve at the Mamaroneck gage no. 01031000 (Figure 14) is more accurate and current than the peak discharge vs. frequency curve in the 1989 GDM (Figure A-14) at that same gage, for the following reasons :

1) The curve in the 1989 GDM was based on only 39 annual peak discharges. The curve in this GRR is based on 68 annual peak discharges. This is a nearly two-fold (74 percent) increase in the historic period, resulting in narrower confidence limits.

2) The curve in the 1989 GDM is based on only one annual peak discharge adjusted for historic urbanization, that of the September 15, 1944 hurricane flood, whereas the curve in this GRR is based on a more thorough historic urbanization adjustment of the annual peak discharges of water years 1944-1973.

3) On the frequency curve in the 1989 GDM, (Figure A-14 in the GDM, Figure 14-A in this report), the highest annual peak flow used was the updated September 15, 1944 hurricane flood peak of 4000 cfs. It has about a 25 year frequency. The 25 to 500 year portion of

the curve above that value is therefore an extrapolation based on the computed standard deviation and adopted skew coefficient.

On the frequency curve in this GRR, (Figure 14), the highest annual peak flow used, the April 15, 2007 nor'easter flood peak of 5340 cfs, has about a 100 year frequency. Therefore, only the 100 to 500 year portion of the curve above that value is an extrapolation, again, based on the computed standard deviation and adopted skew coefficient, a smaller portion than the curve in the GDM.

4) The annual peak flows at Weibull plot positions (plot symbol X) are a tighter fit around the adopted frequency curve on Figure 14 in the current GRR than they are on Figure A-14 in the GDM.

The existing conditions peak discharge vs. frequency curves at the Mamaroneck gage from both this GRR and the 1989 GDM are compared on Figure 14-B.

4.0 FUTURE UNIMPROVED CONDITIONS HYPOTHETICAL PEAK DISCHARGES

Undeveloped areas of land within the basin, mostly parks and golf courses, were identified using available maps. Westchester County planning officials, and representatives of the Village of Mamaroneck, and other communities within the basin, were consulted about possible future development of the undeveloped land areas. The following results were obtained. They represent the most likely future development:

1) Century Country Club, Town of Harrison, sub-basins D and E : 1 dwelling unit per acre, average lot size 1 acre, increase from 2 % impervious to 20 % impervious (NRCS TR-55).

2) Ridgeway Country Club, City of White Plains, sub-basin G : 1.5 dwelling units per acre, 2/3 acre average lot size, increase from 2 % impervious to 22.5 % impervious (NRCS TR-55).

3) Westchester Hills Golf Club, City of White Plains, sub-basin G : 1.5 dwelling units per acre, 2/3 acre average lot size, increase from 2 % impervious to 22.5 % impervious (NRCS TR-55).

4) Fenway Golf Club, City of White Plains, sub-basin I : 1.5 dwelling units per acre, 2/3 acre average lot size, increase from 2 % impervious to 22.5 % impervious (NRCS TR-55).

5) Fenway Golf Club, Village of Scarsdale, sub-basin AA : 0.5 dwelling units per acre, 2 acre average lot size, increase from 2 % impervious to 12 % impervious (NRCS TR-55).

6) Quaker Ridge Golf Club, Village of Scarsdale, sub-basin CC: 0.5 dwelling units per acre, 2 acre average lot size, increase from 2 % impervious to 12 % impervious (NRCS TR-55).

The fractions of the total areas of sub-basins D, E, G, I, AA and CC represented by these country clubs and golf clubs, developable to residential areas, were measured, and used, with the increases in % impervious area given above, to calculate the increase in percent impervious area from present existing conditions to future conditions for these sub-basins. Results are as shown in Table 12. The values at the USGS gage are area-averaged values of the twenty sub-basins upstream of the gage. The above future values of percent impervious area are included with the present existing conditions values in Table 5.

The future values of percent impervious area for the above six HEC-1 model sub-basins were input into the HEC-HMS models of the specific-frequency hypothetical floods to generate HEC-HMS models of future unimproved conditions for each frequency. They were re-run with no other change to compute future unimproved conditions peak discharges for the Mamaroneck and Sheldrake Rivers basin. These future unimproved conditions peak discharges are given in Table 13, and are plotted as peak discharge vs. frequency curves on Figures 19-A through 19-E.

Increases from existing conditions peak discharges were small, 30 cfs at most, for several reasons:

1) Present conditions values of percent impervious area are already fairly high.
2) The estimated increases in the % impervious values from present to future conditions are small, and
3) The infiltration loss parameters for most hypothetical floods are low, so that an increase in the percent of impervious area would have relatively little effect on computed peak flows of runoff.

The largest percent increases in peak flows occur for the Mamaroneck River at the Winfield Avenue gaging station, node WINFLD, 15.35 sq. mi. drainage area, ranging from 30 cfs, 2.3 % increase, for the 1 year flood peak flow, to 10 cfs, 0.1 % increase, for the Standard Project Flood peak flow.

The smallest percent increases in peak flow occur for the Sheldrake River at its mouth, node SHELMO, 6.16 sq. mi. drainage area, ranging from 10 cfs, 1.2 % increase, for the 2 year flood peak flow to zero for the Standard Project Flood peak flow.

The increase in percent impervious area for the long-term gaged basin, Mamaroneck River at Mamaroneck, NY (Halstead Avenue), node 301000, is 3.2 %, as noted above.

At this gaged location, the increases in peak flows from existing to future conditions range from 30 cfs, 1.9 % increase, for the 1 year flood peak flow, to 10 cfs, 0.1 % increase, for the Standard Project Flood peak flow.

There is an estimated plus or minus 10 % uncertainty in the computed increases in sub-basin percent impervious area.

For the long-term gaged basin (Halstead Avenue gage) the area-average percent impervious area increases 3.2 % from existing to future conditions. This results in a 1.9 % increase in the 1 year flood peak flow, as a maximum, decreasing to a minimum 0.1 % increase in the Standard Project Flood peak flow.

There is estimated at least a 1.8 % increase, and, at most, a 2.0 % increase, in the 1 year flood peak flow, from present to future conditions, as a result of estimated future development. The 0.1 % increase in the Standard Project Flood peak flow is so small as to effectively have no such uncertainty bands.

It is estimated there could be twice as much future residential development as has been input to the hydrologic models, but the estimated resulting increase in change in peak flows from existing to future conditions would still be almost negligible: a maximum of 3.8 % for the 1 year flood, decreasing to 0.2 % for the Standard Project Flood.

This twice-as-large future development was not something calculated and then rejected, but is an estimate of the uncertainty of the future development that was calculated and used.

Incremental changes in percent impervious area from current existing to projected future conditions are possible. For example, about six golf courses and country clubs could become residential areas, one by one, but the present analysis only predicts the end result of greatest likely future residential buildout, or development, without attempting to detail or predict any intermediate steps.

Again, the total change is so small, that sudden changes or increments of the total change are virtually negligible.

The above analysis of future unimproved conditions hydrology does not take into account climate change. Such changes are possible, but are not quantified herein due to a current lack of appropriate and relevant Corps of Engineers policy.

5.0 RISK AND UNCERTAINTY OF PEAK DISCHARGE VS. FREQUENCY

The procedure followed to determine the equivalent record length, and 95 % and 5 % confidence limits, for the existing and future unimproved conditions specific frequency hypothetical peak discharges, was taken from Chapter 4, Uncertainty of Discharge-Probability Function, of EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies, 1 August 1996.

The 95 % confidence limit is the lower limit. It means that there is a 95 % probability that the actual value of the specific-frequency peak discharge, for example, the 100 year, or 1 % chance exceedence, peak discharge, is above the 95 % limit value.

The value of the 100 year peak discharge on the plotted peak discharge vs. frequency curve is the most likely value. It is labeled as the expected value in Tables 9 and 13.

The 5 % confidence limit is the upper limit. It means that there is a 5 % probability that the actual value of the specific-frequency peak discharge, for example, the 100 year, or 1 % chance exceedence, peak discharge, is above the 5 % limit value.

This means that there is a 95 % - 5 % = 90 % chance that the actual value of the 100 year peak discharge is between the 95 % and 5 % confidence limits.

Table 4-5, Page 4-5 of Chapter 4 in the EM 1110-2-1619, the "Equivalent Record Length Guidelines:" gives equivalent record length based on the method of frequency function estimation.

The systematic record length of the long-term hydrologic calibration point of this study, the USGS gage # 01301000, Mamaroneck River at Mamaroneck, NY (Halstead Avenue) is 67 years, water years 1944-2010 inclusive. It is 700 feet downstream of the confluence of the Mamaroneck and Sheldrake Rivers. This systematic record length of 67 years was used to determine the confidence limits of the hypothetical peak flows at this gage, and also for the Mamaroneck River at its mouth (node MAMKMO) a short distance downstream, because the drainage areas are so close : 23.46 and 23.63 square miles, respectively, a 0.7 % difference.

The two drainage areas immediately upstream of the gage, Mamaroneck River upstream of Sheldrake River, 17.3 square miles, 73.7 % of the gaged area (23.46 square miles), and Sheldrake River at its mouth, 6.16 square miles, 26.3 % of the gaged area, are not within 20 % of the gaged area, but together, they make up the gaged area completely, and are immediately upstream of, the gage. Therefore, 90 % of the gaged systematic record length, 60.3 years, was

used to determine confidence limits for the hypothetical peak flows at all points of interest on the Mamaroneck and Sheldrake Rivers upstream of their confluence, and the Mamaroneck gage.

Chapter 4 of EM 1110-2-1619 cites Appendix 9: Confidence Limits, of Bulletin # 17B, Guidelines For Determining Flood Flow Frequency, as the source of the procedure used to compute confidence limits for hypothetical peak flows. This procedure was followed in this study. It requires the logarithmic standard deviation, equivalent record length, and frequencies of the hypothetical peak flows at a given point of interest.

An equivalent record length of 60.3 years was used for all points of interest in this study other than Mamaroneck River at its gage (Halstead Avenue) and at its mouth.

Standard deviations were estimated graphically from the existing conditions peak discharge vs. frequency curves by first converting the partial duration curves to annual series curves using the following conversion from the frequency curve at the gage is shown in Table 14.

In the normal distribution used, 68.2 % of the area under the curve is within one standard deviation of the mean (50 % probability annual series) to either side of it. Therefore, the logarithmic standard deviation of the ungaged frequency curves was found by finding the ratio of the annual series 50 % - 68.2 % / 2 = 15.9 % peak flow to the 50 % + 68.2 % / 2 = 84.1 % peak flow, and taking half of the common (base 10) logarithm of this ratio as the standard deviation.

Resulting values of standard deviation are as follows: Mamaroneck River at Winfield Avenue (WINFLD) - 0.188146; Mamaroneck River at New England Thruway (THRUWY) - 0.205624; Mamaroneck River upstream of Sheldrake River (MMUPSH) - 0.190882; Sheldrake River downstream of East Branch (SHDSEB) - 0.194029; Sheldrake River at Fenimore Road (SHDSTE) - 0.148461; Sheldrake River at mouth (SHELMO) - 0.103155

The values of standard deviation for the first four locations above are close to the gaged value of 0.1950. The values at the last two locations are significantly lower because of the relatively large amount of Sheldrake River floodplain storage between the confluence with the East Branch and the mouth of the Sheldrake River.

These logarithmic standard deviations, along with the hypothetical peak flow values, their frequencies, and their equivalent record length of 60.3 years, were used to compute the lower (95 %) and upper (5 %) confidence limits of all the hypothetical peak flows, according to procedures contained in Appendix 9 : Confidence Limits, of Bulletin # 17B, Guidelines For Determining Flood Flow Frequency,

The confidence limits computed for the annual series 63 %, 38 % and 17 % peak flows were taken to be those for the partial duration 1 year, 2 year and 5 year peak flows, respectively.

They are given in Table 9 for existing conditions and in Table 13 for future unimproved conditions. They are plotted on Figures 15 through 18 for existing conditions and as Figures 19-A through 19-E for future unimproved conditions.

6.0 IMPROVED CONDITIONS

6.1 Introduction of Alternatives

Several types of improvements were analyzed to lessen the risk of flooding at the primary damage centers along the Mamaroneck & Sheldrake Rivers. These improvements included, separately or in combination, such structural measures as: channel modification, diversions, detention storage and non-structural measures (i.e. floodproofing, raising, etc.), As part of this analysis, the HEC-HMS model that was used for unimproved conditions discharge estimates was modified using information obtained from improved conditions runs of the HEC-RAS model. Flow and storage values for each flood event at each river station from the initial runs of the HEC-RAS model for improved conditions was used as input to the improved conditions runs of the HEC-HMS model. The results of the modified HEC-HMS runs were utilized to provide better estimates of river discharges with selected improvements in place for further HEC-RAS runs.

6.2 Impacts of the Alternatives on the Hydrology

A total of eight alternatives (6 structural, one non-structural, and one combination of structural and non-structural) were analyzed. They are explained in more detail in Appendix C2

Hydraulics. For most of the structural alternatives, “improved conditions” peak discharges were needed to re-run the RAS model for these alternatives. The previous section explains how the “improved conditions” peak discharges were developed. Below is a brief summary of how the peak discharges for some of the alternatives either increase or decrease based upon the changed storage-discharge data.

All channel improvements proposed will result in loss of natural flood plain storage, and the flood peak reductions associated with it. That is, peak flows will increase downstream of proposed channel improvements, and will occur sooner, under improved conditions, as compared to existing conditions.

The loss of downstream flood peak attenuation (reduction) is NOT LINEAR with regard to loss of natural flood plain storage. That is, a 10 percent loss of storage does not necessarily result in a 10 percent increase in peak flow, under improved conditions, as compared to existing conditions.

The diversion tunnels proposed will result in a decrease in peak flows downstream. The diversion tunnel proposed at Fenimore Road for the Sheldrake River will bring the flood hydrographs of the Sheldrake River from Fenimore Road into the West Basin of Mamaroneck Harbor. These diverted flood hydrographs do not return to either the Mamaroneck or the Sheldrake River.

The Ward Avenue diversion tunnel diverts flood hydrographs from the Mamaroneck River downstream of the Sheldrake River at station or cross section 3554.501. The diverted flood hydrographs re-enter the Mamaroneck River further downstream at station or cross section 2041.332

In the locally requested alternative, flood detention will result in a decrease of peak flows downstream of the Larchmont Reservoir and the Westchester Joint Water Works (Mamaroneck) Reservoir.

6.3 Improved Conditions HEC-HMS Model Result:

Improved conditions peak discharges, for Alternative 1M, are shown in Tables 15 and 16 for both present and future conditions of urbanization and are plotted on Figures 26 through 43. The locations of these reaches of improved channel and floodwalls are shown on Figure 25. Alternative 1 M contained the following hydraulic features:

1. In the Harbor Heights section of the Village of Mamaroneck, on the Mamaroneck River, 1,340 feet of improved trapezoidal channel, with a bottom width of 25 to 30 feet, extending from James Street downstream to Warren Avenue. This reach of improved channel lies within HEC-HMS hydrologic routing reach "RTL", between hydrologic nodes "WINFLD" and "THRUWY".

2. Downstream of the aforesaid Reach 1, 2,400 feet of improved trapezoidal channel, with a bottom width of 45 feet, extending from North Barry Avenue downstream to the confluence of the Mamaroneck and Sheldrake Rivers. This reach of improved channel lies within HEC-HMS hydrologic routing reach "RTM", between hydrologic nodes "THRUWY" and "MMUPSH". This reach of improved channel also contains some floodwalls.

3. On the Sheldrake River, between the downstream face of the Fenimore Road Bridge and the confluence of the Sheldrake River and the Mamaroneck River, 3,470 feet of improved trapezoidal channel, with a bottom width of 25 to 30 feet. . This reach of improved channel lies within HEC-HMS hydrologic routing reach "RTFF, between hydrologic nodes "SHDSTE" and "SHELMO". This reach of improved channel also contains some floodwalls.

4. On the Mamaroneck River, from its confluence with the Sheldrake River, downstream to the Tompkins Avenue Bridge, 2,400 feet of improved trapezoidal channel with a bottom width of 45 feet. This reach of improved channel lies within HEC-HMS hydrologic routing reach "RTO, between hydrologic nodes "301000" and "MAMKMO". This reach of improved channel also contains some floodwalls.

After further design, analysis and formulation, Alternative 1Z was developed and identified as the Tentatively Selected Plan (TSP). The differences between Alternatives # 1M and 1Z are too small to have significant impact on the peak discharges, and no further updating was performed. Alternative 1Z is identical to Alternative 1M with the exception of the channelization in the Harbor Heights reach. Alternative 1Z recommends a non-structural solution in Harbor Heights instead of channelization.

REFERENCES :

US Army Corps of Engineers: New York District is author of all references

1. General Design Memorandum : Streams in Westchester County, NY : Flood Control Project for the Mamaroneck and Sheldrake Rivers Basin in the Village of Mamaroneck NY : Volume 2 of 6 : Appendix A : Hydrology and Appendix B : Hydraulics : January 1989 : Final Report

2. Robinson's Branch of the Rahway River, Rahway NJ : General Design Memorandum : Appendix A : Hydrology : March 1986

3. Elizabeth River at Hillside NJ: Detailed Project Report: Appendix A : Hydrology August 1987.

4. Lower Saddle River NJ GDM : Appendix A : Hydrology : June 1996

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APPENDIX – TABLES & FIGURES

HYDROLOGY

TABLE 1: ANNUAL PEAK FLOWS : MAMARONECK RIVER @ MAMARONECK GAGE (HALSTEAD AVENUE)

Water Year	Annual Peak Flow Date	Annual Peak Flows (cfs)				Updated Annual Peak Flow
		Recorded	Updated	Estimated	Used	Recorded Annual Peak Flow
1938	Jul 23 1938	† 2170	3807	2170	3807	1.75
1944	Sep 15 1944	1760	4000		4000	2.27
1945	Aug 7 1945	738	1845		1845	2.50
1946	May 27 1946	1200	2844		2844	2.37
1947	Apr 5 1947	795	1813		1813	2.28
1948	Apr 1 1948	660	1432		1432	2.17
1949	Jan 6 1949	723	1518		1518	2.10
1950	Mar 23 1950	232	459		459	1.98
1951	Mar 31 1951	1550	2945		2945	1.90
1952	Jun 1 1952	1270	2311		2311	1.82
1953	Mar 13 1953	1620	2900		2900	1.79
1954	Sep 11 1954	900	1521		1521	1.69
1955	Aug 13 1955	1370	2260		2260	1.65
1956	Oct 15 1955	1940	3162		3162	1.63
1957	Nov 1 1956	711	1138		1138	1.60
1958	Feb 28 1958	1260	1915		1915	1.52
1959	Jan 2 1959	723	1085		1085	1.50
1960	Aug 19 1960	1490	2100		2100	1.41
1961	Apr 16 1961	1500	2085		2085	1.39
1962	Mar 12 1962	1500	2010		2010	1.34
1963	Nov 10 1962	975	1277		1277	1.31
1964	Nov 29 1963	1150	1484		1484	1.29
1965	Feb 8 1965	1310	1598		1598	1.22
1966	Feb 13 1966	1170	1392		1392	1.19
1967	Mar 7 1967	1260	1449		1449	1.15
1968	May 29 1968	1840	2060		2060	1.12
1969	Mar 25 1969	1600	1760		1760	1.10
1970	Feb 10 1970	1990	2109		2109	1.06
1971	Aug 28 1971	2260	2328		2328	1.03
1972	Jun 19 1972	3550			3550	1.00
1973	Feb 2 1973	2150	2171		2171	1.01
1974	Sep 3 1974	2840			2840	1.00
1975	Sep 26 1975	3700			3700	
1976	Aug 10 1976	1570			1570	
1977	Feb 25 1977	2000			2000	
1978	Nov 8 1977	3240			3240	
1979	Jan 21 1979	3410			3410	
1980	Apr 10 1980	2790			2790	

† - From July 1938 floodline in GDM

TABLE 1: ANNUAL PEAK FLOWS : MAMARONECK RIVER @ MAMARONECK GAGE (HALSTEAD AVENUE) (CONT.)

Water Year	Annual Peak Flow Date	Annual Peak Flows (cfs)				Updated Annual Peak Flow
		Recorded	Updated	Estimated	Used	Recorded Annual Peak Flow
1981	Sep 9 1981	653			653	
1982	Jan 4 1982	2000			2000	
1983	Apr 10 1983	1810			1810	
1984	Jul 7 1984	2720			2720	
1985	Sep 27 1985	961			961	
1986	Jan 26 1986	1020			1020	
1987	Dec 3 1986	1580			1580	
1988	May 18 1988	572			572	
1989	May 17 1989	1470			1470	
1990	May 17 1990			U 2130	2130	
1991	Oct 24 1990			N 2190	2190	
1992	Aug 18 1992			N 1130	1130	
1993	Sep 27 1993			Y 980	980	
1994	Jan 29 1994			Y 1770	1770	
1995	Jul 26 1995			Y 1320	1320	
1996	Apr 16 1996			U 2250	2250	
1997	Oct 20 1996			U 3060	3060	
1998	Jan 24 1998			N 1010	1010	
1999	Sep 16 1999	2230			2230	
2000	Jun 7 2000			N 960	960	
2001	Mar 30 2001			N 1200	1200	
2002	May 14 2002			N 1050	1050	
2003	Jul 23 2003			Y 2050	2050	
2004	Sep 8 2004			U 4620	4620	
2005	Jun 29 2005			Y 2450	2450	
2006	Oct 9 2005			U 2830	2830	
2007	Apr 16 2007			U 5330	5330	
2008	Feb 13 2008			Y 2000	2000	
2009	Dec 12 2008	1290			1290	
2010	Mar 30 2010	1927			1927	

N – Norwalk
Y – Yonkers
U – unitgraph (HEC-1)

TABLE 2: CORRELATION – ANNUAL PEAK DISCHARGES AT SELECTED USGS GAGES

Water Year	Flood Peak Date	Peak Flows (cfs)	
		Saw Mill River at Yonkers, NY	Mamaroneck River at Mamaroneck, NY (Halstead Avenue)
1944	Sep 15 1944	489	1760
1945	Aug 7 1945	367	738
1946	May 27 1946	678	1200
1947	Apr 5 1947	354	795
1948	Apr 1 1948	369	1660
1949	Jan 6 1949	358	723
1950	Mar 23 1950	294	232
1951	Mar 31 1951	679	1550
1952	Jun 1 1952	518	1270
1953	Mar 13 1953	646	1620
1954	Sep 11 1954	458	900
1955	Aug 13 1955	433	1370
1956	Oct 15 1955	1119	1940
1957	Nov 1 1956	450	711
1958	Feb 28 1958	540	1260
1959	Jan 2 1959	367	723
1960	Aug 19 1960	378	1490
1961	Apr 16 1961	415	1500
1962	Mar 12 1962	733	1500
1963	Nov 10 1962	423	975
1964	Nov 29 1963	412	1150
1965	Feb 8 1965	518	1310
1966	Feb 13 1966	510	1170
1967	Mar 7 1967	317	1260
1968	May 29 1968	536	1840
1969	Mar 25 1969	413	1600
1970	Feb 10 1970	462	1990
1971	Aug 28 1971	539	2260
1972	Jun 19 1972	752	3550
1973	Feb 2 1973	748	2150
1974	Sep 3 1974	403	2840
1975	Sep 26 1975	1277	3700
1976	Aug 10 1976	589	1570
1977	Feb 25 1977	828	2000
1978	Nov 8 1977	816	3240
1979	Jan 21 1979	1075	3410
1980	Apr 10 1980	1155	2790
1980	Apr 10 1980	1155	2790

TABLE 2: CORRELATION – ANNUAL PEAK DISCHARGES AT SELECTED USGS GAGES (CONT.)

Water Year	Flood Peak Date	Peak Flows (cfs)	
		Saw Mill River at Yonkers, NY	Mamaroneck River at Mamaroneck, NY (Halstead Avenue)
1981	Sep 9 1981	515	653
1982	Jan 4 1982	486	2000
1983	Apr 10 1983	670	1810
1984	Jul 7 1984	1662	2720
1985	Sep 27 1985	942	961
1986	Jan 26 1986	1270	1020
1987	Dec 3 1986	932	1580
1988	May 18 1988	454	572
1989	May 17 1989	862	1470
1999	Sep 17 1999	1700	2230
Predictions Made with correlation (Best-Fit-Curve)			
1993	Sep 27 1993	429	980
1994	Jan 29 1994	737	1770
1995	Jul 26 1995	549	1320
2003	Jul 23 2003	858	2050
2005	Jun 29 2005	1040	2450
2008	Feb 13 2008	833	2000

TABLE 3: CORRELATION – ANNUAL PEAK DISCHARGES AT SELECTED USGS GAGES

Water Year	Flood Peak Date	Peak Flows (cfs)	
		Norwalk River at South Wilton, CT	Mamaroneck River at Mamaroneck, NY (Halstead Avenue)
1956	Oct 15-16 1955	12000	† 1940
1965	Feb 8 1965	470	1310
1967	Mar 7 1967	365	1260
1968	May 29 1968	1020	1840
1969	March 25 1969	1100	1600
1972	Jun 19 1972	1690	3550
1973	Feb 2-3 1973	1610	2150
1975	Sep 26-27 1975	1220	3700
1979	Jan 21 1979	1940	3410
1980	Apr 10 1980	2300	2790
1983	Apr 10 1983	1480	1810
1985	Sep 27 1985	969	961
1986	Jan 26 1986	962	1020
1989	May 17 1989	1800	1470
1999	Sep 16-17 1999	1720	2230
Predictions Made with correlation (Best-Fit-Curve)			
		Observed:	**Estimated:**
1991	Oct 24 1990	1360	2190
1992	Aug 18 1992	612	1130
1998	Jan 24 1998	523	1010
2000	Jun 7 2000	495	960
2001	Mar 30 2001	655	1200
2002	May 14 2002	556	1050

† - Outliner point not used in correlation

TABLE 4: HEC-HMS NODES WITH DRAINAGE AREA

HEC-HMS NODE	Description	Drainage Area (sq. mi)
Upper Mamaroneck River		
WINFLD	Winfield Avenue (short term) gaging station	15.35
THRUWY	@ New England Thruway	16.18
MMUPSH	Upstream of Sheldrake River	17.30
Sheldrake River		
SHDSEB	Sheldrake River downstream of its East Branch	5.20
SHUSTE	Sheldrake River Upstream of Fenimore Road Tributary (Upstream side of Fenimore Road Bridge)	5.20
SHDSTE	Sheldrake River downstream of Fenimore Road Tributary (downstream side of Fenimore Road Bridge)	5.57
SHELMO	Sheldrake River at Mouth (confluence with Mamaroneck River)	6.16
Lower Mamaroneck River		
301000	Downstream of Sheldrake River (Long Term USGS Gage at Halstead Avenue)	23.46
MAMKMO	Mamaroneck River at Mouth	23.63

TABLE 5: EXISTING AND FUTURE CONDITIONS HYDROLOGIC MODEL SUB-BASINS PERCENT IMPRERVIOUS AREA

Sub-Basin	Drainage Area (Sq. Miles)	Percent Impervious Area	
		Present (2010)	Future (2060)
A	1.74	32.78	32.78
B	1.18	40.96	40.96
C	0.32	50.70	50.70
D	2.95	14.46	***16.81***
E	2.94	43.08	***44.36***
F	0.92	10.50	10.50
G	1.32	20.21	***25.23***
H	0.43	30.59	30.59
I	2.24	37.44	***38.09***
J	0.63	14.96	14.96
K	0.68	26.05	26.05
L	0.83	19.99	19.99
M	0.68	52.69	52.69
N	0.44	30.37	30.37
AA	2.63	36.96	***37.71***
BB	0.73	34.32	34.32
CC	1.84	17.29	***18.39***
DD	0.37	45.12	45.12
EE	0.24	24.14	24.14
FF	0.35	47.96	47.96
O	0.17	56.71	56.71
301000	23.46	30.29	**† *31.26***
MAMKMO	23.63	30.48	**† *31.44***

† - Area-Average Values
301000: Mamaroneck River at Mamaroneck, NY (USGS Gage at Halstead Avenue)
MAMKMO: Mamaroneck River at Mouth (Boston Post Road)
Change from present to future value indicated by bold italics

TABLE 6: TOTAL RAINFALL AND TOTAL EXCESS (HEC-HMS MODELED HISTORIC AND HYPOTHETICAL FLOODS)

Flood	Total Rain (inches)	Total Excess (inches)	Total Excess/Total Rain
General Design Memorandum (GDM)			
10-year	4.94	2.21	0.447
100-year	7.00	5.07	0.724
Sep 1944 (updated)	4.58	3.11	0.679
Apr 1983	2.74	1.31	0.478
Apr 1980	3.64	1.87	0.514
Nov 1977	4.99	2.97	0.595
Sep 27 1975	4.89	2.42	0.495
Jun 19 1972	4.65	3.15	0.677
General Re-evaluation Report (GRR)			
Sep 27 1975	4.89	2.88	0.589
Jun 19 1972	4.65	3.65	0.785
May 17 1990	2.71	1.80	0.664
Apr 16 1996	3.14	2.07	0.659
Oct 19 1996	4.40	2.95	0.670
Oct 9 2005	5.31	3.78	0.712
July 24 1938	5.45	4.02	0.738
Sep 8 2004	5.55	4.02	0.724
Apr 15 2007	7.90	5.92	0.749
[Total Excess/Total Rain (GRR)]/[Total Excess/Total Rain (GDM)]			
Sep 1975	0.589 / 0.495 =		1.190
Jun 1972	0.785 / 0.677 =		1.160
Average			1.1175

TABLE 7: REVISED POINT RAINFALL DEPTHS FOR HYPOTHETICAL STORMS

Precipitation in inches								
Duration	**5 min**	**15 min**	**1 hour**	**2 hours**	**3 hours**	**6 hours**	**12 hours**	**24 hours**
1-year	0.41	0.78	1.20	1.46	1.61	1.91	2.35	2.64
2-year	0.46	0.88	1.35	1.75	1.92	2.32	2.80	3.31
5-year	0.52	1.03	1.73	2.30	2.48	3.07	3.65	4.40
10-year	0.57	1.15	2.00	2.63	2.88	3.58	4.25	5.08
25-year	0.65	1.31	2.37	3.03	3.33	4.10	4.90	5.85
50-year	0.71	1.45	2.67	3.31	3.71	4.48	5.38	6.38
100-year	0.77	1.58	2.88	3.58	4.10	4.90	5.80	6.80
200-year	0.82	1.70	3.10	3.85	4.47	5.30	6.30	7.25
250-year	0.84	1.75	3.18	3.93	4.58	5.40	6.45	7.40
500-year	0.90	1.89	3.42	4.20	4.95	5.80	6.90	7.90

TABLE 8: INITIAL LOSS AND CONSTANT LOSS RATE FOR HISTORICAL AND HYPOTHETICAL STORMS

Storm	Initial Loss (Inches)	Constant Loss Rate (Inches/hour)
Historical		
July 1938	0.00	0.1310
July 1984	0.00	0.2220
May 1989	2.99	0.1148
April 2007	0.00	0.1291
Hypothetical		
1-year	0.00	0.2700
2-year	0.00	0.2910
5-year	0.00	0.3640
10-year	0.00	0.4000
25-year	0.00	0.2980
50-year	0.00	0.2640
100-year	0.00	0.2072
200-year	0.00	0.1599
250-year	0.00	0.1470
500-year	0.00	0.1050
Standard Project Flood	0.00	0.1000

TABLE 9: EXISTING CONDITIONS – PEAK DISCHARGEWITH CONFIDENCE LIMITS

HMS Node	D.A. (mi.2)	Value	Hypothetical											Hist.
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF	April 2007
		5 % limit	1440	1850	2330	2780	3610	4260	5080	5960	6240	7200		
WINFLD	15.35	EXPECTED	1310	1670	2060	2410	3040	3520	4110	4750	4950	5620	9410	4130
		95 % limit	1180	1520	1860	2150	2660	3040	3510	4010	4160	4670		
		5 % limit	1370	1800	2340	2820	3680	4360	5220	6140	6430	7450		
THRUWY	16.18	EXPECTED	1240	1610	2040	2410	3050	3540	4140	4790	4990	5680	9490	4190
		95 % limit	1110	1450	1820	2130	2640	3010	3480	3980	4120	4640		
		5 % limit	1390	1770	2310	2740	3520	4170	4910	5680	5950	6790		
MMUPSH	17.3	EXPECTED	1260	1600	2040	2370	2950	3430	3960	4510	4700	5280	7960	4060
		95 % limit	1140	1450	1840	2110	2580	2960	3370	3800	3940	4370		
		5 % limit	740	910	1140	1360	1800	2140	2650	3190	3320	3800		
SHDSEB	5.2	EXPECTED	670	820	1000	1170	1510	1760	2130	2520	2610	2940	4670	1480
		95 % limit	600	740	900	1040	1320	1510	1810	2120	2180	2430		
		5 % limit	760	930	1150	1340	1740	1980	2250	2500	2570	2830		
SHUSTE	5.57	EXPECTED	710	860	1040	1200	1520	1700	1900	2090	2140	2330	3650	1530
		95 % limit	650	800	960	1100	1370	1520	1680	1830	1860	2010		
		5 % limit	760	940	1160	1350	1750	2000	2260	2510	2590	2850		
SHDSTE	5.81	EXPECTED	710	870	1050	1210	1530	1720	1910	2100	2160	2350	3680	1550
		95 % limit	650	810	970	1110	1380	1540	1690	1840	1880	2030		

D.A. – Drainage Area (Sub-basin); Hist. - Historical

Note: April 2007 and SPF peak discharge have no confidence limits because they are not specific-frequency hypothetical events.

TABLE 9: EXISTING CONDITIONS – PEAK DISCHARGEWITH CONFIDENCE LIMITS (CONT.)

HMS Node	D.A. (mi.2)	Value	Hypothetical											Hist.
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF	April 2007
		5 % limit	790	910	1060	1180	1340	1470	1560	1680	1720	1820		
SHELMO	6.16	EXPECTED	750	860	990	1090	1220	1320	1390	1480	1510	1590	2190	1280
		95 % limit	710	820	940	1020	1130	1220	1270	1350	1370	1440		
		5 % limit	1790	2390	3240	3830	4840	5670	6530	7420	7680	8670		
301000	23.46	EXPECTED	1620	2140	2870	3370	4150	4740	5350	5990	6200	6860	10130	5340
		95 % limit	1470	1960	2590	3000	3600	4110	4580	5070	5250	5720		
		5 % limit	1800	2390	3250	3840	4840	5670	6540	7420	7690	8680		
MAMKMO	23.63	EXPECTED	1630	2140	2880	3380	4150	4740	5360	5990	6210	6870	10140	5350
		95 % limit	1,480	1,960	2,600	3,010	3,600	4,110	4,590	5,070	5,260	5,730		

D.A. – Drainage Area (Sub-basin); Hist. - Historical

Note: April 2007 and SPF peak discharge have no confidence limits because they are not specific-frequency hypothetical events.

TABLE 10: COMPARISION OF PEAK DISCHARGES (1989 GDM VS. CURRENT GRR)

Frequency	Peak Discharge from 1989 GDM (cfs)	Peak Discharge from current GRR (cfs)
1-year	1230	1620
2-year	1680	2140
5-year	2300	2870
10-year	3000	3370
25-year	3980	4150
50-year	4950	4740
100-year	6000	5350
200-year	7100	5990
250-year	7500	6200
500-year	9050	6860
SPF	9580	10130

TABLE 11: STATISTICAL PARAMETERS OF ANNUAL DISCHARGE VS. FREQUENCY CURVES

Documents	Mean Log.	Antilog	Standard Dev.	Adopted Skew Coef.
1989 GDM	3.173	1490 cfs	0.225	0.253
Current GRR	3.275	1880 cfs	0.195	0.000

TABLE 12: INCREASE IN PERCENT IMPERVIOUS FROM PRESENT TO FUTURE CONDITIONS

Sub-basin	Drainage Area (mi.^{2})	Present Value	Increase	Future Value	Percent increase present to future
D	2.95	14.46%	2.35%	16.81%	16.30%
E	2.94	43.08%	1.28%	44.36%	3.00%
G	1.32	20.21%	5.02%	25.23%	24.80%
I	2.24	37.44%	0.65%	38.09%	1.70%
AA	2.63	36.96%	0.75%	37.71%	2.00%
CC	1.84	17.29%	1.10%	18.39%	6.40%
USGS Gage 01301000	23.46	30.29%	0.97%	31.26%	3.20%

TABLE 13: FUTURE CONDITIONS – PEAK DISCHARGEWITH CONFIDENCE LIMITS

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	1470	1870	2350	2810	3630	4280	5100	5970	6250	7210	
WINFLD	15.35	EXPECTED	1340	1690	2080	2440	3060	3540	4130	4760	4960	5630	9420
		95 % limit	1210	1540	1880	2180	2680	3060	3530	4020	4170	4680	
		5 % limit	1400	1820	2330	2860	3720	4400	5230	6150	6460	7460	
THRUWY	16.18	EXPECTED	1270	1630	2060	2440	3080	3570	4150	4800	5010	5690	9500
		95 % limit	1140	1470	1860	2160	2670	3040	3490	3990	4140	4650	
		5 % limit	1410	1800	2330	2760	3560	4190	4920	5690	5960	6800	
MMUPSH	17.3	EXPECTED	1280	1630	2060	2390	2980	3450	3970	4520	4710	5290	7970
		95 % limit	1160	1480	1840	2130	2610	2980	3380	3810	3950	4380	
		5 % limit	750	910	1140	1370	1800	2150	2650	3190	3320	3830	
SHDSEB	5.2	EXPECTED	680	820	1000	1180	1510	1770	2130	2520	2610	2960	4670
		95 % limit	610	740	900	1050	1320	1520	1810	2120	2180	2450	
		5 % limit	760	940	1150	1340	1740	1990	2250	2500	2570	2830	
SHUSTE	5.57	EXPECTED	710	870	1040	1200	1520	1710	1900	2090	2140	2330	3660
		95 % limit	650	810	960	1100	1370	1530	1680	1830	1860	2010	
		5 % limit	770	940	1160	1350	1760	2000	2270	2520	2590	2850	
SHDSTE	5.81	EXPECTED	720	870	1050	1210	1540	1720	1920	2110	2160	2350	3680
		95 % limit	660	810	970	1110	1390	1540	1700	1850	1880	2030	

D.A. – Drainage Area (Sub-basin)

Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical event.

TABLE 13: FUTURE CONDITIONS – PEAK DISCHARGEWITH CONFIDENCE LIMITS (CONT.)

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	790	910	1070	1180	1340	1470	1560	1680	1730	1820	
SHELMO	6.16	EXPECTED	750	860	1000	1090	1220	1320	1390	1480	1520	1590	2190
		95 % limit	710	820	950	1020	1130	1220	1270	1350	1380	1440	
		5 % limit	1820	2430	3270	3860	4870	5710	6540	7430	7700	8680	
301000	23.46	EXPECTED	1650	2180	2900	3400	4180	4770	5360	6000	6220	6870	10140
		95 % limit	1500	2000	2620	3030	3630	4140	4590	5080	5270	5730	
		5 % limit	1820	2420	3270	3870	4870	5710	6550	7440	7710	8690	
MAMKMO	23.63	EXPECTED	1650	2170	2900	3410	4180	4770	5370	6010	6230	6880	10150
		95 % limit	1500	1990	2620	3040	3630	4140	4600	5090	5280	5740	

D.A. – Drainage Area (Sub-basin)
Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical event.

TABLE 14: CONVERTING PARTIAL DURATION TO ANNUAL SERIES

	Probability			
Partial Duration	100% (1-year)	50% (2-year)	20% (5-year)	10% (10-year)
Annual Series	63%	38%	17%	10%

TABLE 15: PRESENT "IMPROVED" CONDITIONS FOR PLAN 1Z – PEAK DISCHARGEWITH CONFIDENCE LIMITS

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	1440	1850	2330	2780	3610	4260	5080	5960	6240	7200	
WINFLD	15.35	EXPECTED	1310	1670	2060	2410	3040	3520	4110	4750	4950	5620	9410
		95 % limit	1180	1520	1860	2150	2660	3040	3510	4010	4160	4670	
		5 % limit	1460	1880	2340	2820	3680	4360	5220	6240	6430	7450	
THRUWY	16.18	EXPECTED	1290	1660	2040	2410	3050	3540	4140	4870	4990	5680	9490
		95 % limit	1150	1500	1820	2130	2640	3010	3480	4050	4120	4640	
		5 % limit	1430	1870	2370	2840	3700	4380	5170	6070	6330	7180	
MMUPSH	17.3	EXPECTED	1300	1690	2090	2460	3100	3600	4170	4820	5000	5580	8840
		95 % limit	1180	1530	1890	2190	2710	3110	3540	4040	4190	4620	
		5 % limit	740	910	1140	1360	1800	2140	2640	3190	3320	3800	
SHDSEB	5.2	EXPECTED	670	820	1000	1170	1510	1760	2130	2520	2610	2940	4670
		95 % limit	600	740	900	1040	1320	1510	1810	2120	2180	2430	
		5 % limit	760	940	1160	1360	1750	1990	2310	2620	2740	3010	
SHUSTE	5.57	EXPECTED	710	870	1050	1220	1530	1700	1950	2190	2280	2480	3650
		95 % limit	650	810	970	1120	1380	1530	1720	1920	1980	2140	
		5 % limit	770	940	1170	1370	1760	2000	2320	2640	2770	3040	
SHDSTE	5.81	EXPECTED	720	870	1060	1230	1540	1720	1960	2210	2290	2510	3680
		95 % limit	660	810	980	1130	1390	1540	1730	1940	1990	2170	

D.A. – Drainage Area (Sub-basin)

Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical event.

TABLE 15: PRESENT "IMPROVED" CONDITIONS FOR PLAN 1Z– PEAK DISCHARGEWITH CONFIDENCE LIMITS (CONT.)

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	800	970	1190	1360	1570	1710	1810	1950	1990	2140	
SHELMO	6.16	EXPECTED	760	920	1120	1260	1430	1540	1610	1720	1750	1870	2520
		95 % limit	720	880	1060	1180	1320	1420	1470	1570	1590	1690	
		5 % limit	1850	2470	3330	4030	5210	6120	7070	8090	8380	9400	
301000	23.46	EXPECTED	1670	2210	2950	3550	4470	5120	5790	6530	6750	7440	11310
		95 % limit	1520	2020	2660	3160	3880	4440	4960	5530	5720	6200	
		5 % limit	1840	2470	3340	4030	5220	6140	7060	8120	8370	9430	
MAMKMO	23.63	EXPECTED	1670	2210	2960	3550	4480	5130	5790	6540	6760	7460	11320
		95 % limit	1520	2020	2670	3160	3890	4450	4960	5560	5730	6220	

D.A. – Drainage Area (Sub-basin)

Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical event.

TABLE 16: FUTURE "IMPROVED" CONDITIONS FOR PLAN 1Z – PEAK DISCHARGEWITH CONFIDENCE LIMITS

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	1470	1870	2350	2810	3630	4280	5100	5970	6250	7210	
WINFLD	15.35	EXPECTED	1340	1690	2080	2440	3060	3540	4130	4760	4960	5630	9420
		95 % limit	1210	1540	1880	2180	2680	3060	3530	4020	4170	4680	
		5 % limit	1440	1880	2340	2860	3720	4400	5230	6270	6460	7460	
THRUWY	16.18	EXPECTED	1310	1680	2070	2440	3080	3570	4150	4890	5010	5690	9500
		95 % limit	1180	1520	1870	2160	2670	3040	3490	4060	4140	4650	
		5 % limit	1470	1890	2400	2880	3740	4400	5190	6090	6350	7170	
MMUPSH	17.3	EXPECTED	1330	1710	2120	2490	3130	3620	4190	4840	5020	5580	8850
		95 % limit	1210	1550	1890	2220	2740	3130	3570	4080	4210	4620	
		5 % limit	750	910	1140	1370	1800	2150	2650	3190	3320	3830	
SHDSEB	5.2	EXPECTED	680	820	1000	1180	1510	1770	2130	2520	2610	2960	4670
		95 % limit	610	740	900	1050	1320	1520	1810	2120	2180	2450	
		5 % limit	760	940	1170	1370	1750	1990	2310	2630	2740	3020	
SHUSTE	5.57	EXPECTED	710	870	1060	1230	1530	1710	1950	2200	2280	2490	3660
		95 % limit	650	810	980	1130	1380	1530	1720	1930	1980	2150	
		5 % limit	770	950	1180	1380	1770	2000	2320	2640	2760	3040	
SHDSTE	5.81	EXPECTED	720	880	1070	1240	1550	1720	1960	2210	2300	2510	3680
		95 % limit	660	820	990	1140	1400	1540	1740	1940	2000	2170	

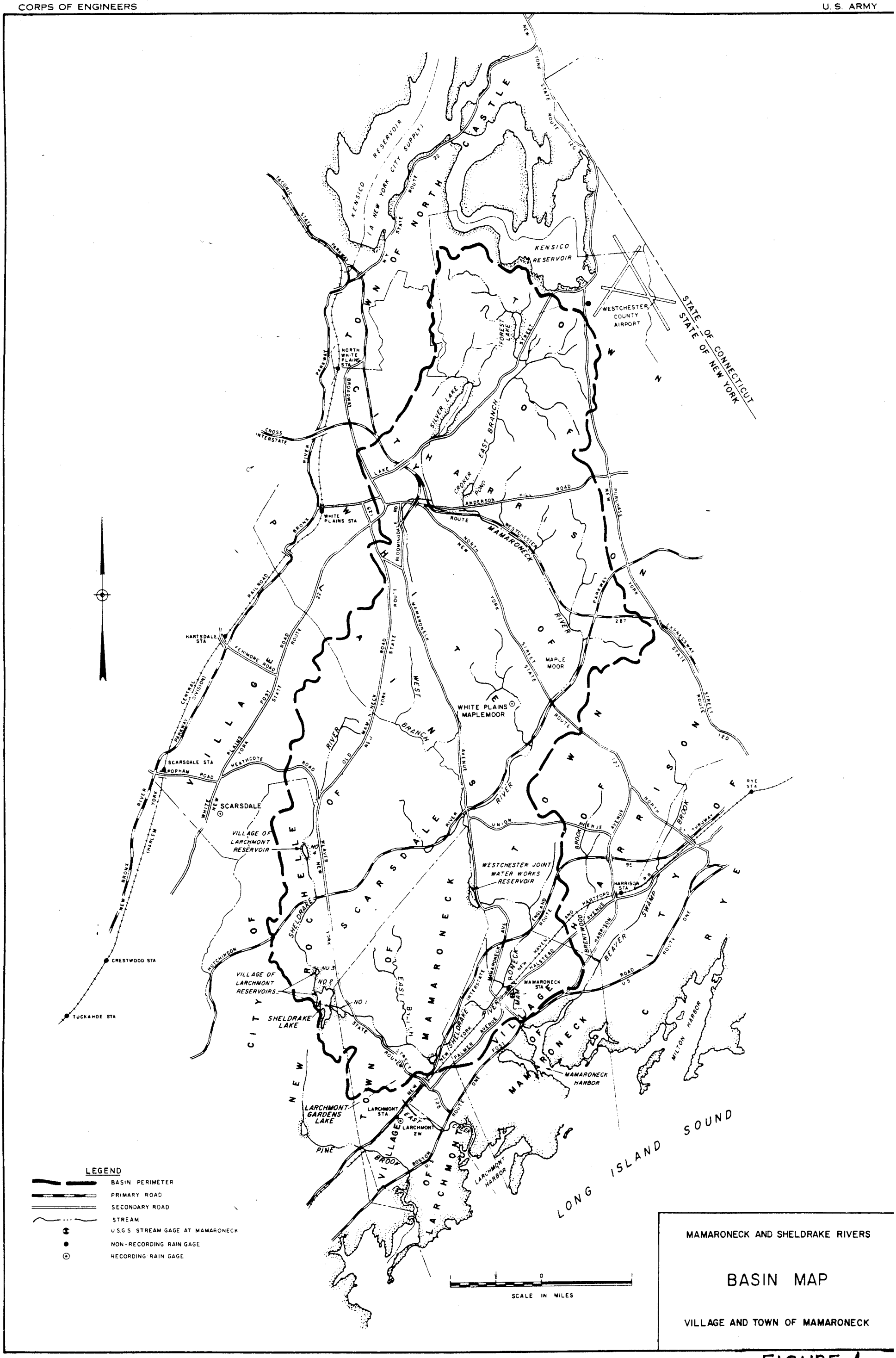
D.A. – Drainage Area (Sub-basin)

Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical event.

TABLE 16: FUTURE "IMPROVED" CONDITIONS – PEAK DISCHARGEWITH CONFIDENCE LIMITS (CONT.)

HMS Node	D.A. (mi.2)	Value	Hypothetical										
			1-year	2-year	5-year	10-year	25-year	50-year	100-year	200-year	250-year	500-year	SPF
		5 % limit	810	980	1200	1370	1570	1710	1810	1950	1990	2140	
SHELMO	6.16	EXPECTED	770	930	1120	1270	1430	1540	1610	1720	1750	1870	2520
		95 % limit	730	890	1060	1190	1320	1420	1470	1570	1590	1690	
		5 % limit	1880	2490	3350	4050	5240	6150	7080	8110	8380	9410	
301000	23.46	EXPECTED	1700	2230	2970	3570	4500	5140	5800	6550	6770	7450	11330
		95 % limit	1550	2050	2680	3180	3910	4490	4970	5550	5740	6210	
		5 % limit	1880	2500	3350	4050	5250	6160	7090	8120	8390	9440	
MAMKMO	23.63	EXPECTED	1700	2240	2970	3570	4510	5150	5810	6560	6780	7470	11330
		95 % limit	1550	2050	2680	3180	3920	4470	5040	5560	5750	6230	

D.A. – Drainage Area (Sub-basin)
Note: SPF peak discharge does not have confidence limits because it is not a specific-frequency hypothetical events.

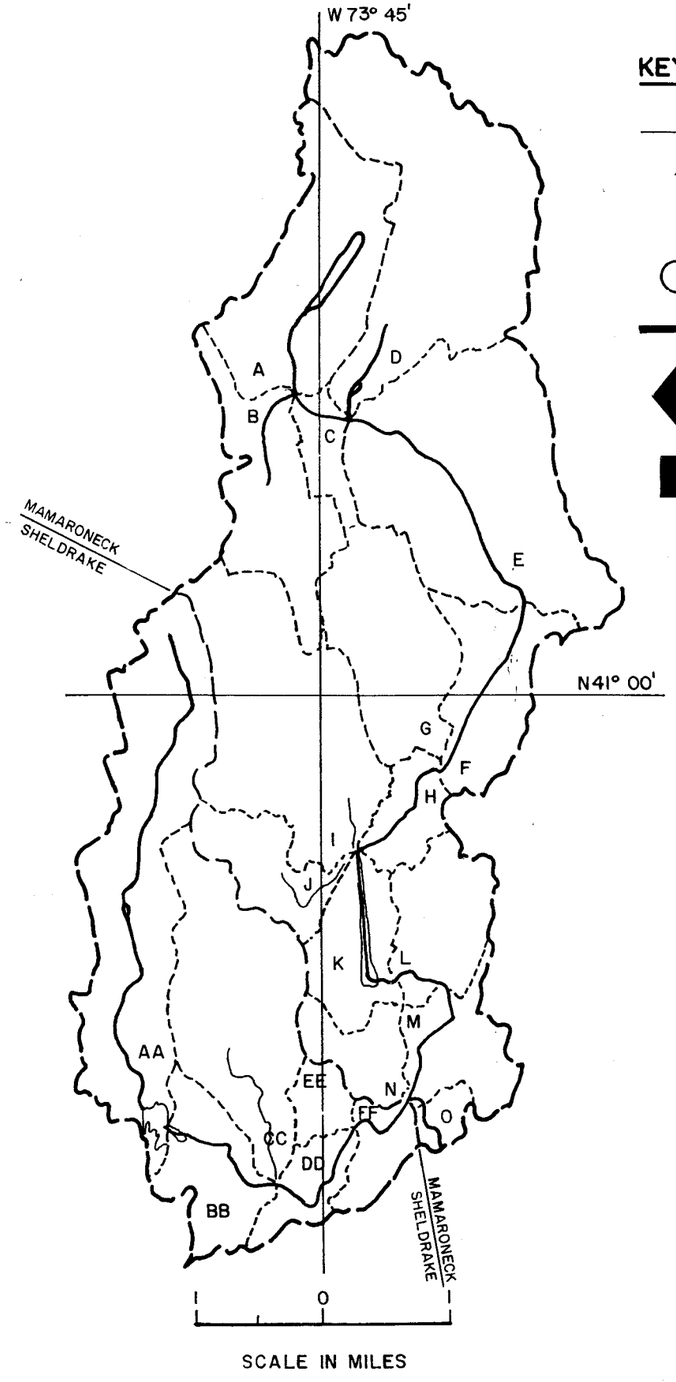


MAMARONECK AND SHELDRAKE RIVERS

BASIN MAP

VILLAGE AND TOWN OF MAMARONECK

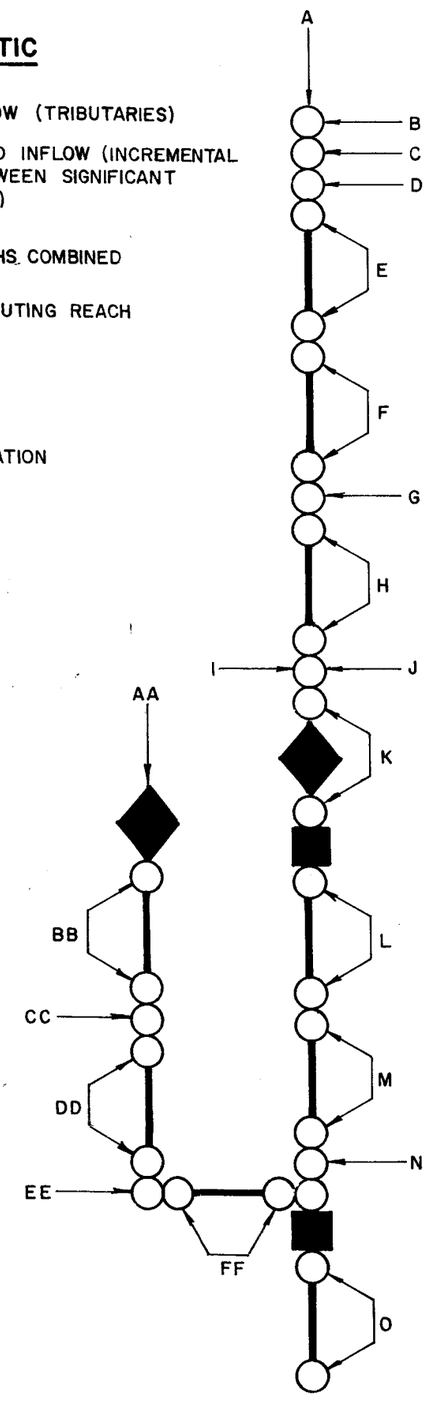
FIGURE 1



SUBAREA MAP

KEY TO SCHEMATIC

- POINT INFLOW (TRIBUTARIES)
- DISTRIBUTED INFLOW (INCREMENTAL AREAS BETWEEN SIGNIFICANT TRIBUTARIES)
- HYDROGRAPHS COMBINED
- CHANNEL ROUTING REACH
- RESERVOIR
- GAGING STATION



SCHEMATIC

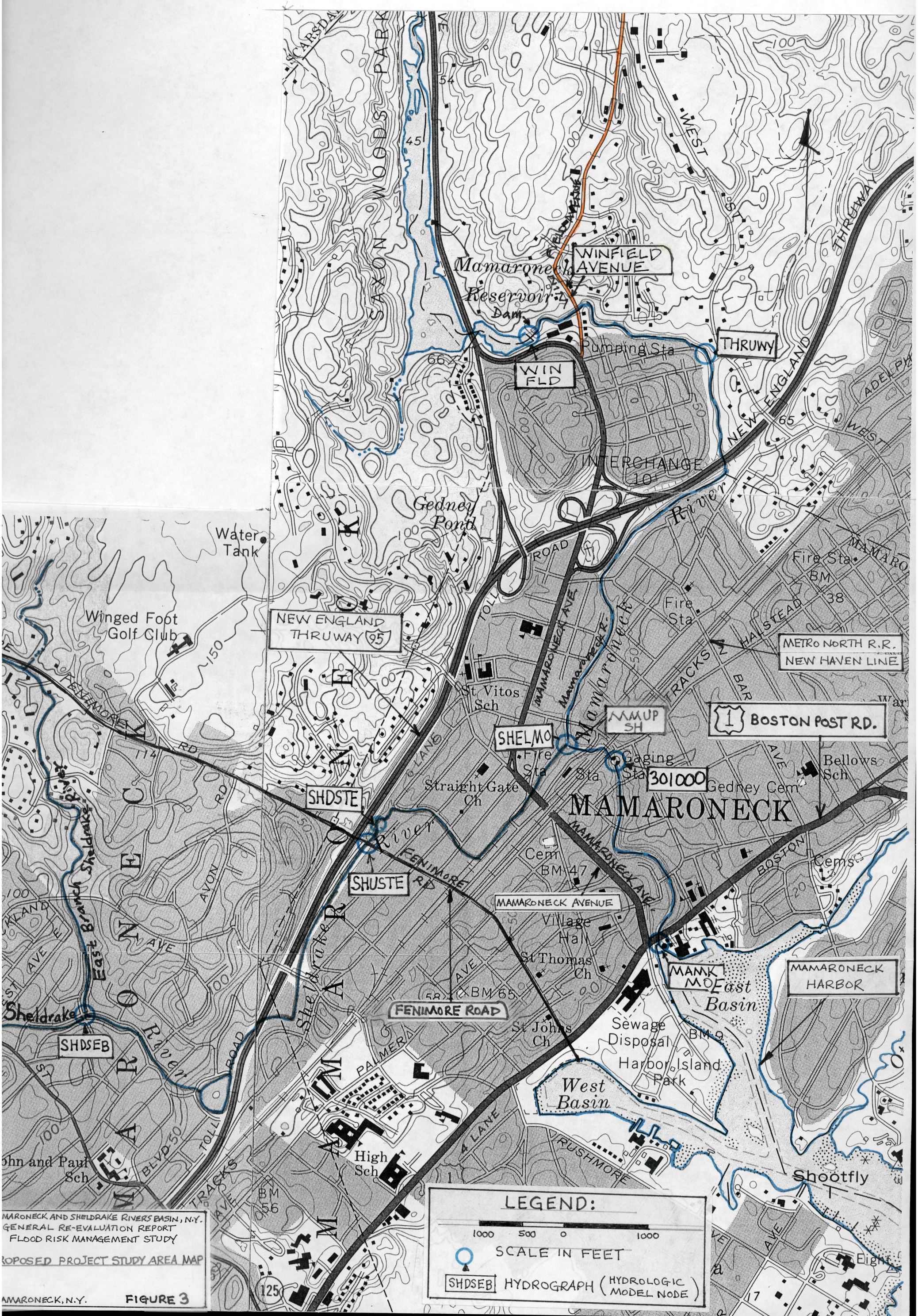
SUBAREA	DRAINAGE AREA (SQ. MILES)	CLARK'S "TC"	CLARK'S "R"	REMARKS
A	1.74	3.14	3.14	
B	1.18	2.02	2.02	
C	0.32	0.68	0.68	
D	2.95	5.58	5.58	
E	2.94	2.44	2.44	
F	0.92	1.52	1.52	
G	1.32	2.16	2.16	
H	0.43	1.10	1.10	
I	2.24	2.52	2.52	WEST BRANCH MAMARONECK
J	0.63	2.70	2.70	
K	0.68	3.20	3.20	
L	0.83	1.08	1.08	
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 2



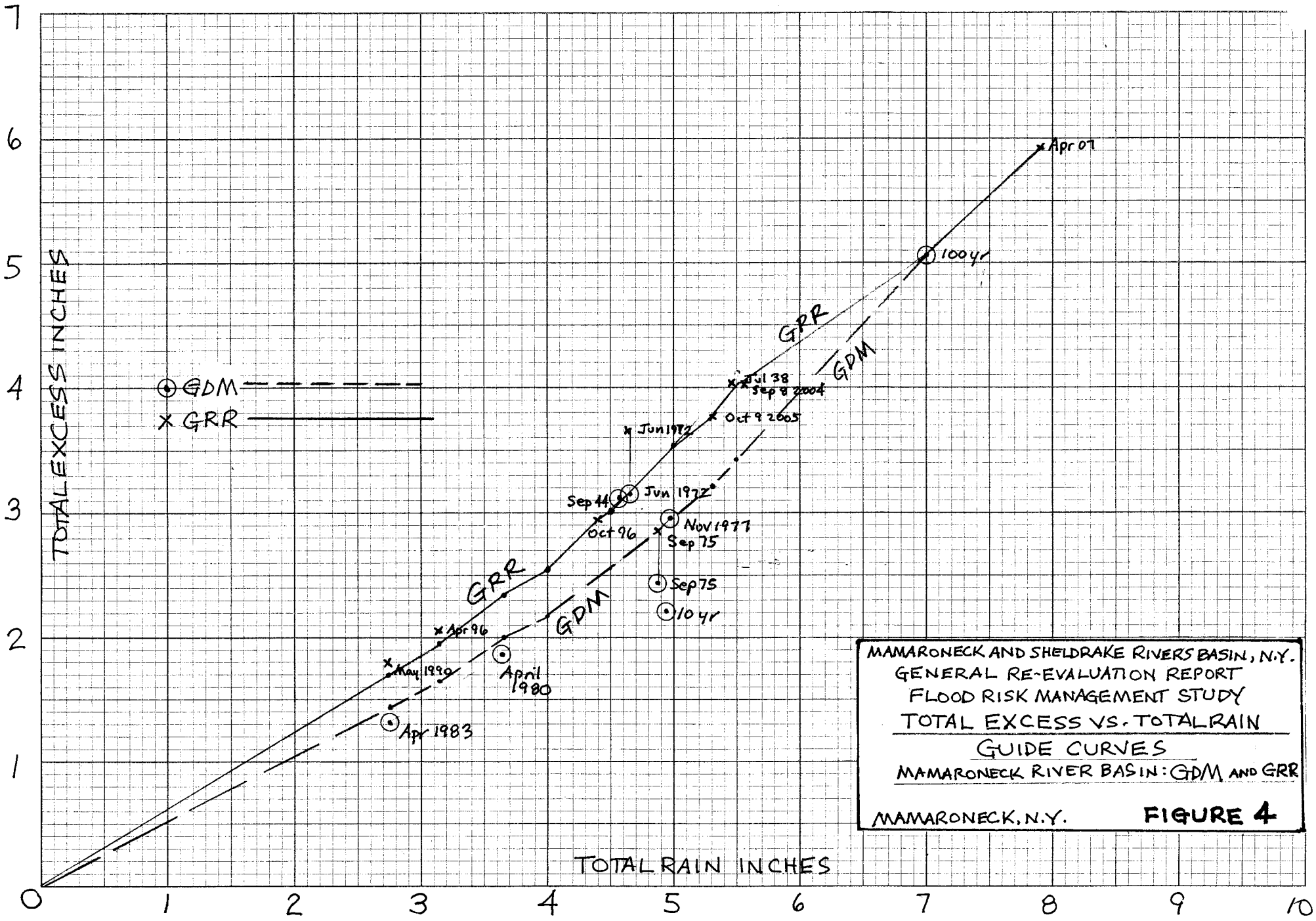
MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

PROPOSED PROJECT STUDY AREA MAP

MAMARONECK, N.Y. FIGURE 3

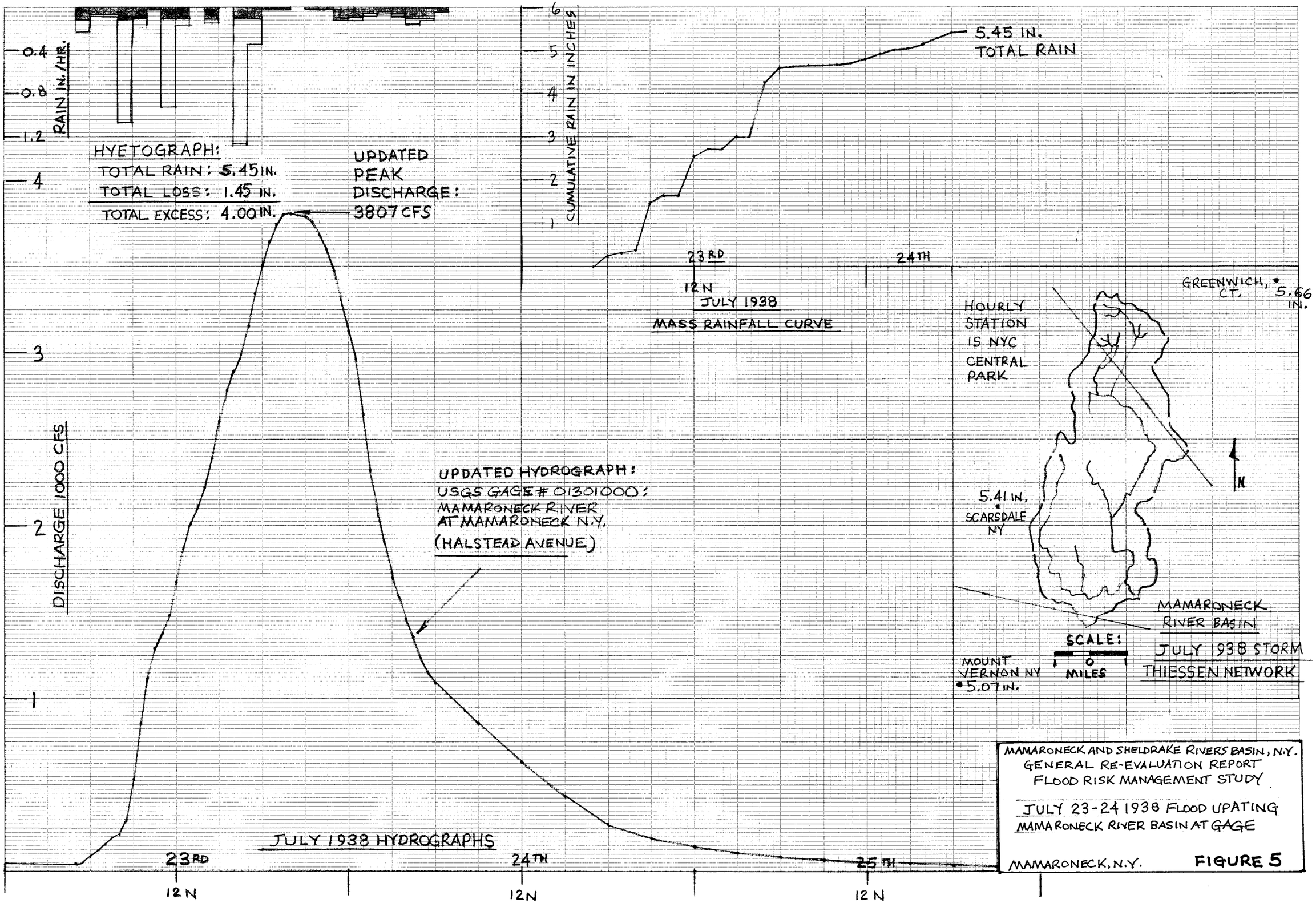
K+E 10 X 10 TO THE INCH • 7 X 10 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 0780



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
TOTAL EXCESS VS. TOTAL RAIN
GUIDE CURVES
MAMARONECK RIVER BASIN: GDM AND GRR

MAMARONECK, N.Y. **FIGURE 4**

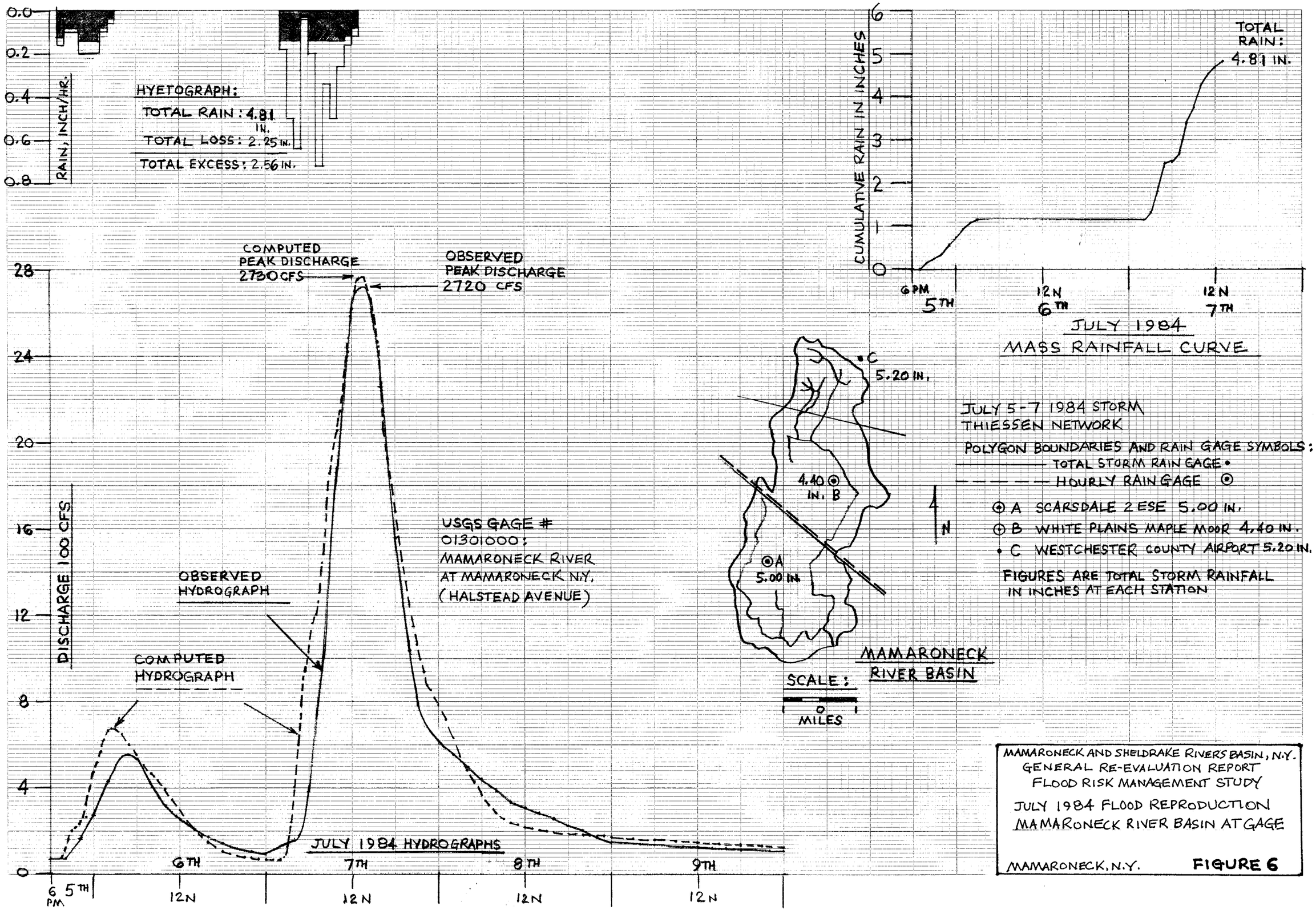


MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

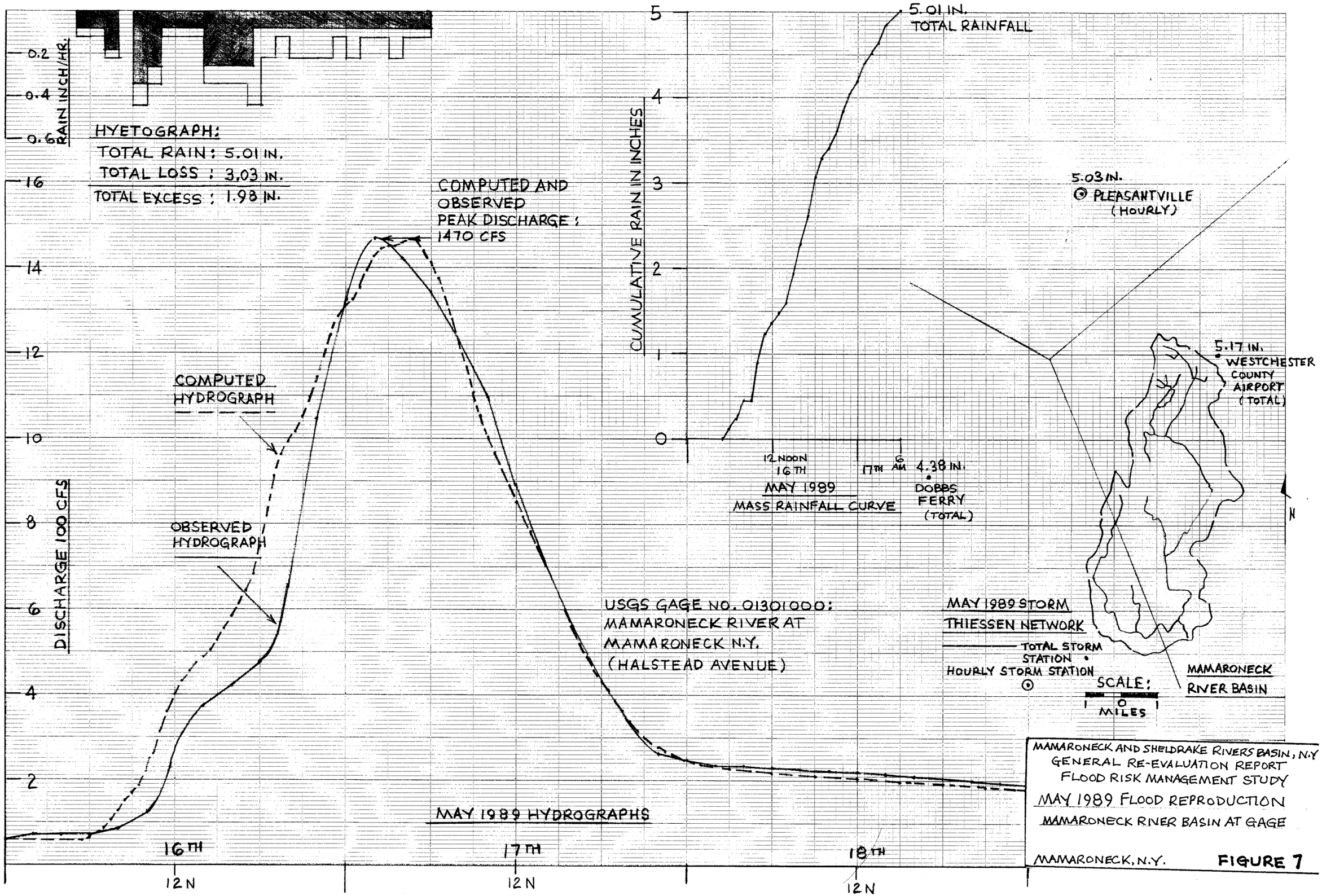
JULY 23-24 1938 FLOOD UPATING
MAMARONECK RIVER BASIN AT GAGE

MAMARONECK, N.Y. **FIGURE 5**

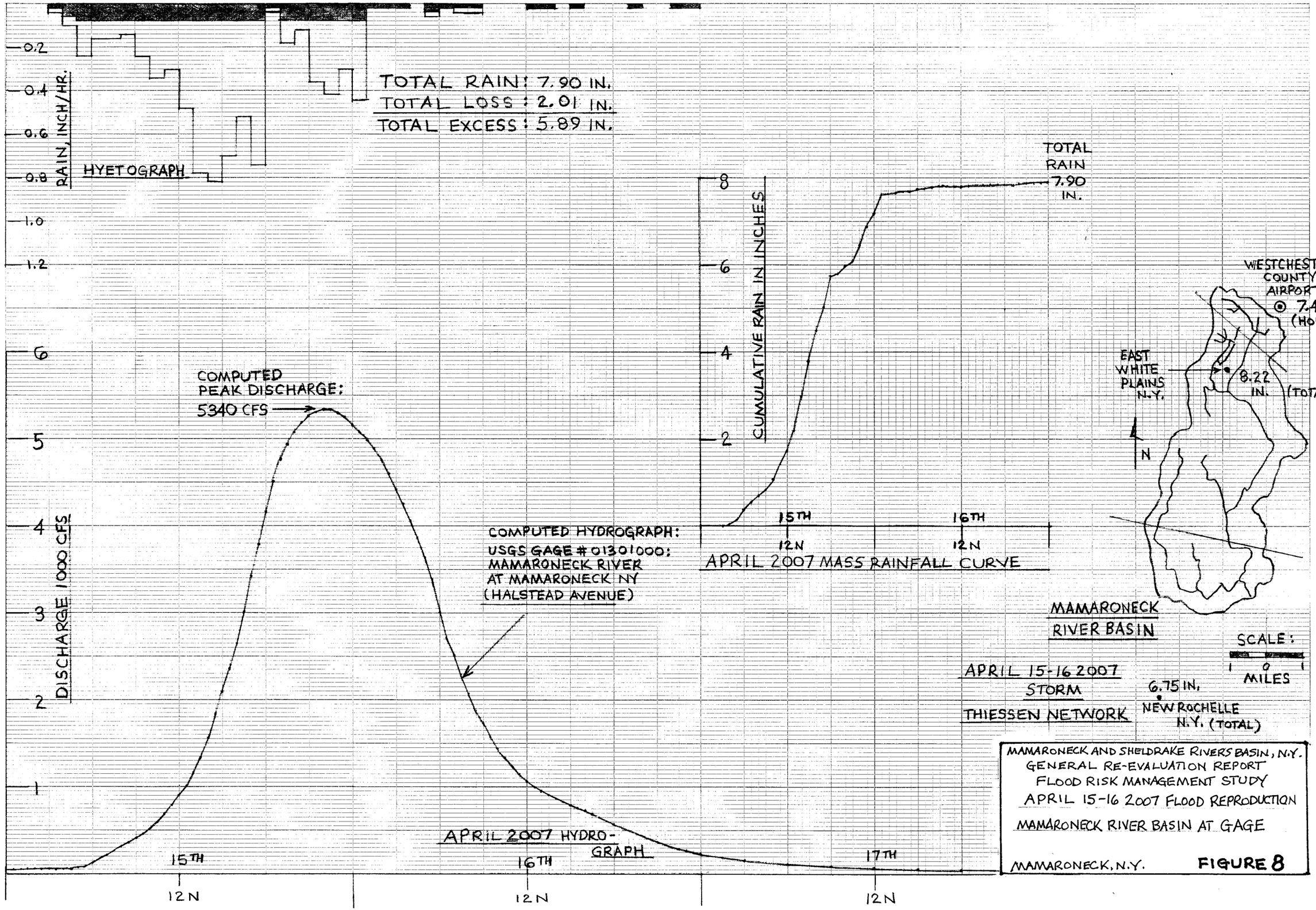
47 1970
K+E 12 x 20 TO THE INCH • 10 X 15 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.



47 1970
K+E 12 x 20 TO THE INCH • 10 x 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.



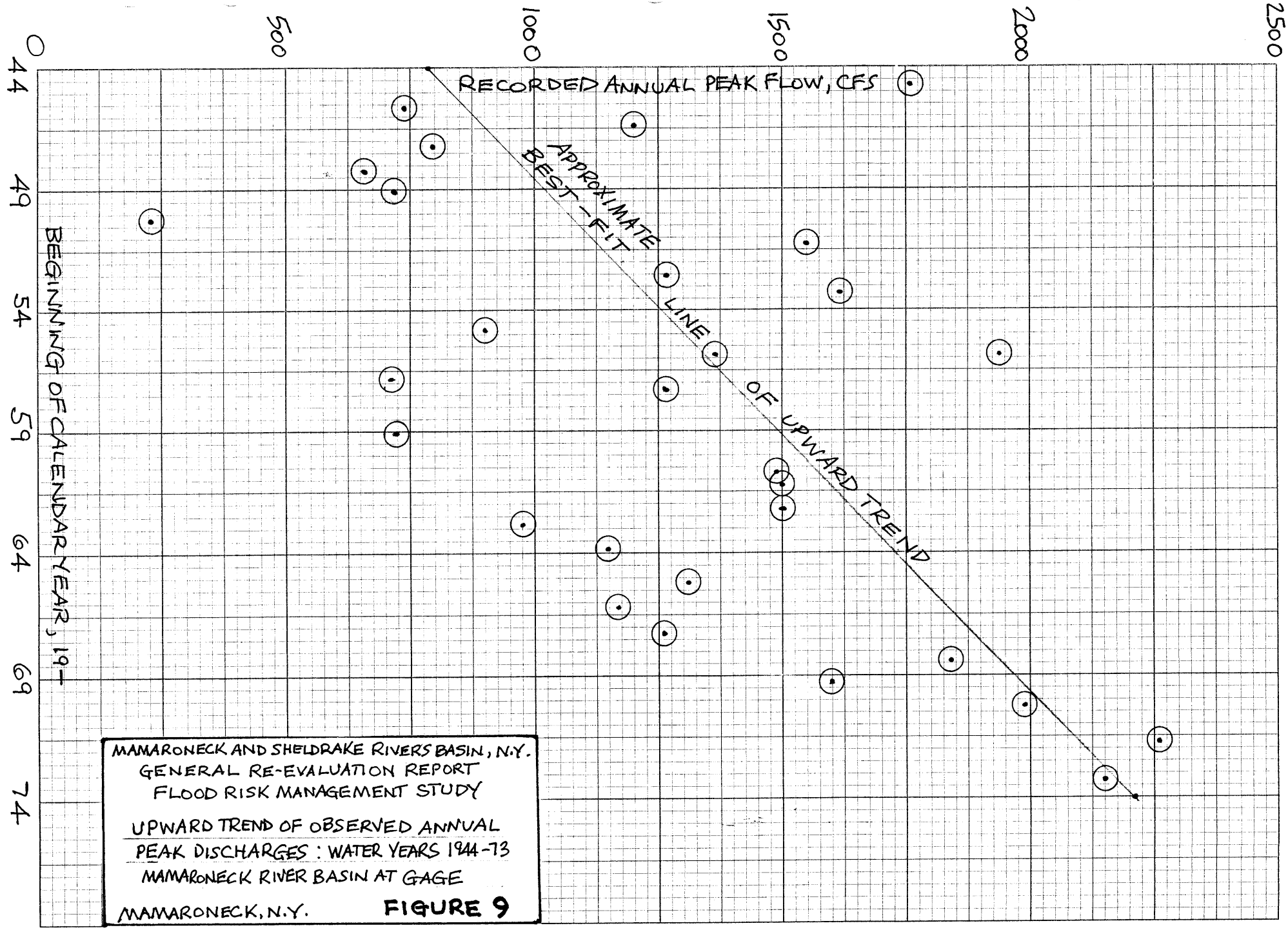
47 1970
K+E 12 X 20 TO THE INCH • 10 X 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.



47 1970
K+E 12 x 20 TO THE INCH • 10 x 15 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.

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

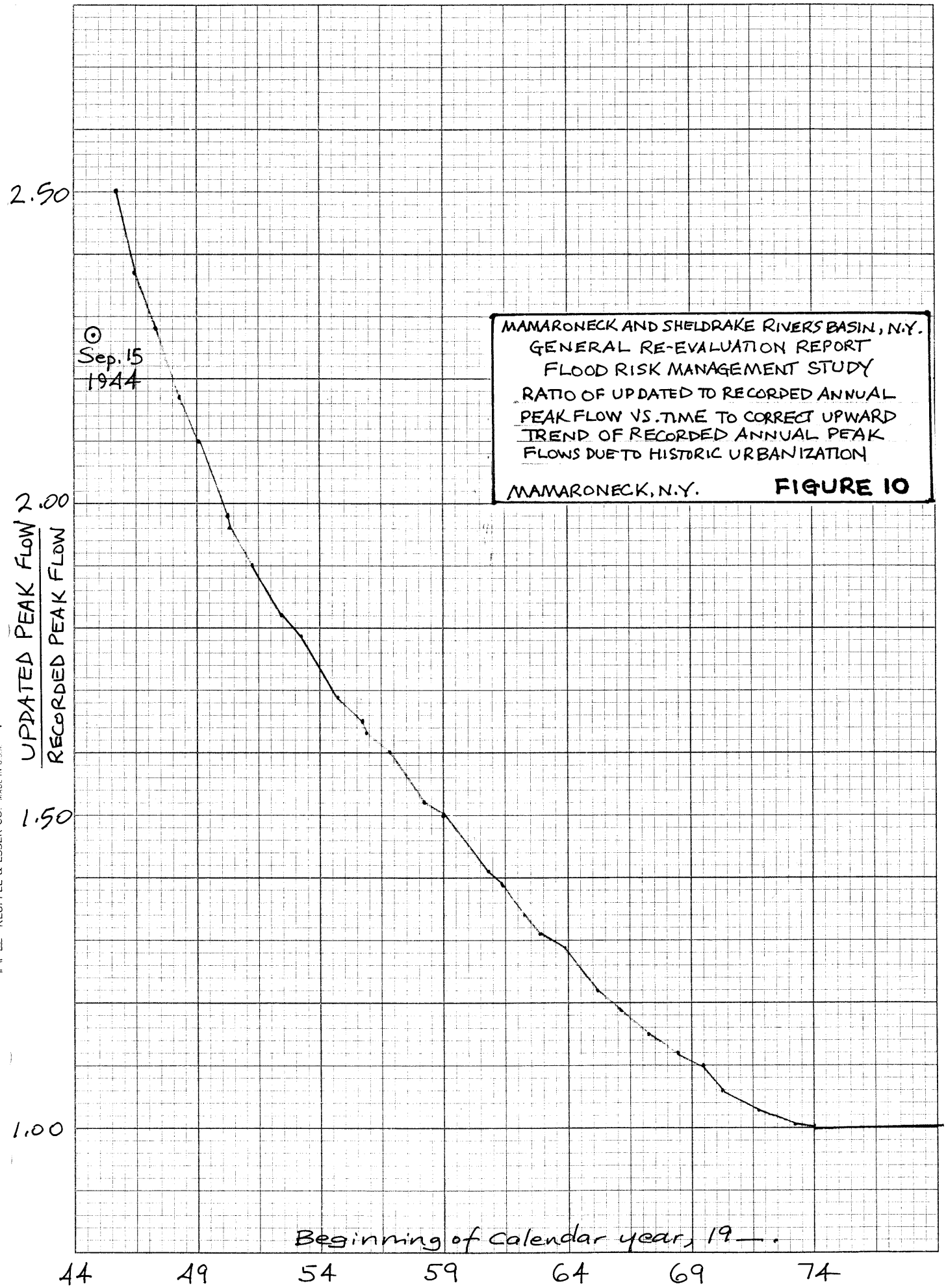
46 0780



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

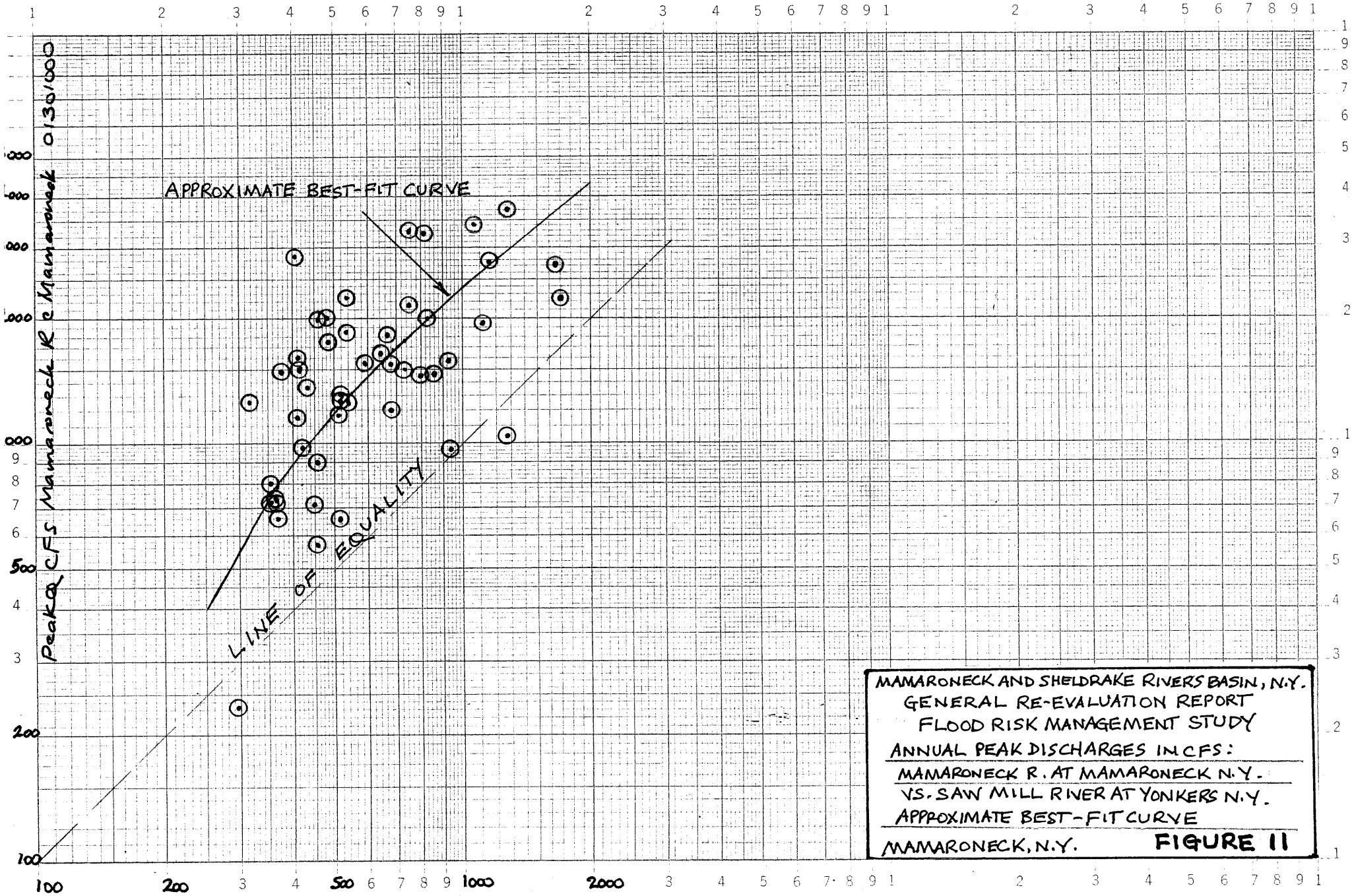
UPWARD TREND OF OBSERVED ANNUAL PEAK DISCHARGES : WATER YEARS 1944-73
MAMARONECK RIVER BASIN AT GAGE
MAMARONECK, N.Y.

FIGURE 9



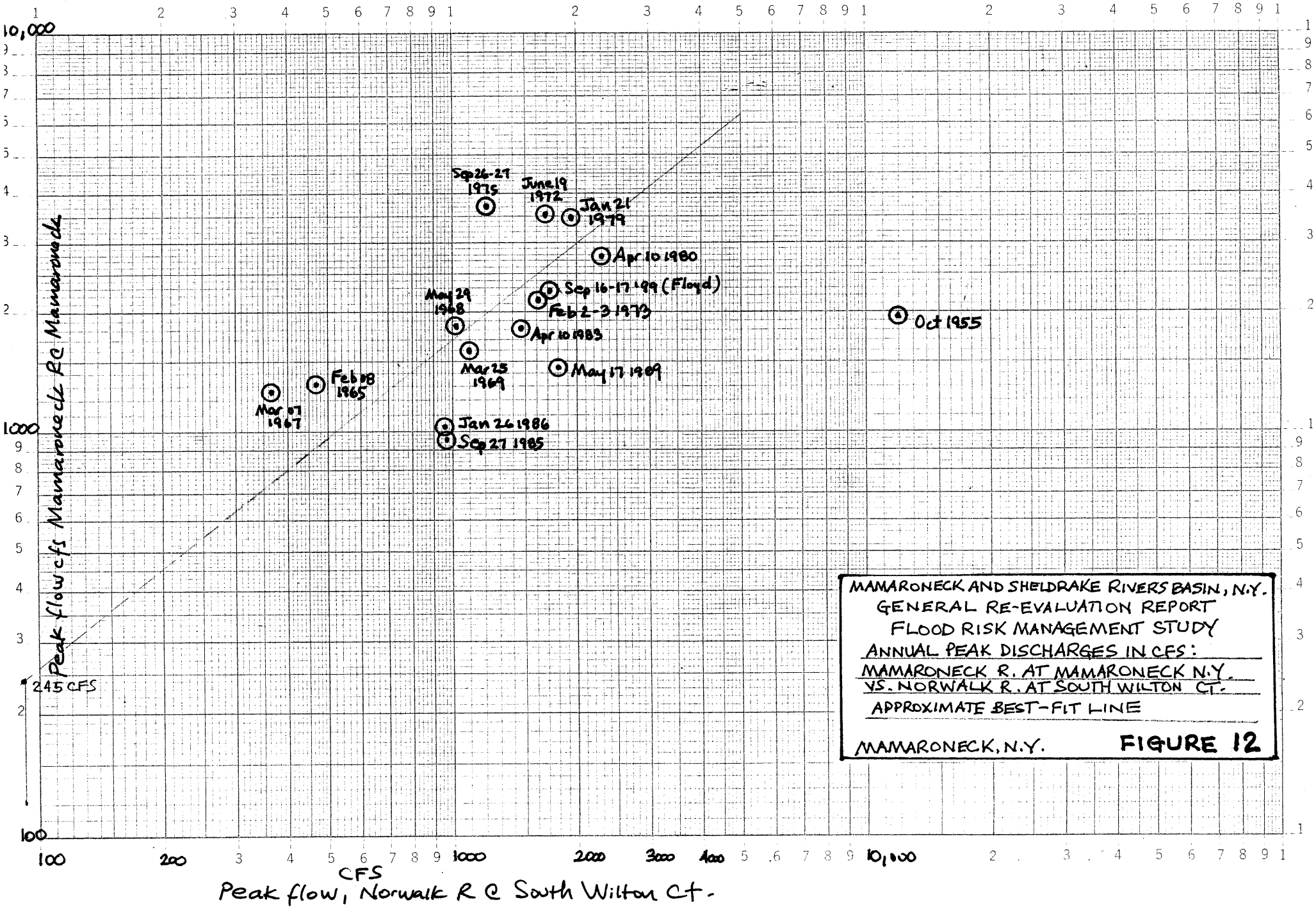
46 0780
K&E 10 X 10 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

K+E LOGARITHMIC • 2 X 3 CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
ANNUAL PEAK DISCHARGES IN CFS:
MAMARONECK R. AT MAMARONECK N.Y.
VS. SAW MILL RIVER AT YONKERS N.Y.
APPROXIMATE BEST-FIT CURVE
MAMARONECK, N.Y. **FIGURE 11**

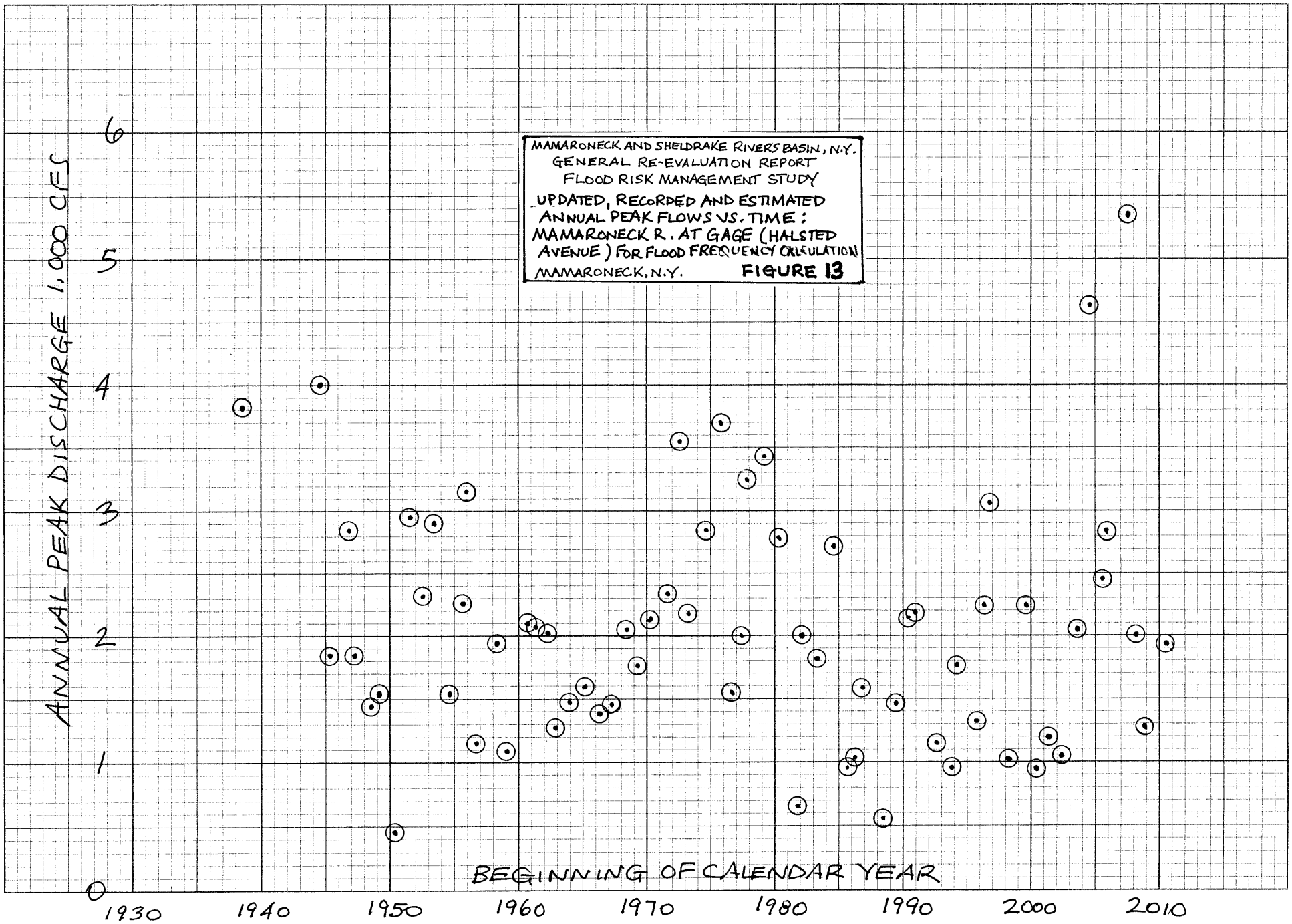
K+E LOGARITHMIC • 2 X 3 CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 7323



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
ANNUAL PEAK DISCHARGES IN CFS:
MAMARONECK R. AT MAMARONECK N.Y.
VS. NORWALK R. AT SOUTH WILTON CT.
APPROXIMATE BEST-FIT LINE

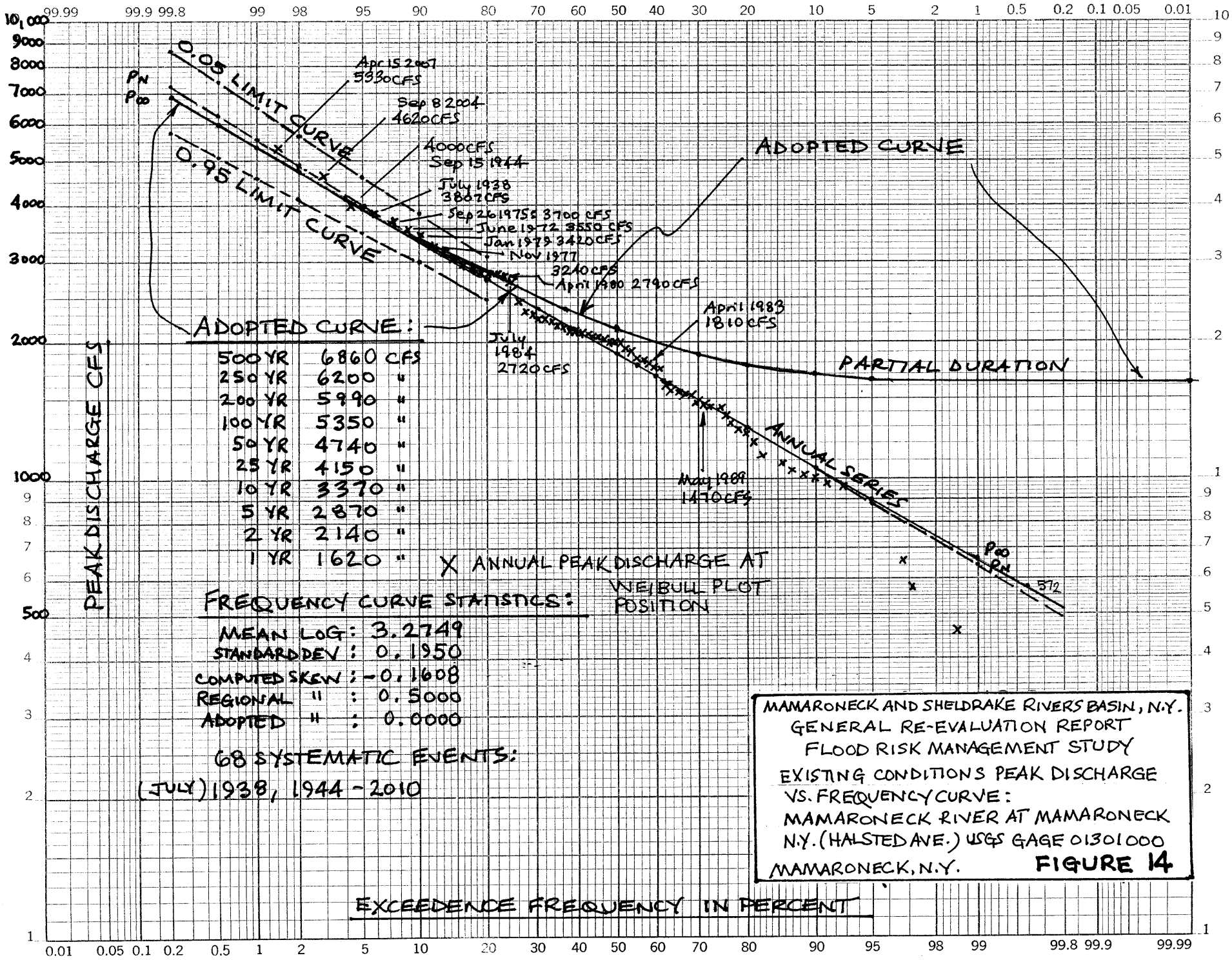
MAMARONECK, N.Y. **FIGURE 12**

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



K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8043



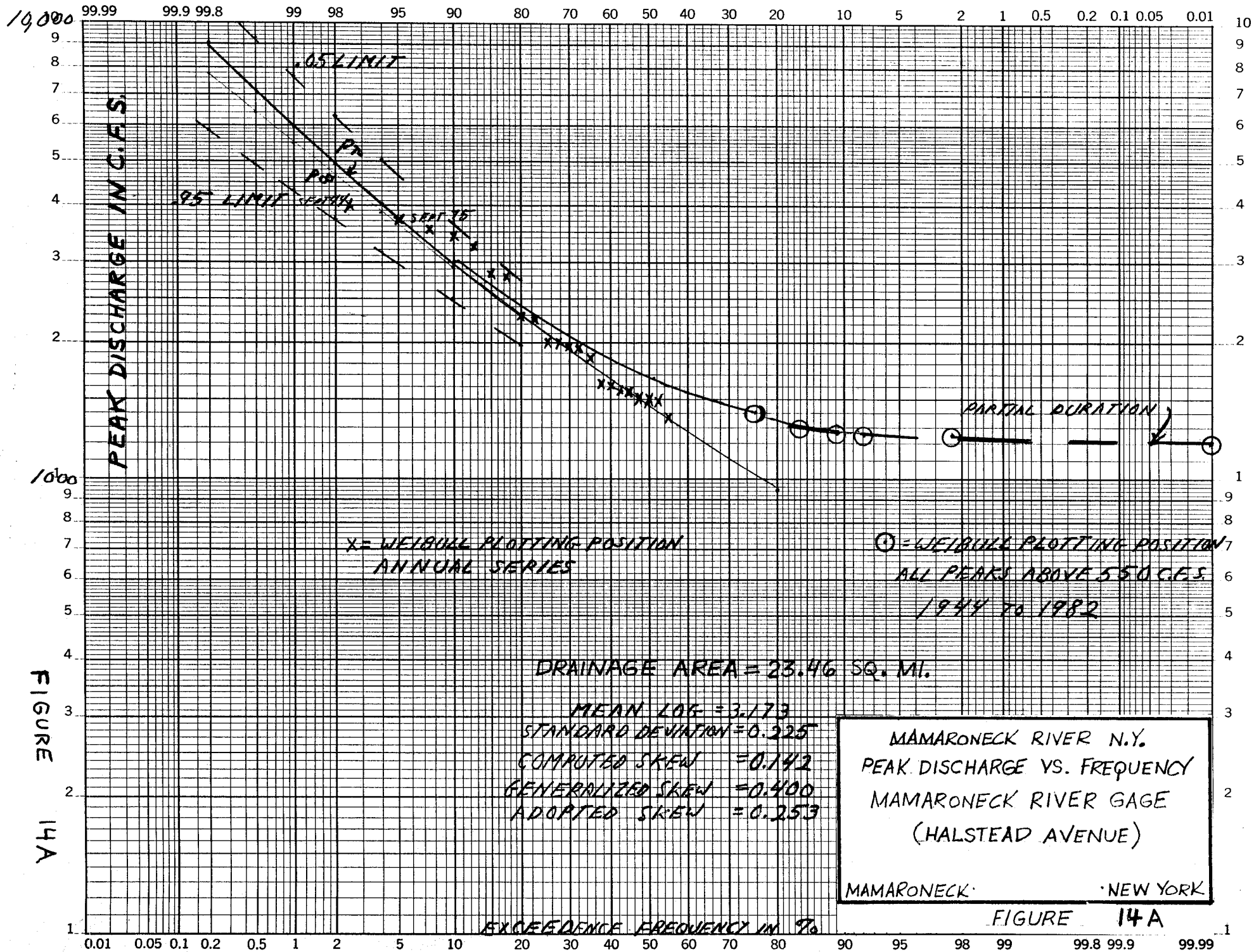

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
PEAK DISCHARGE IN C.F.S.
.05 LIMIT
.95 LIMIT
P_n
P_{00}
SEPT 75
PARTIAL DURATION
X = WEIBULL PLOTTING POSITION
ANNUAL SERIES
⊙ = WEIBULL PLOTTING POSITION
ALL PEAKS ABOVE 550 C.F.S.
1944 TO 1982
DRAINAGE AREA = 23.46 SQ. MI.
MEAN LOG = 3.173
STANDARD DEVIATION = 0.225
COMPUTED SKEW = 0.142
GENERALIZED SKEW = 0.400
ADOPTED SKEW = 0.253
MAMARONECK RIVER N.Y.
PEAK DISCHARGE VS. FREQUENCY
MAMARONECK RIVER GAGE
(HALSTEAD AVENUE)
MAMARONECK · NEW YORK
FIGURE 14A
EXCEEDENCE FREQUENCY IN %

FIGURE 14A

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043

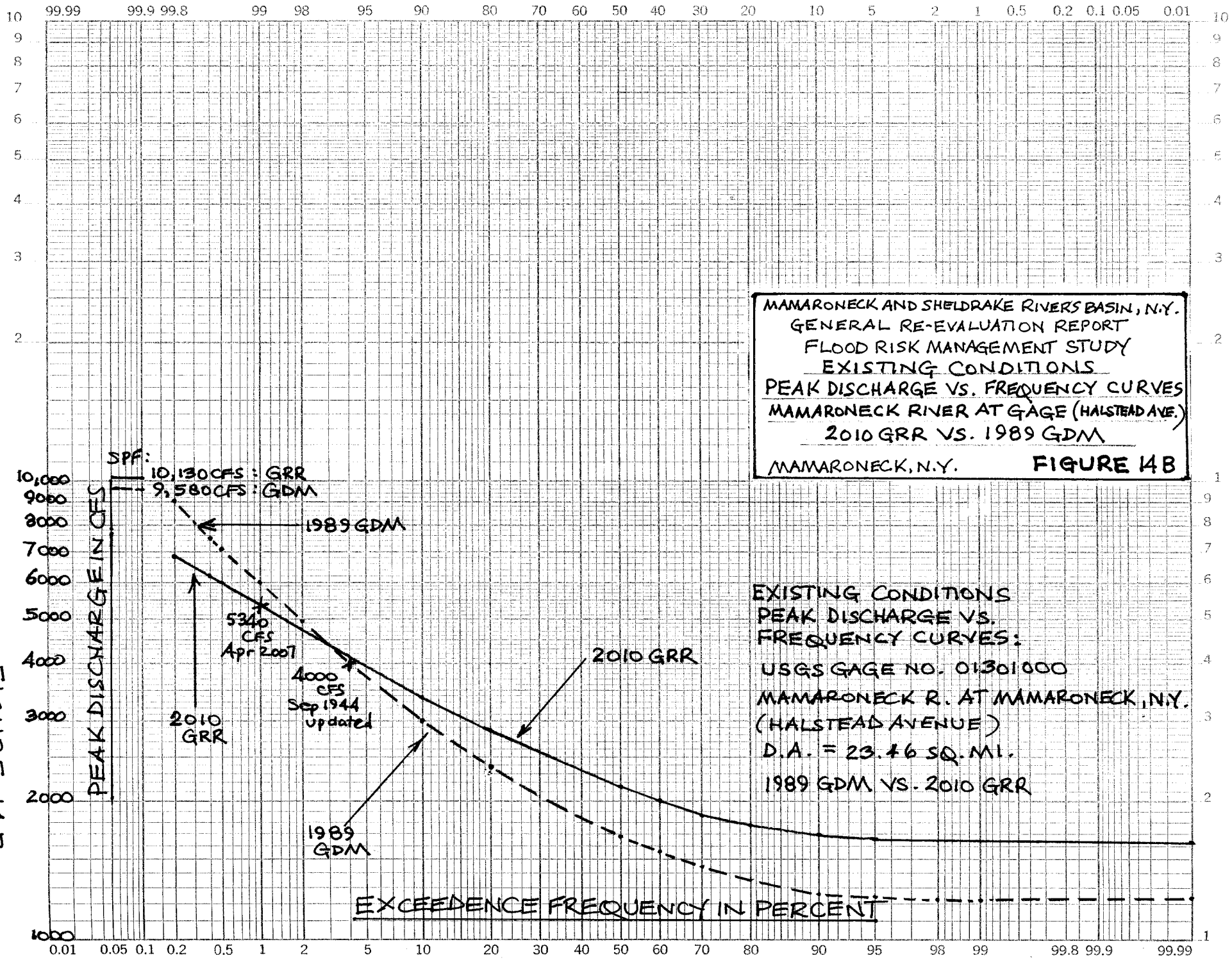
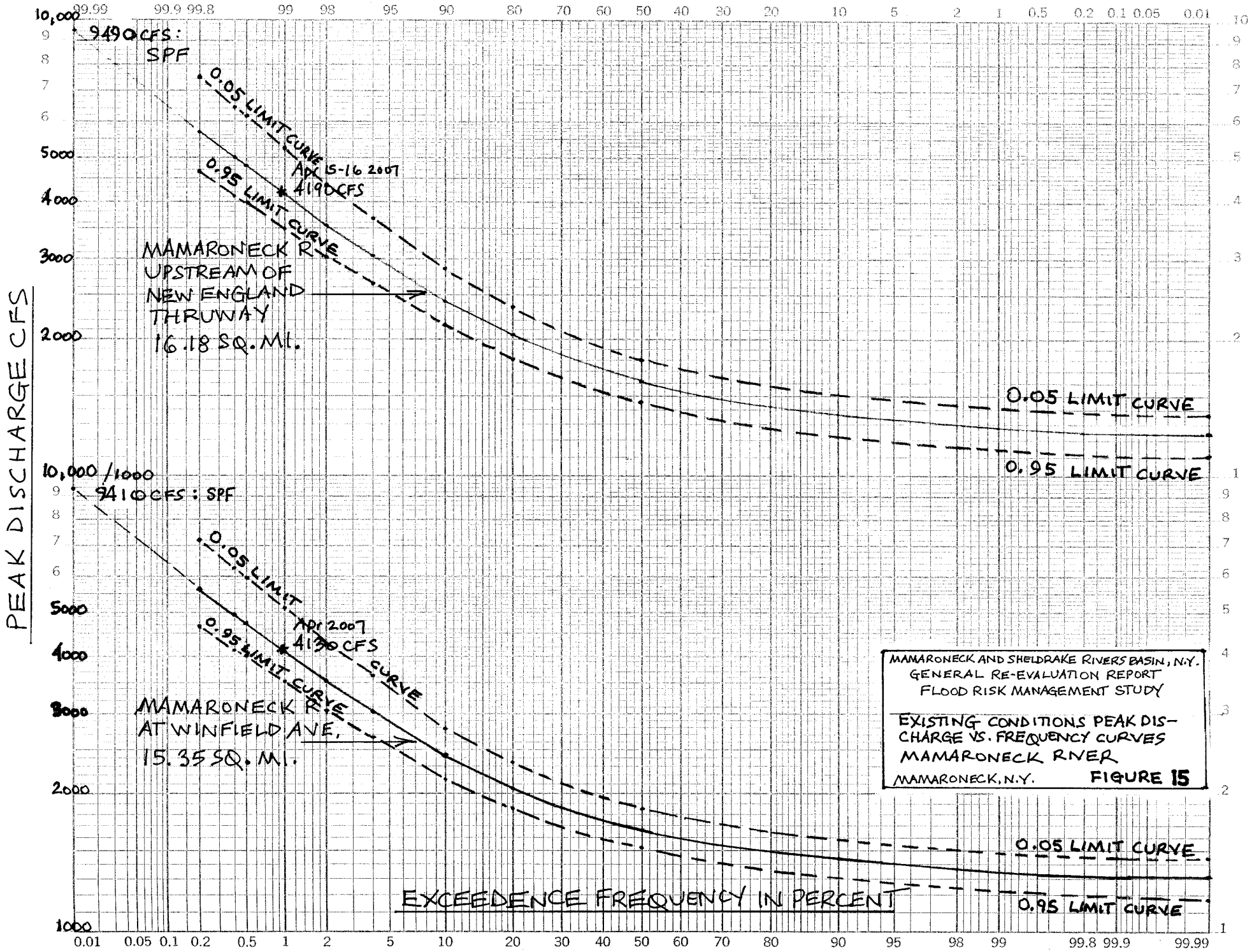


FIGURE 14B

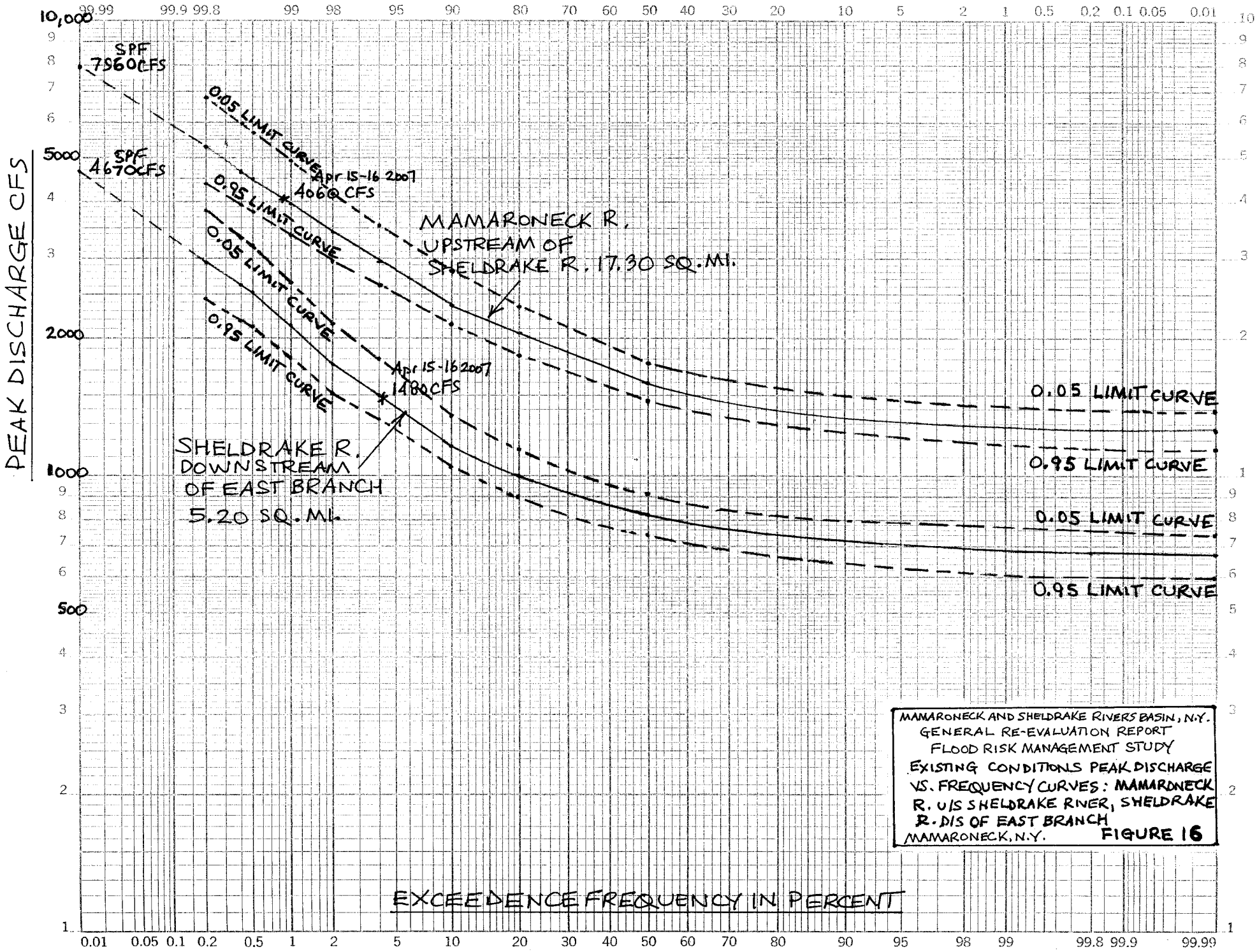
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

EXISTING CONDITIONS PEAK DISCHARGE VS. FREQUENCY CURVES
MAMARONECK RIVER
MAMARONECK, N.Y. **FIGURE 15**

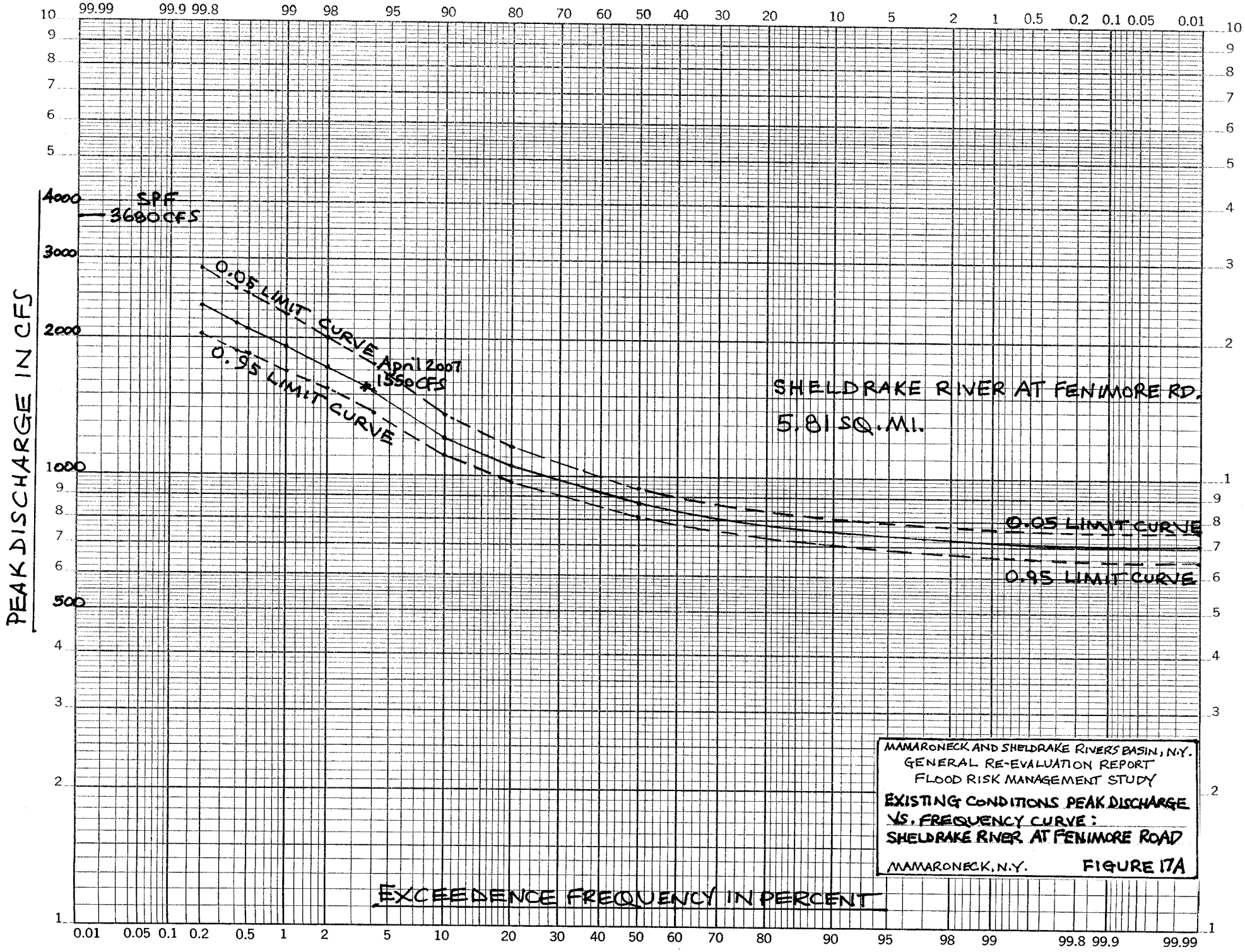
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
EXISTING CONDITIONS PEAK DISCHARGE VS. FREQUENCY CURVES: MAMARONECK R. U/S SHELDRAKE RIVER, SHELDRAKE R. D/S OF EAST BRANCH
MAMARONECK, N.Y.

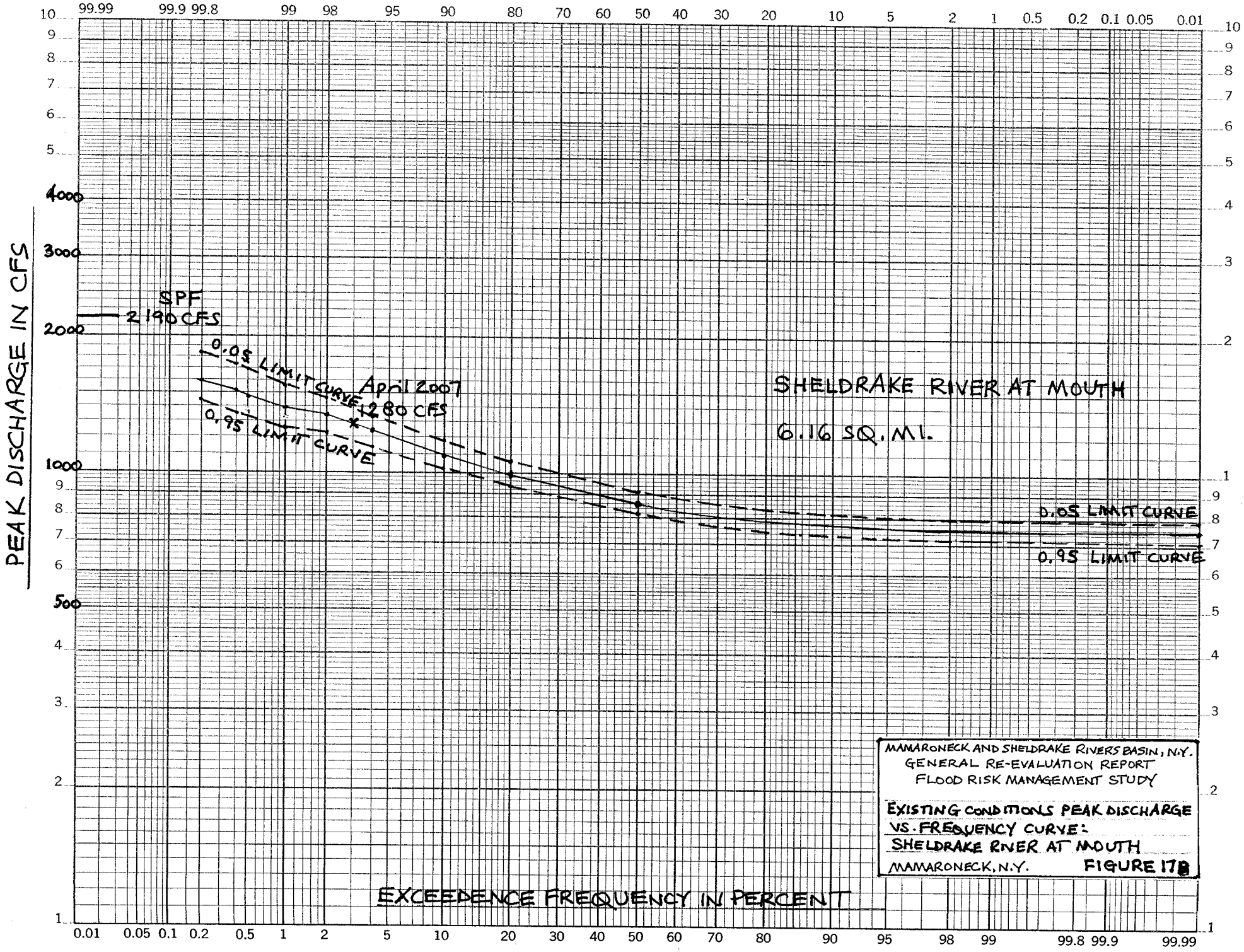
FIGURE 16

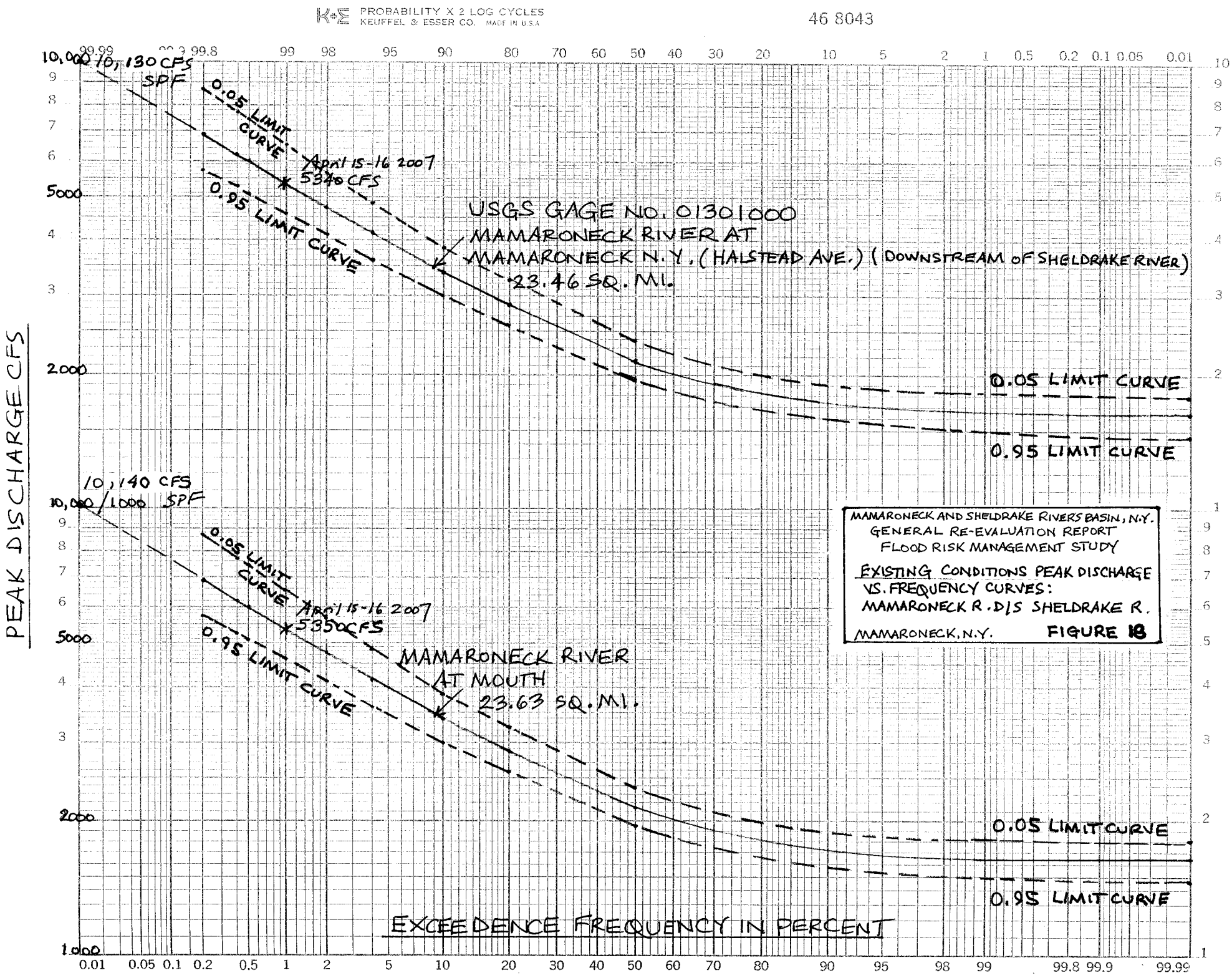
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
EXISTING CONDITIONS PEAK DISCHARGE VS. FREQUENCY CURVE:
SHELDRAKE RIVER AT FENIMORE ROAD
MAMARONECK, N.Y. FIGURE 17A

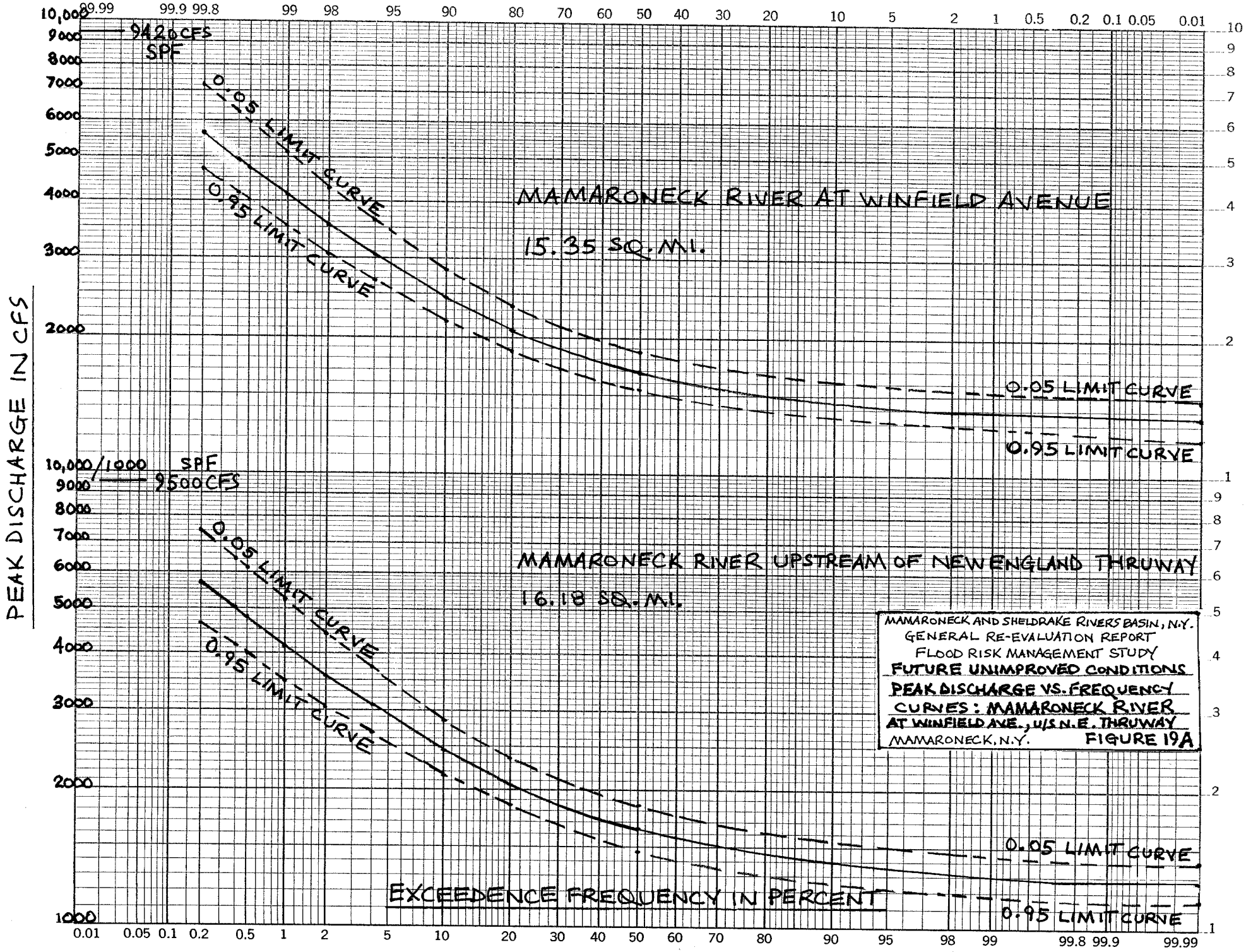
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043




K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043
PEAK DISCHARGE CFS
EXCEEDENCE FREQUENCY IN PERCENT
10,130 CFS SPF
0.05 LIMIT CURVE
April 15-16 2007
5340 CFS
0.95 LIMIT CURVE
USGS GAGE NO. 01301000
MAMARONECK RIVER AT
MAMARONECK N.Y. (HALSTEAD AVE.) (DOWNSTREAM OF SHELDRAKE RIVER)
23.46 SQ. MI.
0.05 LIMIT CURVE
0.95 LIMIT CURVE
10,140 CFS
10,000/1000 SPF
0.05 LIMIT CURVE
April 15-16 2007
5350 CFS
0.95 LIMIT CURVE
MAMARONECK RIVER
AT MOUTH
23.63 SQ. MI.
0.05 LIMIT CURVE
0.95 LIMIT CURVE
MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
EXISTING CONDITIONS PEAK DISCHARGE
VS. FREQUENCY CURVES:
MAMARONECK R. D/S SHELDRAKE R.
MAMARONECK, N.Y.
FIGURE 18

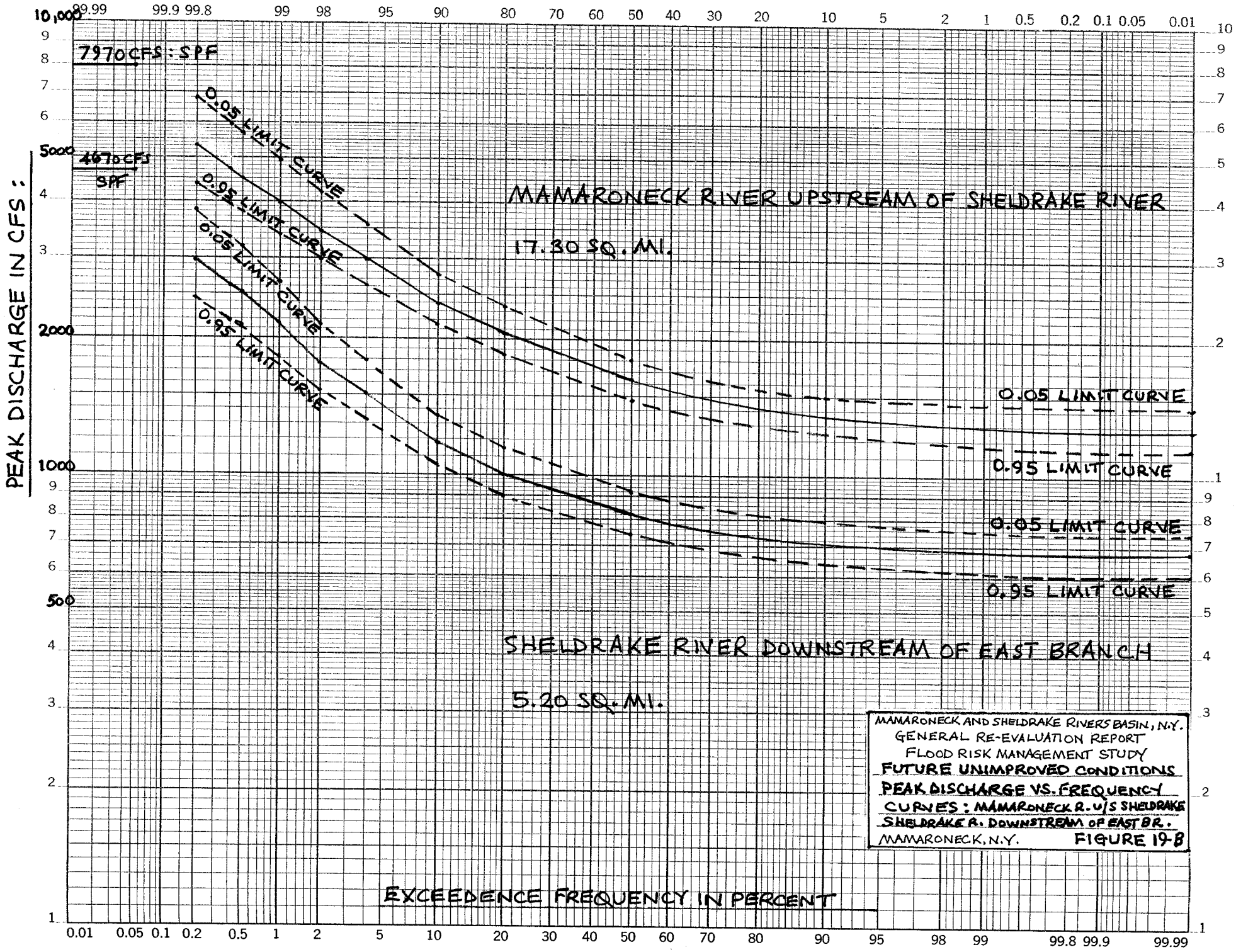
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8043



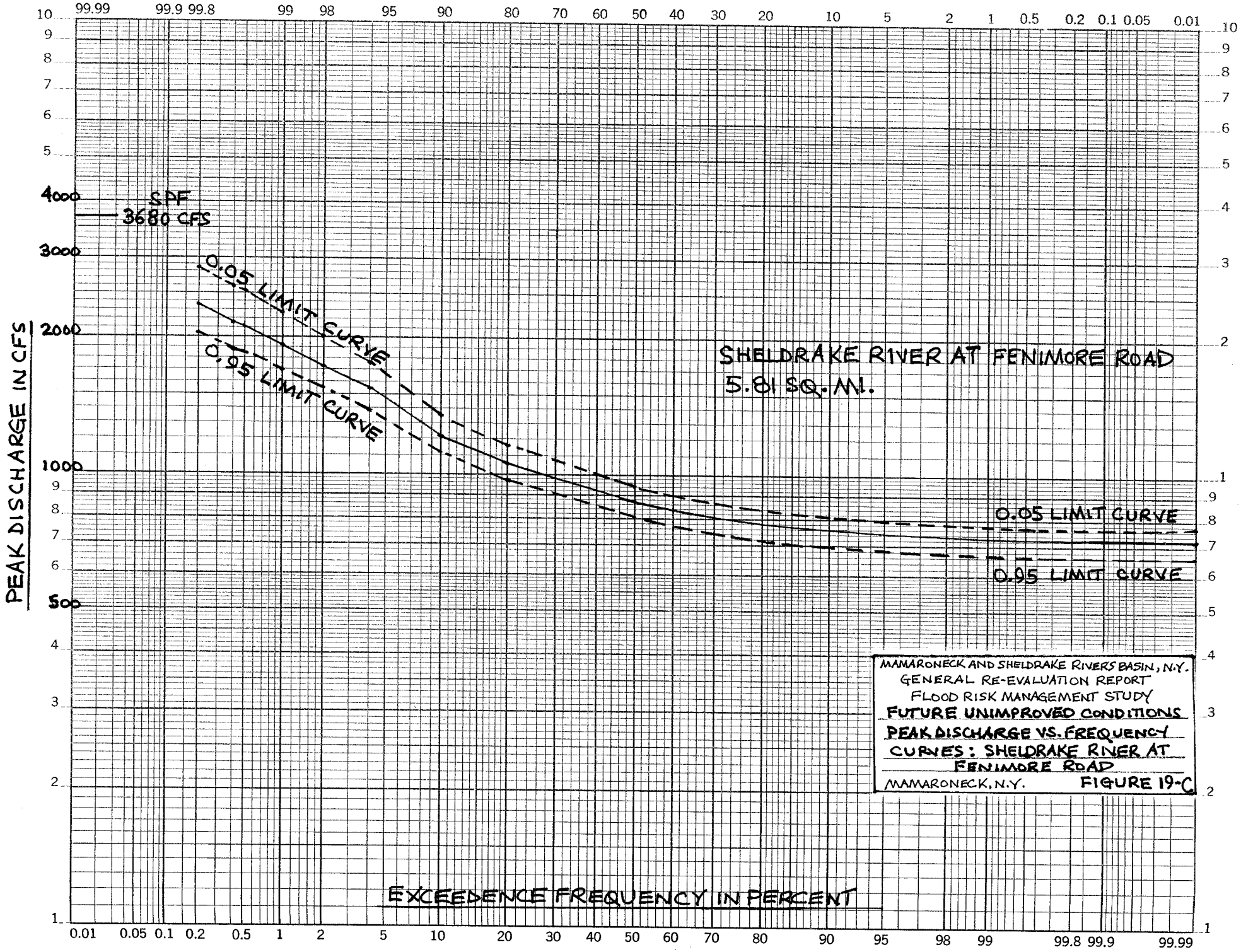
MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
FUTURE UNIMPROVED CONDITIONS
PEAK DISCHARGE VS. FREQUENCY CURVES: MAMARONECK RIVER AT WINFIELD AVE., U/S N.E. THRUWAY
MAMARONECK, N.Y.
FIGURE 19A

K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043

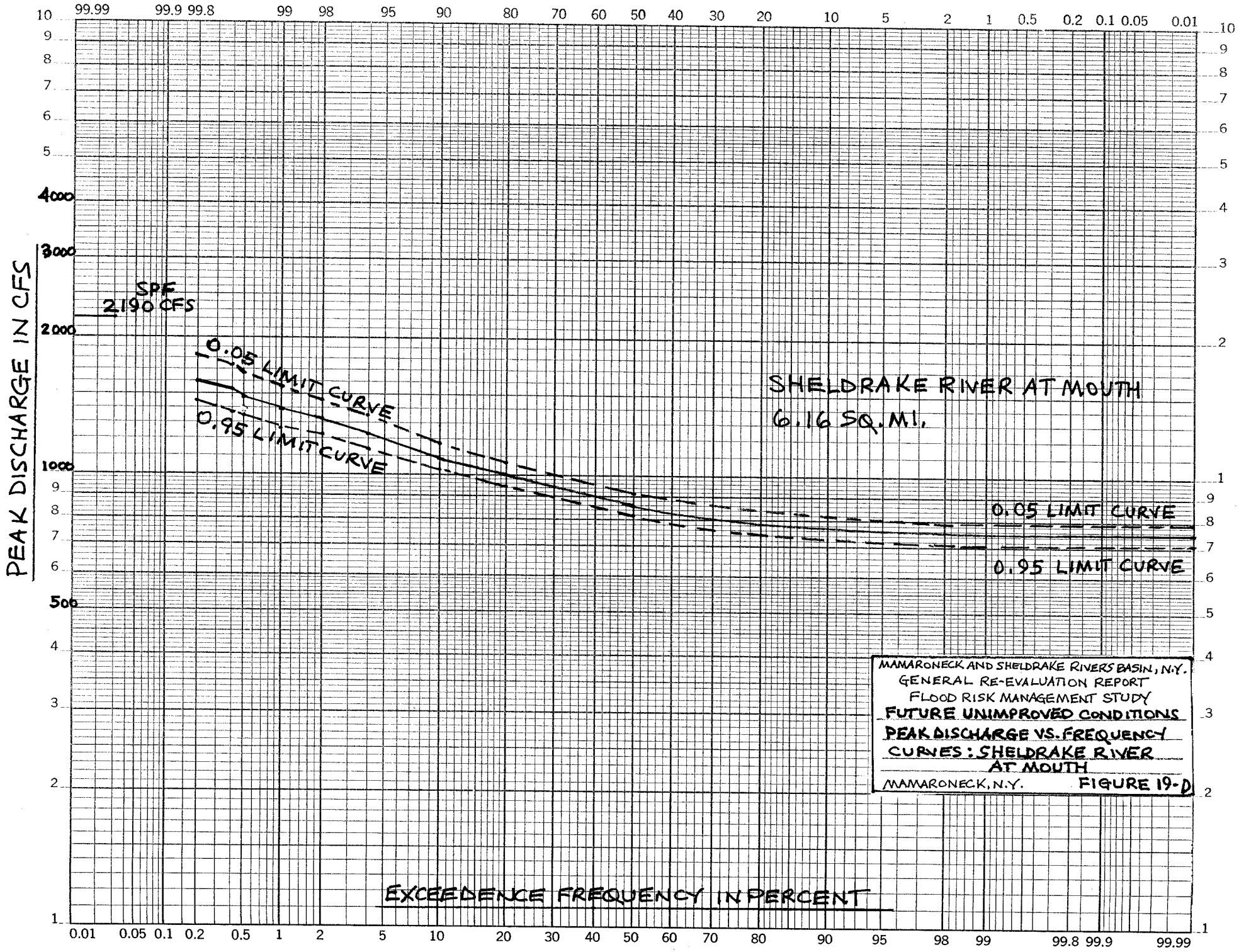


MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
FUTURE UNIMPROVED CONDITIONS
PEAK DISCHARGE VS. FREQUENCY
CURVES: MAMARONECK R. U/S SHELDRAKE
SHELDRAKE R. DOWNSTREAM OF EAST BR.
MAMARONECK, N.Y. FIGURE 19-B

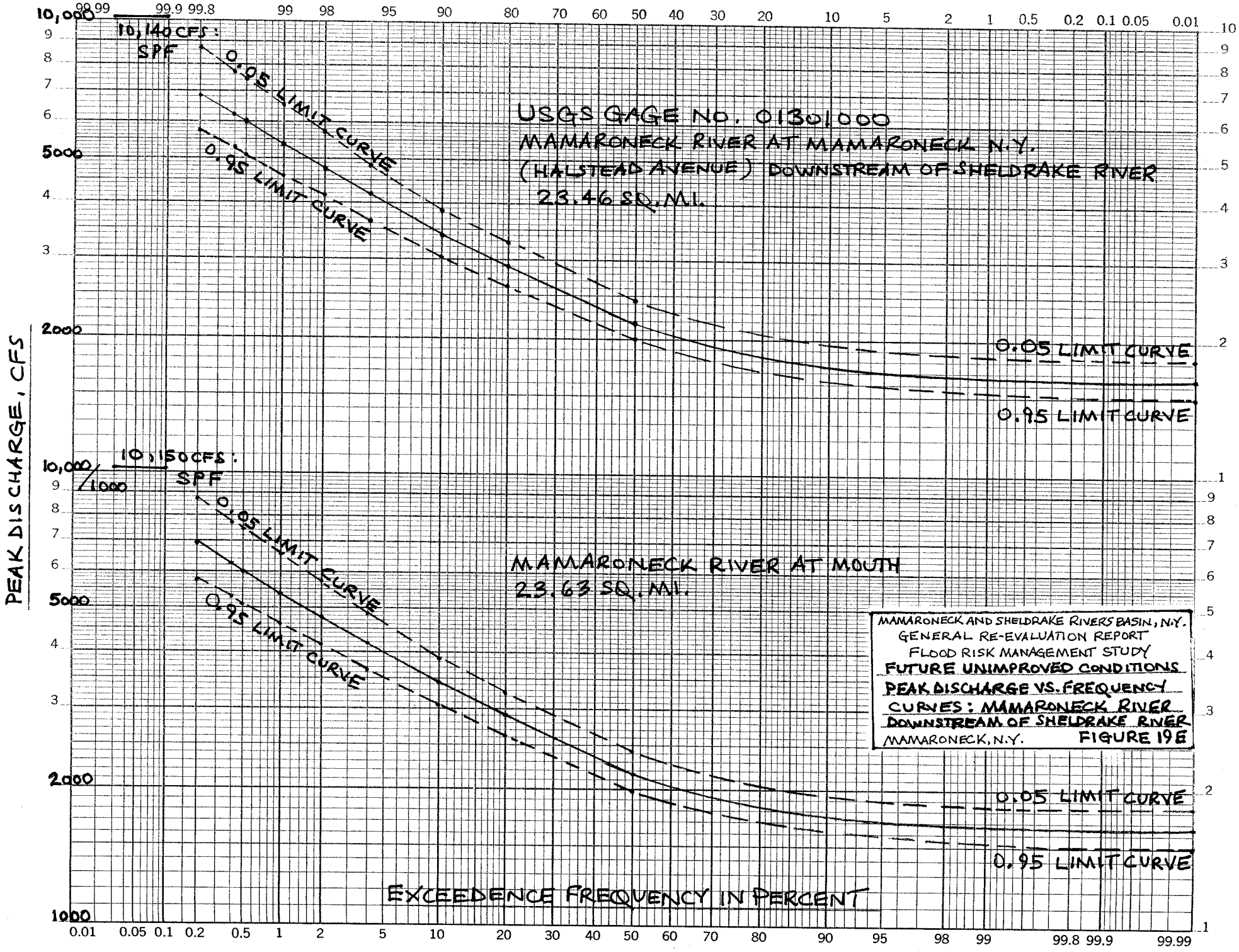
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



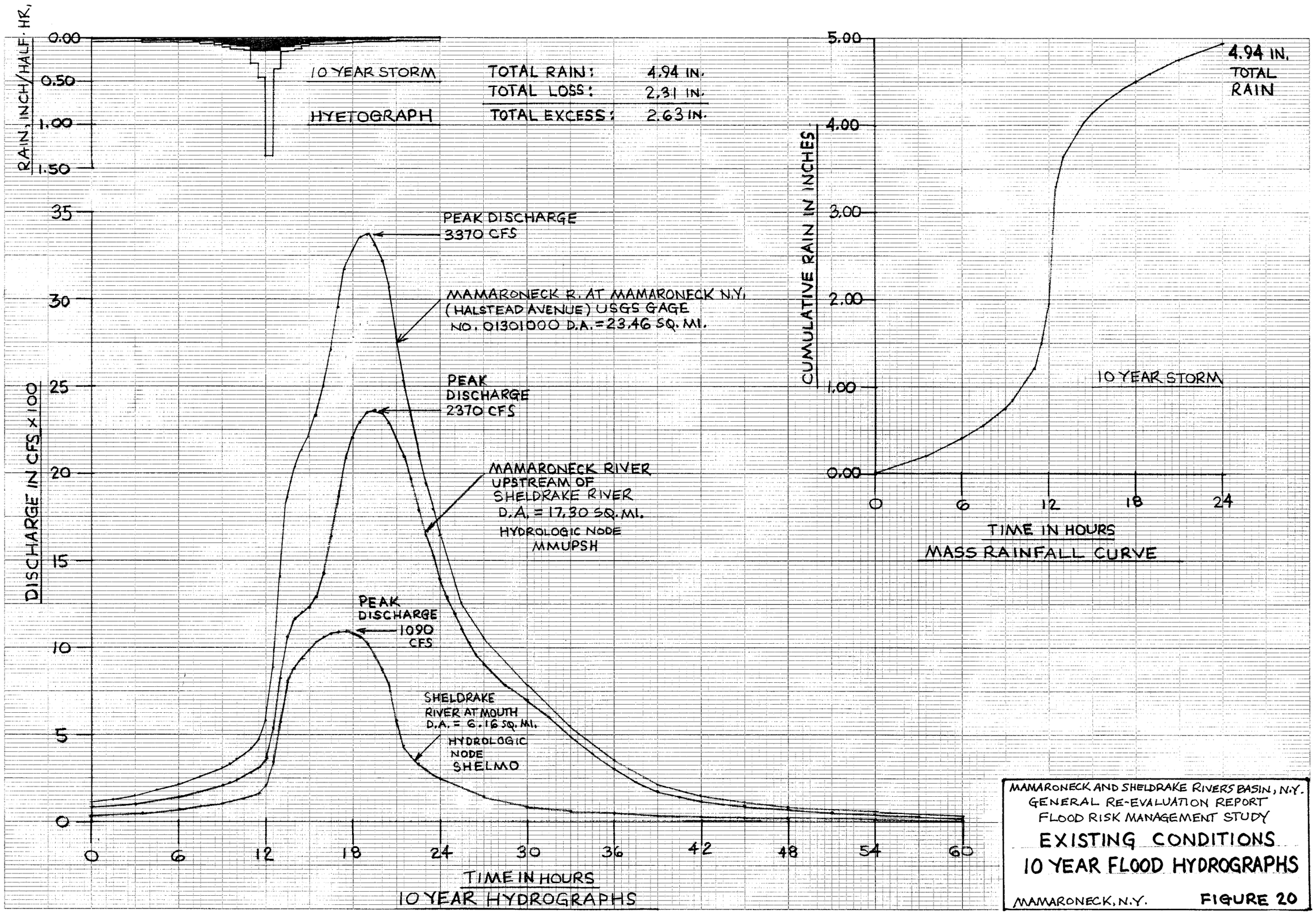
K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8043



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
FUTURE UNIMPROVED CONDITIONS
PEAK DISCHARGE VS. FREQUENCY
CURVES: MAMARONECK RIVER
DOWNSTREAM OF SHELDRAKE RIVER
MAMARONECK, N.Y. FIGURE 19E

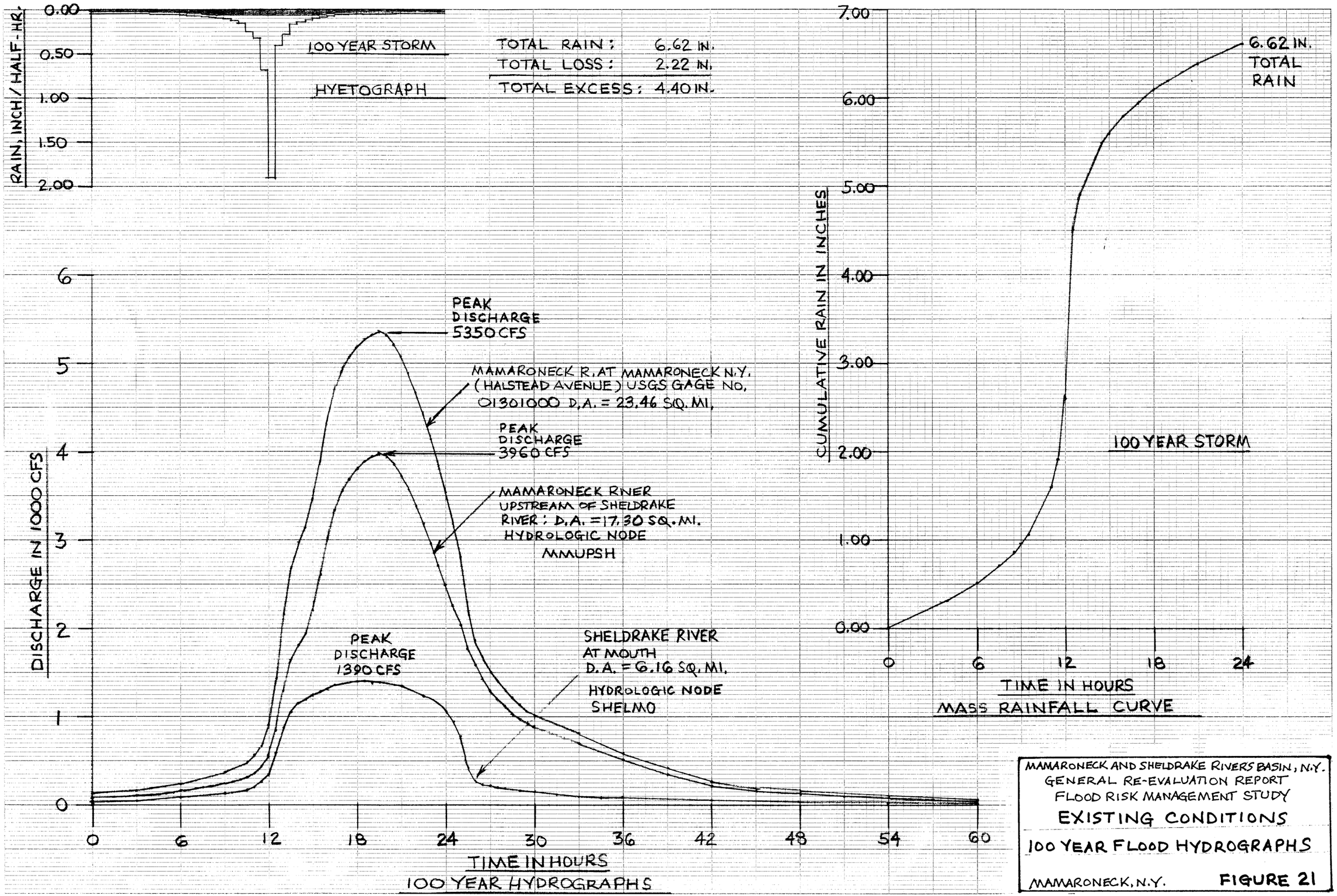


MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

EXISTING CONDITIONS
10 YEAR FLOOD HYDROGRAPHS

MAMARONECK, N.Y. **FIGURE 20**

47 1970
K+E 12 X 20 TO THE INCH • 10 X 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.



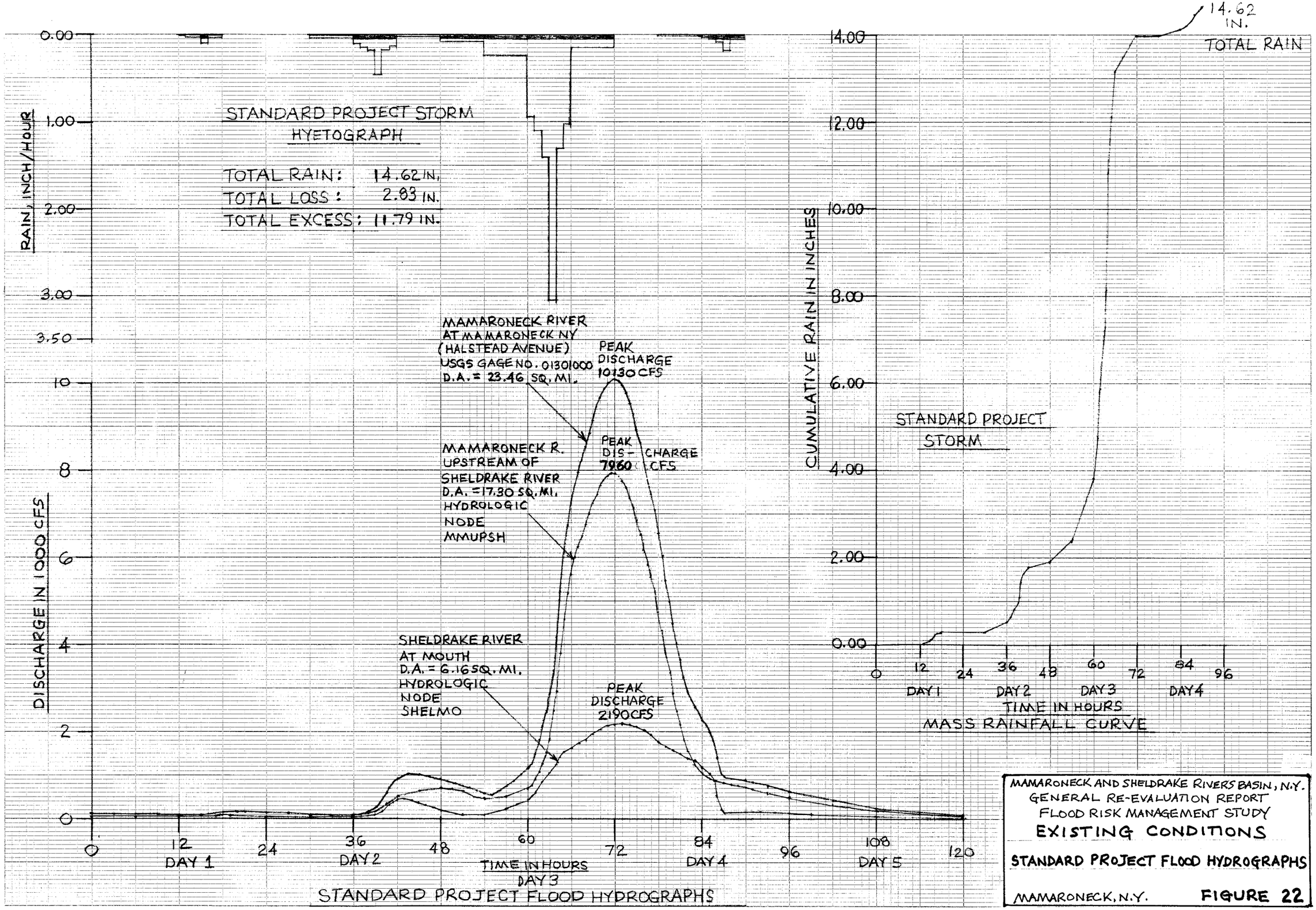
MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY

EXISTING CONDITIONS

100 YEAR FLOOD HYDROGRAPHS

MAMARONECK, N.Y. FIGURE 21

47 1970
K&E 12 x 20 TO THE INCH • 10 x 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.



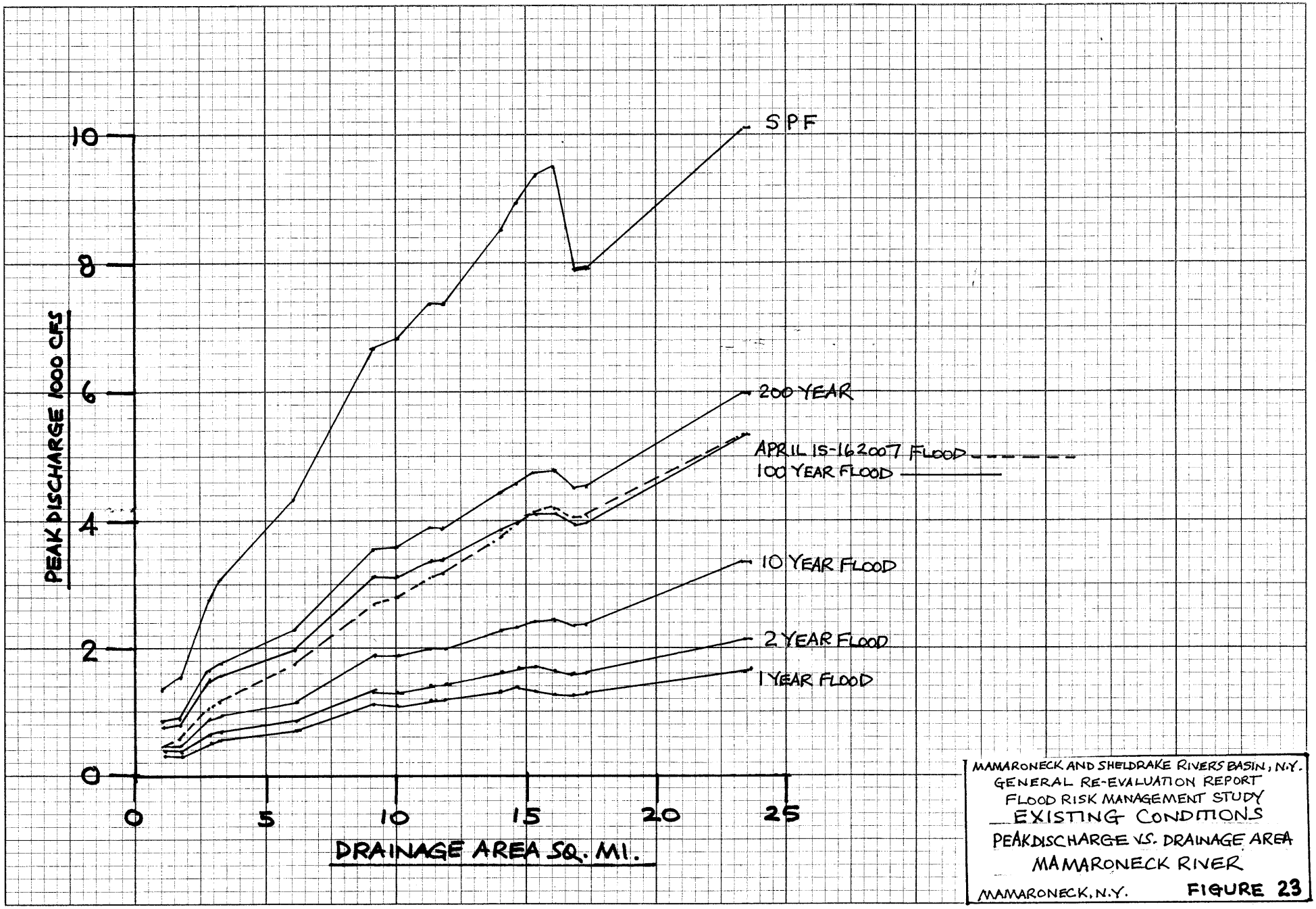
MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
EXISTING CONDITIONS

STANDARD PROJECT FLOOD HYDROGRAPHS

MAMARONECK, N.Y. FIGURE 22

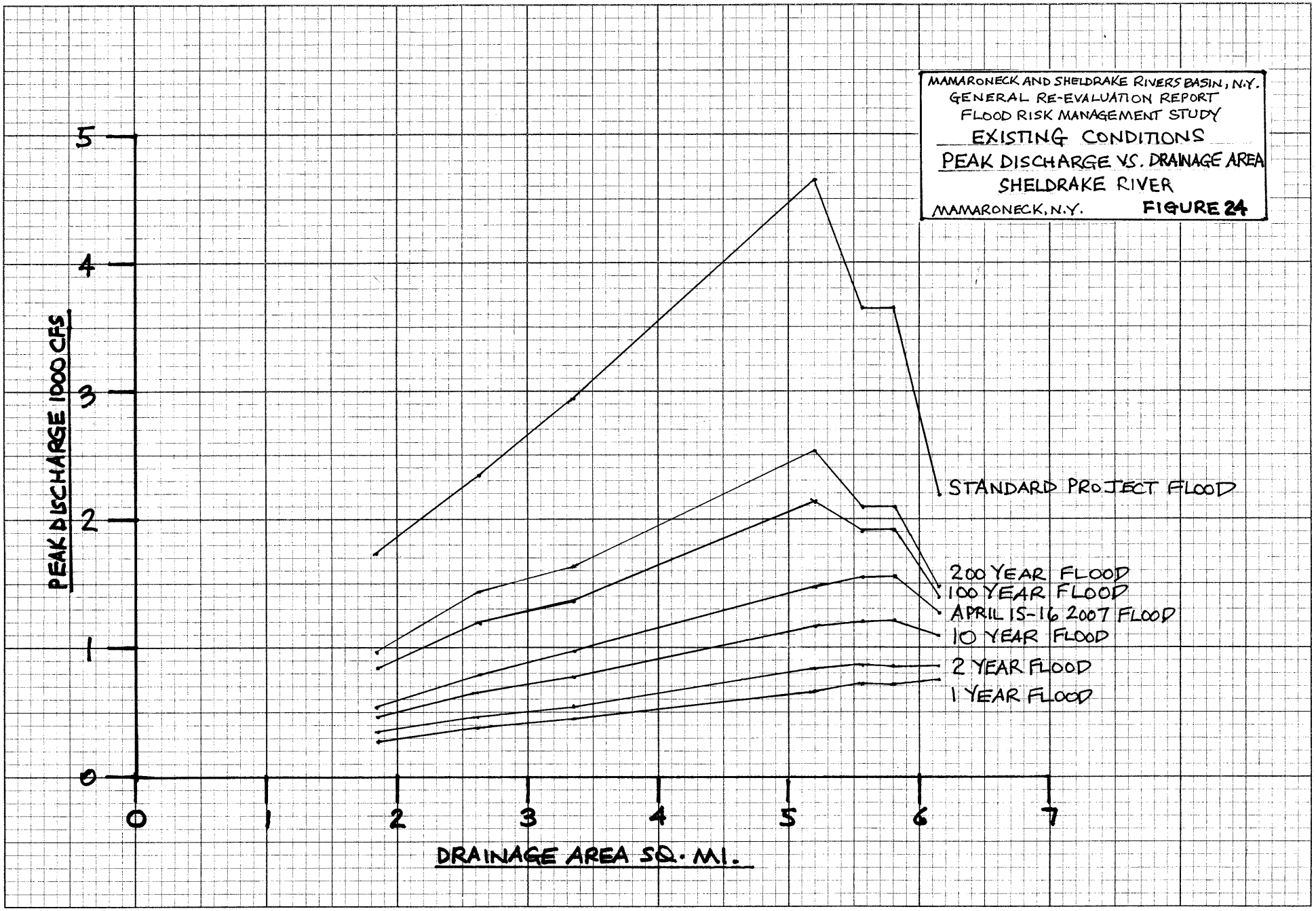
47 1970
K+E 12 X 20 TO THE INCH • 10 X 15 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.

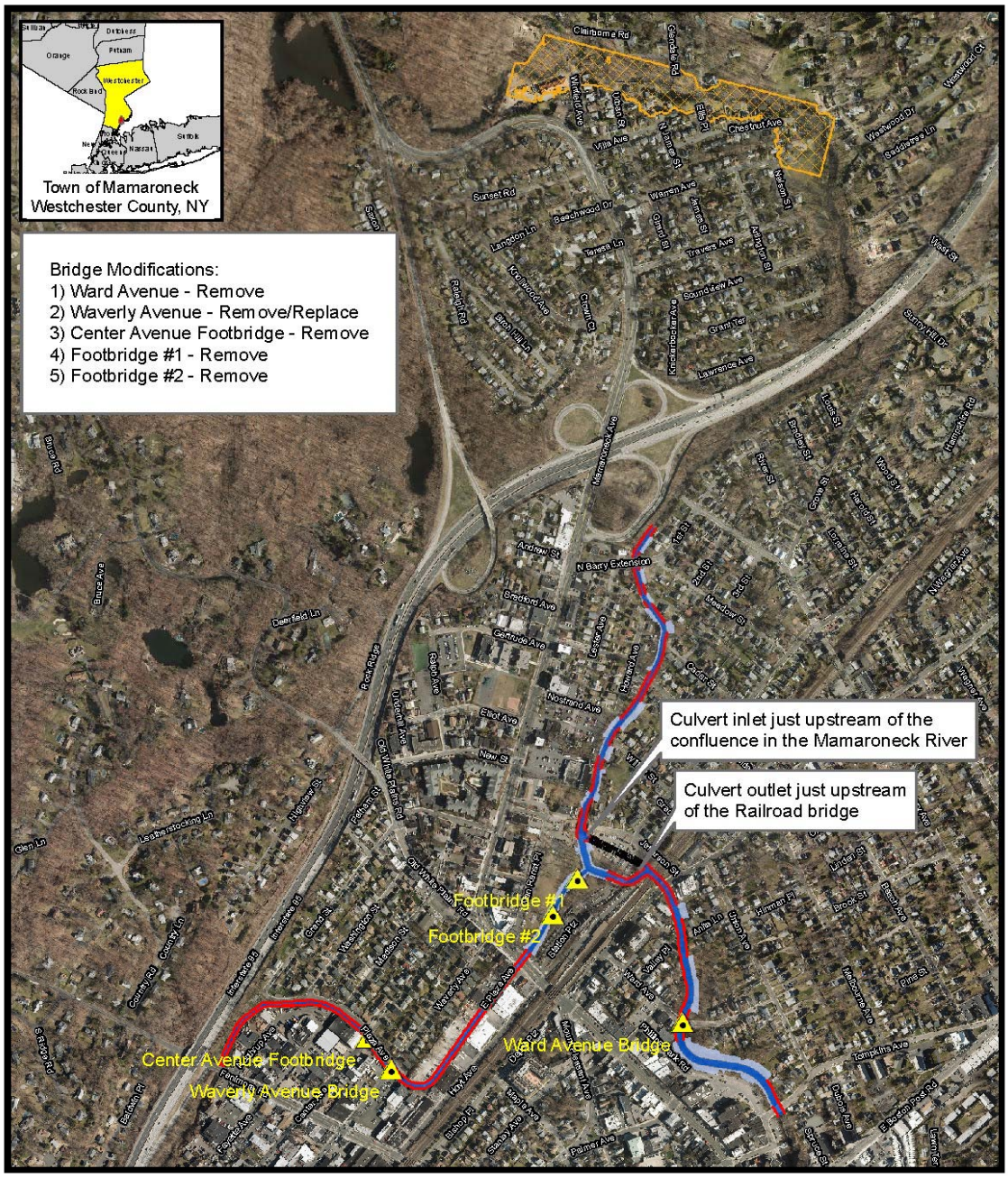
K+E 10 X 10 TO THE INCH • 7 X 10 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.



MAMARONECK AND SHELDRAKE RIVERS BASIN, N.Y.
GENERAL RE-EVALUATION REPORT
FLOOD RISK MANAGEMENT STUDY
EXISTING CONDITIONS
PEAK DISCHARGE VS. DRAINAGE AREA
MAMARONECK RIVER
MAMARONECK, N.Y. FIGURE 23

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





US Army Corps of Engineers
New York District

Mamaroneck and Sheldrake Rivers
Flood Risk Management Study
Alternative #1Z with
VE-Proposed Culvert in Confluence

0 250 500 1,000 1,500 2,000 Feet

N W E S

Legend
Proposed Non-Structural Solution for Select Structures
Bridge Modification
Retaining Walls
Channel Bottom
Culvert
Channel Slope

FIGURE 25: SELECTED PLAN FOR MAMARONECK & SHELDRAKE RIVERS)

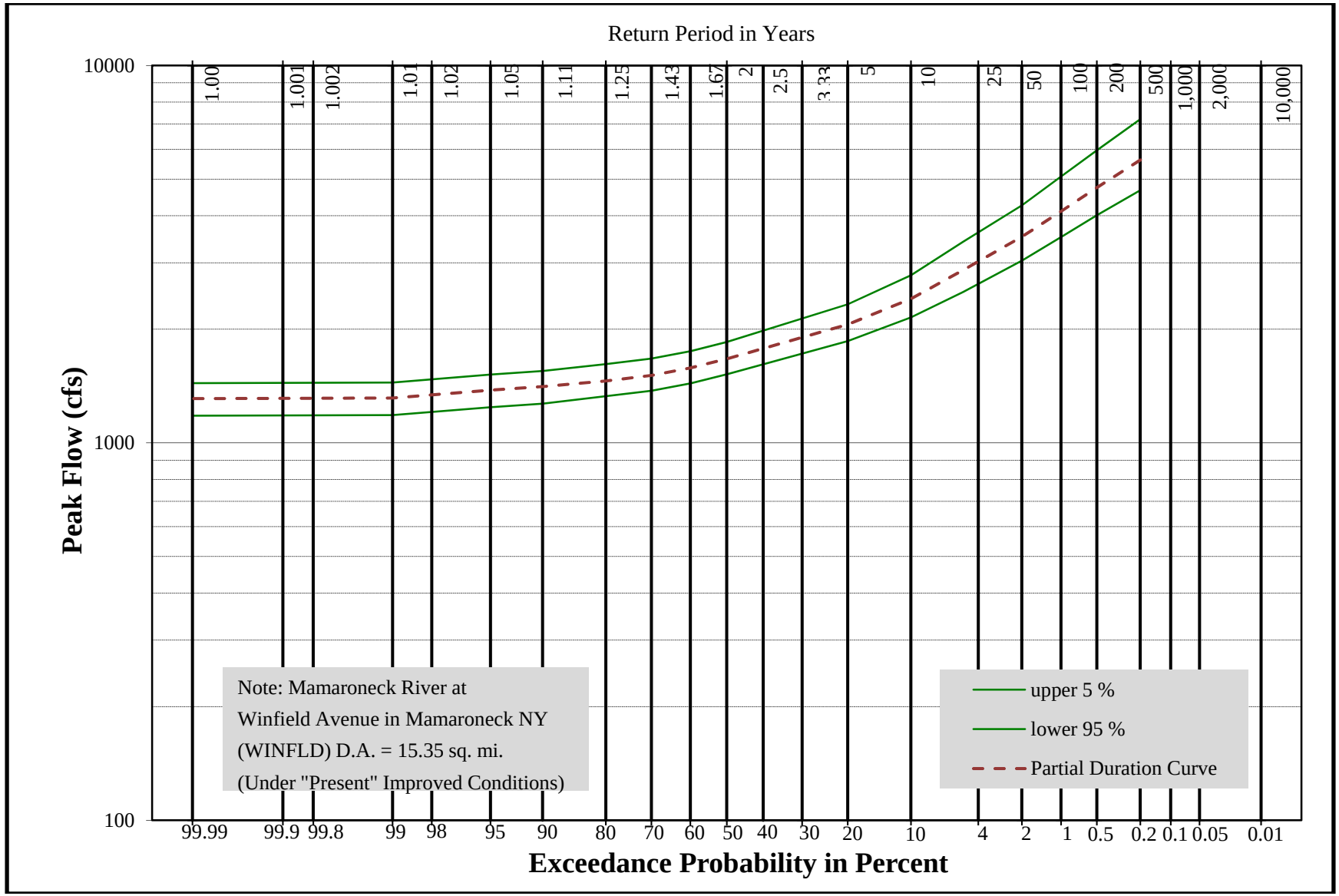


FIGURE 26: PEAK DISCHARGE VS. FREQUENCY CURVE FOR TENTATIVELY SELECTED PLAN AT WINFILED AVENUE

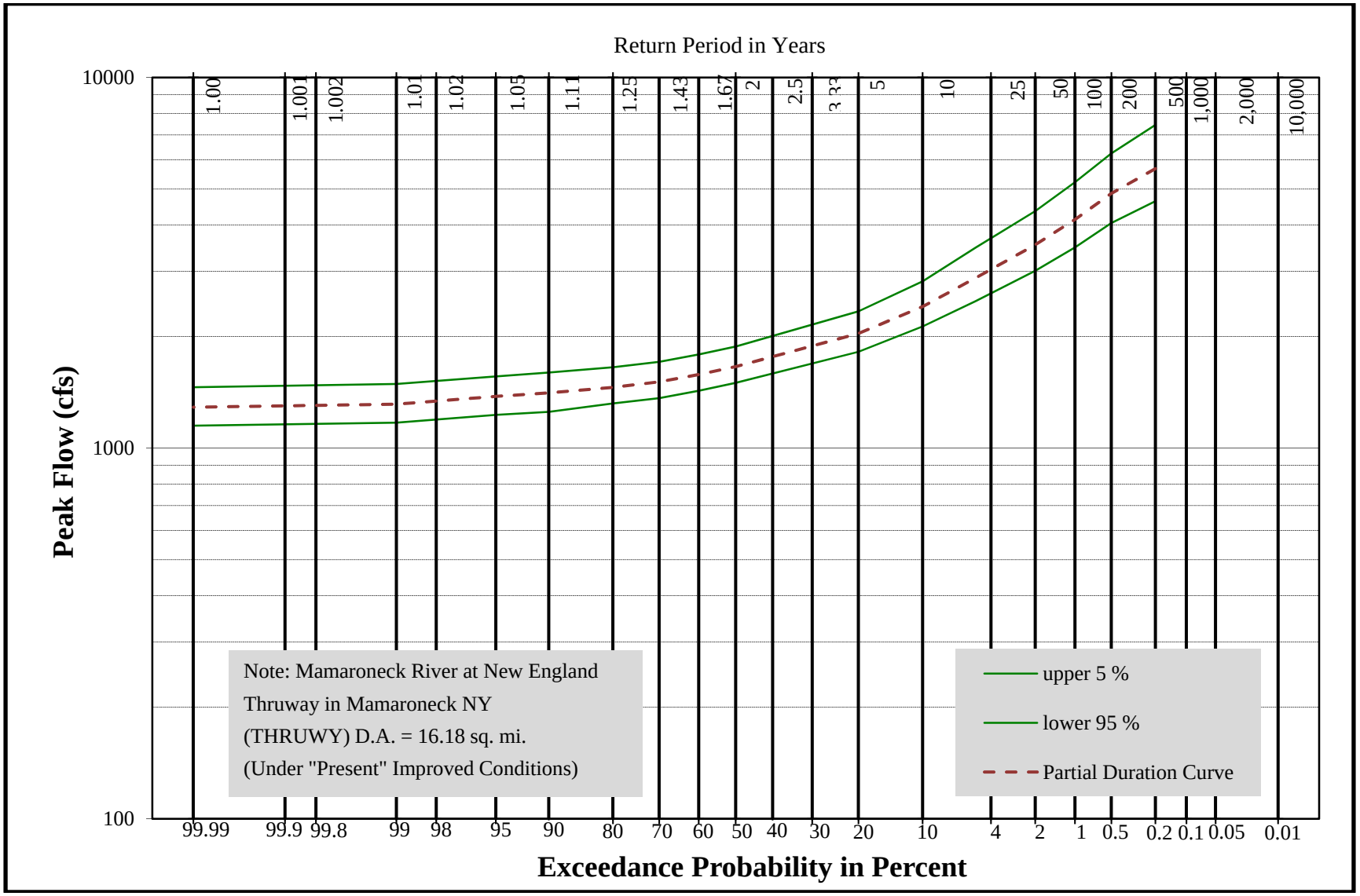


FIGURE 27: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT NY THRUWAY

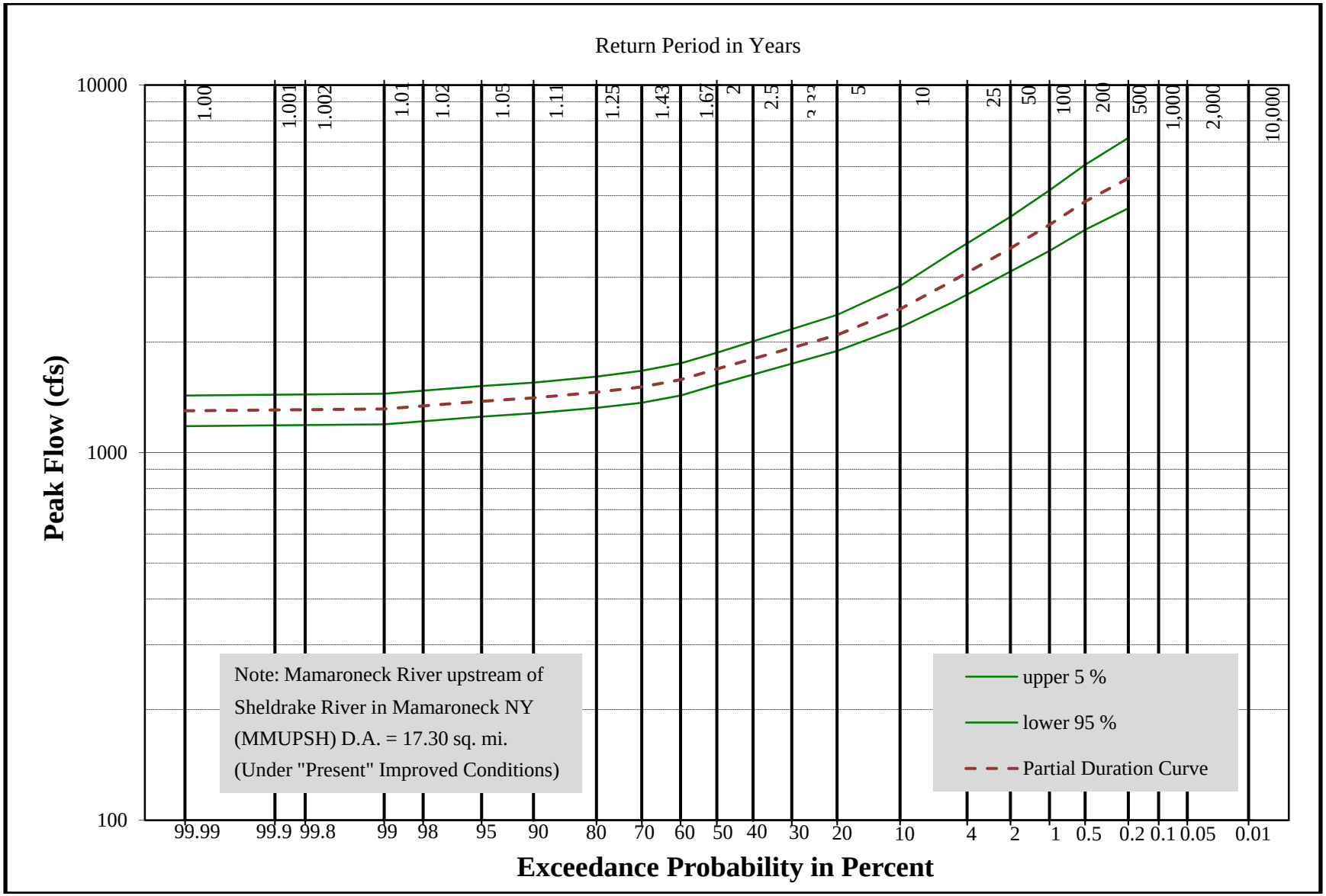


FIGURE 28: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MAMARONECK/SHELDRAKE CONFL.

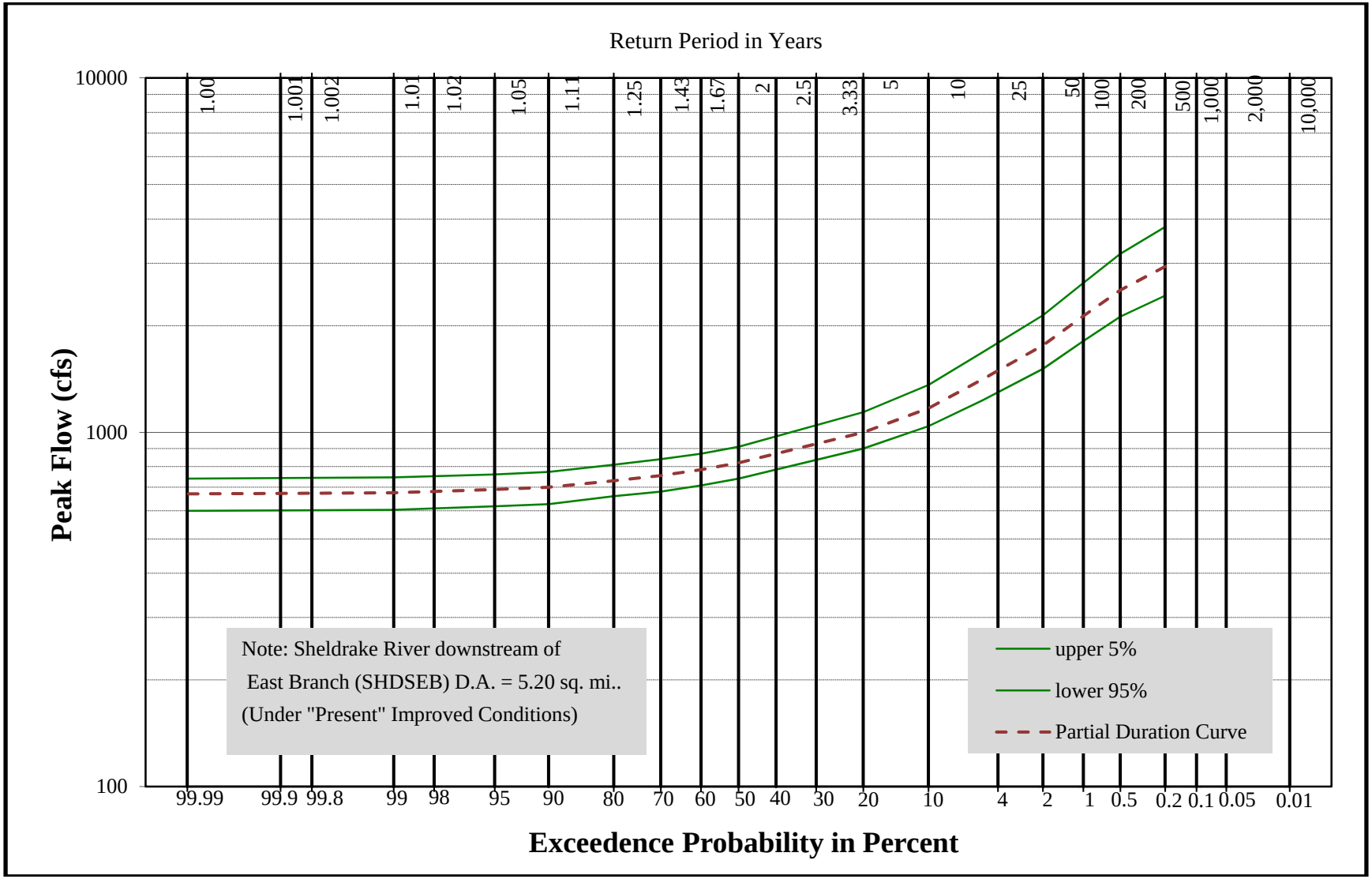


FIGURE 29: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT SHELDRAKE R. & EAST BRANCH

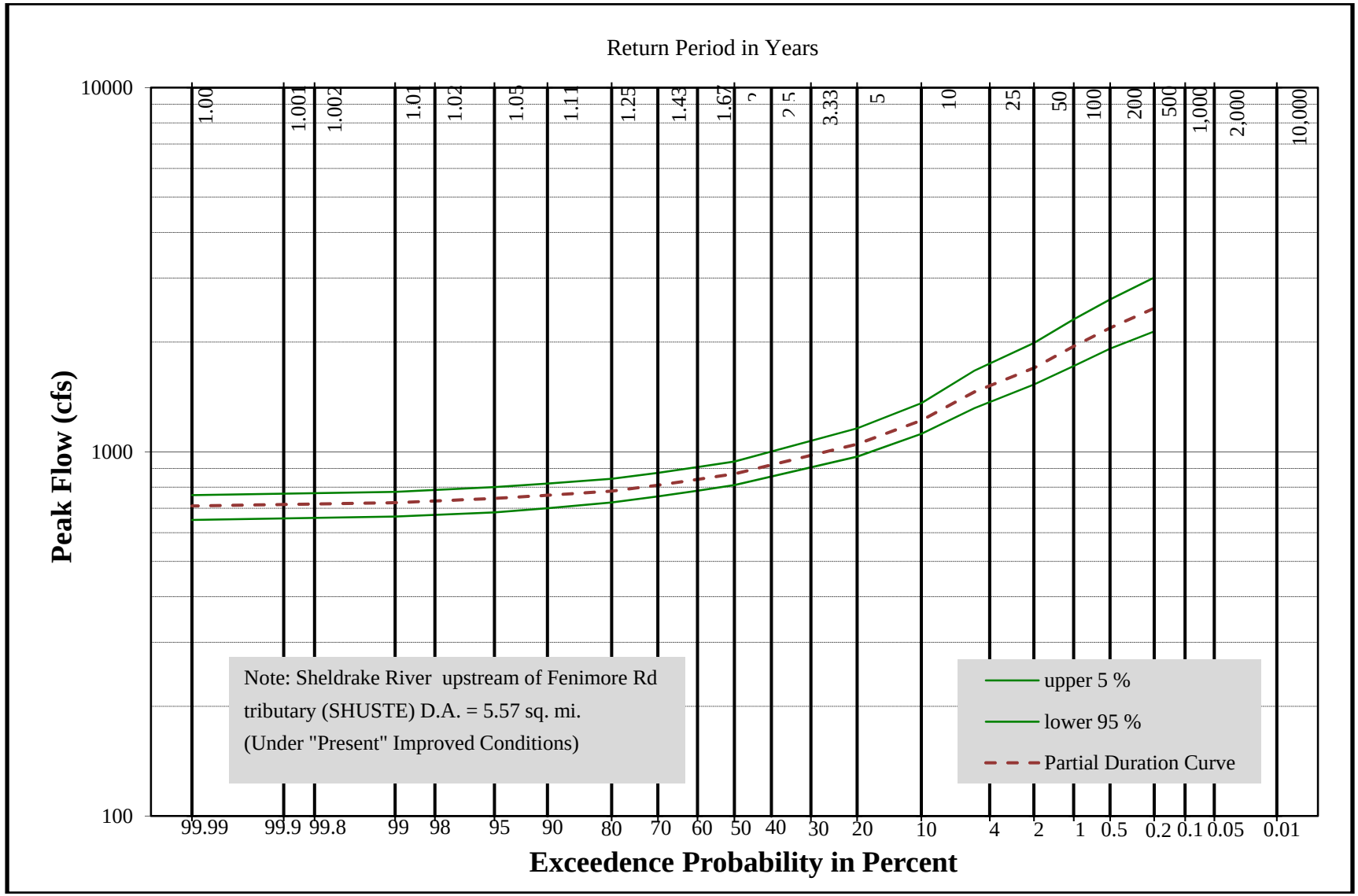


FIGURE 30: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT FENIMORE ROAD BRIDGE (UPST.)

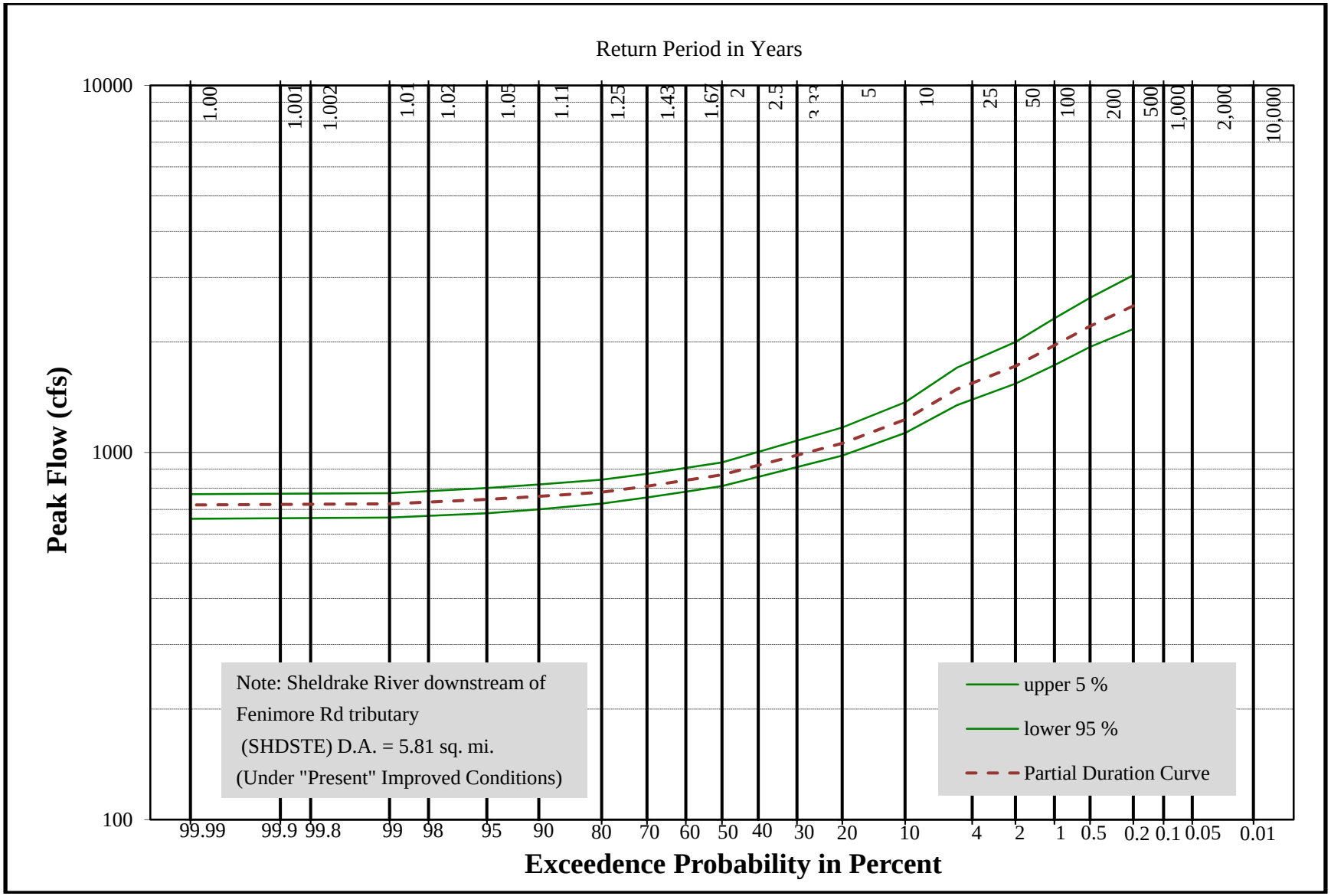


FIGURE 31: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT FENIMORE ROAD BRIDGE (DOWN.)

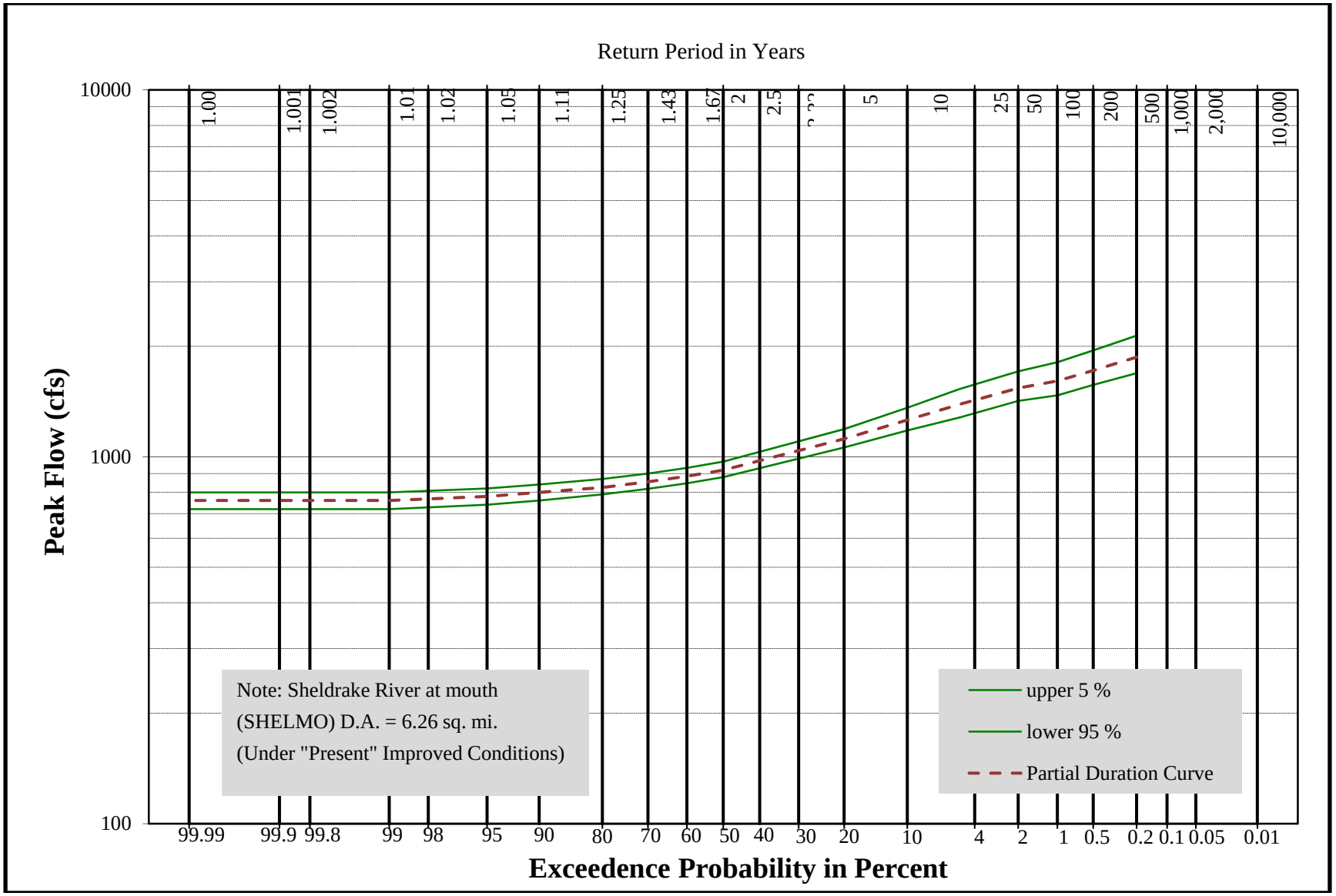


FIGURE 32: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MOUTH OF SHELDRAKE RIVER

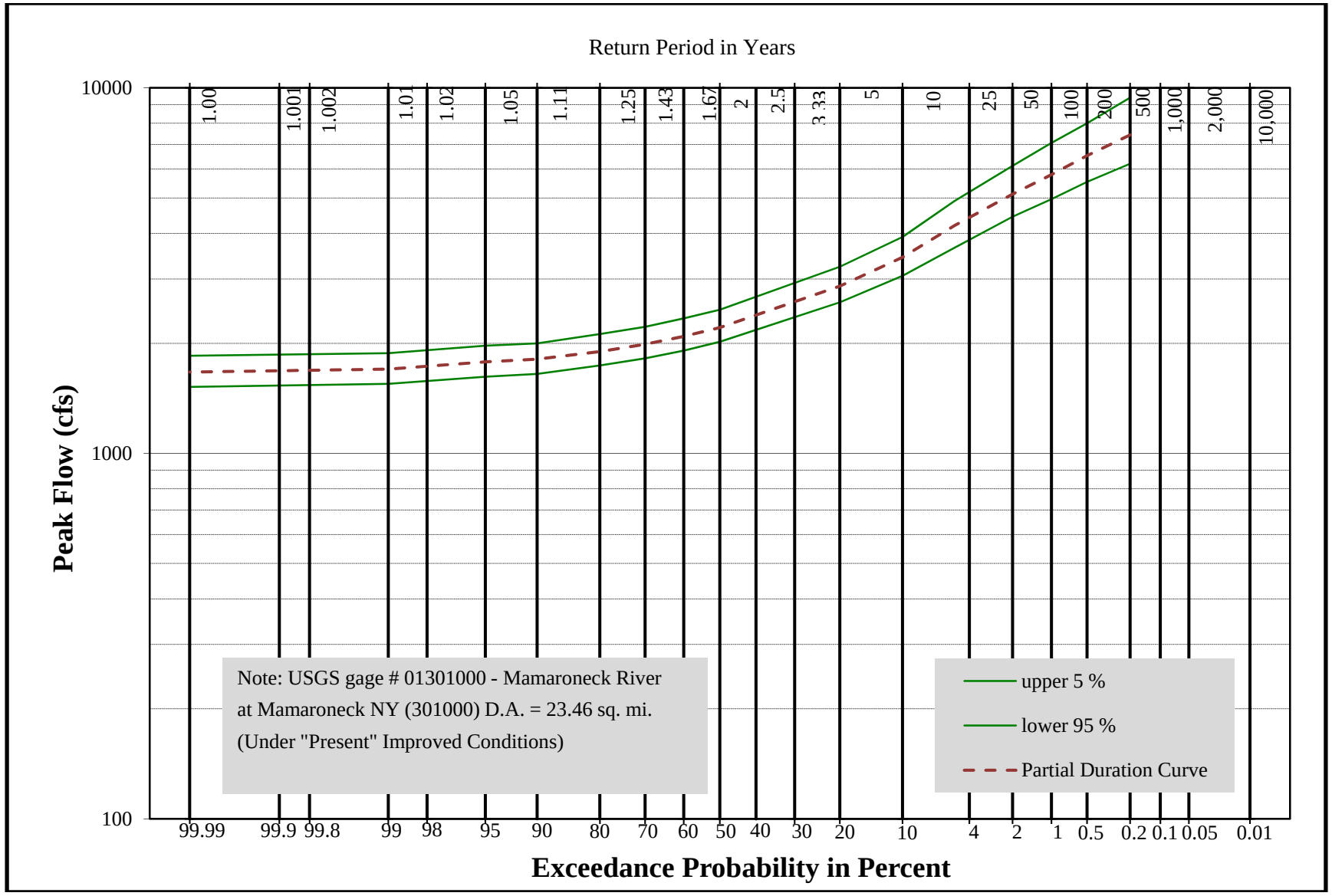


FIGURE 33: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT USGS GAGE 301000

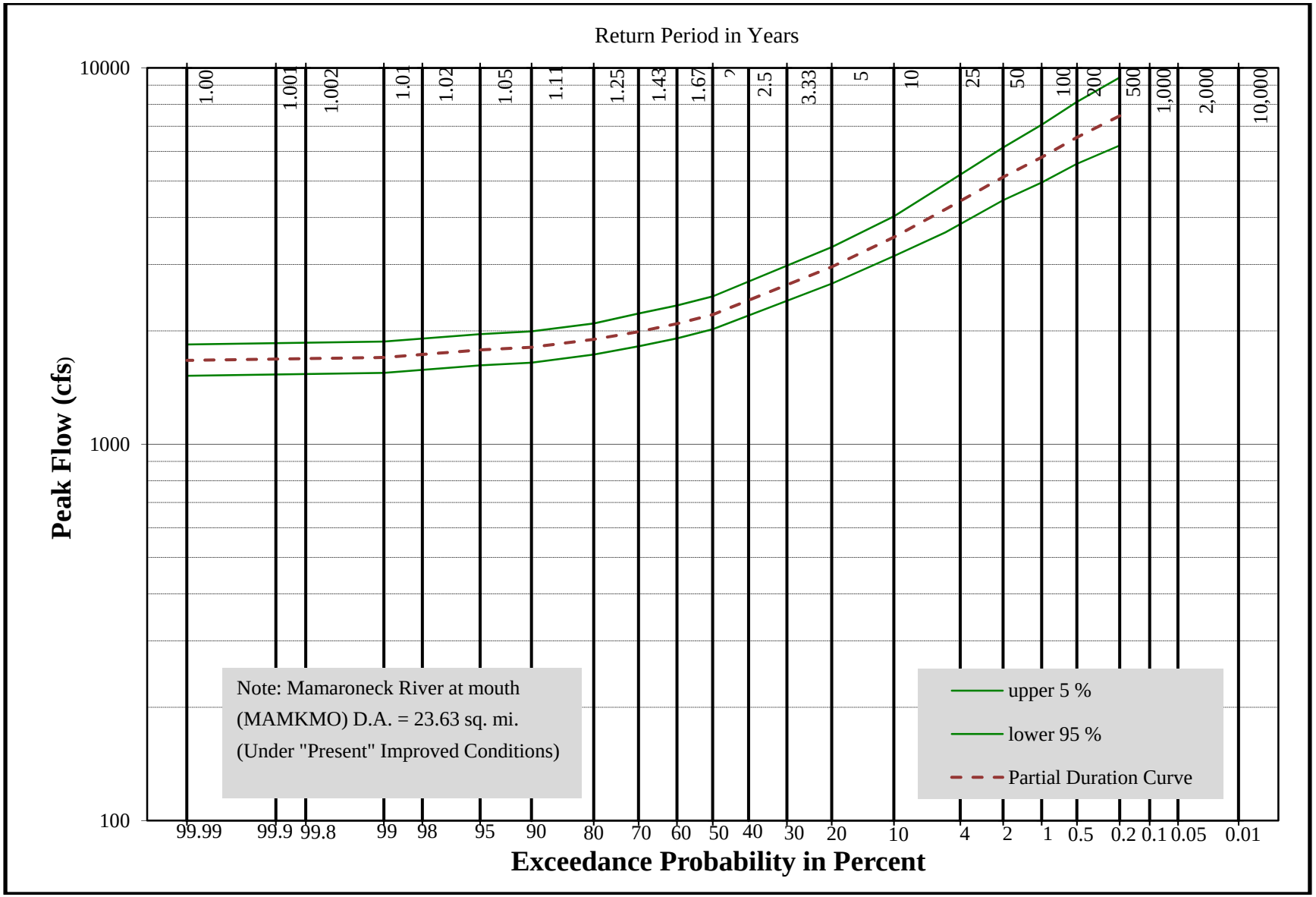


FIGURE 34: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MAMARONECK RIVER AT MOUTH

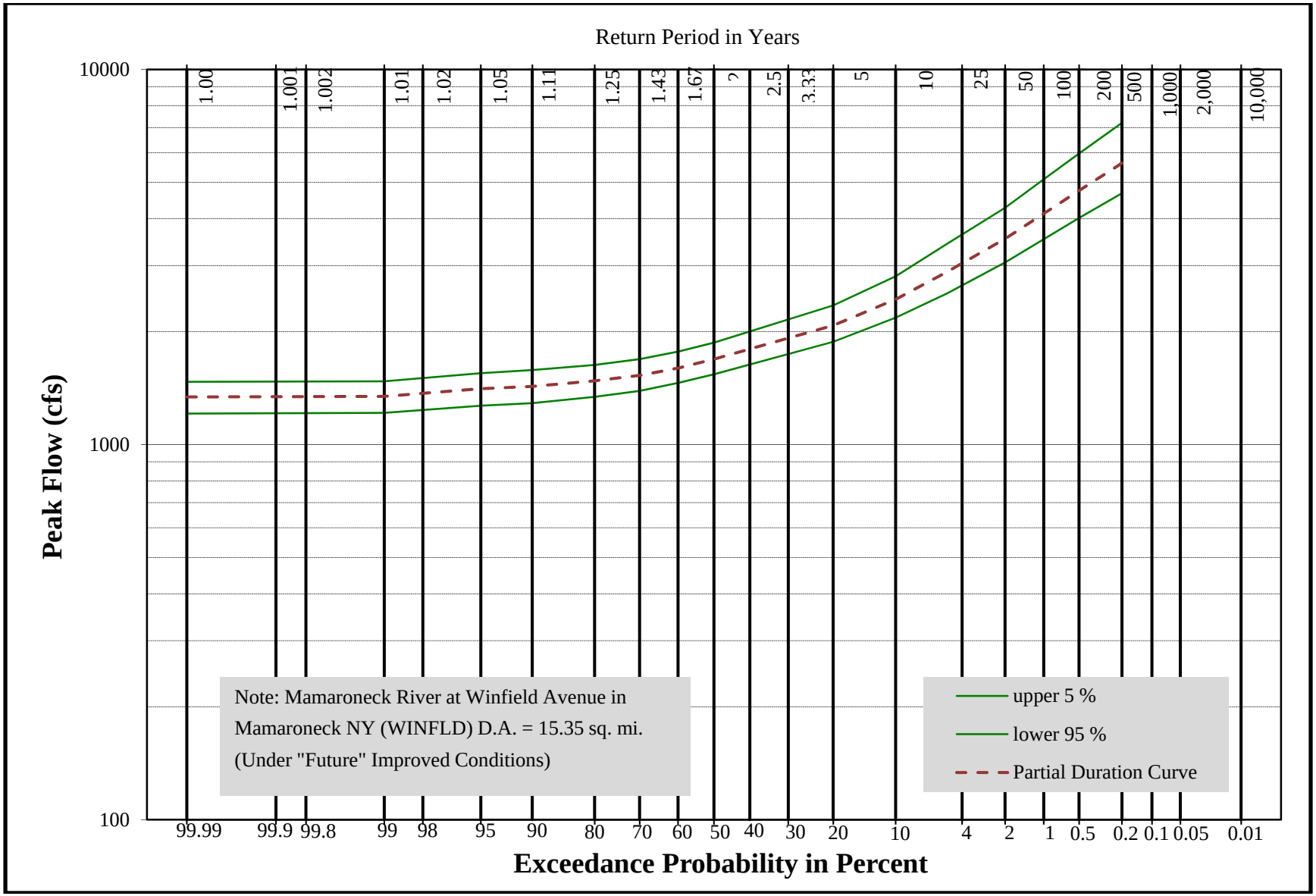


FIGURE 35: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT WINFILED AVENUE

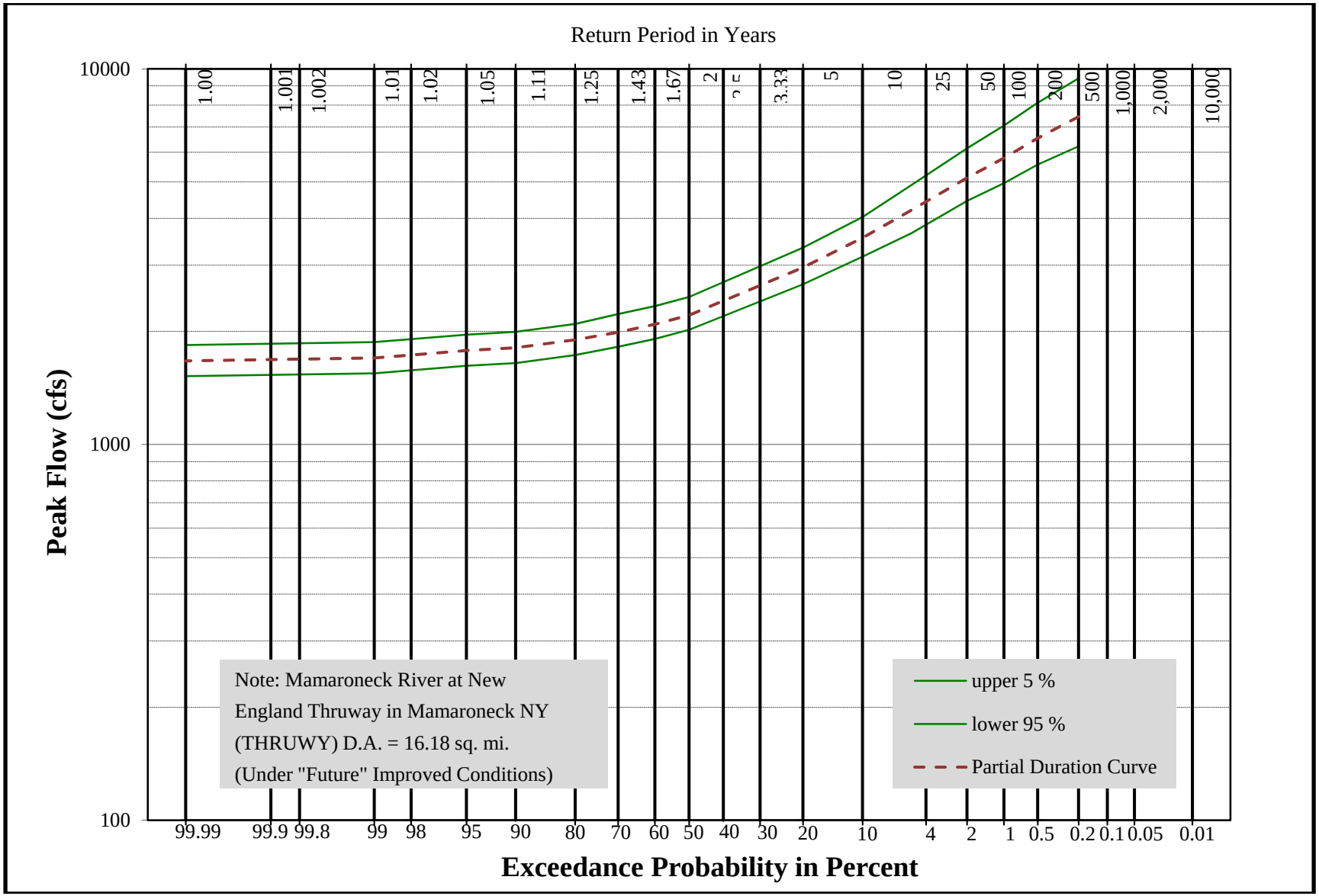


FIGURE 36: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT NY THRUWAY

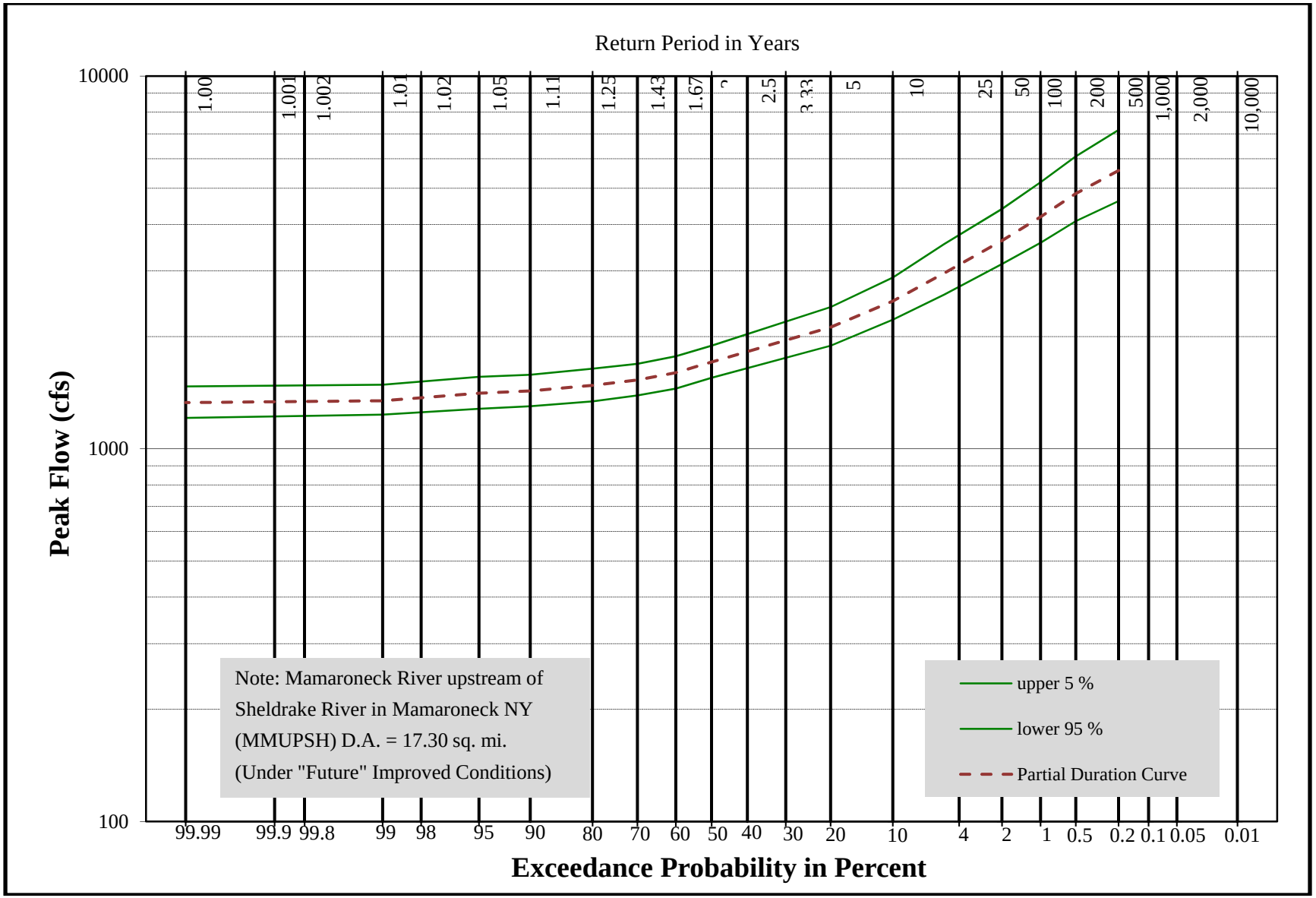


FIGURE 37: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MAMARONECK/SHELDRAKE CONFL.

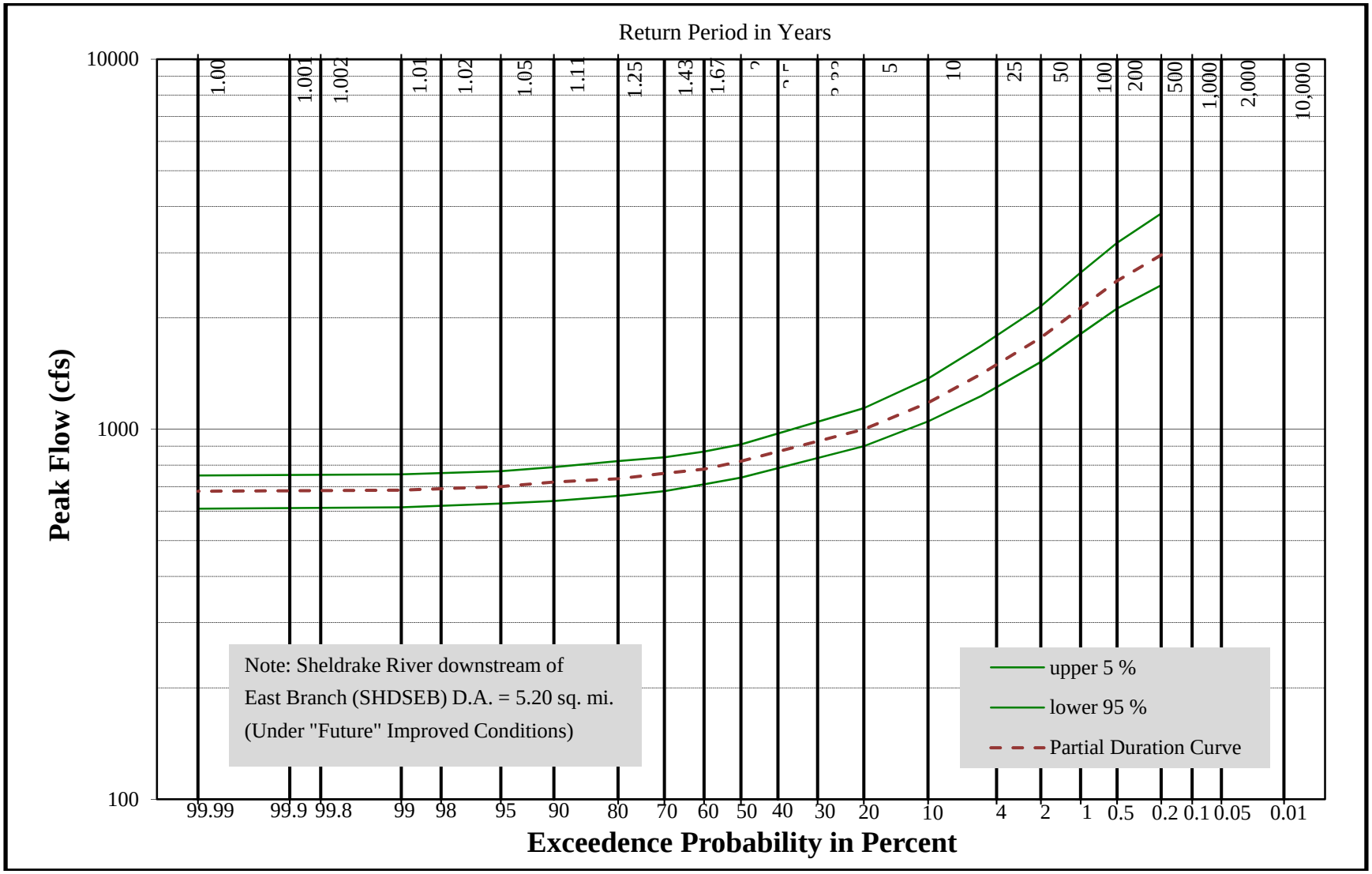


FIGURE 38: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT SHELDRAKE R. & EAST BRANCH

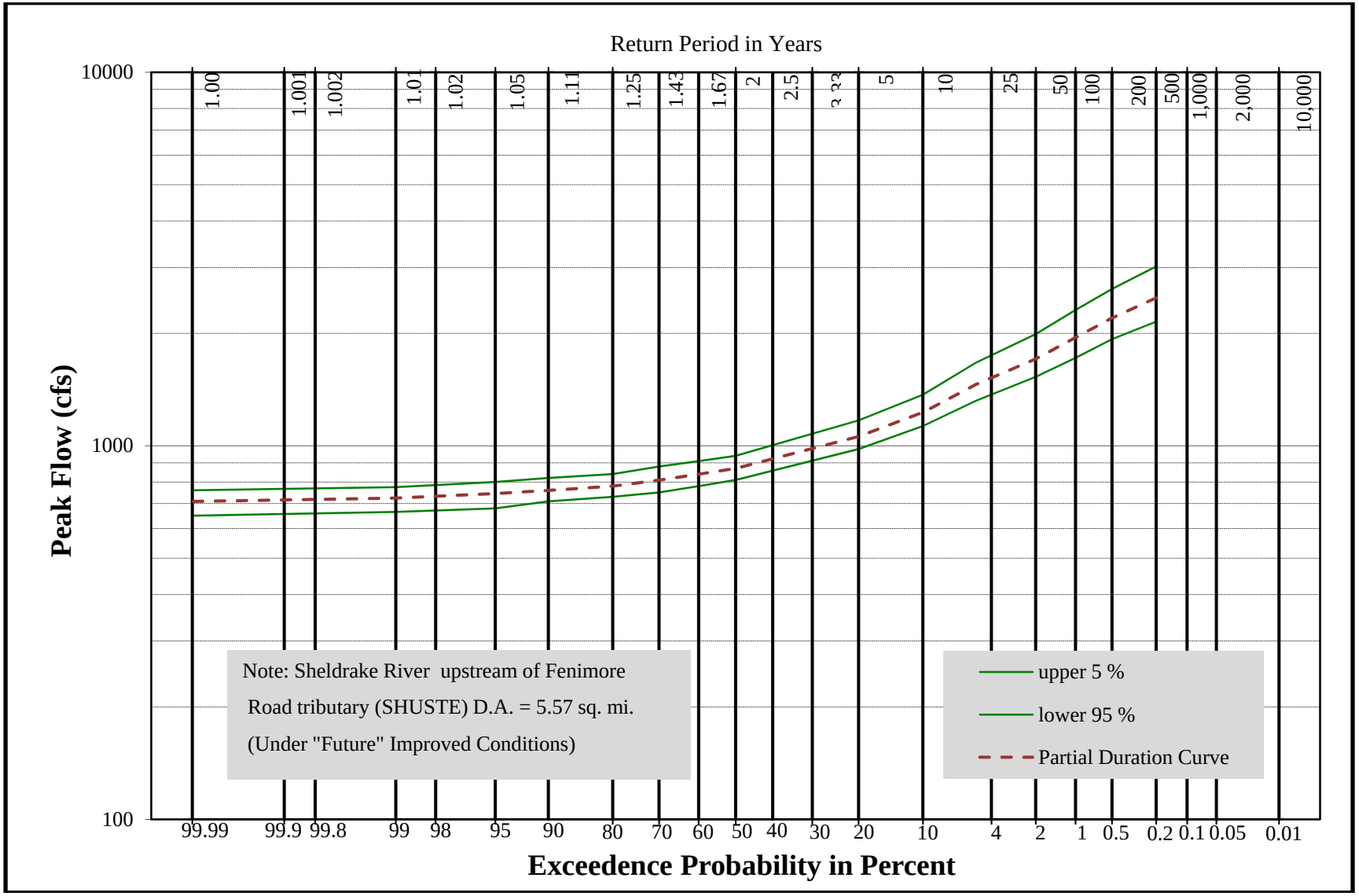


FIGURE 39: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT FENIMORE ROAD BRIDGE (UPST.)

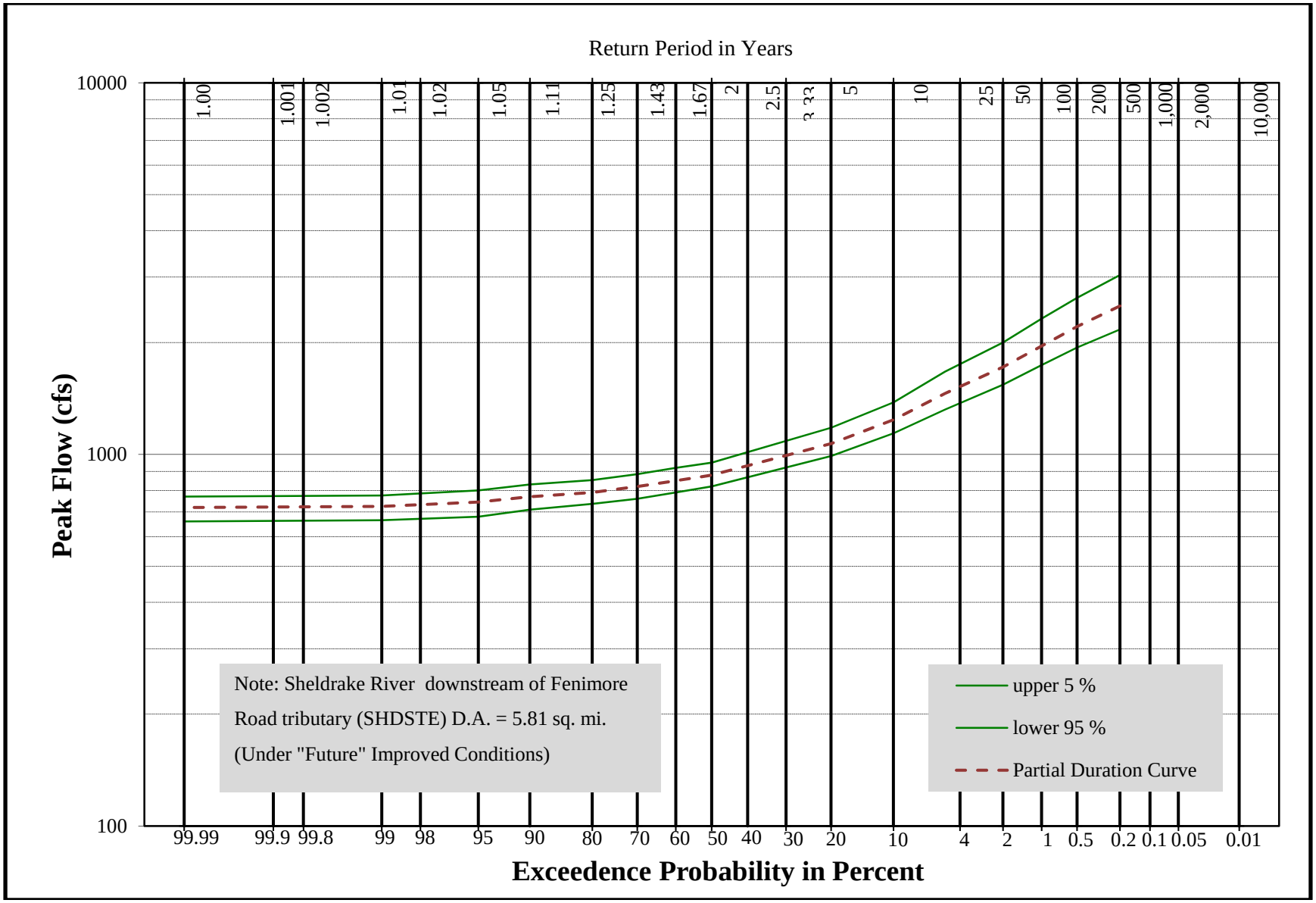


FIGURE 40: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT FENIMORE ROAD BRIDGE (DOWN.)

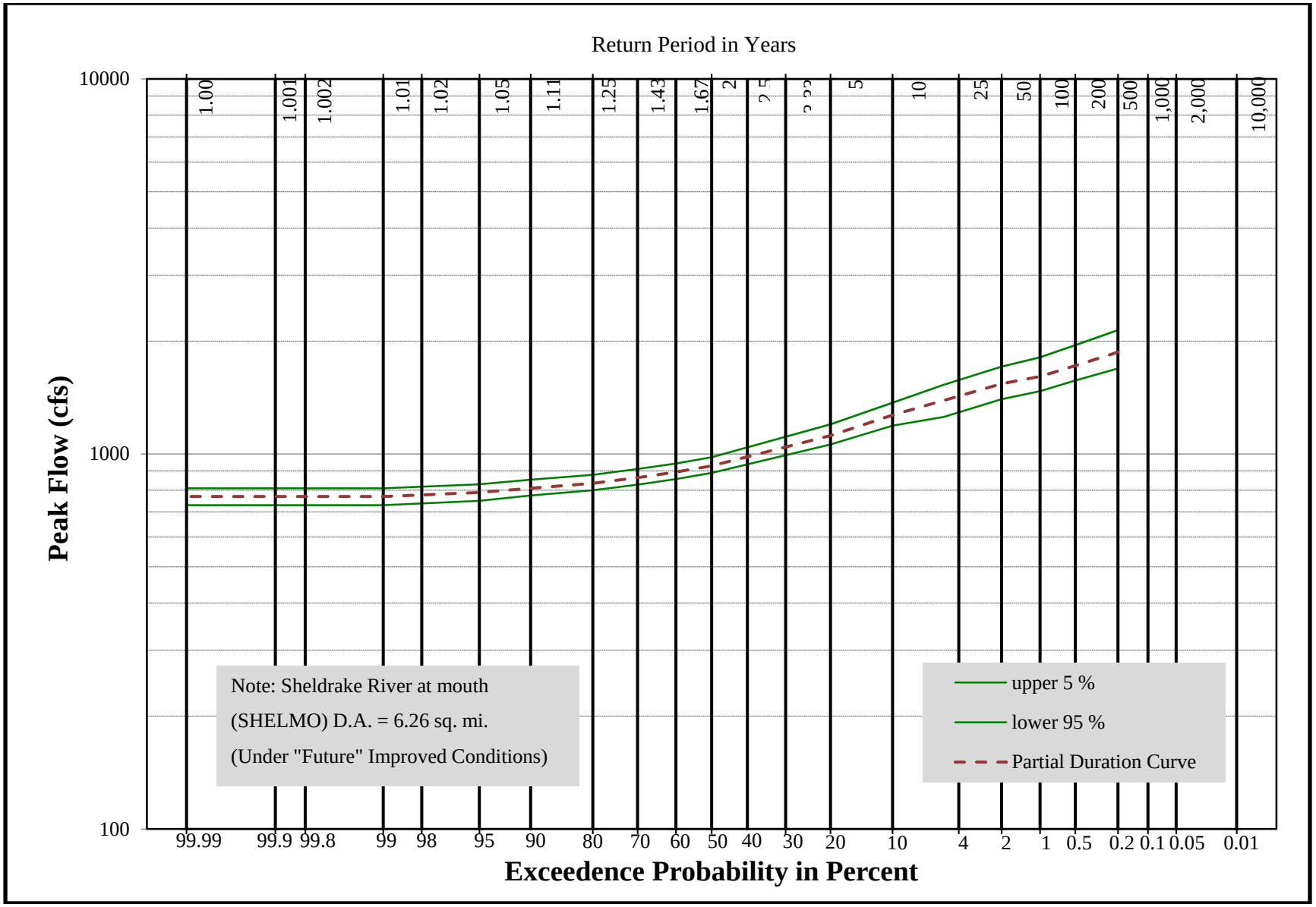


FIGURE 41: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MOUTH OF SHELDRAKE RIVER

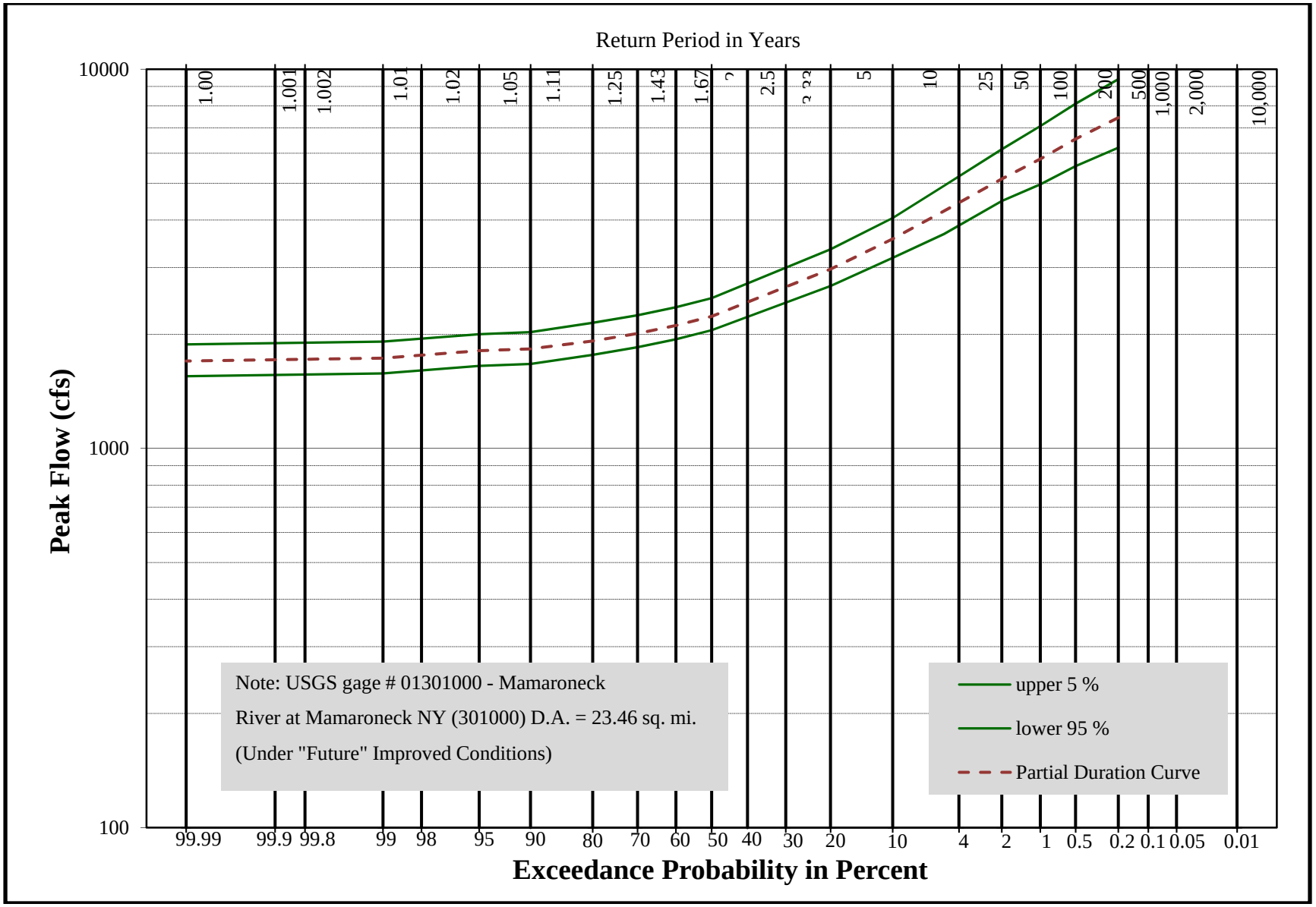


FIGURE 42: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT USGS GAGE 301000

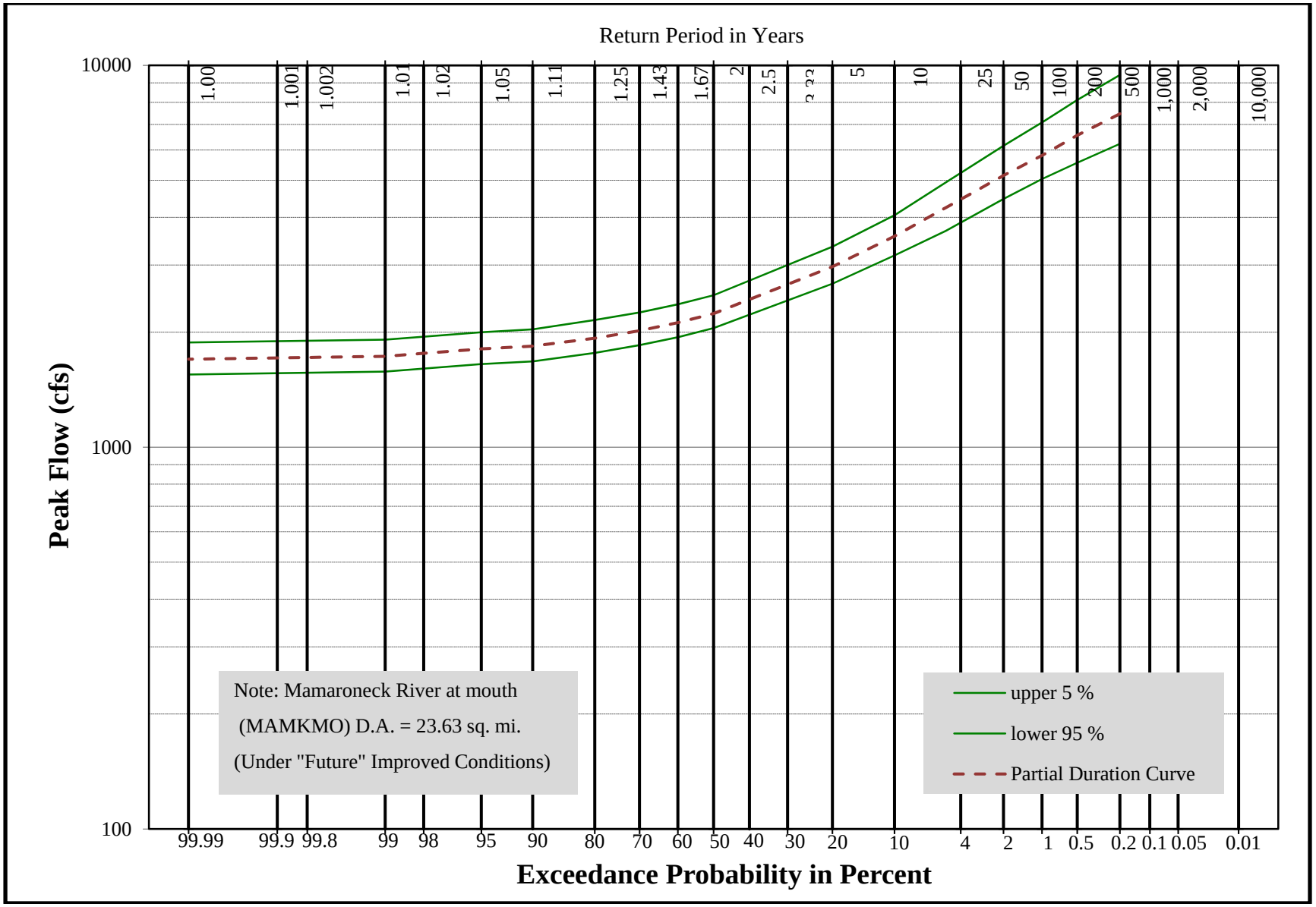


FIGURE 43: PEAK DISCHARGE VS. FREQUENCY CURVE FOR SELECTED PLAN AT MAMARONECK RIVER AT MOUTH