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

APPENDIX C2: HYDRAULICS

US Army Corps of Engineers®
New York District

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1.0 INTRODUCTION

1.1 *General*

1.1.1 Objective

This appendix documents the existing hydraulic analysis 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 Hydrologic and Hydraulic (H&H) analyzes documented in the 1989 GDM.

1.1.2 Description of Project Area

The Mamaroneck and Sheldrake Flood Damage Reduction Project area is primarily within the Village of Mamaroneck in the County of Westchester, New York. The hydraulic study extends a distance of 2.5 miles from the mouth of Mamaroneck River upstream to the Westchester County Joint Water Works Dam. The study also extends a distance of 1.0 mile along the Sheldrake River from its confluence with the Mamaroneck River upstream to the I-95 Bridge. A map of the study area is shown in Figure 1. The hydraulic model limits are defined by flood damage areas located in the Village of Mamaroneck and the Town of Harrison, NY. Along the Mamaroneck River, the model area extends from below the Rt. 1 Bridge to above the Westchester County Joint Water Works Dam. On the Sheldrake River, the model extends from the confluence with the Mamaroneck River to the Village boundary at the New England Thruway (I-95) Bridge.

Figure 1: Aerial Photograph showing the location of Mamaroneck and Sheldrake Rivers, Westchester County, NY

1.1.3 Existing Channel

The project reach on the Mamaroneck River is 2.5 miles in length and contains several major bends. The channel side slopes are moderate and vary from 5 to 15 ft in height. The channel bottom has a moderate slope, averaging 12 ft per mile, and varies in width from 20 ft at the upstream end to 55 ft at the mouth. The mouth of the Mamaroneck River is a short steep reach subject to some tidal inundation. The upstream reaches are subject to fluvial inundation only. The Sheldrake is a major tributary of the Mamaroneck River with its confluence about 3,500 ft upstream of the mouth of the Mamaroneck River, as shown in Figure 1.

The Sheldrake River is narrower than the Mamaroneck River with steeper side slopes. The Sheldrake River is not subject to tidal inundation because it discharges into the Mamaroneck River upstream of the tidal reach. The length of the Sheldrake River in the study area is about 1.0 mile.

1.1.4 Present Flooding Problems

Periodic storms have caused severe flooding along the Mamaroneck and Sheldrake Rivers. Floods are fluvial for Sheldrake River and most of Mamaroneck River. There is a possibility of tidal flooding along the downstream area of Mamaroneck River, outside the scope of this GRR. Flooding along the Mamaroneck River is caused by low channel capacity; two 90 degree bends forming an “S” turn at the Railroad Station just downstream from the confluence between the Mamaroneck and Sheldrake Rivers (Figure 2). Other constrictions at Tompkins, Valley Place and the Halstead Avenue Bridges are the cause for frequent inundations in the downstream reach of the Mamaroneck River and the confluence with Sheldrake River. Flooding along the Sheldrake River is due to a low channel capacity and the backwater effect from the Mamaroneck River.

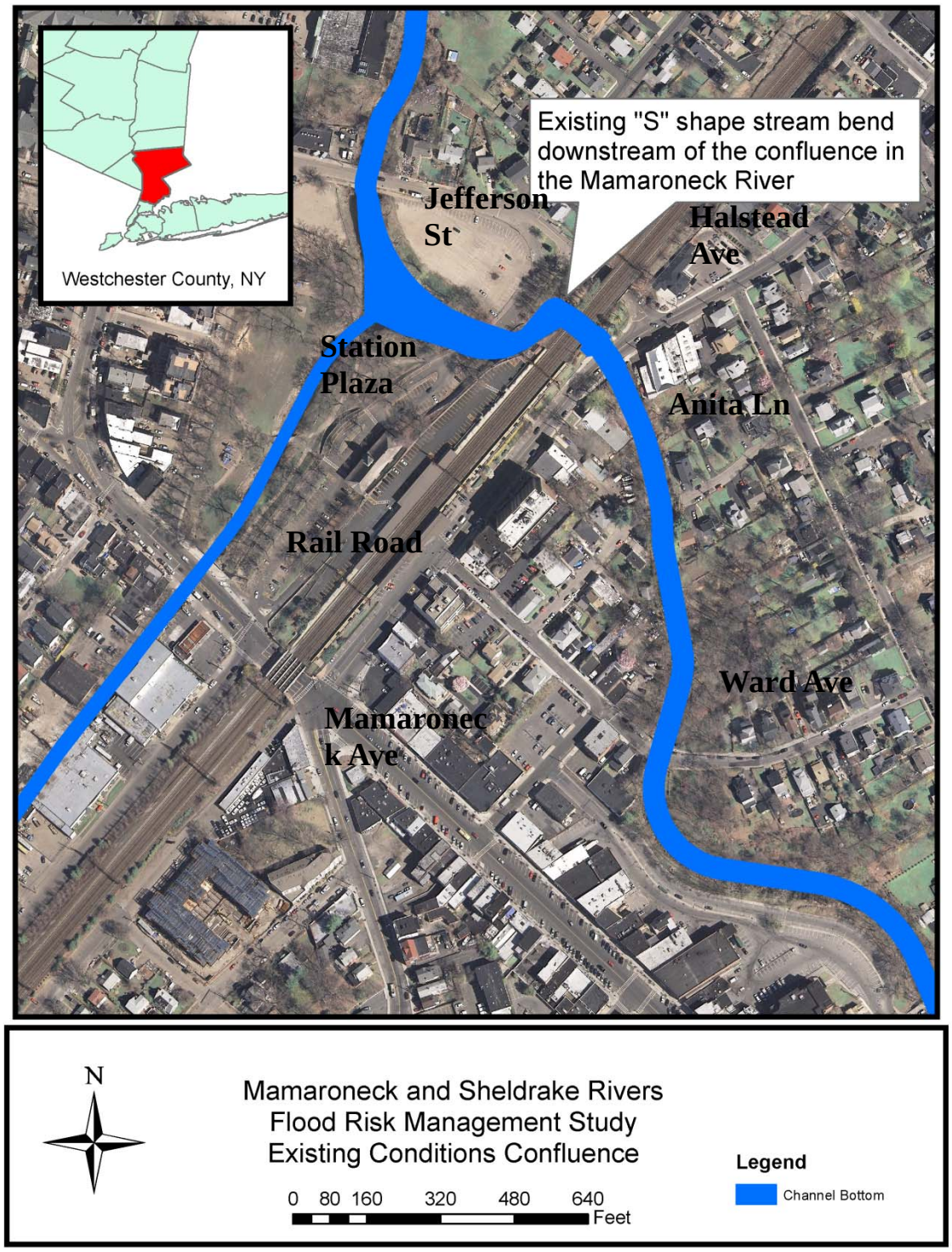

Figure 2: Mamaroneck River - two 90 degree bends forming "S" shape

1.1.5 Study Participants and Coordination

The non-Federal partner for this study is the New York State Department of Environmental Conservation (NYSDEC), with the Westchester County Department of Planning and Village of Mamaroneck serving as the lead stakeholders for the General Reevaluation Study.

2.0 HYDRAULIC ANALYSIS

2.1 General

The hydraulic analysis of the Mamaroneck and Sheldrake Rivers is based on a steady state numerical model using the Hydraulic Engineering Center, River Analysis System (HEC-RAS) software to determine water surface elevation (WSEL) of aforementioned rivers located in the Village of Mamaroneck in the County of Westchester, New York. The technical approach of the numerical modeling involved three phases. The first phase was the development of the actual hydraulic characteristics of the Mamaroneck and Sheldrake Rivers and floodplains based on real storm events and observed water surface elevations (WSEL). The second phase consisted of an evaluation of future land use changes over the project life and its effects on the hydrologic and hydraulic characteristics of the Mamaroneck and Sheldrake River Basins. These two phases facilitated the development of the inundation maps for different frequency events for the present and future without project conditions. The projected extent of flooding over the 50-year period of analysis was used to evaluate a set of improvements and determine a cost-effective level of protection for the Mamaroneck and Sheldrake Rivers. The third and final phase is the evaluation of new alternatives and reevaluation of alternatives contained in the 1989 GDM, including structural and non-structural alternatives. These alternative improvement plans will be combined and the selected plan will be optimized in order to obtain the most feasible and cost-effective set of improvements for the Mamaroneck and Sheldrake Rivers.

2.2 HEC-RAS Model Description

The hydraulic analysis of the Mamaroneck and Sheldrake Rivers is based on a steady state numeric model using the Hydraulic Engineering Center, River Analysis System (HEC-RAS). The existing conditions HEC-RAS mode*l of the study area was divided* into three hydraulic reaches: Mamaroneck Upstream, Mamaroneck Downstream, and Sheldrake, as shown in Figure 1**.** A total of 176 channel cross-sections, 21 bridges, and two dam structures were surveyed and used in the HEC-RAS hydraulic model. Table 1 and Table 2 contain the dam structures analyzed and coded into the hydraulic model of the Mamaroneck and Sheldrake Rivers. The surveyed channel cross sections, no more than 300 ft apart, were supplemented with data of a 2 ft contour topographic map from 2004, which was used to create floodplain cross sections. ArcGIS and HEC tools were used to geo-reference the cross sections in the geometry files of the hydraulic model. Visual observations, Arc-GIS, and aerial photographs were used to characterize the initial cross sections Manning's n-values. The surfaces varied from open spaces and parking lots to areas with high density vegetation or structures. Initial n-values were set at 0.03 for the channel and overbank n-values were estimated to range between 0.03 and 1000. Ineffective flow areas were identified in the overbanks at bridges and bends to better represent the effects of structures and topography on flow conveyance. Contraction and expansion coefficients for the open channel sections were initially set at 0.1 and 0.3, while those for bridge sections were set at 0.3 and 0.5.

Table 1: Mamaroneck and Sheldrake bridge characteristics used in the HEC-RAS model.

Bridge	Reach	Model Station	High Cord* (ft)	Low Cord* (ft)	Width* (ft)	Type of Opening	Existing Capacity (yr)	Improved Capacity (yr)
Mamaroneck Ave.	Upstream	9151	49	45.8	85	Single	2	2
Winfield Ave.	Upstream	7520	37.5	32.8	30	Single	2	1
I-95	Upstream	4322	27	41	115	Triple	10	10
Barry Ave.	Upstream	2136	32.5	26.7	45	Triple	50	500
Hillside Ave.	Upstream	1342	23.7	30	45	Arch	25	100
Jefferson Ave.	Upstream	309	23.5	20	50	Single	5	25
Station Plaza	Downstream	3446	27	24	55	Double	50	500
Railroad Station	Downstream	3200	40	29	55	Arch-Double	500	>500
Halstead Ave.	Downstream	3116	31.3	22.2	67.5	Single	50	500
Valley Place	Downstream	2729	26.9	17.9	15	Arch Double	5	100
Ward Ave.	Downstream	22.11	29.4	23	55	Arch	500	Removed
Tompkins Ave.	Downstream	1282	18.5	15	50	Single-Single	100	100
East Boston Rd.	Downstream	505	28	12.1	73	Arch-Single	> 500	>500
I-95	Sheldrake	6357	50	29.5	150	Double	200	500
Rockland Ave.	Sheldrake	5324	41	26.7	35	Double	25	50
Fenimore Rd.	Sheldrake	3510	30.5	24.9	42	Double	10	150
Centre Ave. Pedestrian Bridge	Sheldrake	2259	21.5	20.2	8	Single	<1	Removed
Waverly Ave.	Sheldrake	2023	22.5	19.5	28	Single	<1	10
Mamaroneck Ave.	Sheldrake	784	28.5	21.3	100	Arch-Single	10	100
Pedestrian Bridge	Sheldrake	388	20.2	18.7	14	Single	2	Removed
Pedestrian Bridge	Sheldrake	105	20.3	18.2	11	Single	2	Removed

*Maximum and minimum observed elevation, considering parapets and other obstructions. Based on Stream Cross Section & Utility Survey by Rogers Surveying, PLLC.

Table 2: Dam structures in the Mamaroneck and Sheldrake River Basins

Dam	Reach	Model Station	*Weir Elevation (ft)	Gates	Opening Area (ft^2)
Water Works	Upstream	8232	42.3	2	49
Larchmont Gardens Lake	Sheldrake	6974	28.0	0	0

*Based on Stream Cross Section & Utility Survey by Rogers Surveying, PLLC.

2.3 HEC-RAS Modeling Approach

The technical approach of the numerical modeling involves four phases; existing conditions, future unimproved conditions, improved conditions and future improved conditions. The first phase was the development of the actual hydraulic characteristics of the Mamaroneck and Sheldrake Rivers and floodplains, calibrated with a real storm event and observed water surface elevations (WSELs). The nor'easter storm of April 15, 2007, roughly a 100-year event in this area, was used to calibrate the steady state HEC-RAS model. The calibrated HEC-RAS model of the Mamaroneck and Sheldrake Rivers was used to determine the WSEL for the 1, 2, 5, 10, 25, 50, 100, 250, and 500-year frequency events. Since the Mamaroneck River flows into Long Island Sound a Tidal-Fluvial correlation was performed to analyze the harbor elevations that may occur due to tide and surge during a typical fluvial event. Based on the Tidal-Fluvial correlation and hydraulic analysis, a weak correlation between the low frequency tidal and fluvial events was found. Also, the influence of high tides during fluvial events on the Mamaroneck River does not extend beyond Tompkins Avenue Bridge. The area just downstream of the confluence between the Mamaroneck and the Sheldrake River, which includes the Station Avenue Bridge, Metro North Railroad Bridge and the Halsted Avenue Bridge, is causing considerable losses and high water surface elevations. The small flow capacity of the channel bends through the bridges and the small size of the Halstead Avenue Bridge are key reasons for the frequent flooding in the Village of Mamaroneck. The second phase

includes an evaluation of future conditions considering the effect of land use changes over the period of analysis on the hydrologic and hydraulic characteristics of the Mamaroneck and Sheldrake River Basins. The potential sea level rise (SLR) over the 50 year period of analysis of the project was also taken into consideration. Future unimproved WSEL for the 1, 2, 5, 10, 25, 50, 100, 250, and 500-year frequency events were computed in the existing conditions hydraulic model. Because there was only a small increase in the flows between existing conditions and the future unimproved conditions, there is no increase in damages expected for the future unimproved conditions due to the change in flows. Since this project will not provide any protection from tidal or coastal surge type of events and because fluvial flooding in the existing channel is not impacted by sea level change, there will be no change in the damage computations due to Sea Level Rise. Finally the third and fourth phases, consist of an evaluation of a set of improvements in which both the actual and expected reduction of flooding over the 50-year period of analysis of the project will be used to determine a cost-effective level of protection for the Mamaroneck and Sheldrake Rivers. The evaluation and reevaluation of new and previous alternatives contained in the 1989 GDM, including structural and non-structural alternatives were studied. Three channel plans were considered: channel work downstream of the confluence between the Mamaroneck and Sheldrake Rivers, channel work downstream and upstream of the confluence between the Mamaroneck and Sheldrake Rivers along the Mamaroneck River, and channel work throughout the Mamaroneck and Sheldrake Rivers. Also two river diversions plans were analyzed, consisting of a tunnel system running beneath Mamaroneck Avenue. from the Sheldrake River just upstream the confluence to the East Basin of Mamaroneck Harbor and a tunnel system running beneath Fenimore Road from the Sheldrake River to the West Basin of Mamaroneck Harbor. These alternative improvement plans were combined and the level of protection provided by the selected plan was optimized in order to obtain the most feasible and effective set of improvements for the Mamaroneck and Sheldrake Rivers. Additional surveys, studies and analysis were conducted for the final optimized plan. In addition risk and uncertainty was included in each phase. The uncertainty in stage related to the computed WSEL was evaluated by conducting a sensitivity analysis. The hydraulic characteristics that will be considered in developing the upper and lower bounds are the

Manning's "n" values, debris jams at bridges, channel bottom changes and changes in the Mamaroneck Harbor sea level which is the downstream boundary condition. The goal or overall approach is to develop realistic upper and lower bounds on the computed stage for a given discharge that will ultimately be used to calculate the reliability of the hydraulic analysis and the feasibility of the project.

2.4 Model Calibration

In order to create an accurate model that represents the Mamaroneck and Sheldrake Rivers and their hydraulic characteristics, the HEC-RAS model was calibrated to a recent storm event in the basin. The nor'easter storm of April 15, 2007, roughly a 100-year event in this area, was used to calibrate the steady state HEC-RAS model. The hydrologic analysis of the Mamaroneck and Sheldrake River Basins provided the flows for the storm event, which are shown in Table 3. These flows were entered into the HEC-RAS hydraulic river model.

Table 3: April 15, 2007 historical flows per hydrologic node and HEC-RAS model stationing.

Reach	HEC-RAS Station	Node ID	15-Apr-07 (cfs)
Upstream	11018.74	WINFLD	4130
Upstream	6930.00	THRUWY	4190
Upstream	2780.19	MMUPSH	4060
Downstream	3524.50	30100	5340
Downstream	1921.84	MAMKMO	5340
Sheldrake	6991.73	SHDSEB	1480
Sheldrake	6357.23	SHUSTE	1530
Sheldrake	3468.01	SHDSTE	1550
Sheldrake	1755.18	SHELMO	1280

Field surveys during the initial phase of this study in the Village of Mamaroneck provided a total of 20 flood marks; 14 in the Mamaroneck River and 6 in the Sheldrake

River. Adjustments to the n-values, contraction and expansion coefficients, weir coefficients, ineffective flow areas, and other loss coefficients were made to compute and reproduce the water surface elevation (WSEL) within 0.5 ft of the observed flood marks for the April 15, 2007 storm. A determination as to which flood marks may be indications of an energy level or an actual WSEL was taken into account at the time of calibration in order to better represent the storm event.

Table 4 shows the flood mark elevations (NAVD88) for the April 2007 nor'easter, as well as the location, and computed WSEL for Mamaroneck and Sheldrake Rivers. Figure 3 and Figure 4 are the calibrated HEC-RAS WSEL profile for April 2007.

Table 4: April 15, 2007 peak observed WSEL and HEC-RAS calibration.

Reach	HEC-STA	FM Elevation (ft, NAVD88)	HEC Calibration WSEL (ft, NAVD88)	Difference (ft)	Location
Upstream	7520.1	39.8	39.7	0.2	490 Winfield Ave.
*Upstream	6665.8	34.8	34.9	0.0	1612 James Street
*Upstream	6665.8	34.9	34.9	0.1	1616 James Street
Upstream	6115.4	33.9	33.7	0.2	511 Chestnut Ave.
Upstream	5711.7	33.0	32.9	0.0	422 Chestnut Ave.
Upstream	3251.1	28.8	28.7	0.2	1587 First Street
Upstream	2008.5	27.8	27.6	0.2	537 Meadow Street
*Upstream	309.2	26.4	26.5	-0.1	200 Jefferson Ave.
*Upstream	309.2	26.5	26.5	0.0	210 Jefferson Ave.
Downstream	2908.5	23.9	23.8	0.2	273 Halstead Ave
Downstream	2729.3	23.8	23.9	-0.1	232 Valley Street
*Downstream	2673.7	21.9	21.9	0.0	233 Valley Street
*Downstream	2673.7	21.8	21.9	-0.1	311 Anita Lane
Downstream	2041.3	18.5	18.5	0.1	355 Philips Park Road
Sheldrake	891	26.5	26.5	0.1	650 Mamaroneck Ave.
Sheldrake	1081.4	26.4	26.5	-0.1	147 East Plaza Ave.
Sheldrake	2067.4	26.9	26.7	0.2	Fayette St, Recycling Plant
Sheldrake	2599.4	26.5	26.8	-0.3	147 East Plaza Ave.
Sheldrake	3156.1	26.8	26.8	0.0	328 Northrup St.
Sheldrake	3392.7	26.7	26.9	-0.3	Northrup St, Geico

*As result of proximity, pairs of flood marks per station were compared and the higher flood mark value was chosen.

To accurately model the Mamaroneck River for the April 2007 storm event, an estimated WSEL in the Mamaroneck Harbor had to be used as a downstream boundary condition. Tide elevation data from the Kings Point gage station, approximately eight miles away from Mamaroneck Harbor, was used as a basis for the observed downstream boundary condition. The elevation is based on the observed tide and surge elevation at the time of the peak Mamaroneck River outflow. The predicted peak outflow at the mouth of the Mamaroneck River (hydrologic node MAMKO) occurred at approximately 10:30PM on April 15, 2007. Eyewitness accounts estimated the storm peaking a few hours earlier. The Kings Point gage station WSEL observation was translated to the closest NOAA tide gage #8518091 at Rye Beach Station, NY, approximately two miles from Mamaroneck Harbor. The translation was done using the recommended NOAA height and time offset methodology. The downstream elevation was calculated to be 1.1 ft (NAVD88). It should be noted however that the maximum WSEL at the harbor for the April 2007 storm was 8.6 ft (NAVD88), approximately four hours later, which is after the estimated peak outflow in the Mamaroneck River.

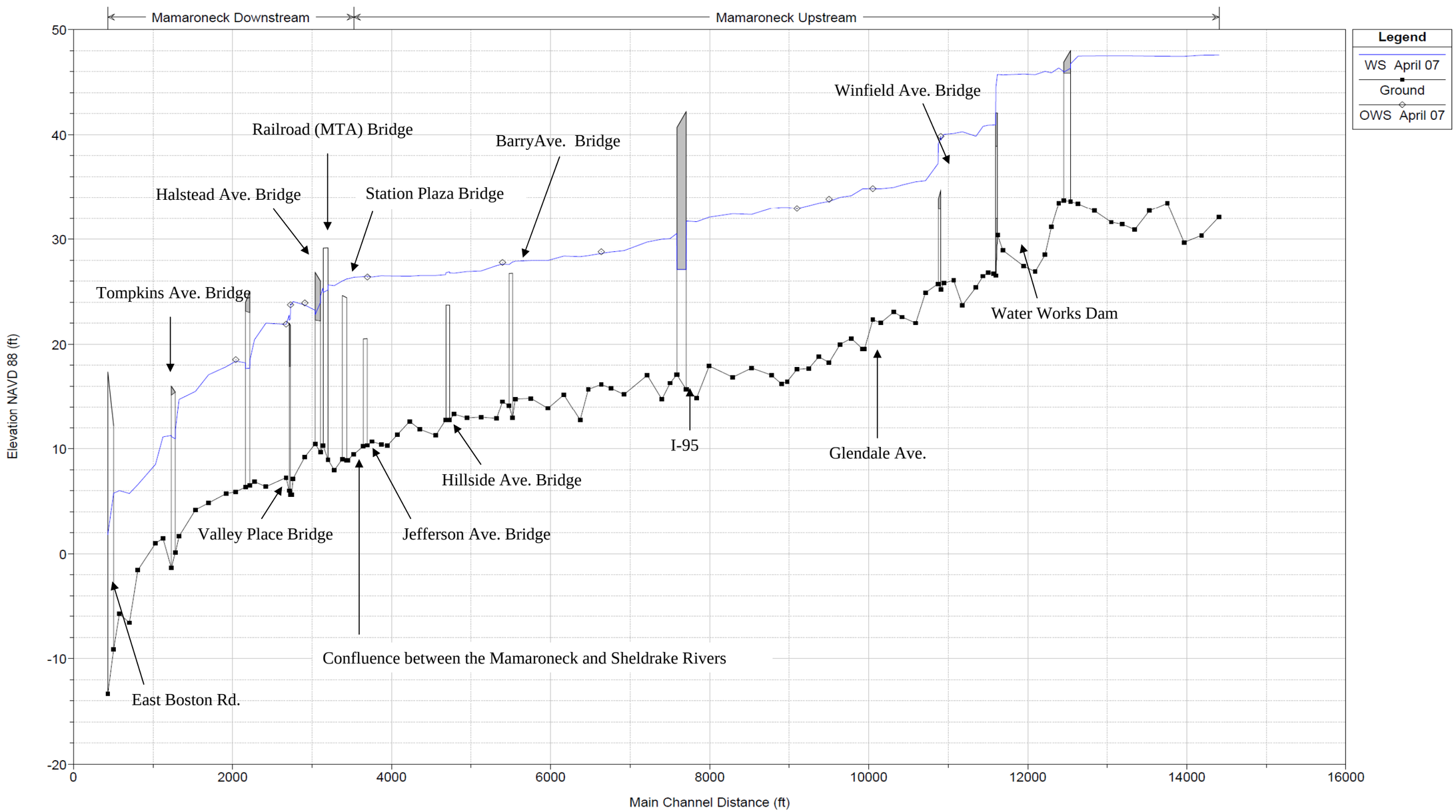

Figure 3: Mamaroneck River calibrated profile to April 2007 storm event

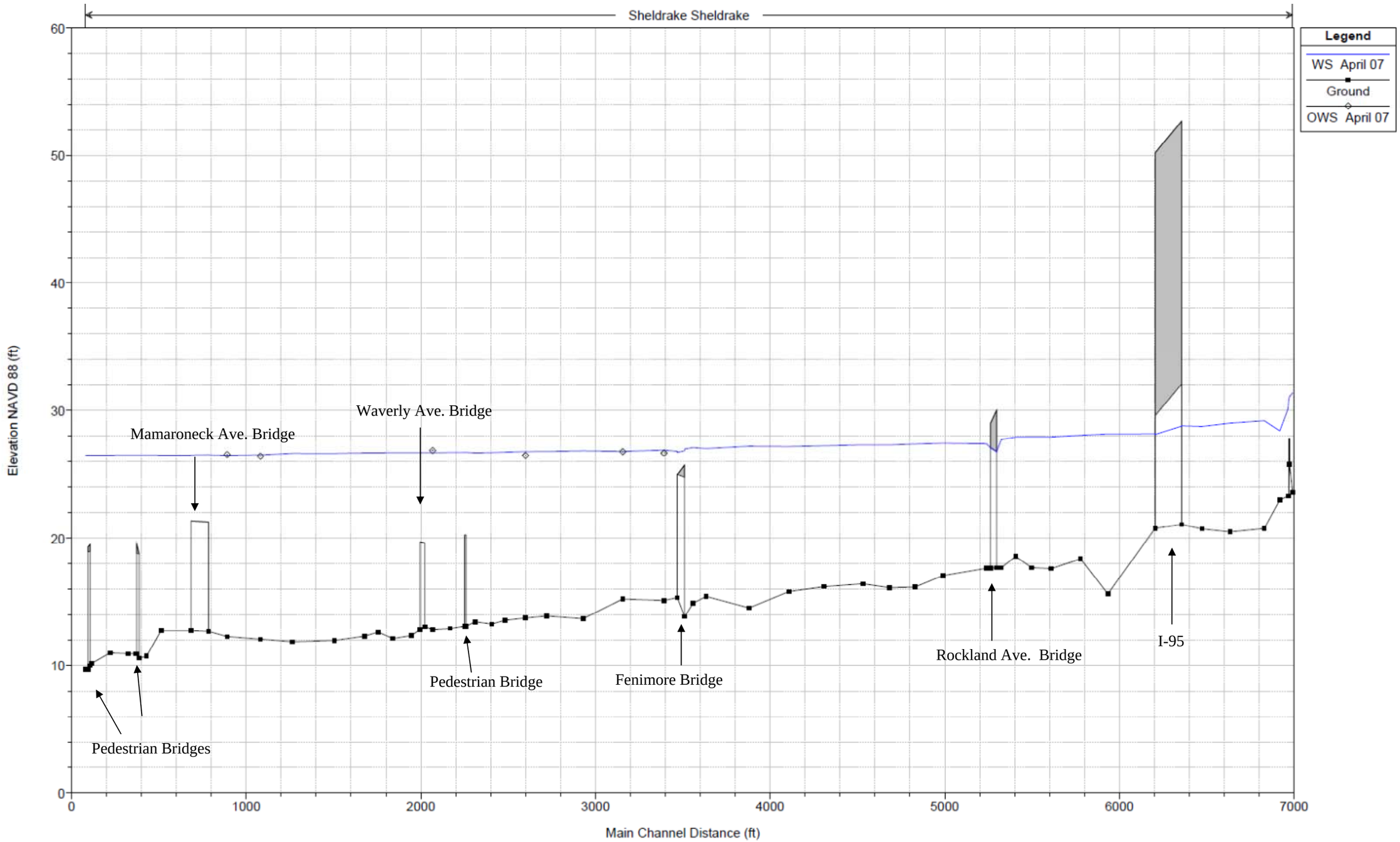

Figure 4: Sheldrake River calibrated profile to April 2007 storm event

2.5 Tidal- Fluvial Correlation

In order to run a hydraulic model of the Mamaroneck and Sheldrake Rivers with a set of hypothetical events, a starting elevation or boundary condition had to be established for Mamaroneck Harbor. Since the Mamaroneck River flows into Long Island Sound, it was necessary to perform a Tidal-Fluvial correlation to analyze the harbor elevations that may occur due to tide and surge during a typical fluvial event. In the 1989 General Design Memorandum, Long Island Sound elevation data for the storm periods was obtained from the Willets Point gauge station. For this analysis, both the Willets Point and the Kings Point gauge stations were used to compile continuous harbor data to match the available fluvial data. Due to periods of gauge inactivity both gages were used in order to develop a continuous data set from 1966 to 2010. Fluvial data begins in 1938 for the Mamaroneck River gage. Because of the close proximity of these gauges, their data is considered interchangeable. Only significant yearly fluvial events and the corresponding tidal information for the specific date of the fluvial event were used in the correlation analysis. Figure 5 is a stage-frequency curve for Willets Point gauge station, used to determine the tidal return periods. Table 5 shows the observed maximum tidal elevations, the predicted astronomical tide elevations and the difference between the two which is the estimated surge at the Willets Point gauge during significant fluvial events from 1966-2000. Most of the observed tides during fluvial peaks, show in Table 5, exceeded the mean high tide but were less than a 2-year tidal event. Figure 6 illustrates the relationship between annual peak flow at the Mamaroneck River gauge and both the astronomically predicted and observed tides at the Willets Point gauge station. The data shown on Figure 7 shows that there is a 1.1 ft average surge height for the selected fluvial events.

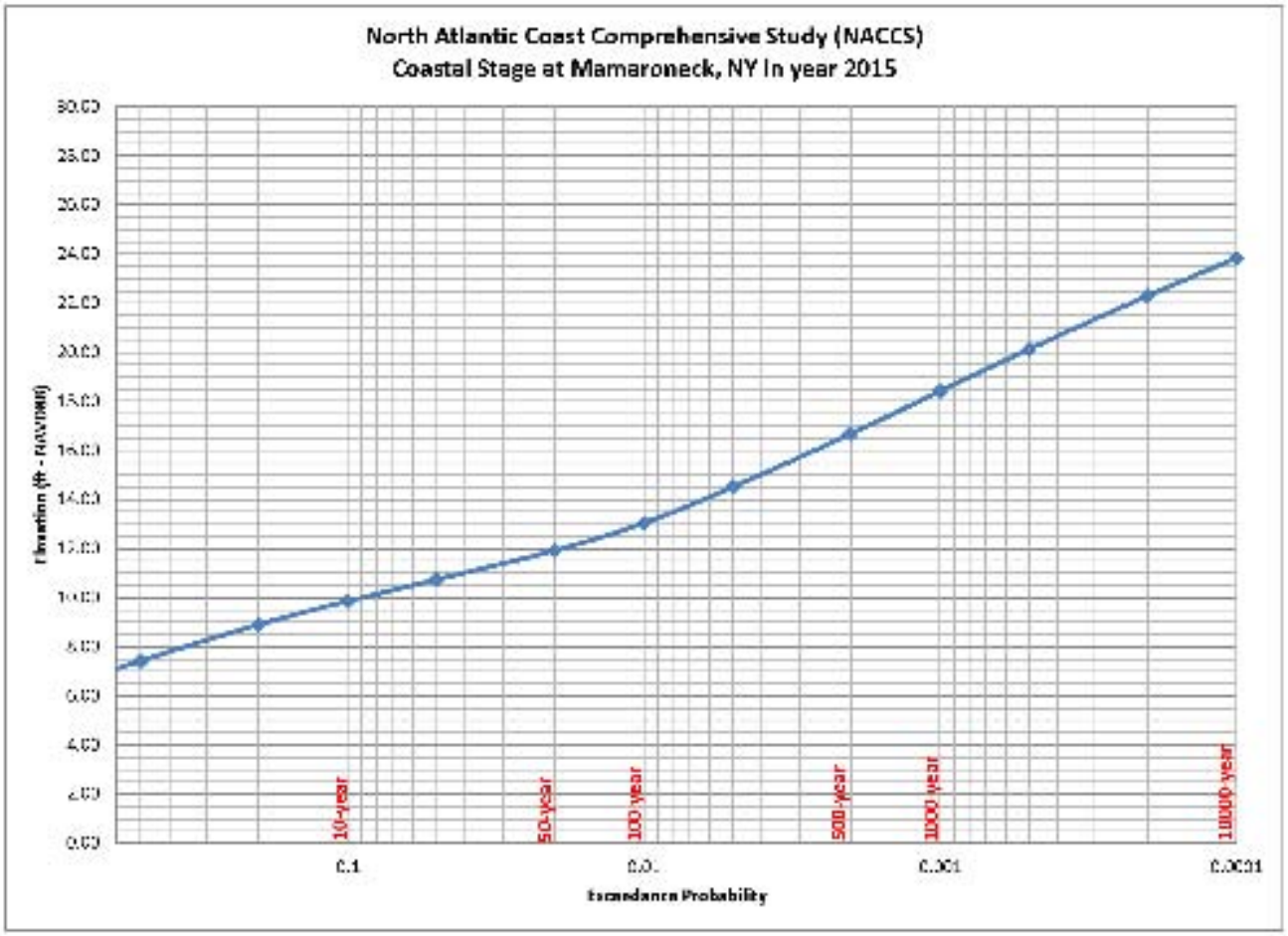

Figure 5: North Atlantic Coast Comprehensive Study (NACCS) Coastal Stage-Frequency Curve at Mamaroneck, NY in year 2015

Table 5: Willets Point gauge recorded sea elevation and Mamaroneck River annual peak flows.

Water Year	Mamaroneck River Gauge		*Willets Point Tide Gauge			**Fluvial Return Periods (yr)	**Tidal Return periods (yr)
	Date	Annual Peak Flow (cfs)	Astronomical Max. tide (ft)	Observed Max. tide (ft)	Max. Surge (ft)		
1966	2/13/1966	1392	7.08	7.85	0.77	1	< 1
1967	3/7/1967	1449	7.19	8.65	1.46	1	< 1
1969	3/25/1969	1760	6.4	7.7	1.3	1	< 1
1670	2/10/1970	2109	8.19	8.95	0.76	2	< 1
1971	8/28/1971	2328	7.06	8.13	1.07	3	< 1
1972	6/19/1972	3550	7.28	7.36	0.08	13	< 1
1975	9/26/1975	3700	7.62	7.71	0.09	15	< 1
1976	8/10/1976	1570	8.25	11.83	3.58	1	5
1977	11/8/1977	3240	7.99	11.06	3.07	8	3
1984	7/7/1984	2720	7.92	8.18	0.26	4	< 1
1985	9/27/1985	961	7.9	11.46	3.56	< 1	3
1986	1/26/1986	1020	7.47	8.3	0.83	< 1	< 1
1986	12/3/1986	1580	8.82	9.66	0.84	1	1
1988	5/18/1988	572	8.01	8.13	0.12	< 1	< 1
1989	5/17/1989	1470	7.5	8.53	1.03	1	< 1
1990	5/17/1990	2130	7.06	7.8	0.74	2	< 1
1990	10/24/1990	2190	6.96	7.77	0.81	2	< 1
1992	8/18/1992	1130	7.74	7.6	-0.14	< 1	< 1
1993	9/27/1993	980	7.71	7.59	-0.12	< 1	< 1
1994	1/29/1994	1770	8.2	7.71	-0.49	1	< 1
1995	7/26/1995	1320	7.49	7.56	0.07	1	< 1
1996	4/16/1996	2250	8.4	9.75	1.35	2	1
1996	10/20/1996	3060	7.55	10.33	2.78	6	2
1998	1/24/1998	1010	6.91	8.6	1.69	< 1	1
1999	9/16/1999	2230	7.05	9.18	2.13	2	1
2000	6/7/2000	960	8.09	9.2	1.11	< 1	1

*Datum: MLLW. The conversion for Willets Point gage station from MLLW to NAVD88 is -4.2 ft.
** Approximate return periods.

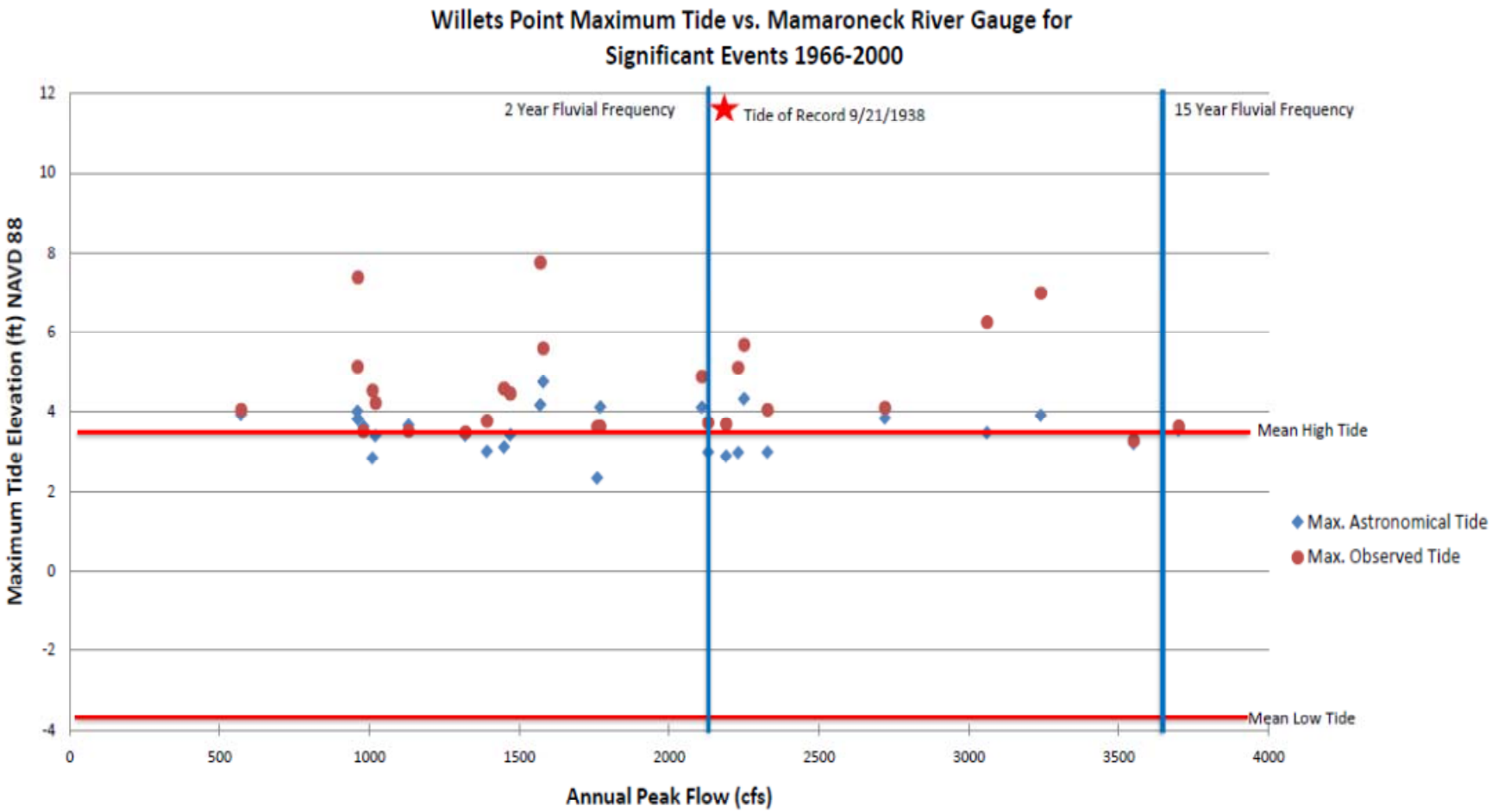

Figure 6: Willets Point maximum tide and Mamaroneck River annual peak flows

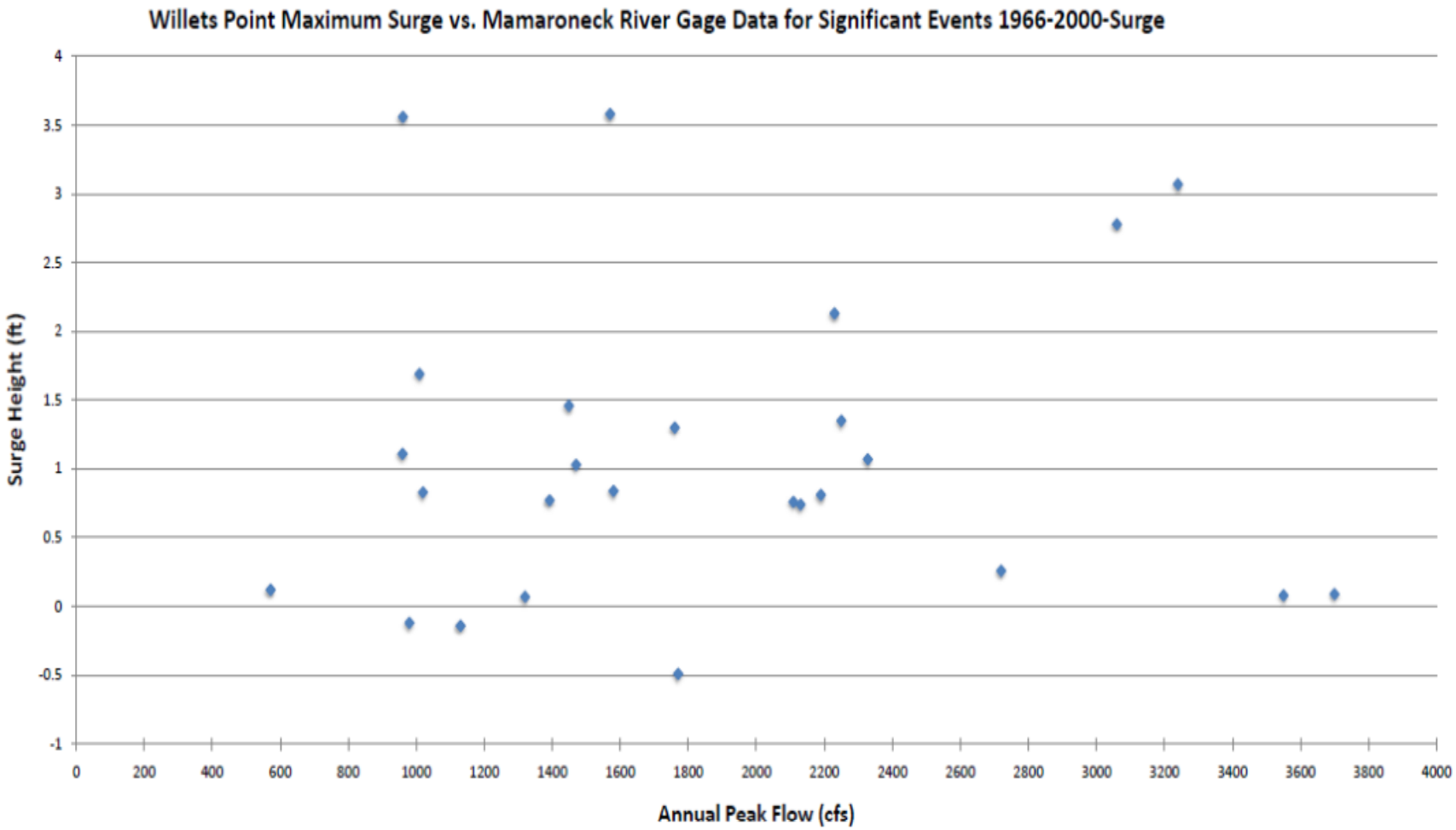

Figure 7: Mamaroneck River gauge annual peak flow and Willets Point surge height.

The same analysis was made for the Kings Point gauge station for significant fluvial events from 1999-2010, as shown in Table 6, Figure 8 and Figure 9. Table 6 shows that most of the astronomically predicted tides exceeded the mean high tide. It was determined that this gauge has a spring tidal fluctuation of about 2.0 ft above normal high tide. The fluvial events generally occurred during a spring tide which caused the both the astronomically predicted tide and the observed tide to be higher than the mean high tide. Figure 7 illustrates the relationship between annual peak flows and both the astronomically predicted and observed tides. Figure 9 shows the surge during these events. The data gathered for Figure 9 shows that there is a 1.0 ft average surge height for the selected fluvial events. During this analysis a weak correlation between the low frequency tidal and fluvial events was found for both the Willets Point and Kings Point data.

Based on this correlation analysis, the use of a mean high water (MHW) elevation with the addition of a mean surge value would be a realist but conservative approach for the computation of maximum WSELs. Using the Kings Point gage station, a MHW of 7.42 ft above mean low water (MLLW) and an average surge of 1.1 ft was set as downstream boundary condition. Translation of the tide from the Kings Point (and Willets Point) gauge station was made using the standard NOAA methodology of height and time offset to determine the boundary condition in Mamaroneck Harbor. The final elevation of 5.35 ft. NAVD88 was used as the downstream boundary for all present conditions hypothetical runs based on the analysis, computations, translations and conversions described above.

Table 6: Kings Point gauge recorded sea level and Mamaroneck River annual peak flows.

Water Year	Mamaroneck River Gage		*Kings Point Tide Gage			**Fluvial Return Periods (yr)	**Tidal Return periods (yr)
	Date	Annual Peak Flow (cfs)	Astronomical Max. tide (ft)	Observed Max. tide (ft)	Max. Surge (ft)		
1999	9/16/1999	2230	7.17	9.19	2.02	2	1
2000	6/7/2000	960	8.19	9.23	1.04	< 1	1
2001	3/30/2001	1200	8.06	8.71	0.65	1	< 1
2002	5/14/2002	1050	8.02	9.29	1.27	< 1	1
2003	7/23/2003	2050	7.04	7.19	0.15	2	< 1
2004	9/8/2004	4620	6.6	6.93	0.33	39	< 1
2005	6/29/2005	2450	8.09	7.93	-0.16	3	< 1
2005	10/9/2005	2830	7.9	8.04	0.14	5	< 1
2007	4/16/2007	5330	8.7	11.43	2.73	100	3
2008	2/13/2008	2000	7.94	8.89	0.95	2	1
2008	12/12/2008	1290	8.95	10.35	1.4	< 1	1
2010	3/30/2010	1927	9.09	10.22	1.13	2	1

*Datum: MLLW. The conversion for Kings Point gage station from MLLW to NAVD88 is -4.2 ft.
** Approximate return periods.

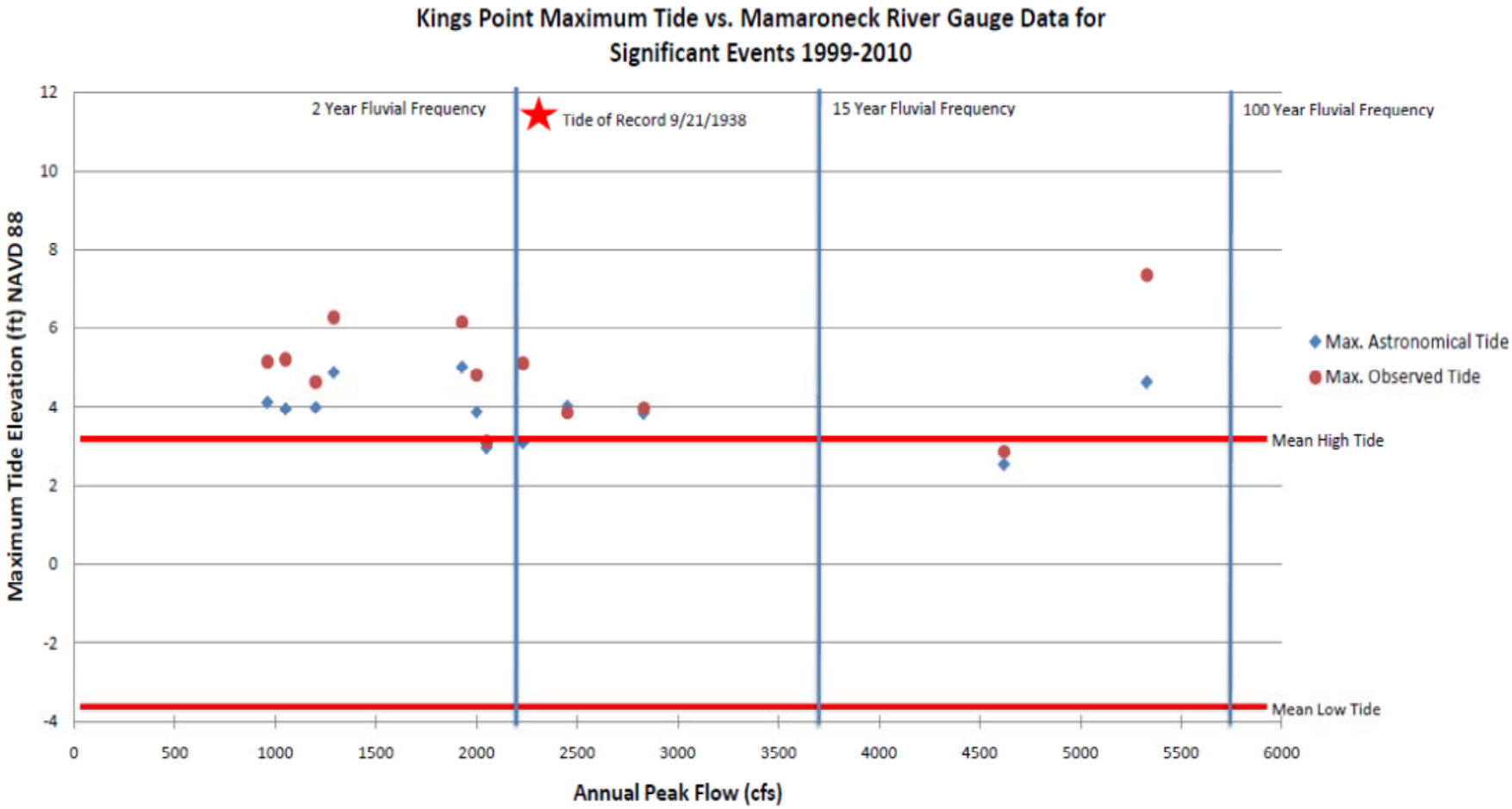

Figure 8: Kings Point maximum tide and Mamaroneck River annual peak flows.

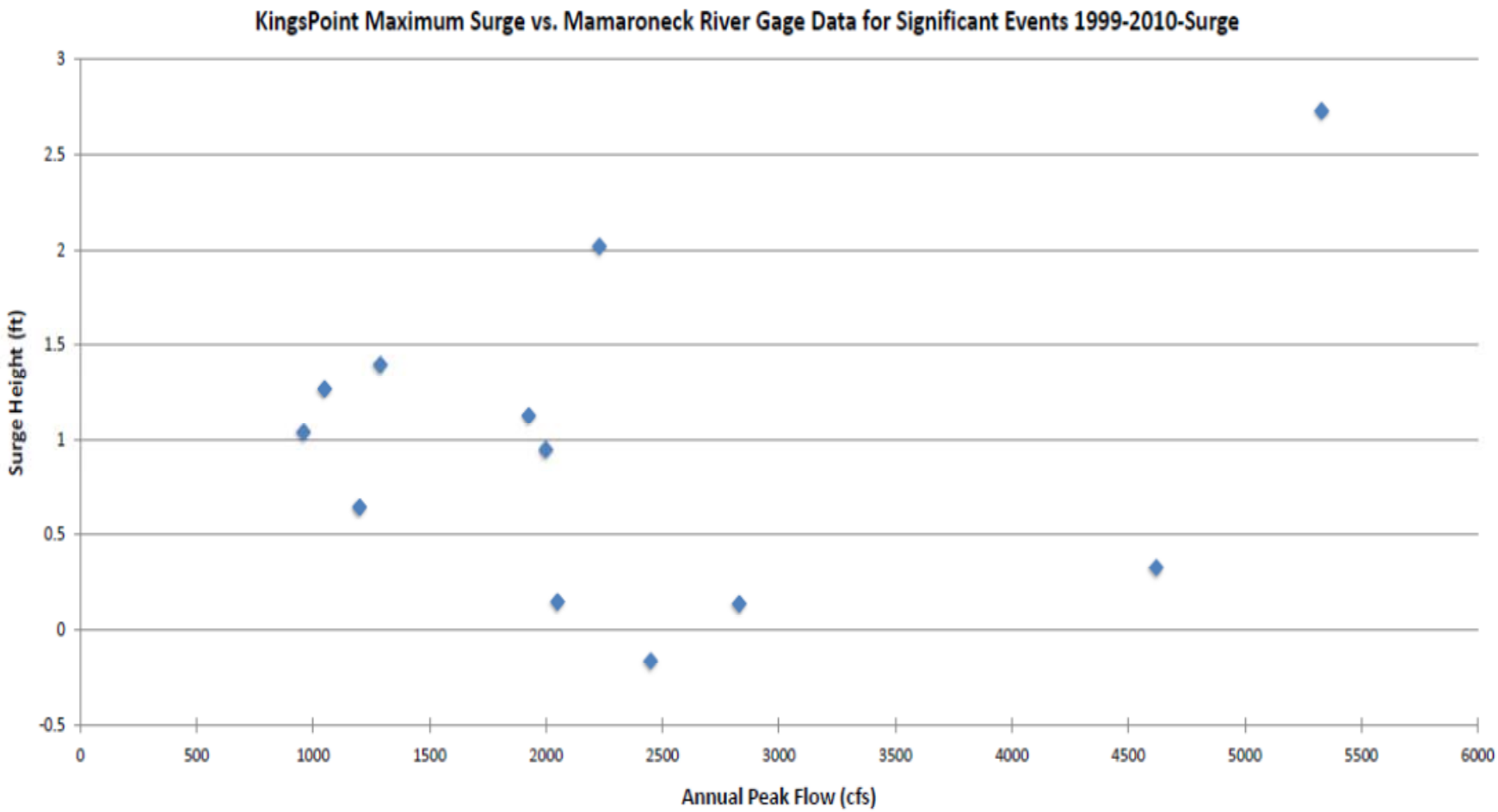

Figure 9: Mamaroneck River gauge annual peak flow and Kings Point surge height.

2.6 Existing Conditions

2.6.1 Flow Line Computation

Flow lines computations were made to develop the hydraulic gradient of the stream in their existing condition with the purpose of determining the current degree of flooding in the Village of Mamaroneck. The calibrated HEC-RAS model of the Mamaroneck and Sheldrake Rivers was used to determine the WSEL for the 1, 2, 5, 10, 25, 50, 100, 250, and 500-year frequency events. Existing conditions peak flows are shown in Table 7. All the existing conditions hypothetical HEC-RAS computations were run with a Mamaroneck Harbor elevation of 5.35 ft (NAVD88) as the downstream boundary condition, as developed and described in the tidal and fluvial correlation analysis.

Table 7: Existing hypothetical events for the Mamaroneck and Sheldrake Rivers.

Reach	Station	Node ID	*Frequency (Years) - Flows (cfs)									
			1	2	5	10	25	50	100	200	250	500
Upstream	11018.7	WINFLD	1310	1670	2060	2410	3030	3520	4110	4750	4940	5610
Upstream	6930.0	THRUWY	1240	1610	2040	2410	3050	3540	4130	4790	4990	5670
Upstream	2780.2	MMUPSH	1260	1600	2040	2370	2950	3420	3960	4510	4690	5280
Downstream	3524.5	30100	1620	2140	2870	3370	4150	4740	5350	5990	6200	6860
Downstream	1921.8	MAMKMO	1630	2140	2880	3380	4150	4740	5350	5990	6210	6870
Sheldrake	6991.7	SHDSEB	670	820	100	1170	1500	1760	2120	2550	2610	2930
Sheldrake	6357.2	SHUSTE	710	860	1040	1200	1520	1700	1900	2090	2140	2320
Sheldrake	3468.0	SHDSTE	710	870	1050	1210	1530	1720	1910	2100	2160	2340
Sheldrake	1755.2	SHELMO	750	860	1000	1090	1220	1320	1390	1480	1510	1590

*Based on the Mamaroneck and Sheldrake hydrologic calibration, Appendix E Part I.

2.6.2 Existing Channel Description

Based on the hydraulic analysis, the influence of high tides during fluvial events does not extend beyond the Tompkins Avenue Bridge. The existing conditions hydraulic profiles for the Mamaroneck River are shown in Figure 10 and Figure 11. Note that the reach of the Mamaroneck River downstream of the confluence is characterized by WSEL in a step-like pattern caused by the bridges and channel contraction. This pattern serves as an impediment to tide controlled floods in the lower reach of the Mamaroneck River, but also indicates that the bridges and channel contractions are causing upstream flooding. Flood "pools" upstream of Tompkins Avenue Bridge, Ward Avenue Bridge and Valley Place Bridge are caused by backwater due to the bridges small size openings and utility pipes under the low cords. Poorly designed wing walls and ineffective transitions from the channel to the constrictions of the bridges are also a cause for floods in this area. At the Valley Place Bridge, the upstream left side wing wall was constructed towards the center of the channel about 10ft upstream the face of the bridge, thereby obstructing the flow and causing a jump in WSEL of approximately 0.25 ft for the 100-year event. High velocities downstream of Tompkins Avenue Bridge exist due to a steep channel bed slope and channel bends; possible scouring is expected during low tides and significant fluvial events. The area just downstream the confluence between the Mamaroneck and the Sheldrake River, which includes the Station Avenue Bridge, Metro North Railroad Bridge and the Halsted Avenue Bridge, is causing considerable losses and high water surface elevations. The small flow capacity of the channel bends through the bridges and the small size of the Halstead Avenue Bridge are key reasons for the frequent flooding in the Village of Mamaroneck. The constriction caused by these three bridges and the "S" shaped channel is causing the most extensive flood "pool" leading to significant flood damages and creating back water upstream of the confluence and into the Sheldrake River.

The effect on the Sheldrake River extends upstream to the Rockland Avenue Bridge for large flood events as seen in Figure 12. Floods in the upstream section of the

Mamaroneck River are caused mostly by poor channel capacity, channel bends and thick vegetation in the overbanks. The Glendale Avenue and the Winfield Avenue Bridges obstructions also cause upstream flooding in this reach of the Mamaroneck River.

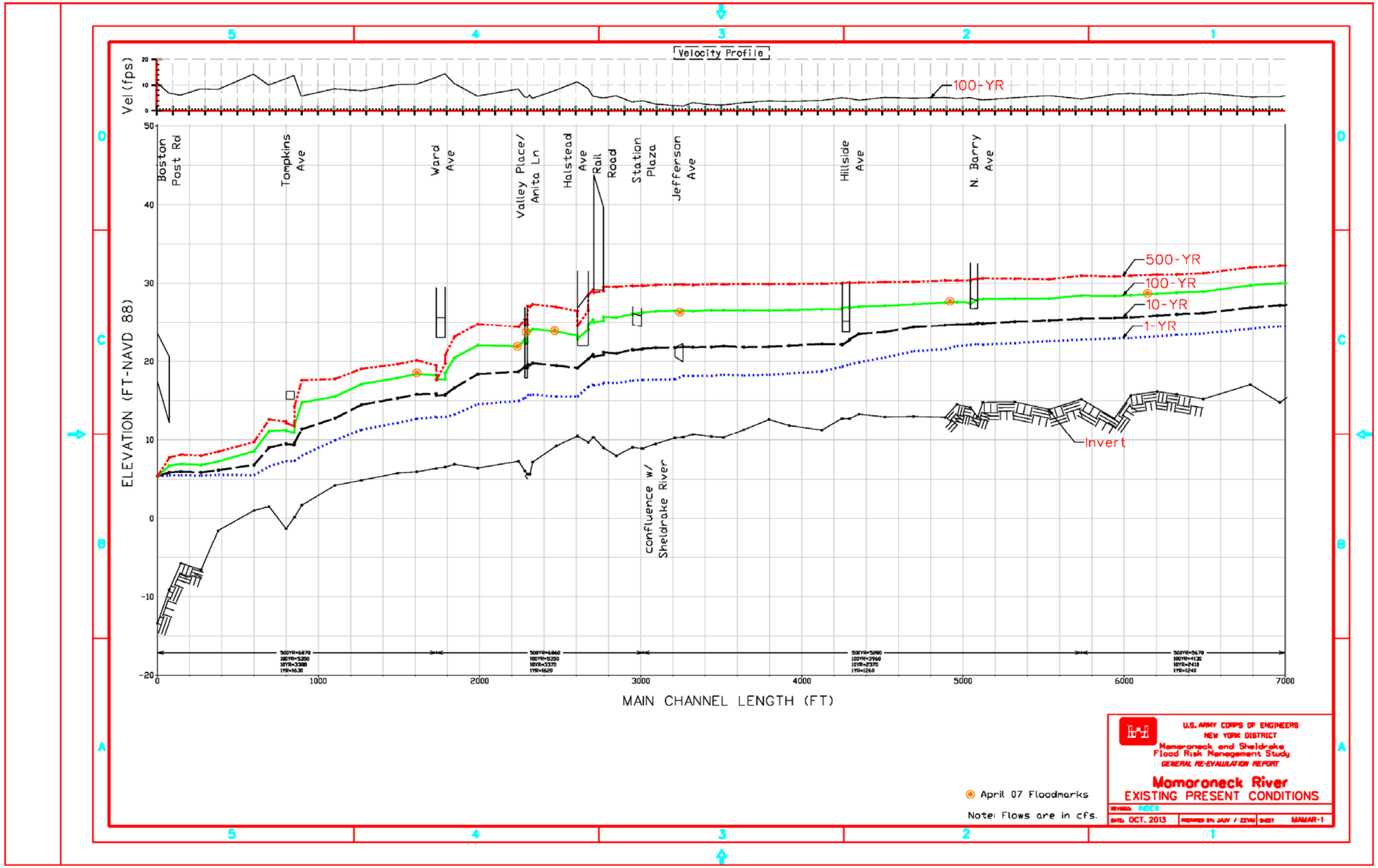

Figure 10: Mamaroneck River existing conditions hypothetical event profiles - ExistingCond_Mamaroneck1

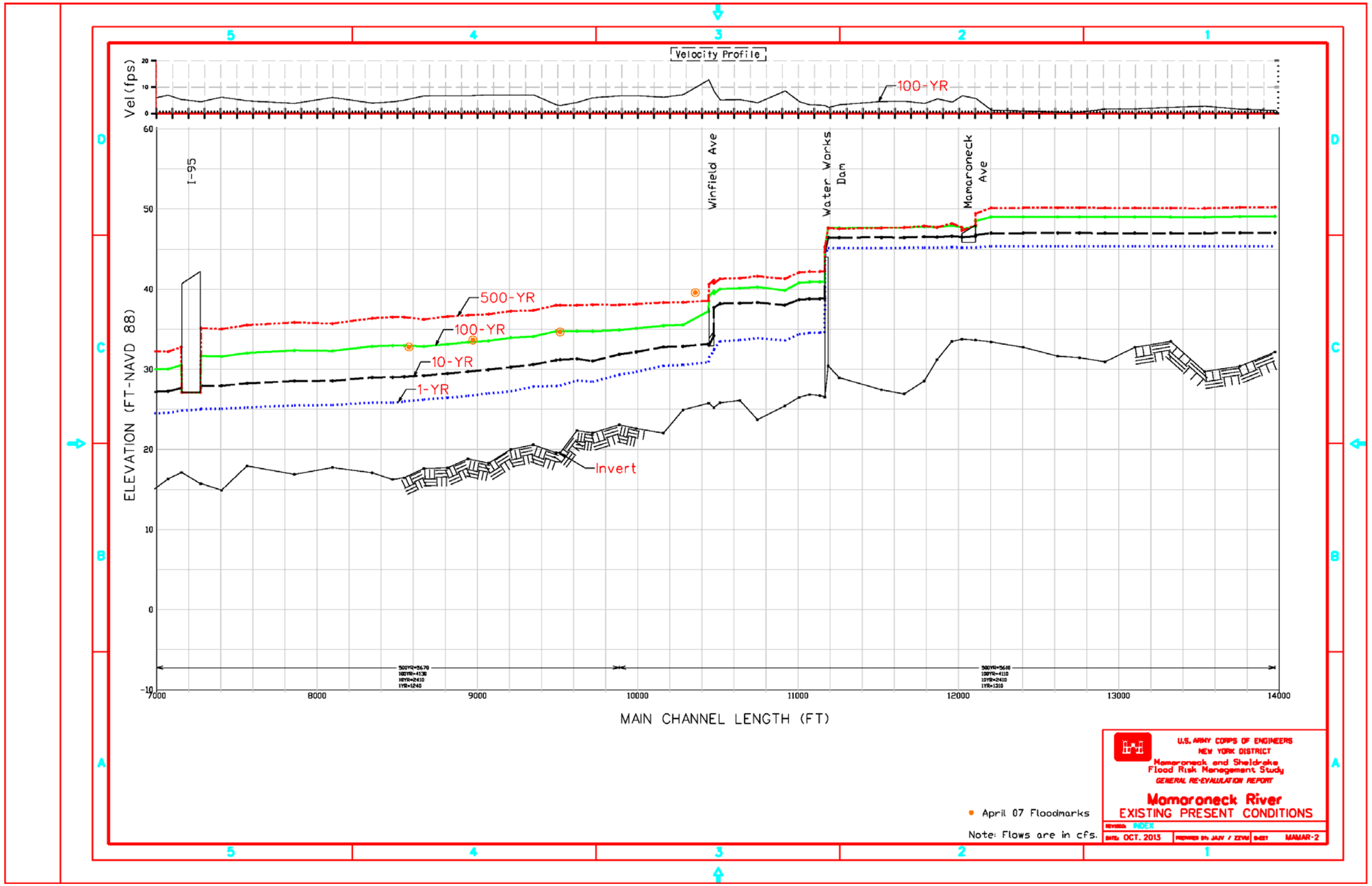

Figure 11: Mamaroneck River existing conditions hypothetical event profiles- ExistingCond_Mamaroneck2

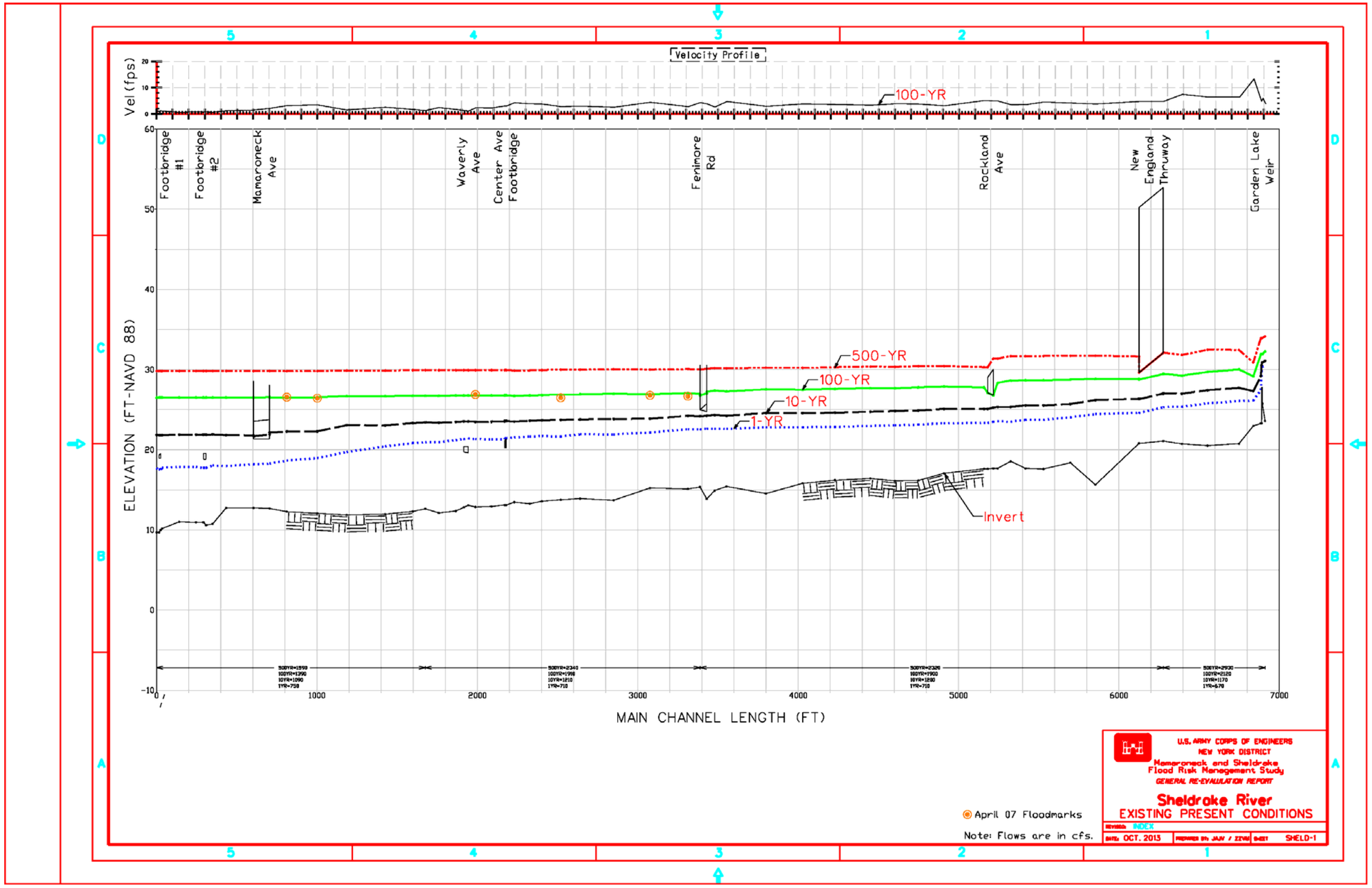

Figure 12: Sheldrake River existing conditions hypothetical event profiles ExistingCond_Sheldrake

2.7 Future Without Project Conditions (No Action Alternative)

2.7.1 Climate Change

In accordance with Corps of Engineers ECB 2014-10 "Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects", no action is required for this GRR. We have however developed some general rough qualitative information concerning precipitation change in the region. The change in future flows associated with climate change is likely to be too small to have an impact on the plan formulation process involving the plan selection and the sizing of that plan (Figure 13 and Figure 14). This is not an area of significant risk for this study or project.

Because the selected plan is a channel deepening plan, the possible impact of climate change on the performance of the improved channel is fairly limited. The proposed plan will reduce flood elevations in the main damage areas by about 2.5 to 3.5 feet for the 2% (50yr) and 1% (100yr) events. If large events like the 2% and 1% events do indeed get larger, the channel will still continue to reduce flooding by several feet. A channel improvement plan is unlike levee or dam features which can result in sudden and catastrophic increases in flood depths once their design capacity is exceeded. Channel plans continue to provide considerable flood risk reduction even after their design capacity is exceeded.

While NOAA Technical Report NESDIS 142-1 states that the historic annual increase in precipitation is statistically significant, this report also states that there is a "substantial decadal-scale variability" associated with extreme precipitation. Therefore estimates of the 50 and 100 year events are very sensitive to the 10-30 year period of record used to compute these estimates. Finally the report states that there is great uncertainty associated with model simulation predictions of future precipitation increases.

Preliminary results presented by Geoff Bonnin of the National Weather Service at a National Dam Safety Technical Seminar in Feb 2014 indicates that while changes in

precipitation associated with climate change appear to be impacting frequent events like the 1 year to 10 year events, there appears to be little precipitation impact on large major events like the 50 year and greater. It is possible that climate change may not be able to make a large "Perfect Storm" more perfect.

This channel improvement plan, like most Flood Risk Management Projects, is designed to manage the risk associated with an estimate of a large and infrequent event. It is possible that climate change may not have a significant impact on the precipitation associated with very large events. Certainly climate change and its' impact on precipitation is very uncertain. Climate change and its' possible impact on precipitation are just another uncertainty involved in the study process. Uncertainties associated with urbanization, gage record lengths, "n" values, erosion, sediment, debris jams, damage curves and sea level change have all been included in the analysis and plan selection.

Possible precipitation impacts associated with climate change appear to be well within the normal range of variability for Flood Risk Management Projects. Without any actionable data or information there is little we can do to address the impacts of possible changes in precipitation. While the perceived project level of protection may change due to changes in precipitation, the actual reduction in flood elevations will not change. It is very unlikely alternative selection would be impacted by a change in future precipitation.

2.7.2 Sea Level Change

Department of the Army Engineering Circular EC-1165-2-211 requires that each civil works project take into consideration the effects of potential sea level change (SLC) over the period of analysis of the project. ER 1100-2-8162 requires the use of three scenarios, at a minimum, to estimate future sea levels. These scenarios are a low rate that shall be based on an extrapolation of the historical tide gauge rate and intermediate and high rates that include future acceleration of global mean sea level (GMSL). It asks that the design team evaluate alternatives for the low, intermediate and high-level future rates of sea

level rise (SLR). The effects of vertical land movement (VLM) must also be considered as a component of sea-level rise. VLM can include the shifting of tectonic plates, the rebounding of the Earth's crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids. The low rate of future sea-level rise is based upon the historic rate of SLR in the vicinity of the project area ("regional" sea-level rise). For our purposes, 43 years of data from the NOAA tide gauge # 8467150 at Bridgeport, Connecticut were used. These data indicate an upward trend of 2.56 mm/year (+/-0.10 in/yr) with a 95% confidence limit of +/- 0.58 mm/year (+/-0.02 in/yr). In consideration of VLM, it is assumed that the regional mean sea level trend is equal to the global, or "eustatic," mean sea level trend, which has been determined by the International Panel on Climate Change (IPCC 2007a) to be 1.7 mm/year (+/-0.07 in/yr), +/- 0.5 mm/year (+/-0.02 in/yr). Removing the eustatic SLR component from the regional sea level rise will isolate the regional rate of VLM. The result obtained is 0.86 mm/year (0.03 in/yr), or a total of 0.12 ft of VLM subsidence over the 50-year period of analysis of the project. The low, intermediate and high rates of future SLR (i.e.; Type I, II and III) are determined from the modified National Research Council (NRC -1987) eustatic sea-level change scenarios and the IPCC (2007). SLR was computed for the period of 50 years from the beginning of the project (2015-2064) with an increment of every 5-years and accounting for the VLM. Changes in SLR and VLM can be seen in Table 8. The net change in SLR taking into account VLM subsidence over the 50-year period of analysis of the project is 0.30, 0.7 and 2.06 ft for the low, intermediate and high rates modified National Rate Council (NRC) respectively. These results are in accordance with NYSDEC as projections based in IPCC were adopted by the New York City of Panel of Climate Change (NYCPCC).

Table 8: Estimates of Sea Level Change and Vertical Land Movement for the period of analysis of 50 years

Year	*SLR (ft)			VLM (ft)
	Type I	Type II	Type III	
2015	0.0	0.0	0.0	0.0
2020	0.05	0.08	0.16	-0.01
2025	0.09	0.14	0.31	-0.02
2030	0.13	0.21	0.49	-0.03
2035	0.17	0.29	0.68	-0.04
2040	0.21	0.37	0.88	-0.06
2045	0.25	0.46	1.11	-0.07
2050	0.29	0.55	1.36	-0.08
2055	0.33	0.64	1.62	-0.09
2060	0.37	0.74	1.91	-0.11
2065	0.42	0.82	2.18	-0.12

* Based on NRC -1987 and the IPCC (2007). Required by ER-1100-2-8162

Higher downstream water levels resulting from future sea level rise will not have an effect on the performance of this flood risk management project. The projected rise in sea level will only extend as far upstream as the Tompkins Avenue Bridge. The main damage areas are approximately 2,000 feet upstream and 10 feet higher than the Tompkins Ave area. The first significant damage elevations for this project are at about elevation 20 ft (NAVD88) which is well above the elevation of any recent coastal storm. Sea level rise is expected to have major impacts on direct coastal flooding and wind-wave erosion along the Mamaroneck Bay shoreline, including impacts to properties and critical infrastructure; however this project does not reduce the risk associated with coastal flooding, and is limited to river flow related events, and is not impacted by sea level rise projections.

2.7.3 Flow Line Computation

The calibrated HEC-RAS model of the Mamaroneck and Sheldrake River was used to determine the future unimproved WSEL for the 1, 2, 5, 10, 25, 50, 100, 250, and 500-

year frequency events. Future hypothetical peak flows from the HEC-HMS model can be observed in

Table 9: The future unimproved model was created using the future hypothetical peak discharges and the calibrated existing conditions HEC-RAS model. Because there was only a small increase in the flows between existing conditions and the future unimproved conditions, there is no increase in damages expected for the future unimproved conditions due to the change in flows. The three predicted modified NRC curves were considered in relation to the downstream boundary condition. The mouth of the Mamaroneck River will be the only area impacted by predicted sea level changes. Because the steep bed slope and topographic characteristics of the overbanks surrounding the mouth of the Mamaroneck River, tidally influenced flooding does not go beyond the Tompkins Ave. Bridge for the future unimproved conditions. It should be noted however that the frequency of floods caused by tidal conditions may increase. Future without project profiles developed with the intermediate modified NRC-II curve for sea level change and accounting for VLM can be observed in Figure 13.Figure 13: Future without project hypothetical events for the Mamaroneck River.

Table 9: Future hypothetical flows for the Mamaroneck and Sheldrake Rivers

Reach	Station	Node ID	* Frequency (Years) – Flows (cfs)									
			1	2	5	10	25	50	100	200	250	500
Upstream	11018.74	WINFLD	1340	1690	2080	2440	3060	3540	4130	4760	4960	5630
Upstream	6930	THRUWY	1270	1630	2060	2440	3080	3570	4150	4800	5010	5690
Upstream	2780.19	MMUPSH	1280	1630	2060	2390	2980	3450	3970	4520	4710	5290
Downstream	3524.5	30100	1650	2180	2900	3400	4180	4770	5360	6000	6220	6870
Downstream	1921.84	MAMKMO	1650	2170	2900	3410	4180	4770	5370	6010	6230	6880
Sheldrake	6991.73	SHDSEB	680	820	1000	1180	1510	1770	2130	2520	2610	2960
Sheldrake	6357.23	SHUSTE	710	870	1040	1200	1520	1710	1900	2090	2140	2330
Sheldrake	3468.01	SHDSTE	720	870	1050	1210	1540	1720	1920	2110	2160	2350
Sheldrake	1755.18	SHELMO	750	860	1000	1090	1220	1320	1390	1480	1520	1590

*Based on the Mamaroneck and Sheldrake land use changes for the 50 year period of analysis, more details in Appendix C1 – Hydrology

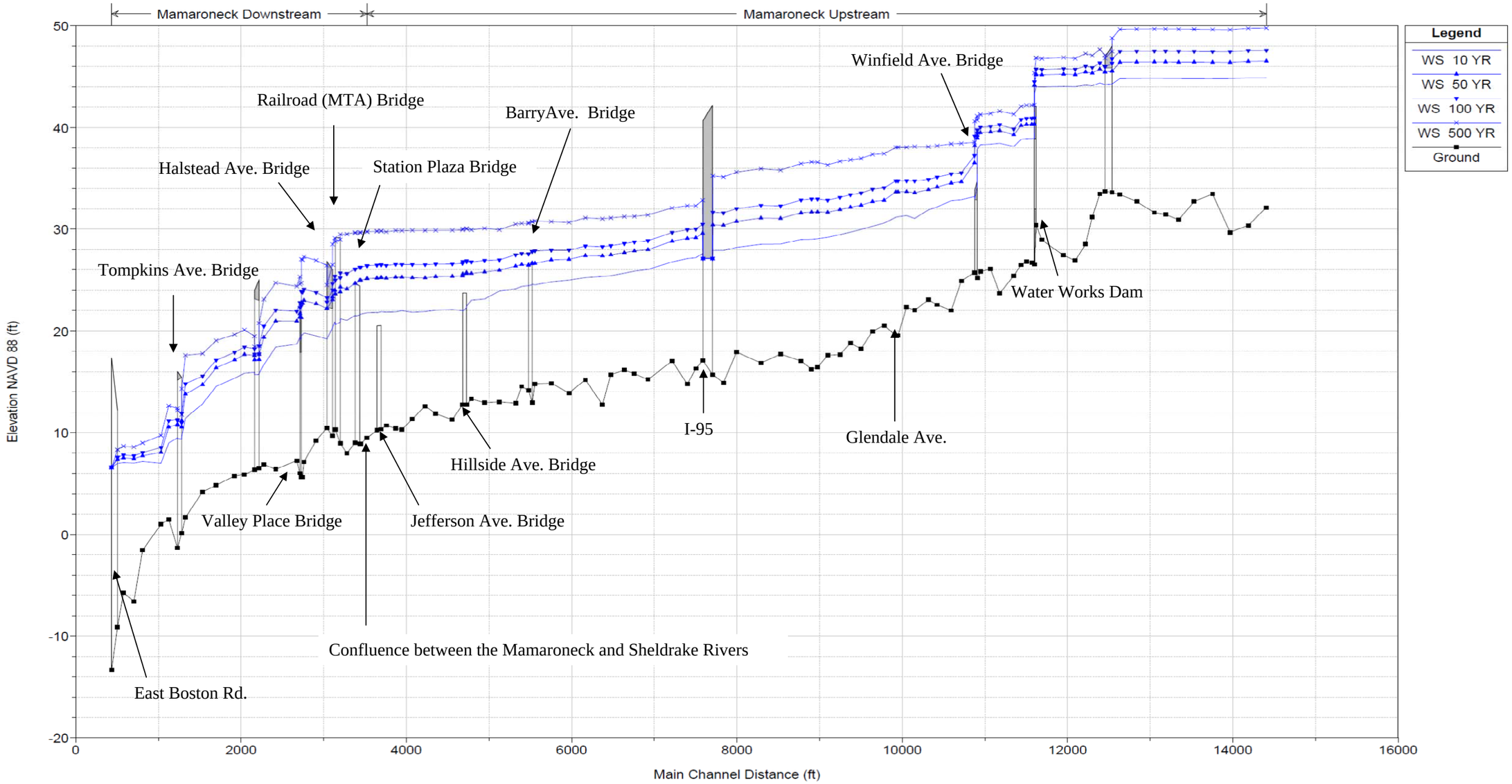

Figure 13: Future without project hypothetical events for the Mamaroneck River.

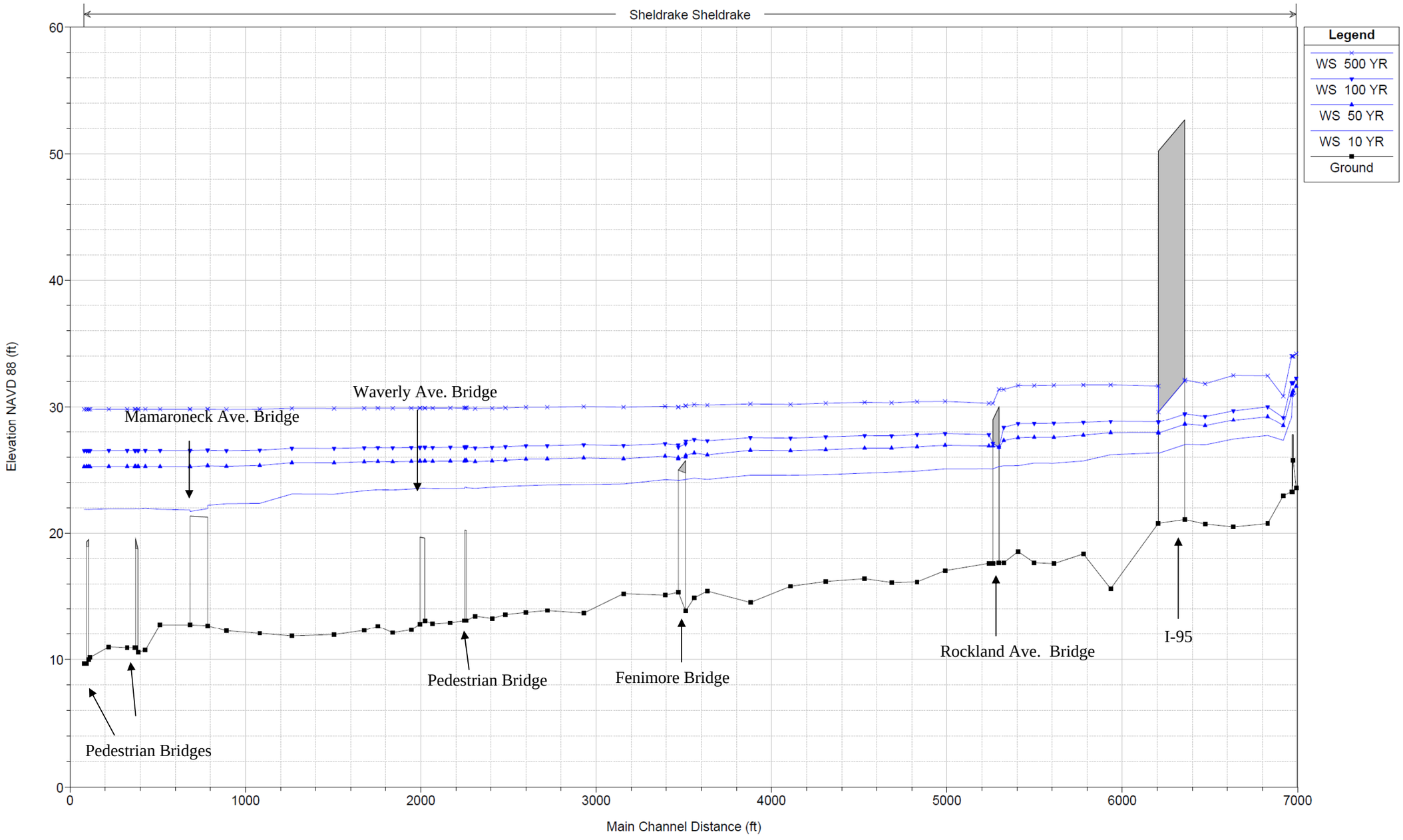

Figure 14: Future without project hypothetical events for the Sheldrake River

2.8 Risk and Uncertainty Analysis Unimproved Conditions

The uncertainty associated with the computed water surface elevations (WSELs) or stage was evaluated by conducting a sensitivity analysis. The goal or overall approach was to develop realistic upper and lower bounds on the computed stage for a given discharge. The hydraulic characteristics considered in developing the upper and lower bounds were the Manning's n-value, debris jams at bridges, channel bed changes and changes in the Mamaroneck Harbor sea level, which is the downstream boundary condition in the HEC-RAS model. A 20% reduction and a 40% increase to the n-values were assigned to help bracket the upper and lower confidence boundaries. This was applied only to the channel and to the majority of cross sections in the hydraulic model. To account for the debris during a storm and to develop the upper uncertainty boundary, most bridge parapets were considered to be blocked. In addition, select bridge low cords were lowered by a foot and all bridge pier sizes were increased by a foot to model debris accumulation. To account for the lack of debris and to develop the lower uncertainty boundary, selected bridge parapets that were blocked in the calibration or base model were opened and the expansion and contraction coefficients of all bridges were lowered by a maximum value of 0.1. A channel bed slope change analysis was based on an invert comparison between the cross-sections surveyed for the 1989 GDM and the cross-sections surveyed for this GRR. The comparison indicated that there was a small but general scouring trend for the Mamaroneck River. The results of the last 20 years was extrapolated 50 years into the future and only the future unimproved model was modified to reflect the scour and help develop the lower uncertainty boundary. Sediment did not appear to be an issue and was not included in the upper boundary computations. To help develop the degree of uncertainty related to the Mamaroneck Harbor boundary condition, different combinations of tide, surge and sea level rise were developed. This combination can be observed in Table 10. All future conditions were run with the maximum and minimum projected SLR. The minimum SLR was calculated using the regional trends at Bridgeport, Willets Point & Kings Point, and the highest SLR by using the NRC Type III curve.

Hydraulic runs that represented the lower and upper uncertainty stage were modeled at a range of tidal water surface elevations. As explained previously, high tides with additional storm surge values superimposed were used to develop the upper limit and best estimate model for existing conditions. Also as previously stated, low or mid tide elevations without superimposed storm surge values were used to develop the lower limit model for existing conditions. Future conditions were modeled by adding high and low sea level rise estimates to the upper and lower limit models respectively.

With the exception of the tidal or downstream boundary condition, the standard deviation for the computed stage uncertainty using the sensitivity analysis and engineering judgment method was determined to be less than the minimum values identified in Table 5-2 of EM 1110-2-1619. Therefore a standard deviation value of 0.5 feet with a normal distribution was used for all reaches except in the downstream tidal area. In the tidal reach a triangular uncertainty distribution was used to express estimates of the maximum, the most likely, and the minimum value of the stage. A triangular distribution was used because the maximum and minimum values were not centered on the most likely value. The WSELs and uncertainty information was provided to the project economist, along with instructions to use either the triangular or the log normal distribution for input into the HEC-FDA model. The procedure to estimate the uncertainty boundary was in compliance with the recommended procedure provided in the EM 1110-2-1916 (USACE 1996).

Table 10: Downstream boundary conditions, Mamaroneck Harbor WSEL

Case	Downstream Boundary	Value			Elevation (NAVD88)		
		Low	Mean	High	Low	Mean	High
Existing	Tide (NAVD88)	-3.93	3.24	3.24	-2.93	5.34	7.01
	High Surge (April 07)	0.00	1.10	2.77			
	Sea Level Rise	0.00	0.00	0.00			
	Gage translation	1.00	1.00	1.00			
Future With and	Tide (NAVD88)	-3.93	3.24	3.24	-2.51	6.16	9.19
	Surge	0.00	1.10	2.77			
	Sea Level Rise	0.42	0.82	2.18			

Without Project	Gage translation	1.00	1.00	1.00			

2.9 Hydraulic and Economic Reaches

To better facilitate the collection of flood damage data and the calculation of flood reduction benefits, the Mamaroneck and Sheldrake Rivers were divided into twelve separate economic reaches (see Appendix D – Economic Appendix, Section 4.1.1.1, Table 3). These reaches were established based on the most logical hydraulic and economic break points for existing and improved conditions. Reach delineations were made where there are significant changes in the hydrologic, hydraulic and land use characteristics. These economic reaches were used in the HEC-FDA model and are not to be confused with the hydraulic reaches identified in the HEC-RAS model. The hydraulic reaches as identified in the HEC-RAS model simply identify confluences between rivers and tributaries.

3.0 DEVELOPMENT AND EVALUATION OF ALTERNATIVES

3.1 General

As discussed previously, the objective for the development of improvements is to better manage the risk of flooding in the project area. The following sections describe a series of improvements evaluated as part of this GRR.

3.2 Plan Formulation

As part of the GRR a series of alternatives was considered, including the reevaluation of improvements proposed in the 1989 GDM. The alternatives evaluated can be classified as No Action (same as Future without Project Conditions), Structural and Non-structural alternatives. The Structural alternatives involve, channel work, reservoirs modifications, diversion tunnels and/or a combination of the above. Non-structural measures are permanent or temporal procedures applied to a structure and/or its parts preventing or resisting damage from a flood event. Examples of such measures are, dry flood proofing,

wet flood proofing, elevate/raise the structure, ring walls/levees and buyouts. Buyouts were not evaluated at this stage of the project.

3.2.1 No Action Alternative (Future Unimproved Conditions)

This plan involves no Federal action to manage the flood risk in the Village and Town of Mamaroneck. The No Action alternative avoids environmental and other impacts associated with implementation of other plans for flood damage reduction. Since future trends do not indicate a significant drop in land use or rainfall this plan fails to meet any of the study objectives. The result would be the continuation and future increase of flooding problems in the study area. This alternative represents the default condition if no other plan is recommended for further action and is a basis of comparison for all other plans.

3.2.2 Alternative #1: Channel works along the lower Mamaroneck and through the confluence area.

This plan includes channel deepening and widening along both the Mamaroneck and Sheldrake Rivers from a little above the confluence to the Tomkins Ave Bridge (tidal limit) as seen in Figure 15. Five bridges will be removed and/or replaced including the two Columbus Park pedestrian bridges, Station Plaza, Halstead Ave., and Ward Avenue. Along the Mamaroneck River, channel work extents from the Tompkins Ave. Bridge to 400 ft above the Hillside Avenue Bridge, for an approximated length of 4,200 ft. Along the Sheldrake River, channel work extents from the confluence to 700 ft above the Mamaroneck Avenue Bridge, for an approximated total length of 1,400 ft. The river will be significantly realigned at the confluence and below the Ward Ave Bridge. Trapezoidal channel improvements will consist of a natural bed channel with a 30 to 50 ft width and side slopes of one vertical on two and a half horizontal (1:2.5). Concrete retaining walls will be used where space is limited. The existing channel side slopes range from one vertical on one horizontal (1:1), to one vertical on three horizontal (1:3). The width of the existing channel varies from 30 to 50 ft for the Mamaroneck River and from 20 to 40 ft in the Sheldrake River. The channel bottom will be lowered from 2 to 4 ft. The channel

bottom has a moderate slope, approximately 12 ft per mile. Water-surface elevations for various hypothetical events with Alternative 1 were computed using the HEC-RAS computer program (version 4.1) and drawn as shown in Figure 16, 17, 18.

Figure 15: Alternative 1 | Improved Map showing channel

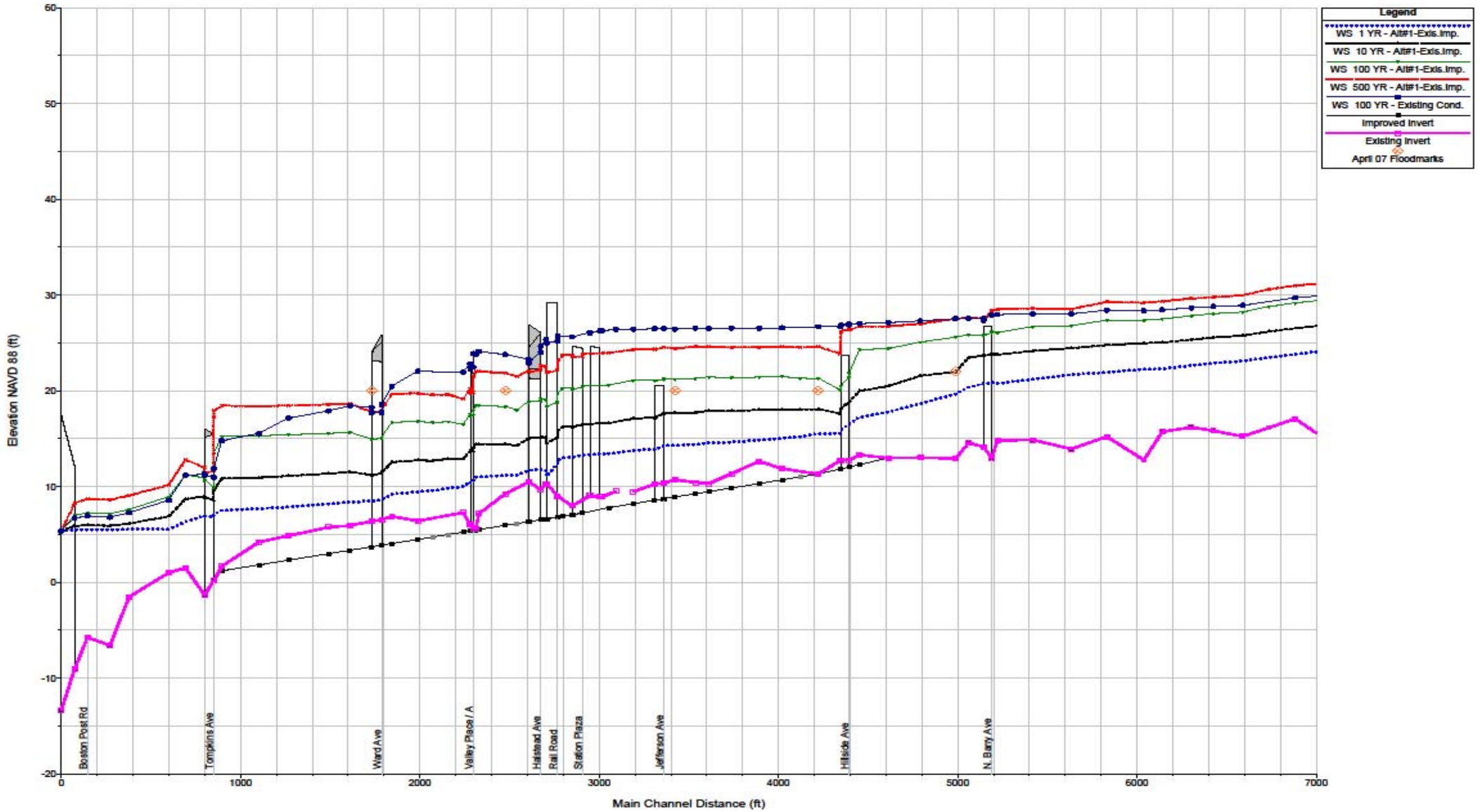

Figure 16: Alternative 1 | Mamaroneck1- Channel works along the lower Mamaroneck and Confluence area

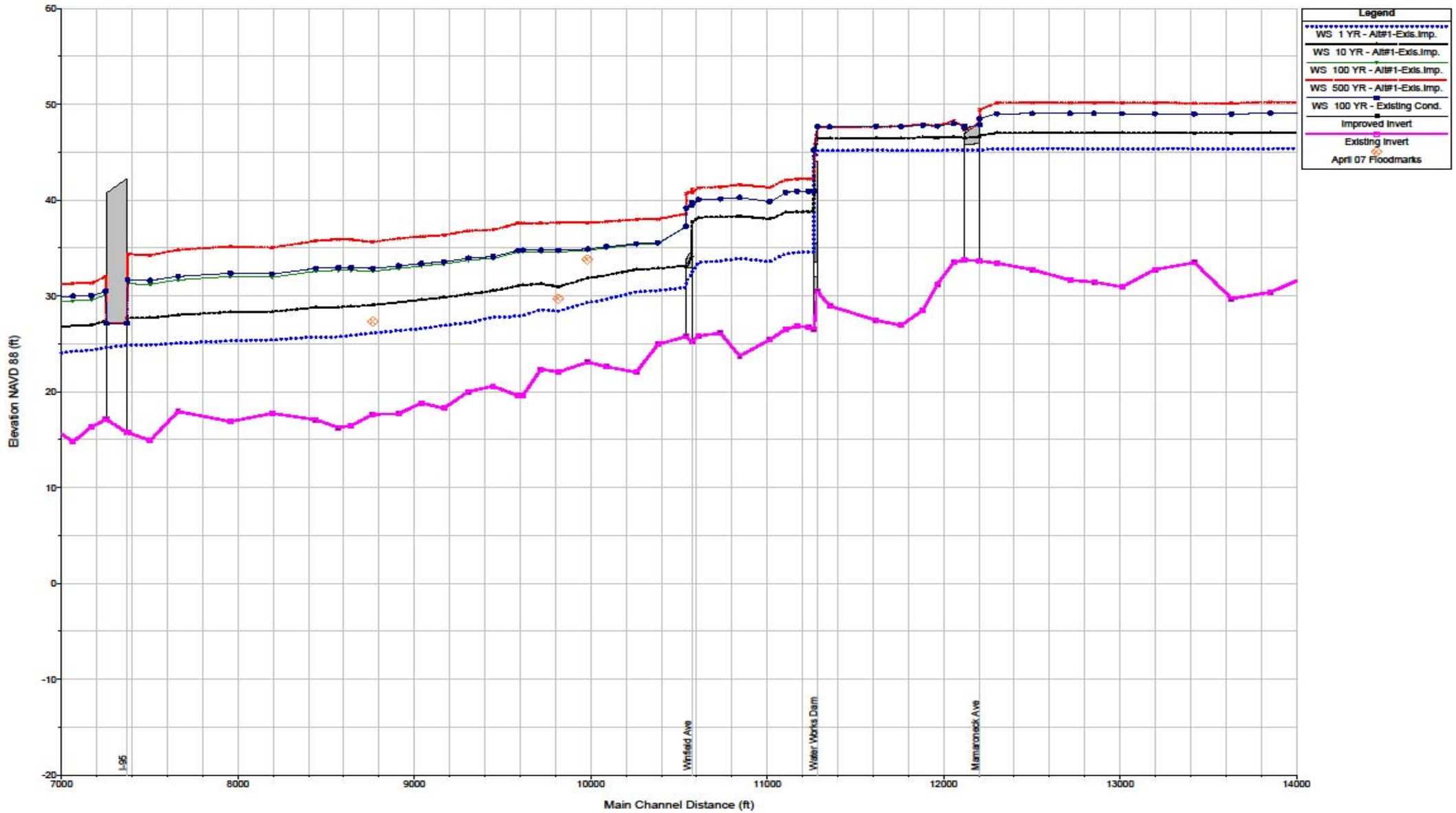

Figure 17: Alternative 1 | Mamaroneck2 - Channel works along the lower Mamaroneck and Confluence area

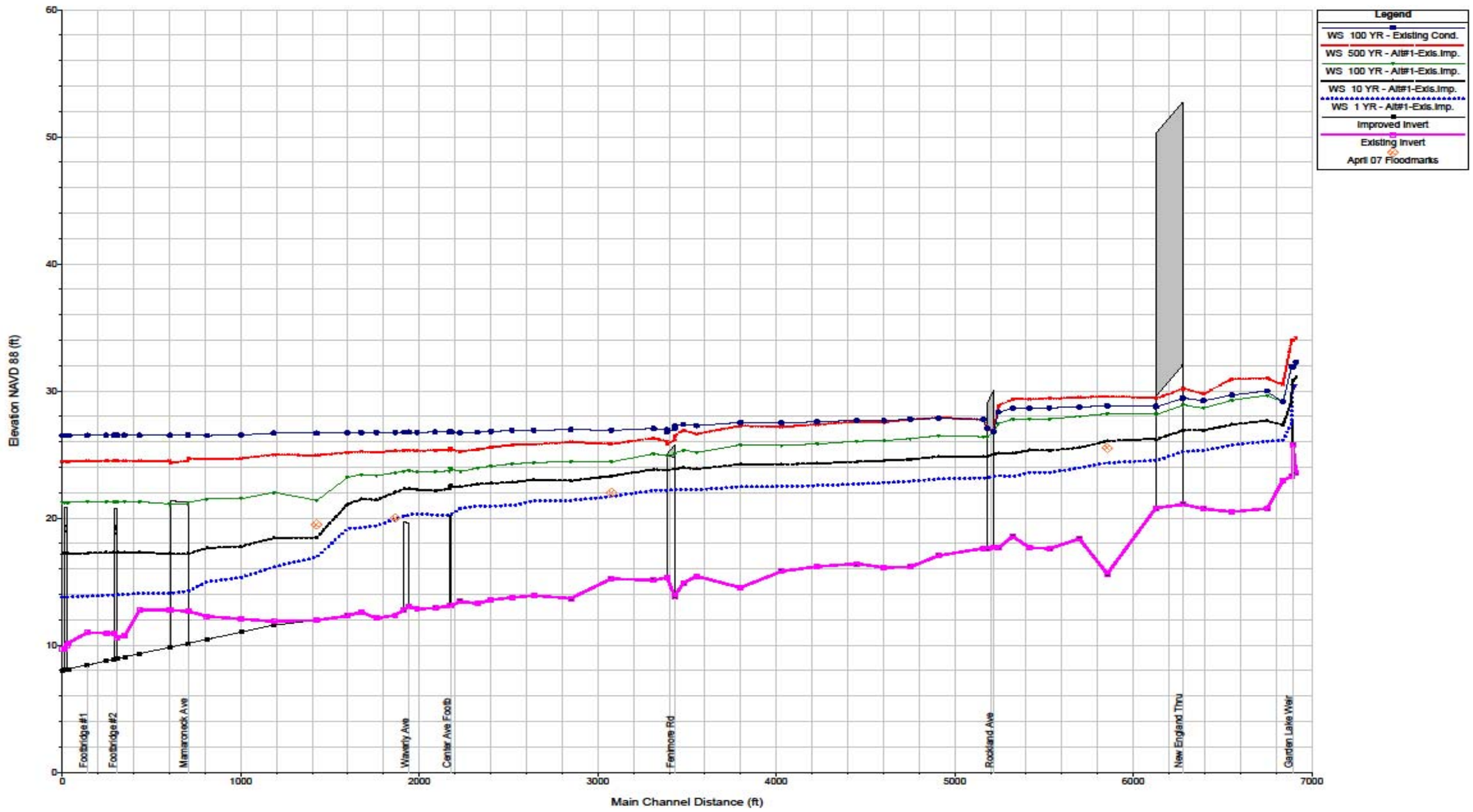

Figure 18: Alternative 1 | Sheldrake Channel works along the lower Mamaroneck and Confluence area

3.2.3 Alternative 2: Channel work throughout much of the Mamaroneck River in addition to the Confluence area.

This plan includes Alternative 1 and additional work along the Mamaroneck River up to the Winfield Ave. Bridge as seen in Figure 19. Six bridges will be removed and/or replaced including Hillside Ave, the two Columbus Park pedestrian bridges, Station Plaza, Halstead Ave. and Ward Avenue. Along the Mamaroneck River, channel work extents from the Tompkins Ave. Bridge to 270 ft above Winfield Ave. Bridge, for an approximated length of 6,700 ft. Along the Sheldrake River, channel work extents from the confluence to 750ft above the Mamaroneck Avenue Bridge, for an approximate length of 1,500ft. The river will be significantly realigned at the confluence and just below the Ward Ave Bridge. Trapezoidal channel improvements will consist of a natural bed channel with side slopes of one vertical on two and a half horizontal (1:2.5), with retaining walls where space is limited. Columbus Park will be used as the stage area during construction. Articulated concrete mats block may be used throughout and just downstream of the Winfield Avenue Bridge due to the high stream velocities. Water-surface elevations of various hypothetical events with Alternative 2 were computed using the HEC-RAS computer program (version 4.1) and drawn as shown in Figure 20, 21, 22.

Figure 19: Alternative 2 | Map showing channel works throughout the Mamaroneck River and the Confluence area in the Sheldrake River

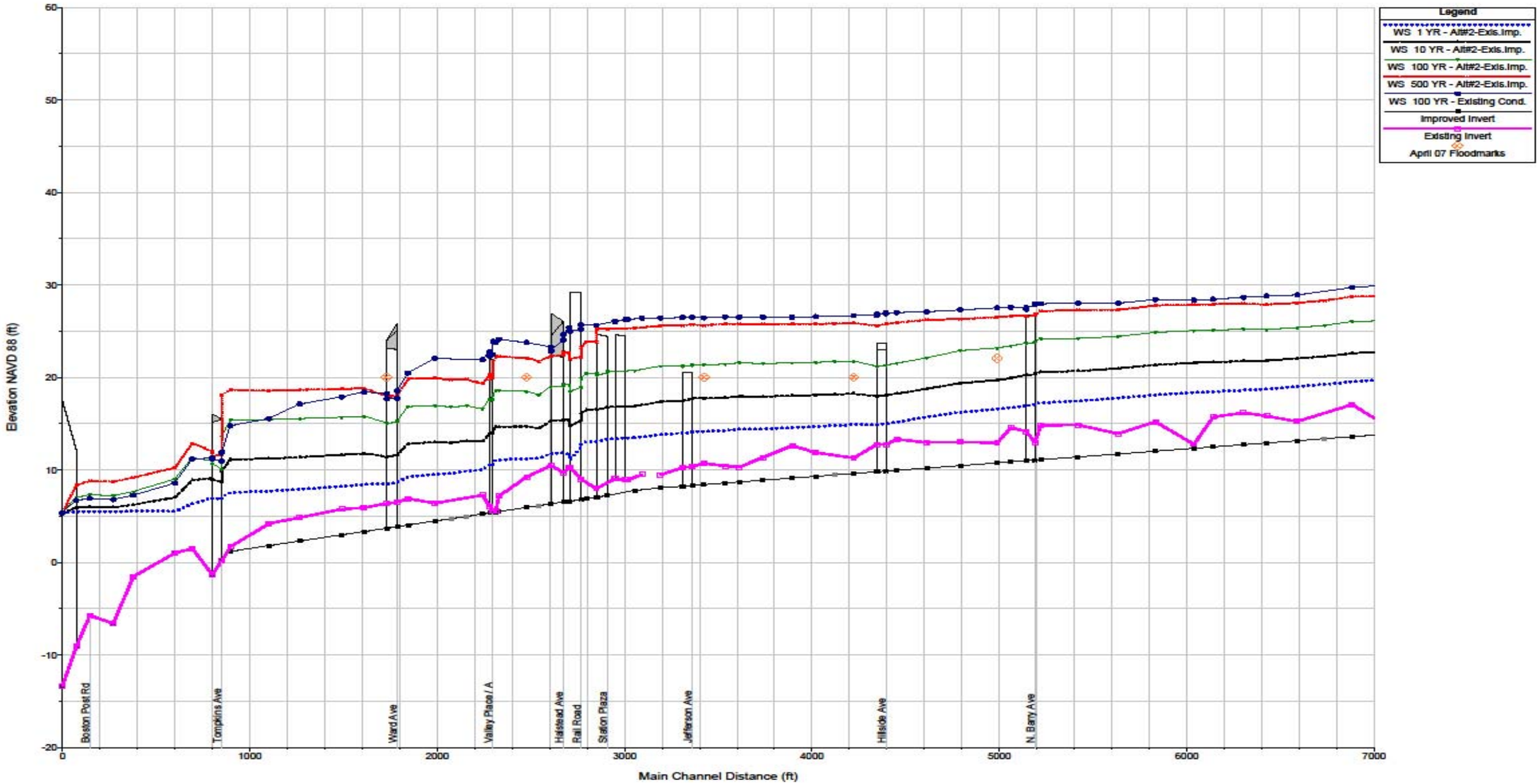

Figure 20: Alternative 2 | Mamaroneck1 - Improved Mamaroneck Channel

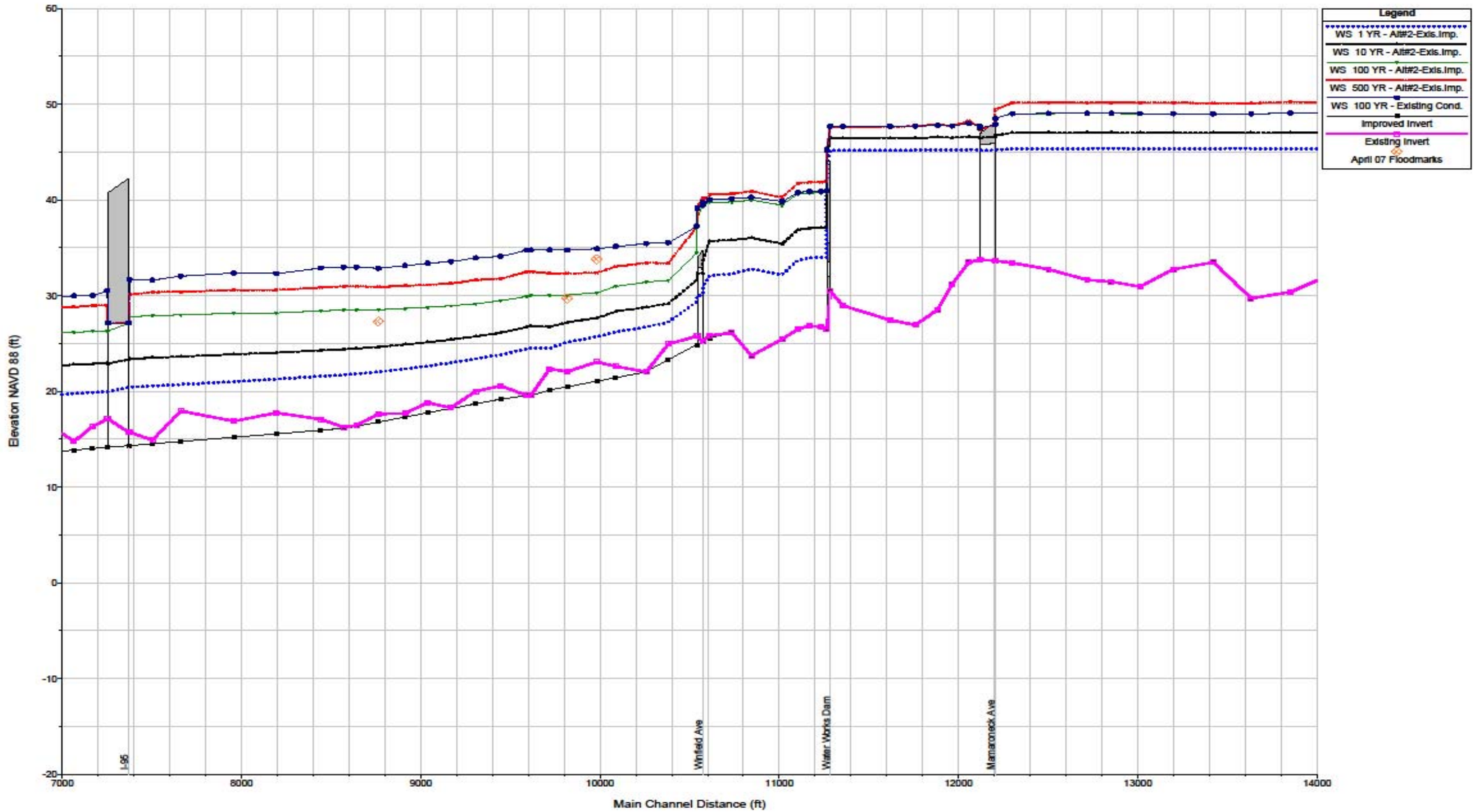

Figure 21: Alternative 2 | Mamaroneck2 - Improved Mamaroneck Channel

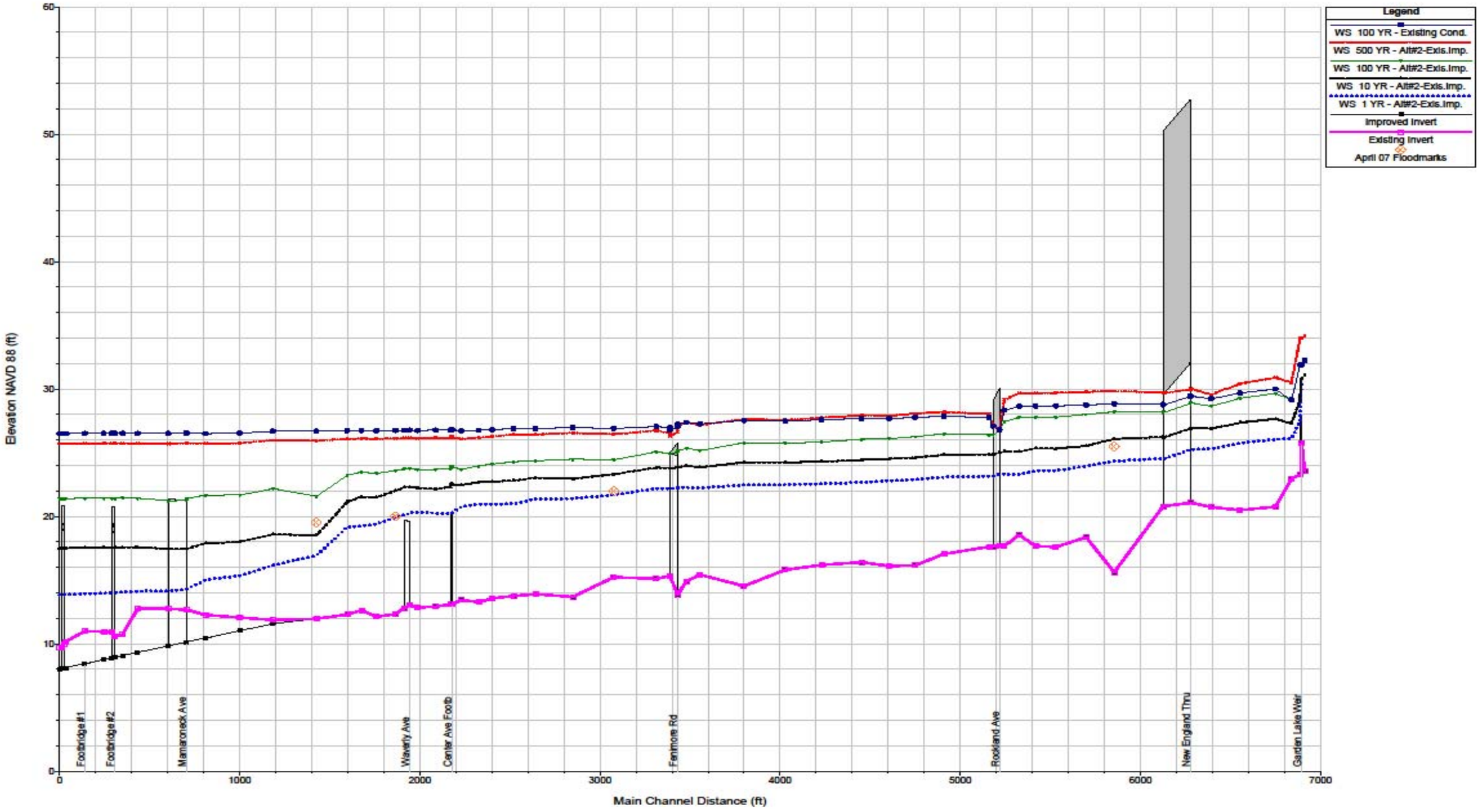

Figure 22: Alternative 2 | Improved Sheldrake Channel

3.2.4 Alternative 3: Channel work along both the Mamaroneck and Sheldrake Rivers

This plan includes Alternative 2 with additional channel deepening along the Sheldrake River as seen in Figure 23. Eight bridges will be removed and/or replaced including Centre Avenue, the two Columbus Park pedestrian bridges, Hillside Avenue, Station Plaza, Halstead Avenue, Valley Place (Anita Lane Sewer Bridge); and Ward Avenue. Along the Mamaroneck River, channel work extents from Tompkins Avenue Bridge to 270ft above Winfield Avenue Bridge, an approximated total length of 6,700ft. The Sheldrake River channel work extents from the confluence to 450ft above the Rockland Avenue Bridge, for an approximated total length of 6,700ft; a significant amount of retaining walls will be used for this alternative. The river will be significantly realigned throughout the confluence and just below the Ward Avenue Bridge. Trapezoidal channelization will consist of a natural bed channel with side slopes of one vertical on two and a half horizontal (1:2.5), and concrete retaining walls will be used where space is limited. Articulated concrete mats will be used throughout and just downstream of Winfield Avenue Bridge due to the high stream velocities. A rectangular channel with concrete retaining walls and channel bottom is needed from the Railroad Bridge to the Halstead Ave. Bridge. Columbus Park will be use as the staging area during construction. Water surface elevations of the various hypothetical events with Alternative 3 were computed using the HEC-RAS computer program (version 4.1) and drawn as shown in Figure 24, 25, 26.

Figure 23: Alternative 3 | Map showing Channel work along both the Mamaroneck and Sheldrake Rivers

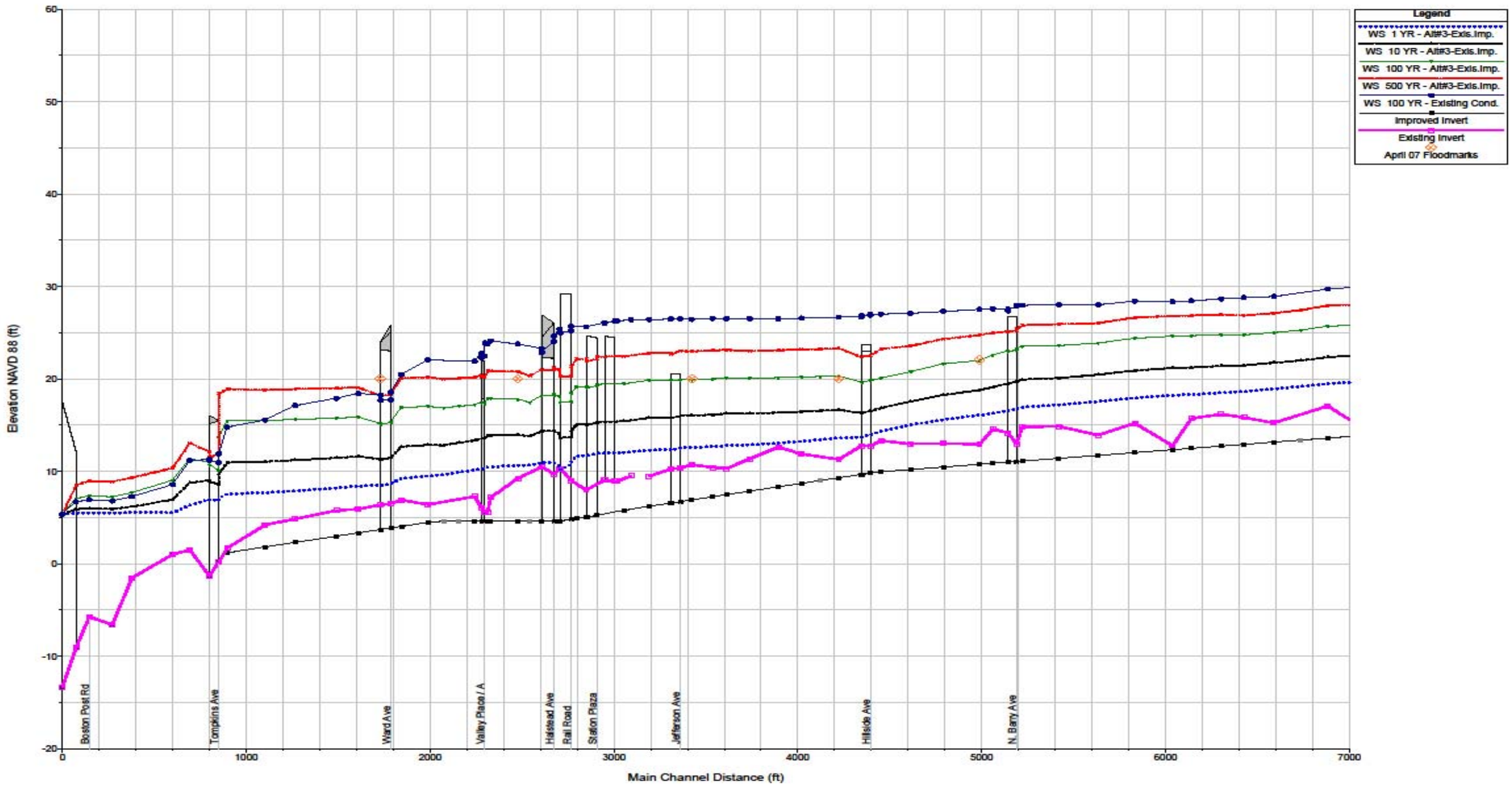

Figure 24: Alternative 3 | Mamaroneck1 - Improved Mamaroneck Channel

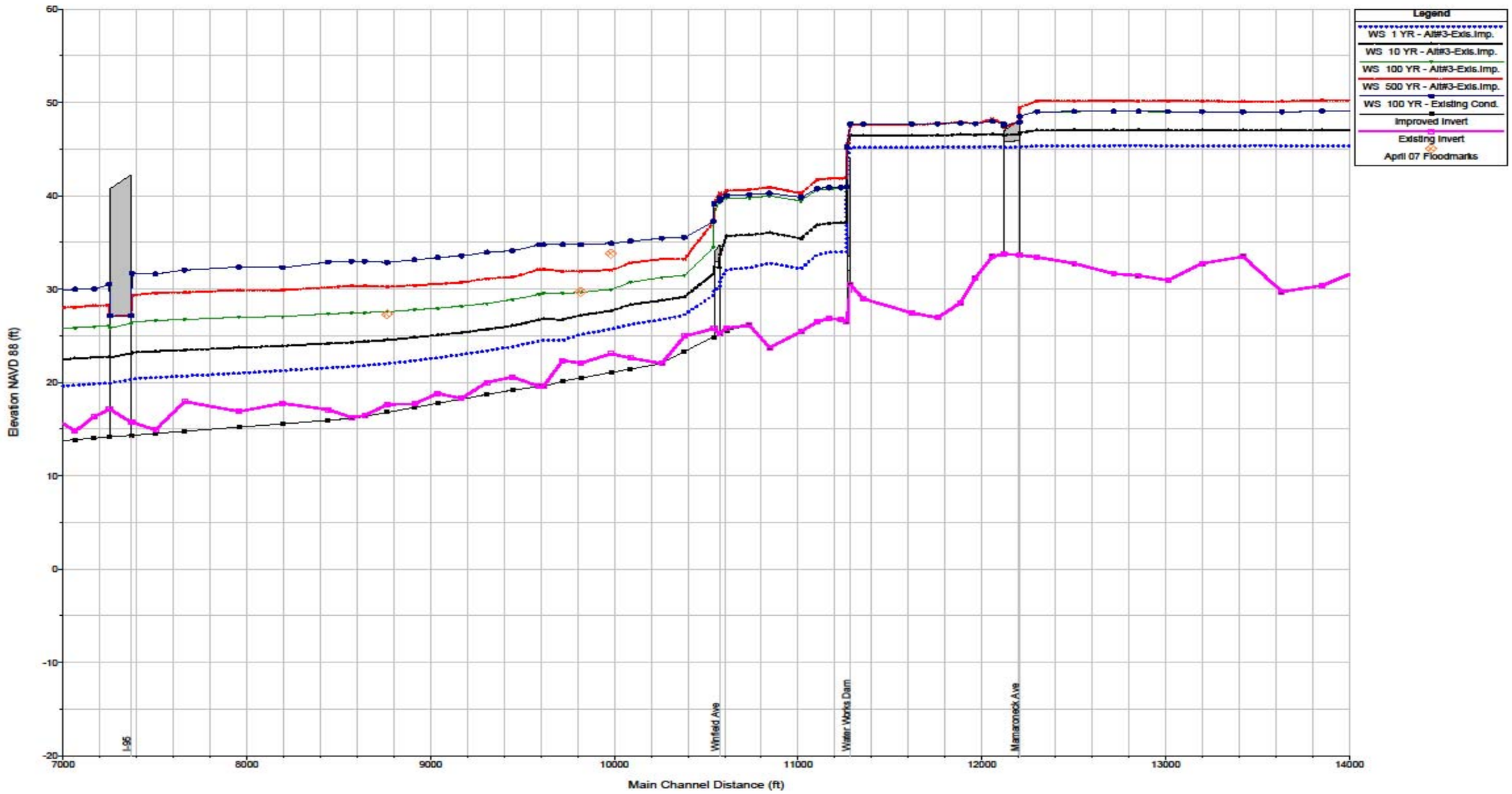

Figure 25: Alternative 3 | Mamaroneck2 - Improved Mamaroneck Channel

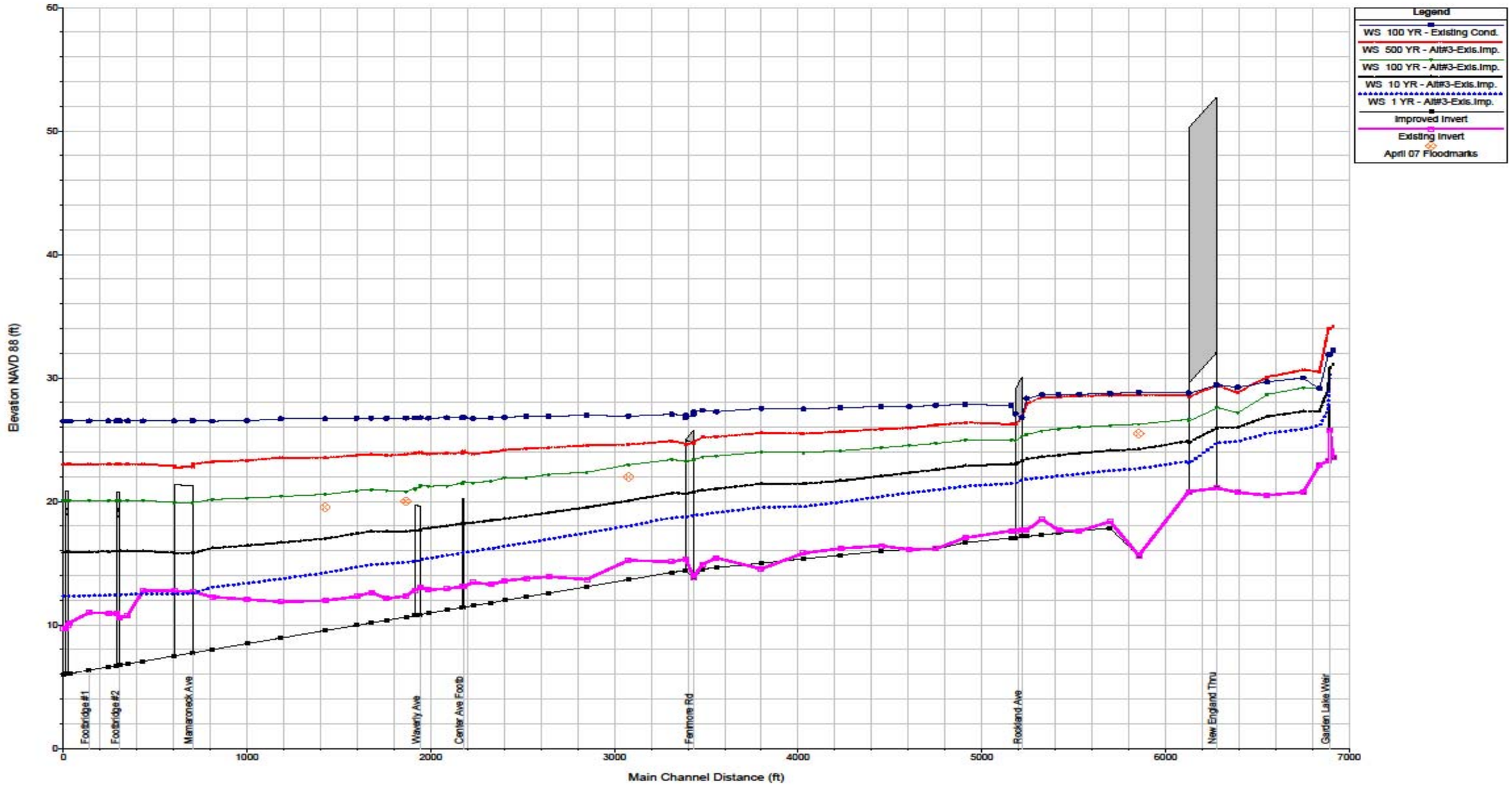

Figure 26: Alternative 3 | Sheldrake Improved Channel

3.2.5 *Alternative 4: Fenimore Avenue Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers.*

The 1989 GDM river diversion and channel improvements plan consists of a tunnel system running beneath Fenimore Rd. from the Sheldrake River to the West Basin of Mamaroneck Harbor as seen in Figure 27. This 16 ft wide by 16 ft high tunnel system which was approximately 4,010 ft in length, is comprised of an inlet structure, the tunnel works and the outlet structure. Channel work in the Mamaroneck River includes a trapezoidal channelization consisting of a natural bed channel, 45 to 60 ft wide. Side slopes of one vertical on three horizontal (1:3) with concrete retaining walls where space is limited. Sheldrake improvements extend from the Mamaroneck Avenue to I-95 with a trapezoidal channel with a natural bed channel 30 ft wide. The 1989 GDM improvements on the Mamaroneck and Sheldrake Rivers were expected to contain the 200-year frequency flood, or 0.5% exceedance probability event. Water surface elevations of various frequency events for Alternative 4 were computed using the HEC- RAS computer program (version 4.1) and drawn as shown in Figure 28, 29, 30.

Figure 27: Alternative 4 | Map showing Fenimore Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

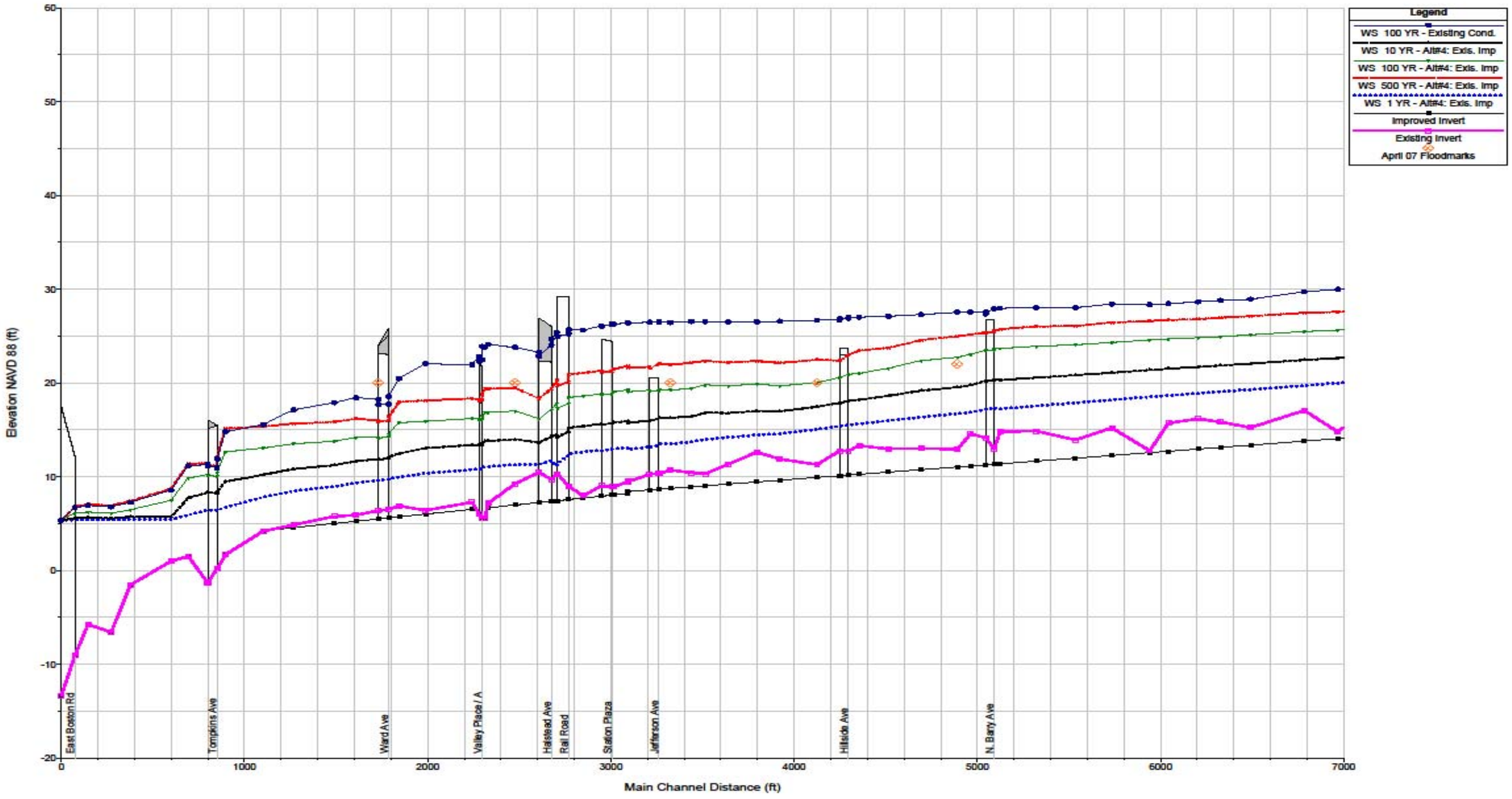

Figure 28: Alternative 4 | Mamaroneck1 - Fenimore Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

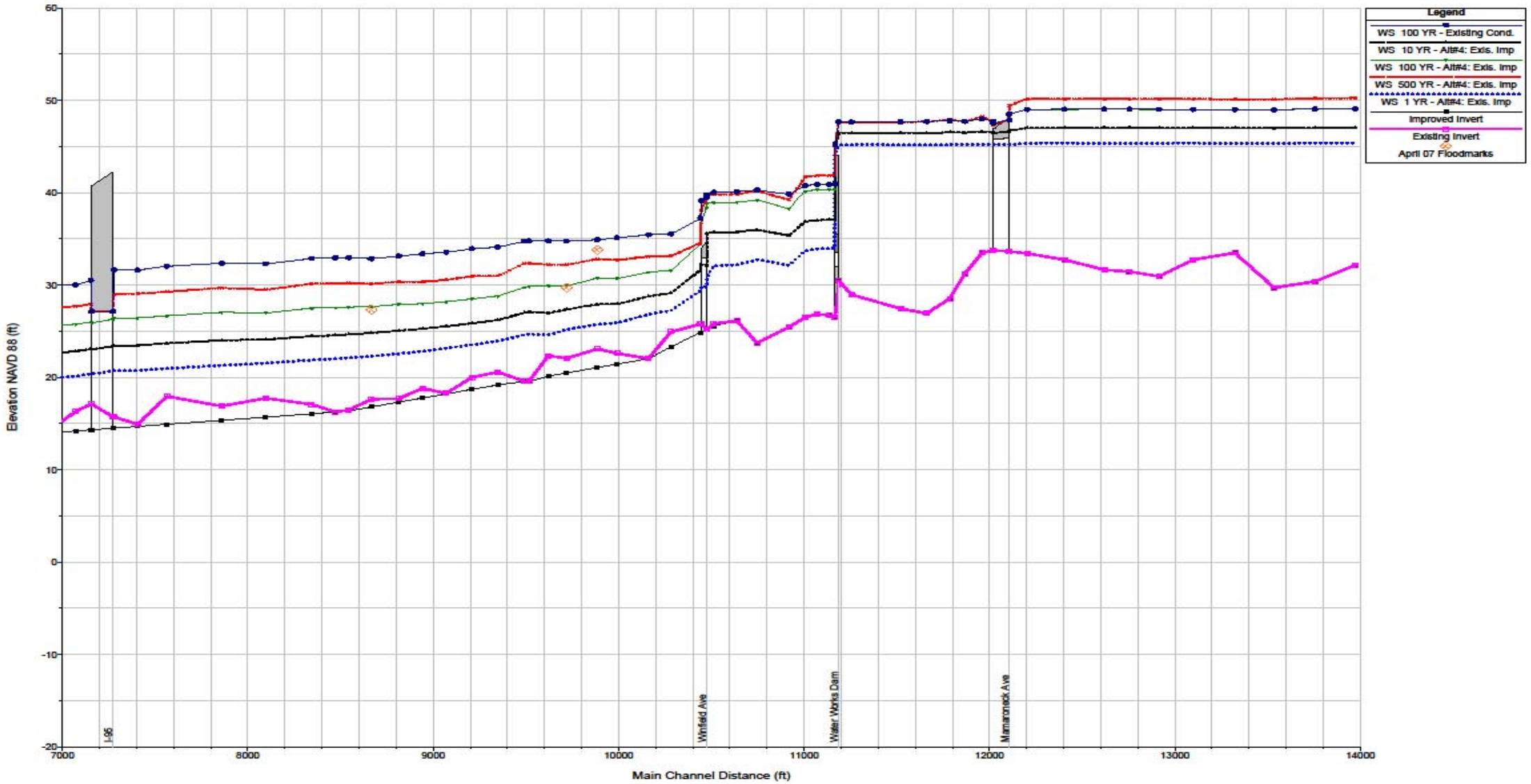

Figure 29: Alternative 4 | Mamaroneck2 - Fenimore Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

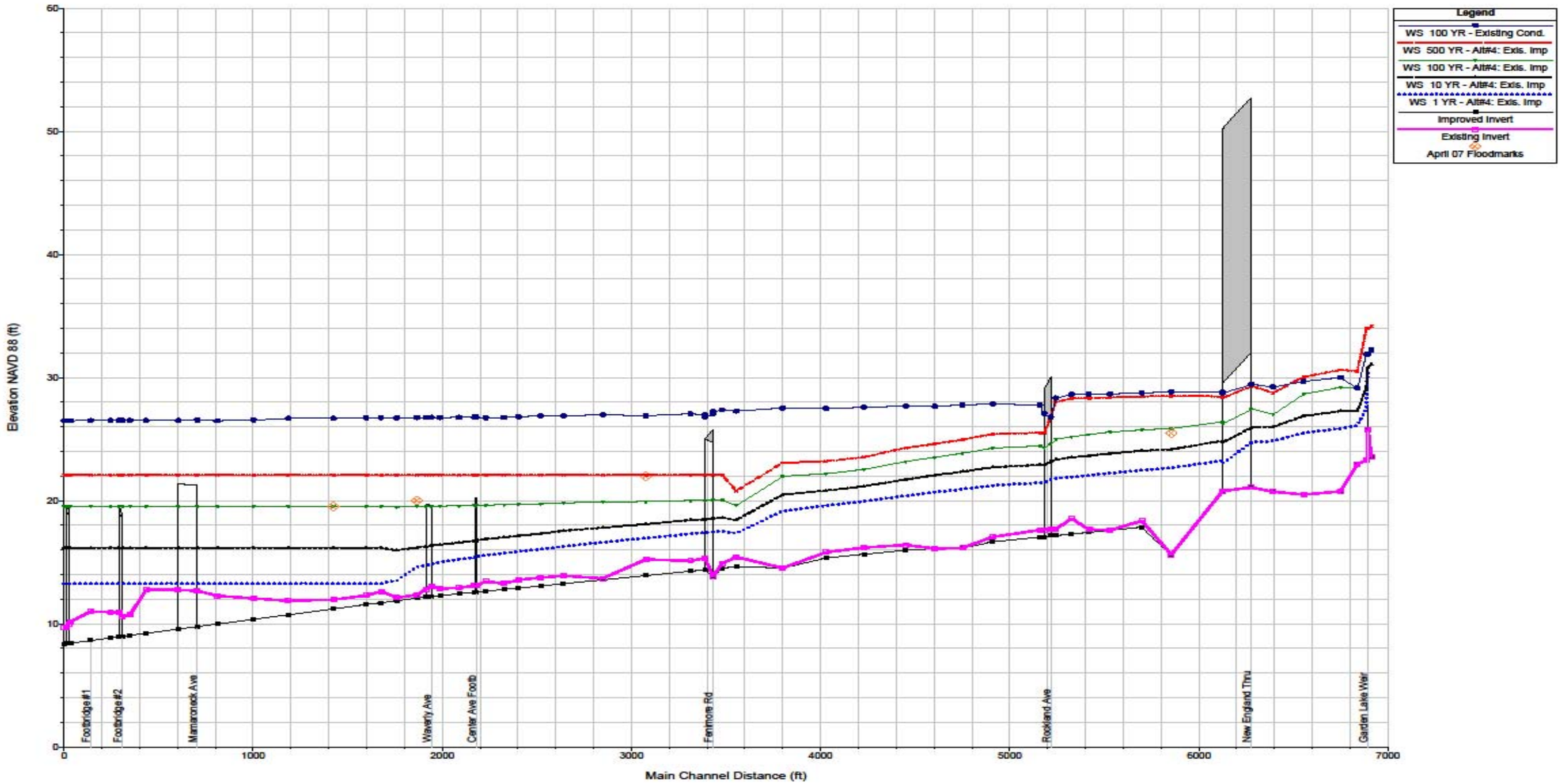

Figure 30: Alternative 4 | Sheldrake - Fenimore Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

3.2.6 Alternative 5: Ward Avenue Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers.

This plan includes channel works throughout the Mamaroneck and Sheldrake Rivers. Five bridges will be removed and/or replaced including Centre Avenue, and the two Columbus Park pedestrian bridges, Hillside Avenue, Ward Avenue. The Ward Avenue Bridge will be relocated approximately 20 ft upstream of its current location to allow the proposed tunnel to discharge downstream of the bridge. Because of the relocation, approximately 130 ft of approach road on each of the bridge side is going to be impacted by construction. Along the Mamaroneck River, channel work extents from Tompkins Ave. Bridge to 270 ft above Winfield Avenue Bridge, for an approximate length of 6,700 ft. In the Sheldrake River, channel work extents from the confluence to 450 ft above the Rockland Avenue Bridge, for an approximate length of 5,700 ft. Trapezoidal channel improvements will consist of a natural bed channel with side slopes of one vertical on two-and-a-half horizontal (1:2.5), with concrete retaining walls where space is limited by structures and private properties. Columbus Park will be use as the staging area during construction. A diversion tunnel with an ogee spillway is to be constructed just downstream of the confluence between the Mamaroneck and Sheldrake Rivers. Ward Avenue tunnel was designed for a 1 % event. The ogee spillway will be approximately 5.3 ft in height and 40 ft long. The diversion tunnel, approximately 1,050 ft in length and 13 ft in diameter, will start at the confluence and run under the railroad and under Ward Ave., discharging back into the Mamaroneck River just downstream of a new Ward Ave. Bridge as seen in Figure 31 and Figure 32. Water surface elevations of various hypothetical events for Alternative 5 were computed using the HEC-RAS computer program (version 4.1) and drawn as shown in Figure 33 through 35.

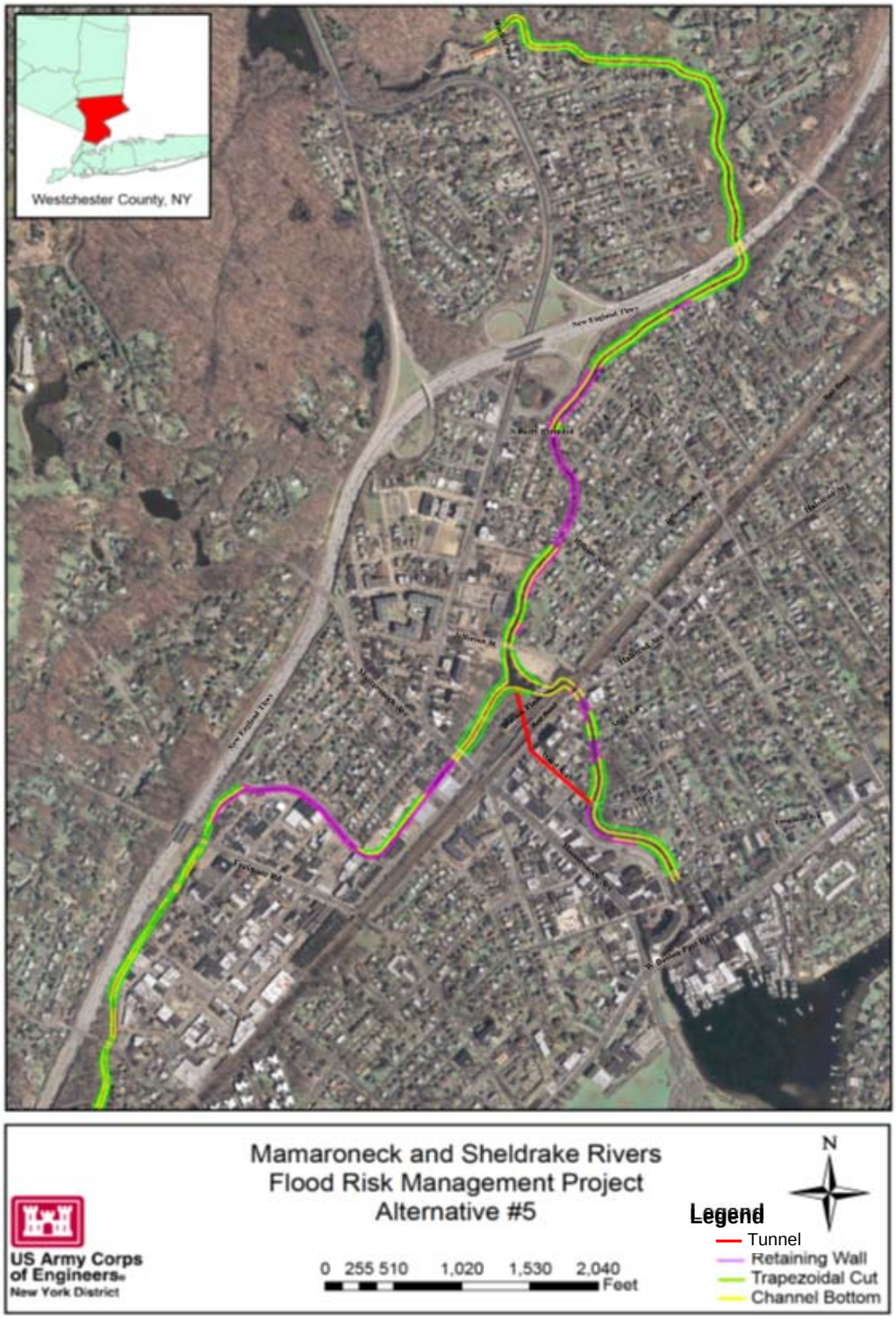

Figure 31: Alternative 5 | Map showing Ward Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

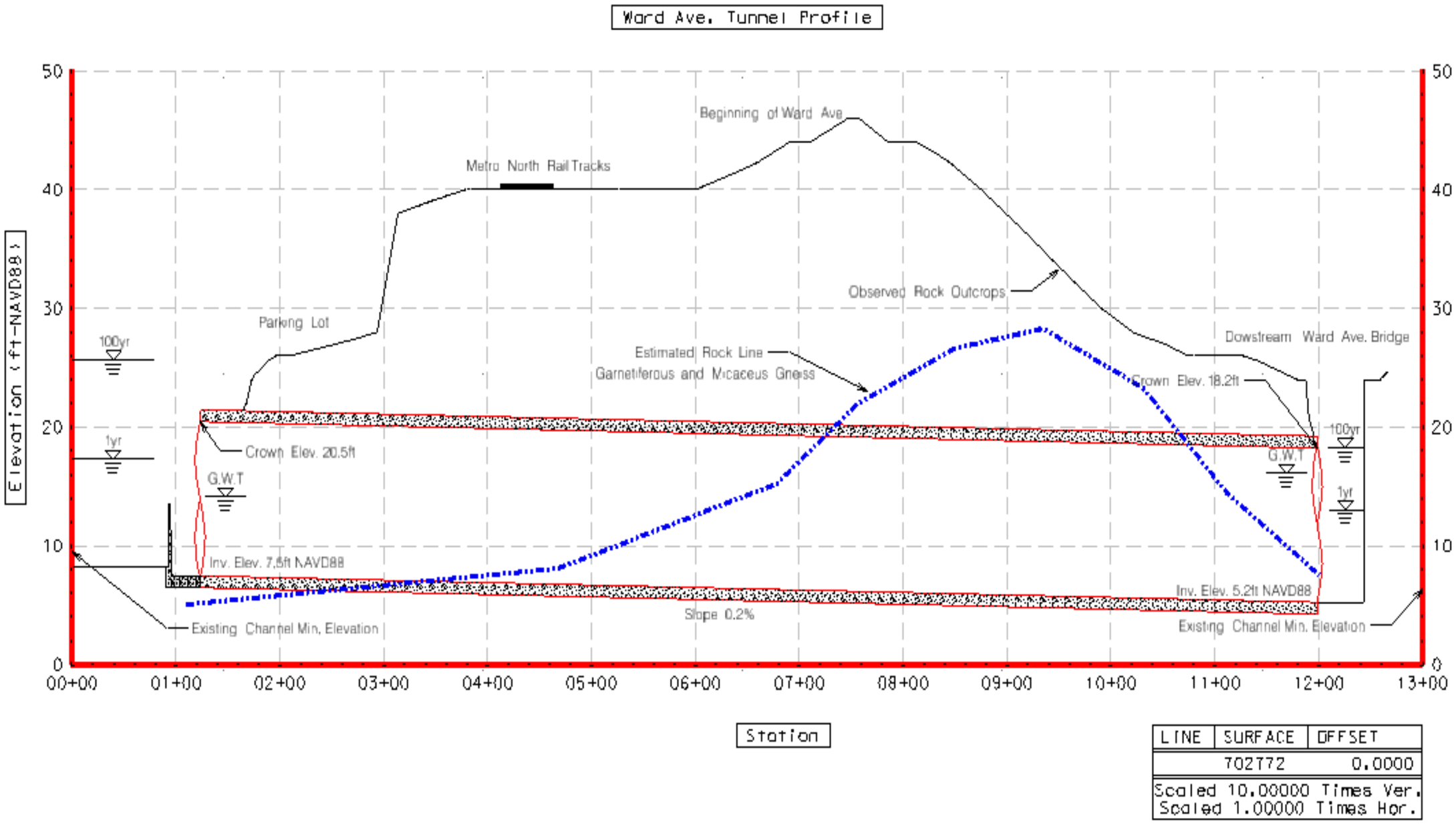

Figure 32: Alternative 5 | Ward Tunnel Preliminary Profile

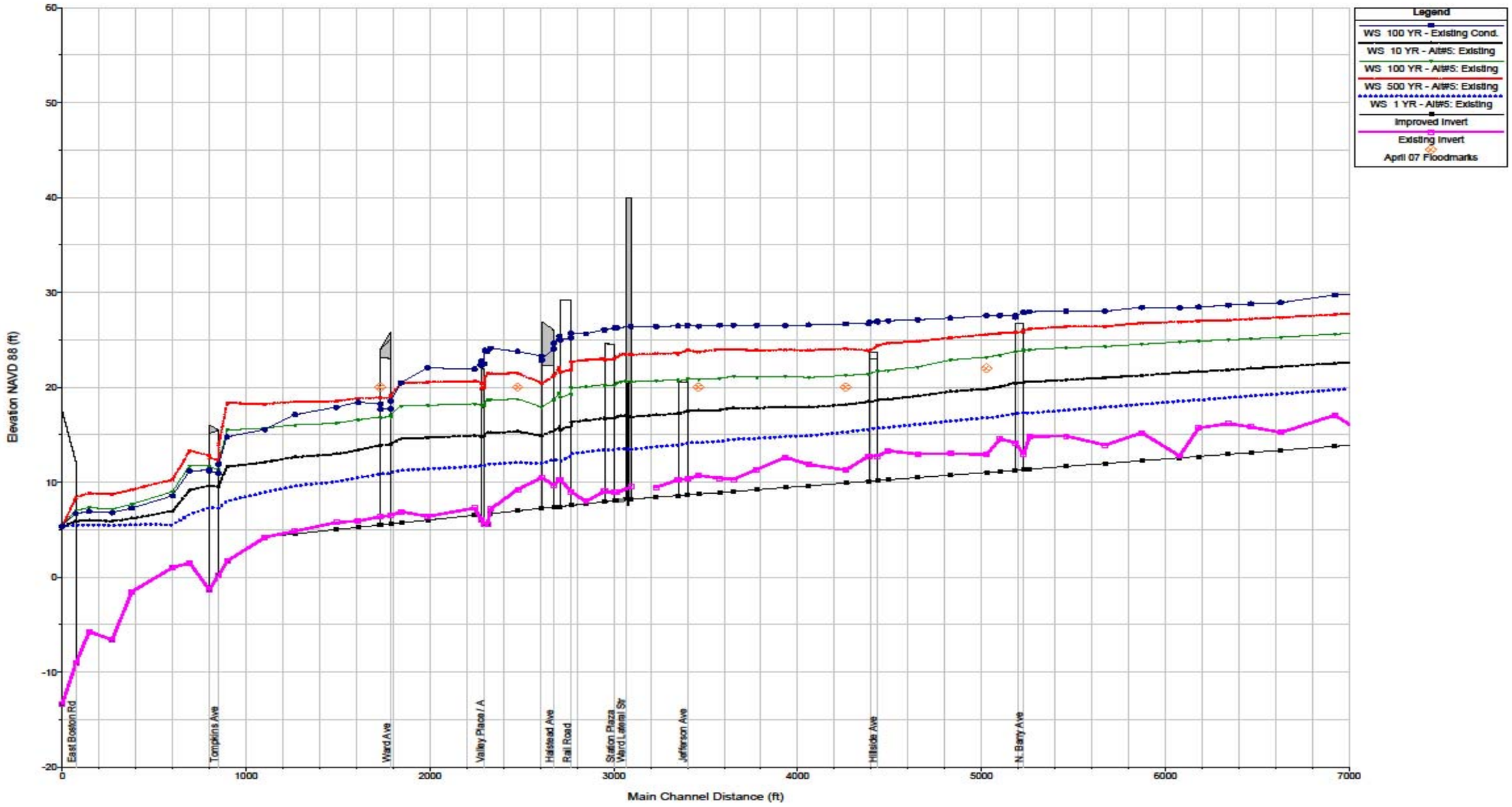

Figure 33: Alternative 5 | Mamaroneck1 - Ward Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

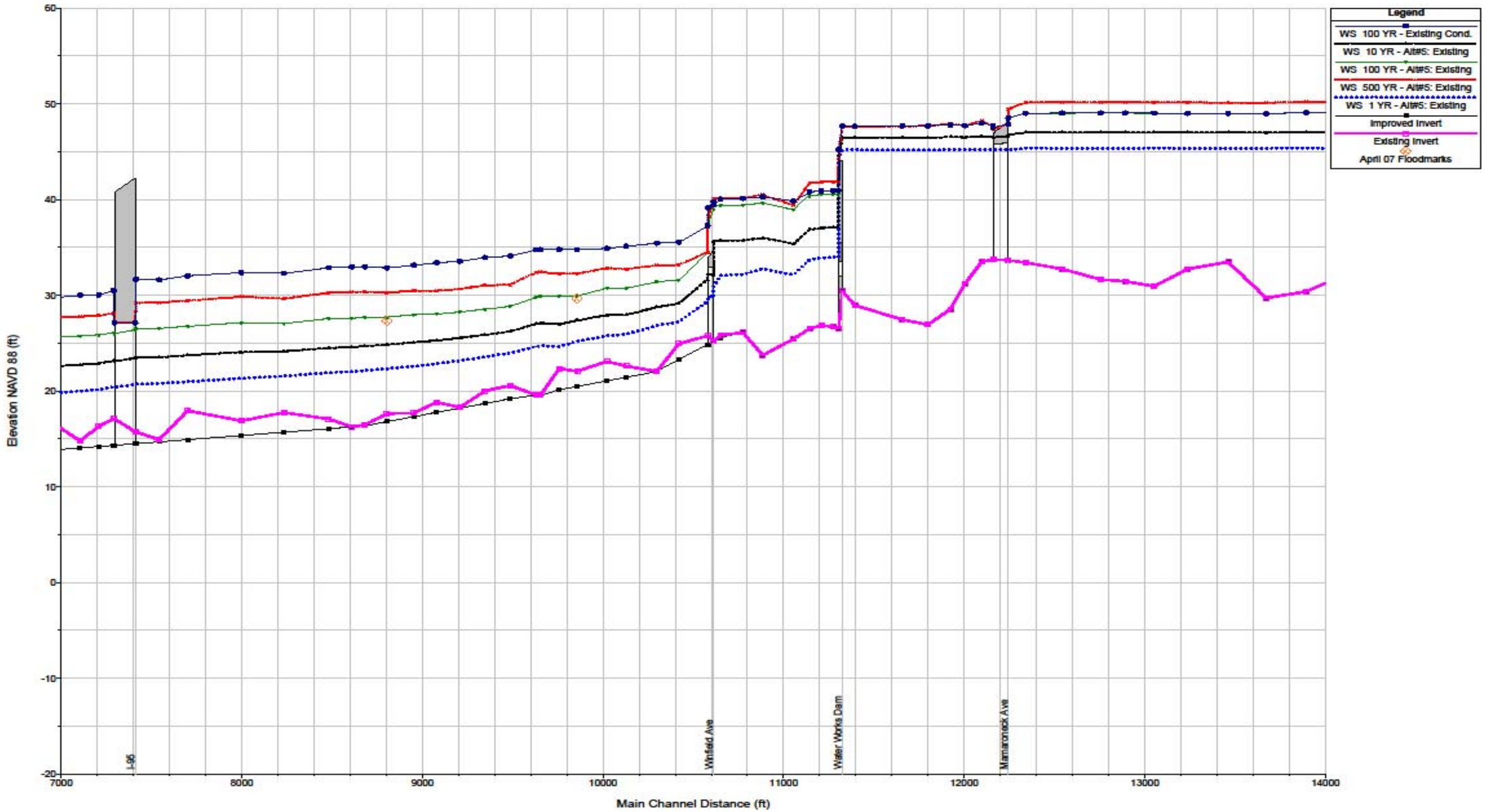

Figure 34: Alternative 5 | Mamaroneck2 - Ward Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

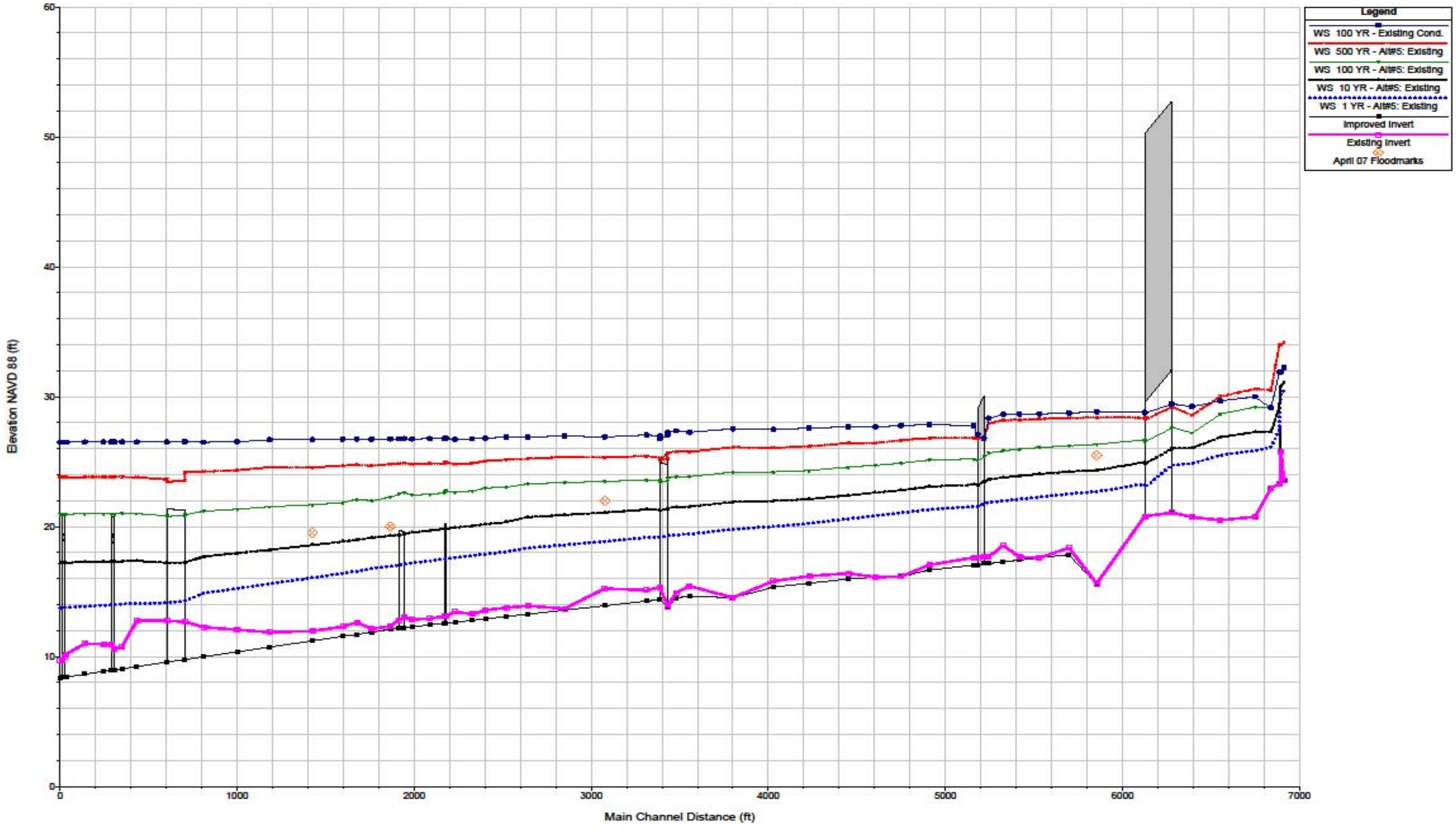

Figure 35: Alternative 5 | Sheldrake - Ward Ave. Tunnel with channel work throughout the Mamaroneck and Sheldrake Rivers

3.2.7 Alternative #6: Non-Structural Alternatives

Non-structural flood risk management measures are authentic techniques for reducing accountable flood damages within floodplains. These techniques mainly consist of measures such as relocation, acquisition, flood proofing (wet/dry), elevation, barriers (ring berms and floodwalls), flood warning system, flood emergency preparedness plans and public education. Some of the measures (i.e., flood proofing and rising) maintain residential, commercial and industrial areas, reducing flood damages through modifications of the existing structures. Others are more invasive non-structural measures like buying and removing low lying high risk properties from the floodplain. These non-structural measures are generally used for the reduction of damages for frequently flooded properties events (i.e., 25 year event or below).

The non-structural measures to be considered in the feasibility study of the Mamaroneck and Sheldrake River project includes dry flood proofing (e.g., sealing basement windows on residential properties), wet flood proofing, elevation (raising buildings) and barriers (ring floodwalls/ring berms). There is a well-established and functional flood warning system, flood emergency preparedness and public education plan in the Village of Mamaroneck, therefore, no further analysis of this matter was performed. Relocations and acquisitions were not considered in this stage of the analysis. Floodplains corresponding to a flood frequency of 2, 10, 100 years were evaluated without considering future sea level change.

3.2.7.1 Level of Protection

A non-structural component was formulated into specific alternative plans for evaluation. The village of Mamaroneck specifically requested that a 100-year level-of-protection (LOP) be evaluated; therefore one of the nonstructural plans was designed to withstand inundation for up to and including a 100-yr return period event (with 1 foot of "freeboard"). This alternative would protect most of the residential and nonresidential

structures on the right bank and some of the residential and nonresidential structures on the left bank from a 100-year flood on Mamaroneck and Sheldrake Rivers.

3.2.7.2 Type of Existing Structures on Mamaroneck and Sheldrake Rivers Floodplain

The types of structures located in the 100-year flood plain of Mamaroneck and Sheldrake river study area are mostly residential and commercial.

3.2.7.2.1 Residential. The predominant land use within the study area is primarily residential with a combination of residential and commercial structures. In the case where the land had both commercial and residential use, a residential use was assumed for the purpose of determining flood protection measures. Structure types were divided into the following categories:

3.2.7.2.1.1 Slab-on-Grade Foundation Type. This structure is constructed on a slab foundation at grade (see Figure 36).

Figure 36: Typical Slab-on-Grade Foundation Type

3.2.7.2.1.2 Raised (Crawlspace) Foundation Type. This structure is on a raised foundation, typically concrete masonry, not high enough for a basement (see Figure 37).

Figure 37: Typical Raised (Crawlspace) Foundation Type

3.2.7.2.1.3 Subgrade Basement. All basements were assumed to be subgrade full (not partial) basements. Typically, one floor equivalent of space is located under the main floor on a slab. The foundation walls may be poured concrete or concrete masonry. The basement may be finished or unfinished. The "subgrade" basement slab is completely below grade on all four sides (Figure 38).

Figure 38: Typical Subgrade Basement Foundation Type

3.2.7.2.1.4 Bi-Levels and Raised Ranches (Slab-on-Grade only). The bi-level structure consists of two stories. In most cases, the first story is partially below grade, consisting of living space or a garage or both. The main floor of the bi-level tends to be above the first story of the structure, with the main entrance located between the lower and main floor (see Figure 39).

Figure 39: Typical Bi-Level Structure Type

3.2.7.2.1.5 The Raised Ranch Structure is similar to a Bi-Level. The lower level is built slab-on-grade and the main entrance is usually at the main floor (2nd level) (see Figure 40).

Figure 40: Typical Raised Ranch Structure Type

Due to the similarities between the characteristics of bi-levels and raised ranches, these structure types were considered identical for flood proofing alternatives screening. Elevation methodology and costs are generally similar to structures with basements.

3.2.7.2.1.6 Split Levels (Slab-on-Grade only). This structure consists of three levels: a stacked lower and upper level, with an adjacent main floor between the upper and lower floor levels. Each floor (lower, main, and upper) has a different elevation, and is connected by short stairways. The lower level is generally on a slab foundation and the main floor is usually raised. The lower level may be living space and/or a garage (see Figure 41). The main entrance is at the main floor level. There was assumed to be no basement below the main level.

Figure 41: Typical Split Level Structure Type

3.2.7.2.1.7 Larger Residential. This category included multi-family units (>2 families), garden apartments, and townhouses. Foundation types for this category included slab-on-grade, raised crawlspace, or subgrade basement (see Figure 42).

Figure 42: Typical multi-family units

3.2.2.7.2.2 Non-Residential. This category includes commercial, industrial, and municipal structures, where persons would not reasonably be expected to sleep. Construction materials included: Brick, Wood and Metal Frame, Masonry, Measures Considered (see Figure 43).

Figure 43: Typical *Non-Residential unit*

3.2.7.3 Nonstructural Flood Proofing measures considered in this project were dry and wet flood proofing, elevation (aka. raise) and barriers (aka. ringwall).

3.2.7.3 .1 Dry Flood Proofing Methods. Dry Flood proofing using veneer walls involves the construction of a waterproofed wall attached to the structure with sealed openings at all entrances. A ring wall or levee may also be constructed to protect an individual structure or small group of structures. Dry flood proofing methods allow flood waters to reach the structure but diminish the flood threat by preventing the water from getting inside the structure walls. Dry flood proofing measures considered in this assessment make the portion of a building that is below the flood level watertight by attaching watertight closures to the structure in doorway and window openings. It is suitable for flooding of depths less than 2 feet, and is also referred to as "Sealant and Closures" in this document (see Figure 44)

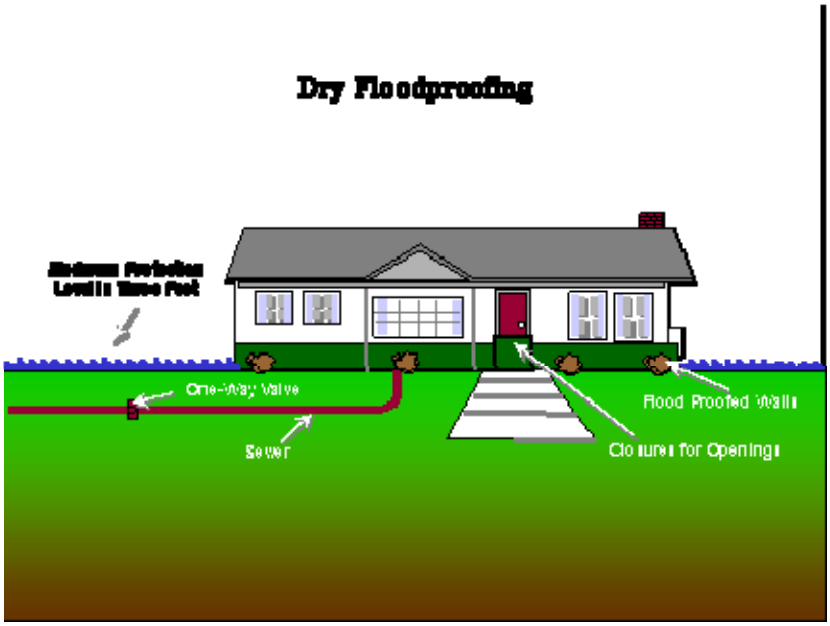

Figure 44: Dry Flood Proofing Diagram

3.2.7.3 .2 *Wet Flood Proofing*. Wet flood proofing measures allow flood water to get inside lower, non-living space areas of the structure via vents and openings in order to reduce the effects of hydrostatic pressure, in turn, flood-related damages to the structure's foundation are reduced (see Figure 45).

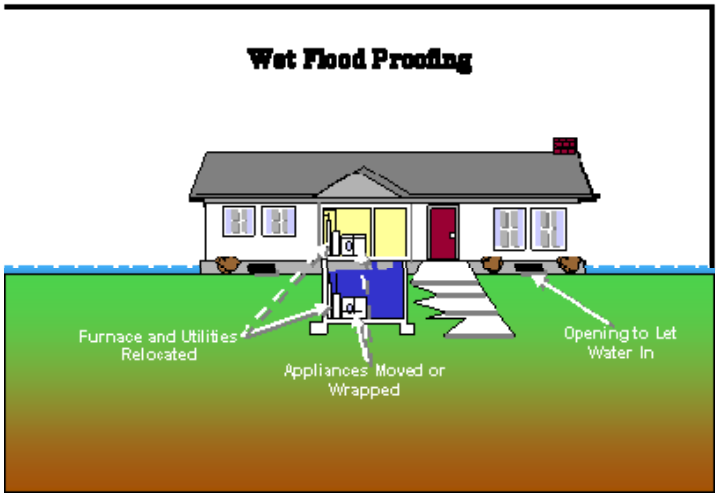

Figure 45: Wet Flood Proofing Diagram

The wet flood proofing *for utilities* would protect the Utilities and other critical equipment. This can involve raising machinery, critical equipment, heating and cooling units, electrical outlets, switches, and panels and merchandise/stock permanently above the estimated flood water height (see Figure 46).

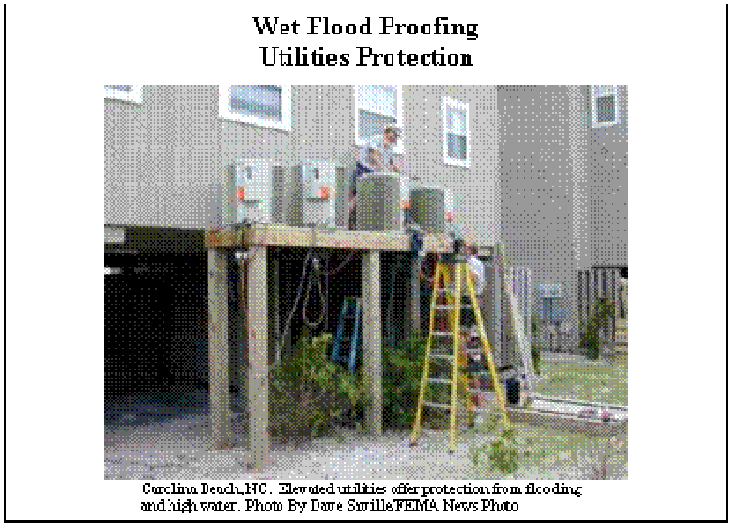

Figure 46: Wet Flood Proofing Utilities Protection

It can also involve construction of interior or exterior floodwalls, utility rooms, or additional living space (to compensate for space lost due to floods), and the use of flood resistant materials wherever possible to further reduce damages. It is suitable for all flooding depths and can involve a range of features such as raising the air conditioning (A/C) unit only, raising the A/C unit and adding flood louvers, adding a utility room, filling the basement, and raising exterior utilities; and raising and rebuilding main floor (in bi-level type structures, etc.).

3.2.7.3 .3 Elevation on Extended Foundation Walls. Elevation involves raising the lowest finished floor of a building to a height that is above the flood level (see Figure 47).

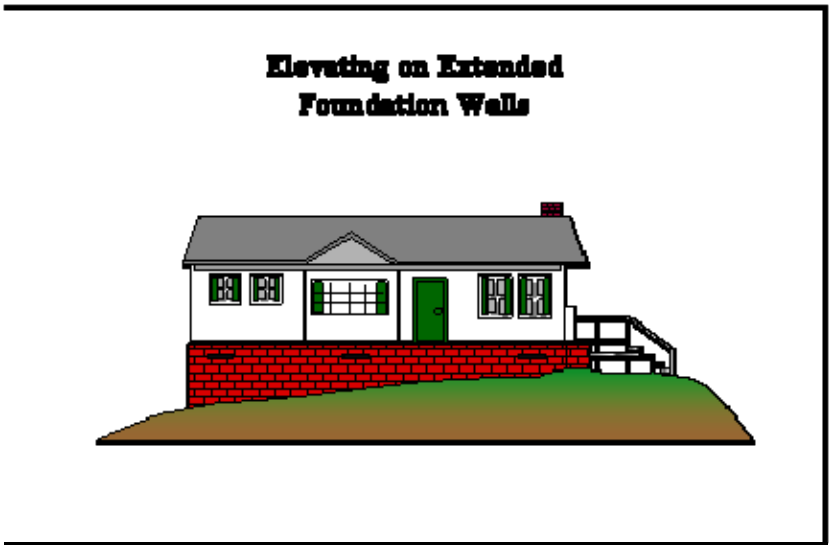

Figure 47: Elevation on Extended Foundation Walls

Under the structural raising option, the damageable structure would be raised to a target elevation. In some cases, the structure is lifted in place and foundation walls are extended up to the new level of the lowest floor. In this study area, no structures are elevated on piers, posts, or piles. The elevation process differs for different foundation types: slab-on-grade, sub grade basement, walkout basement (the majority of structures in the study area had subgrade basements, the assumption was made that all structures with a

basement were of sub grade type for this study), raised (crawlspace) foundation, bi-levels or raised ranches, or split levels.

3.2.7.3 .4 *Barriers*. Usually surround the building but are not attached, such as in the case of ring walls, levees, or berms. It is used where the elevation isn't feasible. Due to the density of structures in the study area, only ring walls were considered (see Figure 48)

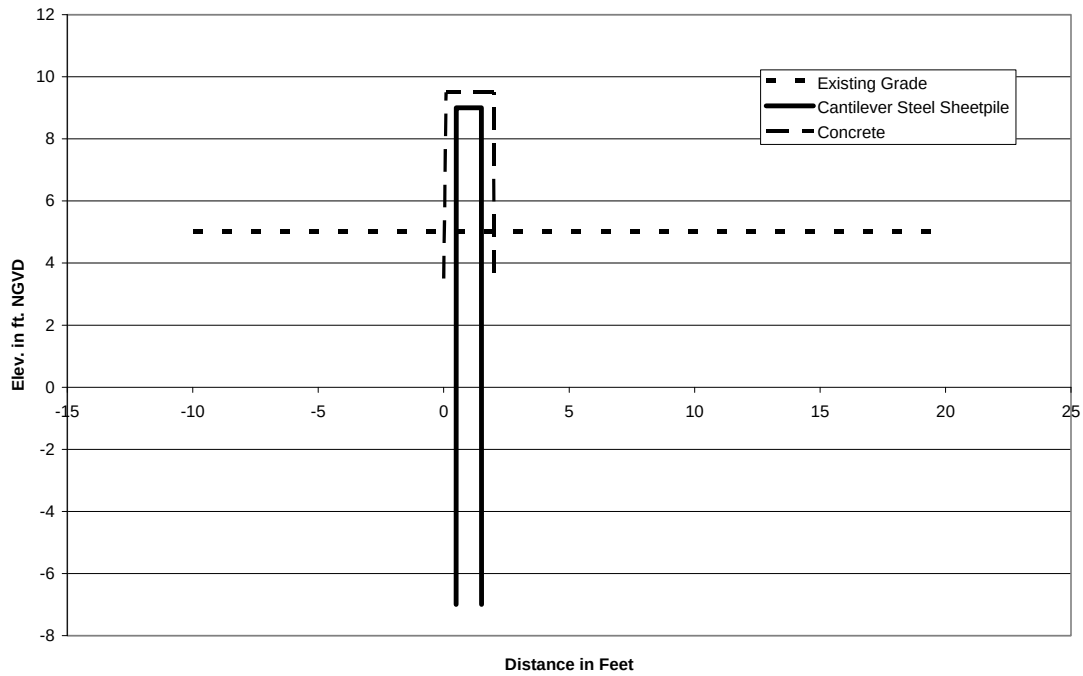

Figure 48: Barriers typical cross-section

3.2.7.4 *Non-Structural Screening Level Analysis Design Assumptions*

Several assumptions were made for design and unit cost development because this was a screening level analysis as described below in Table 11.

3.2.7.4.1 *Screening Level Algorithms*. The analysis applied generalized algorithms to a database of structures. The algorithms use flood levels along with information about

each structure (i.e., ground elevation, main floor elevation, type of construction, etc.) to determine the appropriate method of flood proofing. It should be noted that this was a screening level analysis. Actual determination of which type of flood proofing is most appropriate for a specific building would need to be determined by examining individual structures and site specific conditions.

Some abbreviations used throughout the report are defined as follows:

- Flood Elevation (FE) is the water surface elevation of the storm event (not including sea level rise).
- Protection Elevation (PE) is one foot above the flood elevation
- Main Floor Elevation (MFE) is the elevation of the main living area
- Ground Elevation (GE) is the minimum elevation of the grade surrounding the structure
- MFE+3 is the elevation 3 feet above the main floor elevation
- GE+3 is the elevation 3 feet above the ground elevation

Table 11: Assumptions Inherent to the Screening of Non-Structural Alternatives for Representative Buildings

General Assumptions	• Flood velocity is negligible. • Debris impacts will not be considered. • The area is considered non-coastal and thus not subject to wave and erosion impacts. No areas were designated as "V-zone" by FEMA, subject to 3-foot breaking waves. • Buildings elevated will be raised (finished floor elevation) to the 100-year water surface plus 1 foot of freeboard. • Flooding is gradual (no flash flooding).
Foundation Walls	• All basement foundation types are assumed to be unreinforced, 8" concrete masonry units (CMUs).
Raised Structures (Crawlspace)	• No utilities are located in the crawlspace. • Wet flood proofing of raised structures includes the elevation of utilities only.
Slab-On-Grade Structures	• Wet flood proofing is possible if the expected flood elevation is below the main floor (shallow flooding). This alternative includes the elevation of utilities only. • Consistent with Corps' flood proofing guidance, structures will not be dry flood proofed for flooding depths greater than 2 feet with a maximum 3 feet of dry flood proofing protection (See Attachment 1 for supporting calculations).
Structures With Basements	• All basements are unfinished and contain major utilities.
Bi-Levels	• The lower portion of the first floor walls are masonry construction. • The foundation is slab-on-grade. • The main floor can be raised separately from the lower level by lifting off the sill of the masonry wall.
Raised Ranches	• The first floor (lower) walls are masonry. • The foundation is slab-on-grade. • The main floor can be raised separately from the lower level (similar to a structure with a basement).
Split-Levels	• The lower level is slab-on-grade. • The lower portions of the lower level walls are masonry construction. • The main floor level is raised over a crawl space. • The main floor and upper level can be separated from the lower level by rising at the sill.

3.2.7.5 Screening Level Results. Results of the screening levels analysis using the algorithms by structure type are shown on Table 12 for all three floodplains (100, 10, and 2-year). Table 12 identifies the number of residential and non-residential structures targeted for treatment in the 50%, 10% and the 1% annual chance of exceedance non-structural plans. Table 12 also identifies the number of structures identified for each of the different types of non-structural treatments. All structures will be treated to the 100 year (1% chance of exceedance event) level plus an additional foot of freeboard regardless of the size of the non-structural plan, therefore while the number of structures treated under each plan changes, the design water level of treatment for each structure does not vary by plan. Finally, the identification of structures and types of treatment is only a computer screened identification at this point; should a non-structural features be selected for implementation then a more detailed analysis of each structure and each treatment would have to be conducted. The home owners would also be consulted before final determination on any non-structural treatment was implemented.

Table 12: Structure and Treatment Type as identified in the Screening Process of the Non-Structural Alternatives for the 50% (2 yr.), 10% (10 yr.) and 1% (100 yr) events

Nonstructural Flood Proofing Measure	**50% (2-yr) Annual Exceedance**			**10% (10-yr) Annual Exceedance**			**1% (100-yr) Annual Exceedance**		
	Residential	**Non-Residential**	**Sub Total**	**Residential**	**Non-Residential**	**Sub Total**	**Residential**	**Non-Residential**	**Sub Total**
Dry Flood proofing	0	2	2	0	4	4	9	19	28
Wet Flood proofing	7	0	7	29	0	29	100	1	101
Barriers	1	34	35	11	61	72	16	73	89
Raise	35	0	35	105	0	105	145	0	145
Total of Structures	43	36	79	145	65	210	270	93	363

3.2.8 Alternative 7: Combination Plan

This plan was originally intended to combine non-structural features with structural features. Once Alternative 1 was identified as the TSP and considerable effort was invested in the Alternative 8 Series (described below) the Project Team decided to combine non-structural features with Alternative 1 as part of optimization rather than identifying Alternative 7 it as a separately screened alternative. Alternative 1F and 1L described under Section 4.3 "Optimization of Alternative 1" below are combination plans.

3.2.9 Alternative 8: Concept Identified by Locals for Evaluation – Reservoir and Bridge Plan

This alternative was proposed by NYDEC and Westchester County. It consists of a combination of detention areas, the realignment of the confluence and bridge removal and/or replacement. The plan was designed to limit the improvements or changes to public lands and thereby avoiding the real estate costs associated with purchasing private property. The two primary areas identified for possible detention were the Mamaroneck Reservoir and Sheldrake Lake/Larchmont Reservoir. For the Mamaroneck Reservoir, the plan includes the removal of sediment accumulation near Mamaroneck Avenue Bridge, which is a major cause of ineffective or dead storage within Mamaroneck Reservoir. The new slope in this area is 0.0015ft/ft. In order to maximize storage, which would allow low flow to go through without filling Water Works Dam and Mamaroneck Reservoir, a new lower level outlet design is required. For the existing Sheldrake Lake/Larchmont Reservoir, the plan includes dredging and sediment removal. This adds approximately 85.4 MGa of volume capacity approximately a 50% increase in the reservoir volume. The dam will require an additional 30in diameter pipe below the existing outlet and 5ft above the reservoir is lowest bottom elevation. The last component of the alternative is bridge Modification and/or Removal. The Ward and Winfield Avenue Bridges were completely removed as well as the Glendale Ave abutments (Road to No-where). The Halstead Ave, Valley Place (Anita Lane), Hillside Ave and Jefferson Ave Bridges were modified. The

confluence was re-aligned and Station Plaza Bridge was aligned with the Rail Road and Halstead Ave Bridges. This alternative has been subdivided into five parts: Alternative 8A (a larger Mamaroneck Reservoir with modifications to the Water Works Dam), alternative 8B (a larger Sheldrake Lake/Larchmont Reservoir with modification to the dam, alternative) 8C (Bridge Modifications and/or Removal plan), Alternative 8D (Combination Plan), Alternative 8E (All Inclusive Plan).

3.2.9.1 Alternative 8A: Enlarge the Mamaroneck Reservoir

Alternative 8A involves enlarging the Mamaroneck Reservoir (see Figure 49) to maximize the storage of flood waters from the Mamaroneck River. This modification to the reservoir includes the removal of sediment accumulation near the Mamaroneck Avenue Bridge which is a major cause of ineffective or dead storage within Mamaroneck Reservoir. This will make Mamaroneck Reservoir into a dry detention area. The new slope in this area is 0.0015 ft/ft. The new outlet design was based on passing a 5-yr event or an extra 1000 cfs before causing a flood. The area has an approximate flow area of $45ft^2$ (or 4,20in pipes) located 5 ft below the existing pipe invert.

Figure 49: Alternative 8A | Mamaroneck Reservoir

3.2.9.2 Alternative 8B: Enlarge and dredge Sheldrake Lake

Alternative 8B involves enlarging and dredging the Sheldrake Lake/Larchmont Reservoir to maximize the storage of flood waters from the Sheldrake River. Modifications to this site include dredging and sediment removal, which adds approximately 85.4MG of volume, an increase of about 50%. Another modification includes providing an additional 30in diameter pipe below the existing outlet 5ft above the reservoir's lowest lake bottom.

3.2.9.3 Alternative 8C: Removal/Modification of Bridges

Alternative 8C involves removing and/or modifying some bridges to reduce the water surface elevations throughout the project area. The Ward and Winfield Avenue Bridges as well as the private road "Road to No Where" were completely removed without

replacement**.** The Halstead Ave, Valley Place (Anita Lane), Hillside Ave and Jefferson Ave Bridges were modified. The confluence was relocated and realigned and Station Plaza Bridge was aligned with the Rail Road and Halstead Ave Bridges.

3.2.9.4 Alternative 8D: Combination Plan

Alternative 8D involved combining Alternatives 8A, 8B, and 8C. The Mamaroneck Reservoir Water Works Dam Modifications, Sheldrake Lake/Larchmont Reservoir Modifications and Bridge Modification and/or Removal were all combined into a single hydraulic model.

3.2.9.5 Alternative 8E: All Inclusive Plans

Alternative 8E modeled the Mamaroneck Reservoir in both HEC-RAS & HMS. Sheldrake Lake was modeled in HEC-HMS with additional storage. Gardens Lake Pond was modeled in HEC-HMS as a reservoir with additional storage based on provided topographic data. Golf Courses were modeled in HEC-HMS with a change in Muskingum Routing ("X" from 0.2 to 0.0). Carpenters Pond and Goodliffe were modeled in HEC-HMS as reservoirs with additional storage. This alternative includes all of Alternative 8D "combination plan", and added a series of smaller storage/retention areas including Garden Lakes Pond, Golf course modifications, Carpenters Pond and Goodliffe Pond. The following Figure 50 shows the location of bridges and pond/reservoirs to be modified, areas to be used as storage as well as the realignment of the Mamaroneck/Sheldrake confluence.

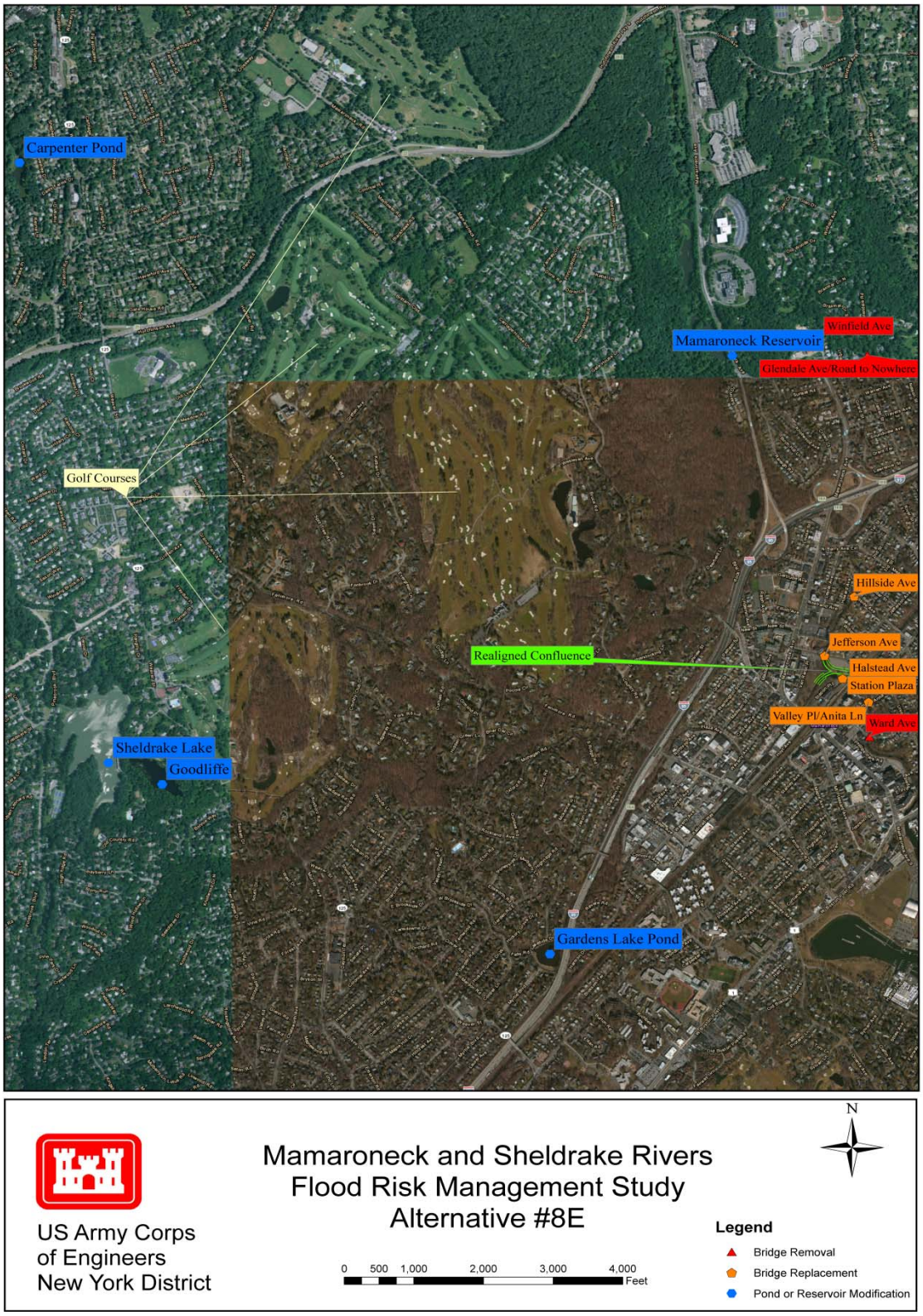

Figure 50: Alternative 8E | All Inclusive Plans

3.2.9.6 Summary Comparison of Alternative Actions

Tables 13 and 14 below include potential actions under the Alternative 8 Series combinations and Alternative 1 to existing condition for the 10 yr and 100 yr event.

Table 13: Water surface reduction (from the existing conditions) for the Alternative 8 Series in comparison with Alternative 1 for the 10 yr event

River	Location	Index Location	*Existing Water Depth (ft)	Reduction in the 10yr WSE ft (Existing -Alternatives)					
				Alt #1	Mamar #8a	Sheldrake #8b2	Bridges #8c	Comb #8d	Ponds #8e
Mamaroneck	Upstream of I-95	5711.7	11.59	0.15	0.88	0.03	0.02	0.90	0.97
Mamaroneck	Columbus Park	841.4	9.23	3.85	0.90	1.41	0.73	1.88	2.32
Mamaroneck	Downstream of Halstead Ave. Br	2420.0	11.98	5.65	0.80	1.31	0.71	2.07	2.50
Sheldrake	Upstream of Mamaroneck Ave Br	1755.2	10.78	1.92	0.39	1.12	0.31	1.39	1.48
Sheldrake	Upstream of Fenimore Ave Br	4309.1	8.42	0.31	0.14	0.97	0.09	1.01	1.12

*Existing Water Depth = WSE - Thalweg

Table 14: Water surface reduction for the Alternative 8 Series in comparison with Alternative 1 for the 100 yr event

River	Location	Index Location	*Existing Water Depth (ft)	Reduction in the 100yr WSE ft (Existing -Alternatives)					
				Alt #1	Mamar #8a	Sheldrake #8b2	Bridges #8c	Comb #8d	Ponds #8e
Mamaroneck	Upstream of I-95	5711.7	15.23	0.28	0.26	0.06	0.09	0.42	0.53
Mamaroneck	Columbus Park	841.4	13.89	5.08	0.30	0.59	1.72	2.63	2.92
Mamaroneck	Downstream of Halstead Ave. Br	2420.0	15.64	5.23	0.38	0.60	0.99	1.79	2.06
Sheldrake	Upstream of Mamaroneck Ave Br	1755.2	14.11	3.30	0.26	0.58	1.27	1.80	1.89
Sheldrake	Upstream of Fenimore Ave Br	4309.1	11.4	1.76	0.21	0.88	0.90	1.73	1.90

*Existing Water Depth = WSE - Thalweg

The conclusion drawn from engineering judgment and these tables was that the Alternative 8 plans would be more costly than Alternative 1 and would not reduce water surface elevation in the main damage areas as much as Alternative 1. A benefit to cost ratio was determined only for Alternative 8e, for the sole purpose of confirming this conclusion.

3.3 Improved Conditions Stage Uncertainty Analysis

The uncertainty in the computed WSELs for the channel plans was assumed to be the same as the uncertainty computed for existing conditions because there was no significant change in the factors impacting stage uncertainties. Alternatives 4, 5 and 8 however, involved either diversions or reservoir operation which introduced an additional factor affecting the stage computation, which required addition uncertainty analysis.

3.3.1 Alternative 4: Uncertainty Analysis for the Fenimore Road, diversion tunnel and channel improvements throughout the Mamaroneck and Sheldrake Rivers.

A physical model of the Fenimore tunnel was constructed and tested in 1986. A Hydraulic Model Investigation (Technical Report HL-86-7) generated by WES in 1986, contains rating curves developed for the Fenimore Rd. tunnel with two different "n" values, 0.01 and 0.013. The diversion flow uncertainty for the Fenimore Rd. tunnel was calculated by using the rating curve for an "n" value of 0.01 as the best diversion scenario and then computing the difference in stage between the low n-value of 0.01 and the high "n" value of 0.013 and adding that difference on to the 0.013 "n" rating curve to establish the worse case diversion scenario. Since this alternative was not expected to be and ultimately was not an economically justified alternative, the uncertainty bands for this alternative, were not further refined.

3.3.2 Alternative 5: Uncertainty Analysis for a tunnel under Ward Avenue and channel improvements throughout the Mamaroneck and Sheldrake Rivers.

Uncertainty associated to the performance of a flow diversion was taken into account. The Ward Ave. tunnel was modeled as a Lateral Structure in HEC-RAS where the culvert was coded and the split flow calculation optimized. The uncertainty of the tunnel was calculated using the following:

From the HEC-RAS split flow calculation, an inflow vs. outflow relationship for the Ward Ave. tunnel was developed for both an upper and lower head loss boundary range. To establish the range of performance for the diversion tunnel the parameters presented in Table 15 were varied to establish the upper and lower performance range. The values presented in the tables apply to base year and future conditions. Three head losses associated with the tunnel were varied from high loss values to low loss values such as Manning's "n" values, entrance minor losses and sediment deposits.

Table 15: Variables to be modified in HEC-RAS for the uncertainty analysis

Band	n-value		*Depth (ft)	Losses		Type of Pipe	Entrance Shape
	Top	Bottom	Blockage	Entrance	Exit		
Worst (upper)	0.020	0.025	3	0.5	1	Concrete Pipe	Square Edge
Best (lower)	0.012	0.012	0	0.2	1	Concrete Pipe	Bevel Ring

*From bottom to top.

3.3.3 Alternative 8: Uncertainty Analysis, with a combination of detention and structural measures in the Mamaroneck and Sheldrake Rivers Basin.

The maximum and minimum outflow values for the upstream reservoirs were determined by routing a set of starting reservoir elevation and reservoir storage capacities using the HEC-HMS model. The actions taken to establish the best and worst case scenarios are shown in Tables 16 and 17 for the Larchmont and Mamaroneck Reservoirs. Storage discharge data are included in Tables 18 and 19. The resulting band of possible outflows was input into HEC-FDA for each frequency.

Table 16: Water Works Dam /Mamaroneck Reservoir uncertainty proposed actions for the low and high bounds

Cases/Band	Action
Worst Case/ High Band	Full reservoir, with WSEL at the spillway crest (i.e. 44ft NAVD88)
	Remove 10% of storage capacity
Best Case/ High Band	Reservoir at the target elevation of 27 ft NAVD88
	Add 10% of storage capacity

Table 17: Larchmont Reservoir uncertainty proposed actions for the low and high bounds

Cases/Band	Action
Worst Case/ High Band	Full reservoir, with WSEL 1ft above spillway crest (i.e. 129ft NAVD88)
	Remove 20% of storage capacity
Best Case/ High Band	Reservoir at the target elevation of 109 ft NAVD88
	Add 1% of storage capacity

Table 18: Storage-outflow relationship for Mamaroneck Reservoir

WSEL	Volume (ac-ft)			Flow (cfs)	Weir (cfs)	Gate (cfs)
	Alt#8a	10%	-10%			
29.1	16.6	18.3	14.8	100.0	0.0	100.0
30.3	31.8	35.0	28.3	200.0	0.0	100.0
32.1	58.9	64.8	52.4	400.0	0.0	200.0
33.7	90.6	99.6	80.6	600.0	0.0	400.0
35.0	122.5	134.8	109.1	800.0	0.0	600.0
36.0	149.9	164.9	133.4	1000.0	0.0	800.0
37.1	183.8	202.2	163.6	1200.0	0.0	1000.0
37.8	204.5	225.0	182.0	1300.0	0.0	1300.0
39.3	254.7	280.2	226.7	1510.0	0.0	1510.0
42.2	363.6	400.0	323.6	1740.0	0.0	1740.0
44.4	456.5	502.2	406.3	1970.0	16.0	1954.0
45.9	536.6	590.3	477.6	2790.0	203.9	2586.1
46.3	564.7	621.1	502.5	3370.0	320.9	3049.1
46.9	601.4	661.5	535.2	4020.0	566.9	3453.1
46.6	594.3	653.8	528.9	4780.0	456.0	4324.0
46.8	604.2	664.7	537.8	4970.0	520.9	4449.1
47.2	638.3	702.2	568.1	5640.0	760.3	4879.7
49.6	908.2	999.0	808.3	9200.0	2864.1	6335.9

Table 19: Storage-outflow relationship for Larchmont Reservoir

Elevation	Volume (ac-ft)			Outflow
(ft NAVD 88)	Alt#8b	+1%	-20%	(cfs)
104.0	0.0	0.0	0.0	0.0
105.0	16.5	16.7	13.2	0.0
106.0	32.4	32.8	25.9	0.0
107.0	48.9	49.3	39.1	0.0
108.0	65.8	66.4	52.6	0.0
109.0	83.2	84.1	66.6	1.3
110.0	101.2	102.2	81.0	24.5
111.0	119.7	120.9	95.8	34.6
112.0	138.7	140.1	111.0	42.3
113.0	158.3	159.9	126.6	48.9
114.0	178.4	180.2	142.7	59.9
115.0	199.1	201.1	159.3	71.9
116.0	220.3	222.6	176.3	80.9
117.0	242.2	244.6	193.7	88.6
118.0	264.6	267.2	211.7	95.6
119.0	287.6	290.4	230.1	102.1
120.0	311.2	314.3	248.9	108.1
121.0	335.4	338.7	268.3	113.8
122.0	360.2	363.8	288.1	119.2
123.0	385.6	389.5	308.5	124.4
124.0	411.7	415.8	329.3	129.3
125.0	438.4	442.7	350.7	134.1
126.0	465.7	470.3	372.6	138.7
127.0	493.8	498.7	395.0	143.1
128.0	522.6	527.9	418.1	150.3
129.0	552.3	557.9	441.9	323.2
130.0	582.9	588.7	466.3	617.3
131.0	614.3	620.4	491.4	993.4
132.0	646.5	653.0	517.2	2465.5

4.0 OPTIMIZATION OF ALTERNATIVE 1, THE TSP

4.1 The Tentatively Selected Plan (TSP)

As a result of the plan formulation process which used stages, frequencies, costs, benefits and an economic analysis indicated that Alternative #1 was the plan with the highest net benefits. Basic environmental, cultural and local considerations were evaluated and Alternative #1 was identified as the Tentatively Selected Plan (TSP).

4.2 Value Engineering (VE)

The findings of this study were evaluated by a value engineering team after the TSP was identified and when optimization of the TSP was nearly complete. The most significant recommendation by the team was to keep the existing Station Plaza Bridge as is and substitute the replacement of that bridge with an overflow diversion culvert under the commuter parking lot. The culvert was to start just downstream of the Jefferson Ave Bridge pass under the parking lot located on the left bank (north side) of the river.and discharge almost directly into the Rail Road Bridge opening.

A hydraulic and an economic analysis of the VE recommendation was performed and the diversion culver should save between $3 and $5 million. The culvert was designed not to change the improved water surface elevation and can be added to most any plan without changing the benefits.

The culvert will be about 390 feet long with a slope of 0.36 feet per 100 feet, 25 feet wide, 8 feet high, will be about 3 feet above the proposed bottom of the river and about 3.5 feet under the finish grade of the parking lot. Therefore, the culvert will be (high and) dry during normal flows but it will divert a portion of the Mamaroneck River flows from just downstream of Jefferson Ave directly to the Railroad Bridge approximately once or twice a year. The channel from the Rail Road Bridge to the Jefferson Ave. Bridge will

still be deepened to the same depths previously specified but the alignment of the river will remain as it currently is. While the two 90 degree bends will remain, the impact of those losses will be reduced since a portion of the river will now flow "straight" through the culvert.

4.3 *Optimization of Alternative 1*

Alternative 1 was simultaneously optimized in size and combined with Non-structural features to determine the size with the maximum net benefits. The plan and size with the maximum net benefits would become the National Economic Development (NED) plan. The width, depth and extent of the initial Alternative 1 channel improvements were varied along with the number of bridge replacements and limited non-structural measures to develop plans that were larger and smaller than the original Alternative 1. After the first three variations of the initial Alternative 1 sizes were completed, another size (Alternative 1F) was developed to help determine the most cost effective variation version of this alternative. After coordination with the Non-Federal and Local Sponsors, Agency Technical Review (ATR) and additional Plan Formulation, one more variation (Alternative 1Z) was developed. Alternative 1Z is the plan size with the highest net benefits. Table 20 is a comparison of the optimized features of each alternative. The river was originally realigned throughout the confluence, but the VE investigation later changed that. Trapezoidal channelization will consist of a natural bed channel with side slopes of one vertical on two-and-a-half horizontal (1:2.5), and concrete retaining walls will be used where space is limited. In addition, riprap will be used in areas with high velocities. Several bridges will be removed and/or replaced. See Structural and Civil layout for final features and quantities. Some of the features involved in the optimization were:

- The medium alternative (Alt. 1M) has some channel work in the Harbor Heights area and the removal of a portion of the Glendale Ave. ("road to nowhere").
- Alternatives 1L, 1F and 1Z are combination plans that have the some nonstructural components along both the Mamaroneck and Sheldrake Rivers.

Along the Sheldrake River, there is a proposed barrier or ringwall around the commercial building located near Fenimore Rd; while along the Mamaroneck River, there are eight proposed raisings or elevated structures in the Harbor Heights residential area.

See Figure 51 for plan view and Figure 52, 53 and 54 for the hydraulic profiles of Alternative #1Z.

Table 20: Comparative Features for Combinations and Optimization

Feature			Alternatives												Units
Bridge Name	**River/Reach**	**Hydraulic River Stationing**	**Alt. 1**		**Alt. 1S - Small**		**Alt. 1M - Medium**		**Alt. 1L - Large**		**Alt. 1F - Final**		**Alt. 1Z**		
Bridge Modification:															
Ward Ave.	Mamaroneck/DS	17+86	Remove/replace		No action		Remove		Remove		Remove		Remove		-
Halstead Ave.	Mamaroneck/DS	26+75	Remove/replace		No action		No action		Remove/replace		No action		No action		-
Station Plaza	Mamaroneck/DS	29+84	Remove/relocate		Remove/relocate		~~(Remove/relocate)~~ *		Remove/relocate		~~(Remove/relocate)~~ *		~~(Remove/relocate)~~ *		-
Footbridge #1 (near confluence)	Sheldrake/Sheldrake	00+25	Remove		Remove		Remove		Remove		Remove		Remove		-
Footbridge #2	Sheldrake/Sheldrake	03+07	Remove		Remove		Remove		Remove		Remove		Remove		-
Waverly Ave.	Sheldrake/Sheldrake	19+43	No action		Remove/replace		Remove/replace		Remove/replace		Remove/replace		Remove/replace		-
Centre Ave. Footbridge	Sheldrake/Sheldrake	21+71	Remove		Remove		Remove		Remove		Remove		Remove		-
Channel Work Length:															
Mamaroneck River at Harbor Heights		93+50 (midpoint)	N/A		N/A		1,340.00		N/A		N/A		N/A		ft
Mamaroneck River Upstream of Confluence		42+50 (midpoint)	1,800.00		2,400.00		2,400.00		2,400.00		2,400.00		2,400.00		ft
Mamaroneck River Downstream of Confluence		20+00 (midpoint)	2,400.00		2,400.00		2,400.00		2,400.00		2,400.00		2,400.00		ft
Sheldrake River		17+50 (midpoint)	1,400.00		3,470.00		3,470.00		3,470.00		3,470.00		3,470.00		ft
Channel Width Size:															
Mamaroneck River at Harbor Heights			N/A		N/A		25.00 - 35.00		N/A		N/A		N/A		ft
Mamaroneck River Upstream of Confluence			45		30.00 - 40.00		45		45		30.00 - 40.00		45		ft
Mamaroneck River Downstream of Confluence			45		45		45		45		45		45		ft
Sheldrake River			30		20.00 - 30.00		25.00 - 33.00		30		20.00 - 33.00		25.00 - 33.00		ft
Channel Cut Volume (Soil/Rock):			47,000.00	10,000.00	60,990.00	11,250.00	89,210.00	11,250.00	87,230.00	11,250.00	60,990.00	11,250.00	89,210.00	11,250.00	cy
Walls (average height/Length):			8.5	4,955.00	8.4	4,085.00	8.5	4,360.00	8.7	7,210.00	8.4	4,085.00	8.5	4,360.00	ft
Non-Structural:															
Raise			No action		No action		No action		8 structures		8 structures		8 structures		-
Ringwall			No action		No action		No action		1 structure		1 structure		1 structure		-

***** Subsequent design change as a result of the Value Engineering Study resulted in the inclusion of a short diversion culvert instead of replacing the Station Plaza Bridge.

** All quantities are preliminary. Final quantities can be found in Appendix C4- Structural.

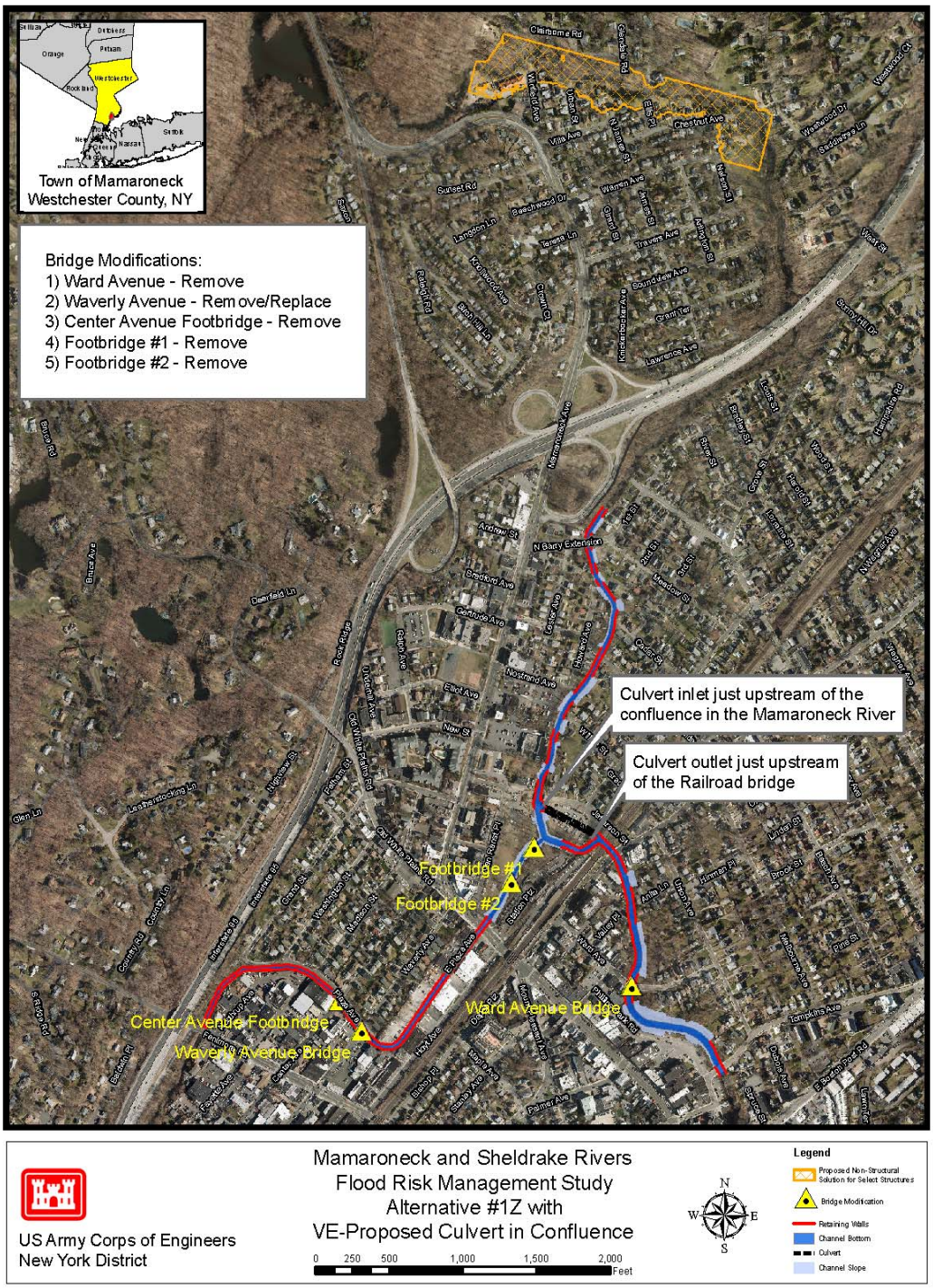

Figure 51: Alternative 1Z | Plan view of alternative 1Z with the VE proposed culvert in confluence

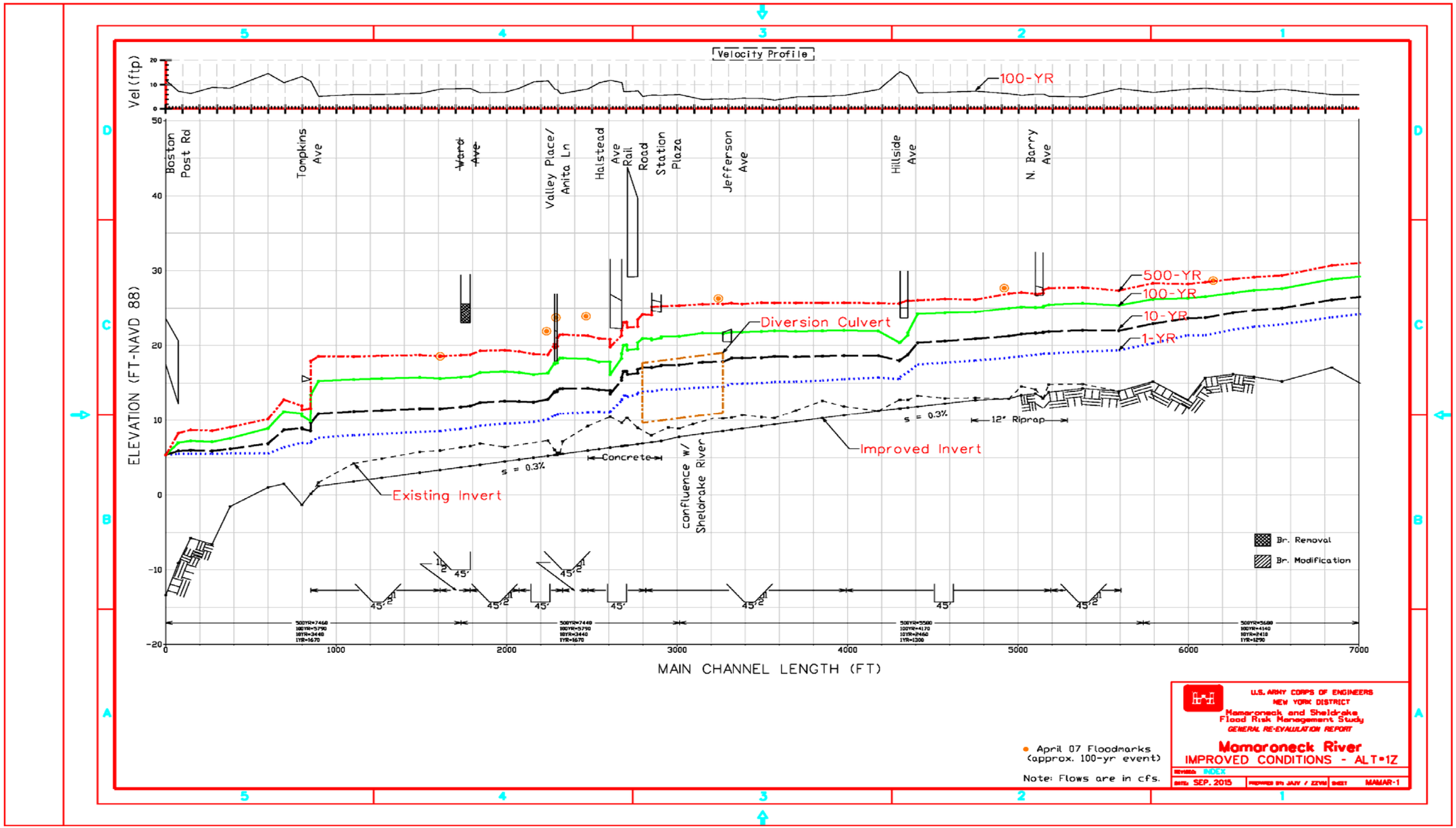

Figure 52: Alternative 1Z | Mamaroneck1 Profile

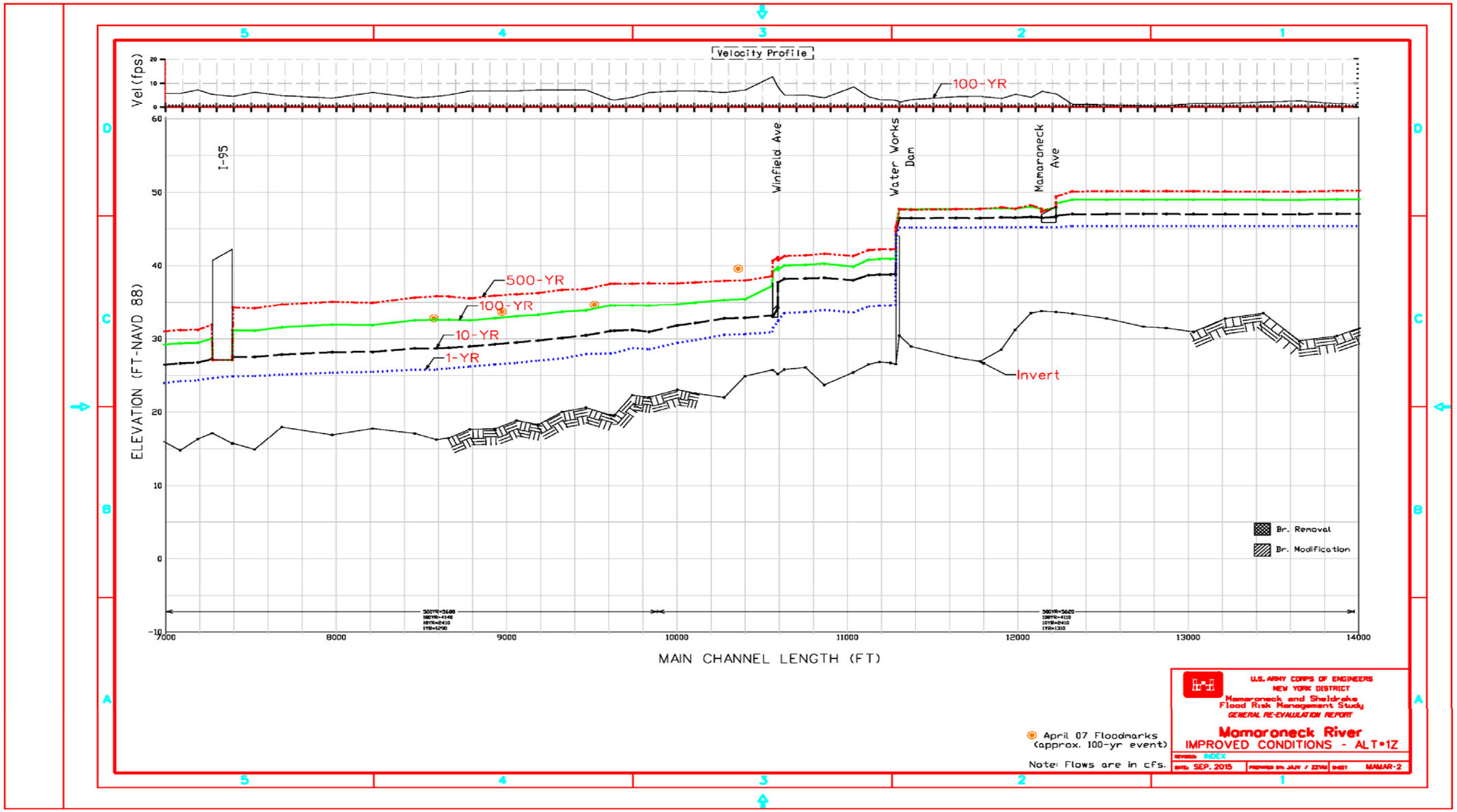

Figure 53: Alternative 1Z | Mamaroneck2 Profile

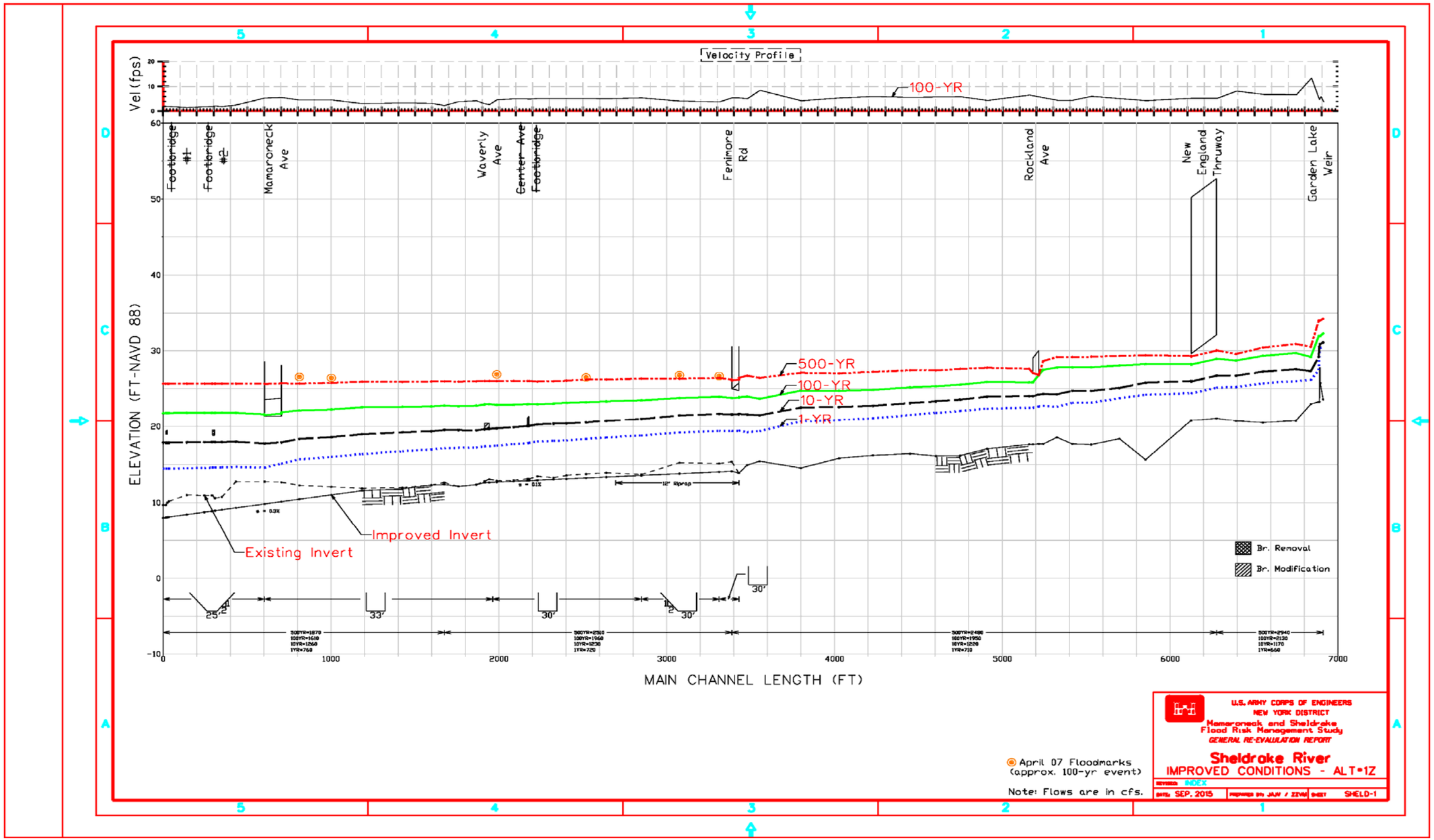

Figure 54: Alternative 1Z | Sheldrake Profile

4.5 Erosion Protection and Bank Stabilization

Riprap and concrete was selected to protect the Mamaroneck and Sheldrake Rivers from erosion. This solution will stabilize the stream bank using techniques consistent with the requirements of the US Army Corps of Engineers, NYDEC, Westchester County and the Town of Mamaroneck. The size and gradation of the riprap was determined following Corps of Engineers' procedures and methodology presented in EM 1110-2-1601, 1 July 1991, revised 30 June, 1994. Approximately 1,200 linear feet of riprap (i.e.; 13,000 square feet, 600 cubic yards) will be used mainly on the bottom of the Mamaroneck and Sheldrake Rivers. About 500 feet of riprap will be located roughly 200 feet both upstream and downstream of the N. Barry Ave Extension Bridge over the Mamaroneck River and 700 feet of riprap will be placed at the 90 degree turn in the Sheldrake River located downstream of the Fennimore Rd. Bridge. Also, due to high velocities and structural considerations along the Mamaroneck River from the Station Plaza Bridge to just downstream of the Halstead Ave Bridge, 300 LF concrete will placed along the bottom of the stream prevent scour under and around the footings of these three bridges.

All the riprap evaluated for the Mamaroneck and Sheldrake River consists of a 12 inch thick layer of riprap applied over a 6 inch stone bedding layer. If a geo-textile material is used instead of a 6 inch granular bedding layer for the bottom riprap a non-woven or geo-web product will be specified. The riprap on the side slopes will be extended to the top of the riverbank. A series of velocities from the 1 year to the 100 years where used to determine the necessary riprap stone sizes. These stone sizes are based on an assumed specific stone weight of 165 lbs per cubic foot. A Summary of riprap schedule for Alternative 1Z is provided in Table 21.

Table 21: Mamaroneck River Improved Conditions Alternative 1Z Riprap and Concrete Protection: Thickness, Areas, and Volumes

Mamaroneck River Improved Conditions Alternative 1Z Riprap and Concrete Protection Design: Thickness, Areas, and Volumes								
River	River Station	Channel Section	Reference Bridge/Ave.	Channel Protection	Length (ft)	Width (ft)	Placement Thickness (in)	Riprap/Concrete Volume (cy)
Mamaroneck River Upstream	53+00 to 47+50	Trapezoidal	450 feet Upstream North Barry Ave. 100 feet Downstream	Riprap @ Bottom	550	45	12"	611
Mamaroneck River Downstream	29+00 to 24+75	Rectangular	Station Plaza to Halstead Ave Bridge	Concrete @ Bottom	300	45	N/A	500*
Sheldrake River	35+00 to 33+00	Rectangular	Fenimore Rd	Riprap @ Bottom	200	25	12"	185
Sheldrake River	30+50 to 28+50	Semi-Trapezoidal	200 feet Upstream Bend	Riprap @ Bottom & Side	200	25	12"	333
Sheldrake River	28+50 to 27+75	Rectangular	75 feet Downstream Bend	Riprap @ Bottom	75	25	12"	69
							Total Volume Riprap	1198

*Volume not included in Total.

4.6 Residual Risk & Performance

As part of the continued coordination with the local sponsors and the public, the following concepts of flood risk management will continue to be presented:

- Design Limitations: No Flood Risk Management project can eliminate the risk of flooding. Given a long enough period of time, all projects will experience an event both larger and different than what they were designed for.
- Flood Risk Reduction: Flood Risk Management Projects can only reduce the frequency and/or severity of flooding and can provide additional time to respond.
- Awareness: Communication of accurate and timely information about the risk of living in a flood prone area is critical.
- Many Other Components: Physical features are only a single component of a flood risk management approach. (Insurance, Zoning, Emergency Action Plans, Storm Forecasting, Evacuation)
- Team: Flood safety is a shared responsibility and a collaborative approach is required to effectively manage the risk of flooding and to save lives. (National Weather Service, Corps, FEMA, State, County, Local Government Emergency Personnel & Residents)

Since the complete project consists of channel deepening & widening, retaining walls, bridge removal, a bridge replacement, a short diversion culvert and some erosion protection there will be a significant reduction in the depth and frequency of flooding but there will not be a significant change in the pattern of flooding should the design capacity be exceeded. Specifically the depth of flooding will be reduced about 2.5 to 3.5 feet for almost all events regardless of whether the channel design capacity is exceeded or not and there will be a longer period of time from the start of rainfall to the initiation of overbank flooding.

- <u>Design Exceedance</u>: This project will provide protection from small to medium sized rainfall events.

- Historic Comparison: The April 15, 2007 flood elevations was approximately a 100 year event and Alt 1Z would have reduced the water depths by about 4 feet in Columbus Park, 3-4 feet along the Sheldrake, about 3 feet at Barrie Ave., about 0.5 feet in the Harbor Heights Area and tapered to no change in all other areas.
- Non-structural Measures: The non-structural features in the Harbor Heights area of the project have a 1% annual chance of being exceeded (100 yr).
- Climate Change: Weather trends currently indicate that there may be a general increase in the size and frequency of both rainfall and coastal events.
- Project Type: This fluvial project will not provide any protection from the coastal/tidal flooding which mainly occurs downstream or seaward of Tompkins Ave.
- Local Drain Issues: The Corps of Engineers is not authorized to address local rainfall runoff issues and this project does not directly reduce the risk of flooding associated with poor street and storm sewer drainage.
- Maintenance: Maintenance of this project is important to the performance of the system. The amount of flood risk reduction and the performance of this project will be adversely impacted by poor maintenance. (see Operation and Maintenance section below.)
- Life Loss: Population at risk and estimated life loss was computed using the HEC-FDA program. While the with-project conditions provide a reduction in the estimated life-loss, the overall life-loss is still very low for both the existing and with-project conditions. While there is always a threat to human life when people live near a river the threat is too low to be considered significant project feature.

A summary of the Residual Risk and Performance can be found in Table 22 below.

Table 22: Summary of Residual Risk and Performance

Hypothetical Uniform Rainfall Total in inches	Storm Size express as a return period in years	Annual Chance of Exceedance (in any one year)	Chance of Exceedance in 30 years (mortgage)	Chance of Exceedance in 75 years (lifetime)	Likely Project Performance
5.9	25	4%	70%	95%	Good
6.4	50	2%	45%	78%	Fair
6.8	100	1%	26%	53%	Inadequate
Good – Flooding of streets and structures is unlikely. Fair – Flooding of streets and structures will be limited but both gradual and less than the without project condition. Inadequate – Flooding of streets and structures will be significant but both gradual and less than the without project condition.					

The Village of Mamaroneck tracks forecasted and actual rainfall amounts to help coordinate their flood risk management efforts. Rainfall amounts are never uniformly distributed over an entire drainage basin. The same amount of rainfall can result in very different amounts of river flow due to ground water conditions, vegetation cover, reservoir levels and temperature. We have included uniform hypothetical rainfall amounts in Table 27 above to assist the Village in their flood risk management efforts. The rainfall amounts in Table 27 probably have an accuracy of plus or minus one (1) inch. The hypothetical 100 yr flood was developed using a total of 6.8 inches of uniform rainfall over the entire basin. The April 2007 event, which was about equal to a 100 yr flood, experienced a non-uniformly distributed rainfall that averaged about 7.9 inches across the entire basin.

In addition, “wet/dry” inundation maps have been presented to both the local sponsors and the public as a means of illustrating both the amount of flood risk reduction and the amount of residual risk. These “wet/dry” inundation maps depict rough estimates of whether or not a main floor will be flooded for a specific hypothetical event. These maps portray the residual risk associated with the various alternatives and frequencies. See the 100 year wet/dry inundations for Alt 1Z in Figure 55.

Alternative 1Z would have reduced the April 15, 2007 flood elevations by 4.5 feet in Columbus Park, 3 feet along the lower Sheldrake and 0.5 feet in the Harbor Heights and tapers to no change in all other areas. As a point of reference, flood depths of 8 feet in Columbus Park would have been reduced to about 3.5 feet and street flooding in the same general area would have dropped from 6 feet to about 1.5 feet.

The water surface reduction can also be seen in a series of Rating Curves developed at three different locations, these locations are Waverly Ave. on the Sheldrake River, Columbus Park at the confluence and Harbor Heights at Glendale Ave. on the Mamaroneck River (see Figure 56, Figure 57 and Figure 58, respectively).

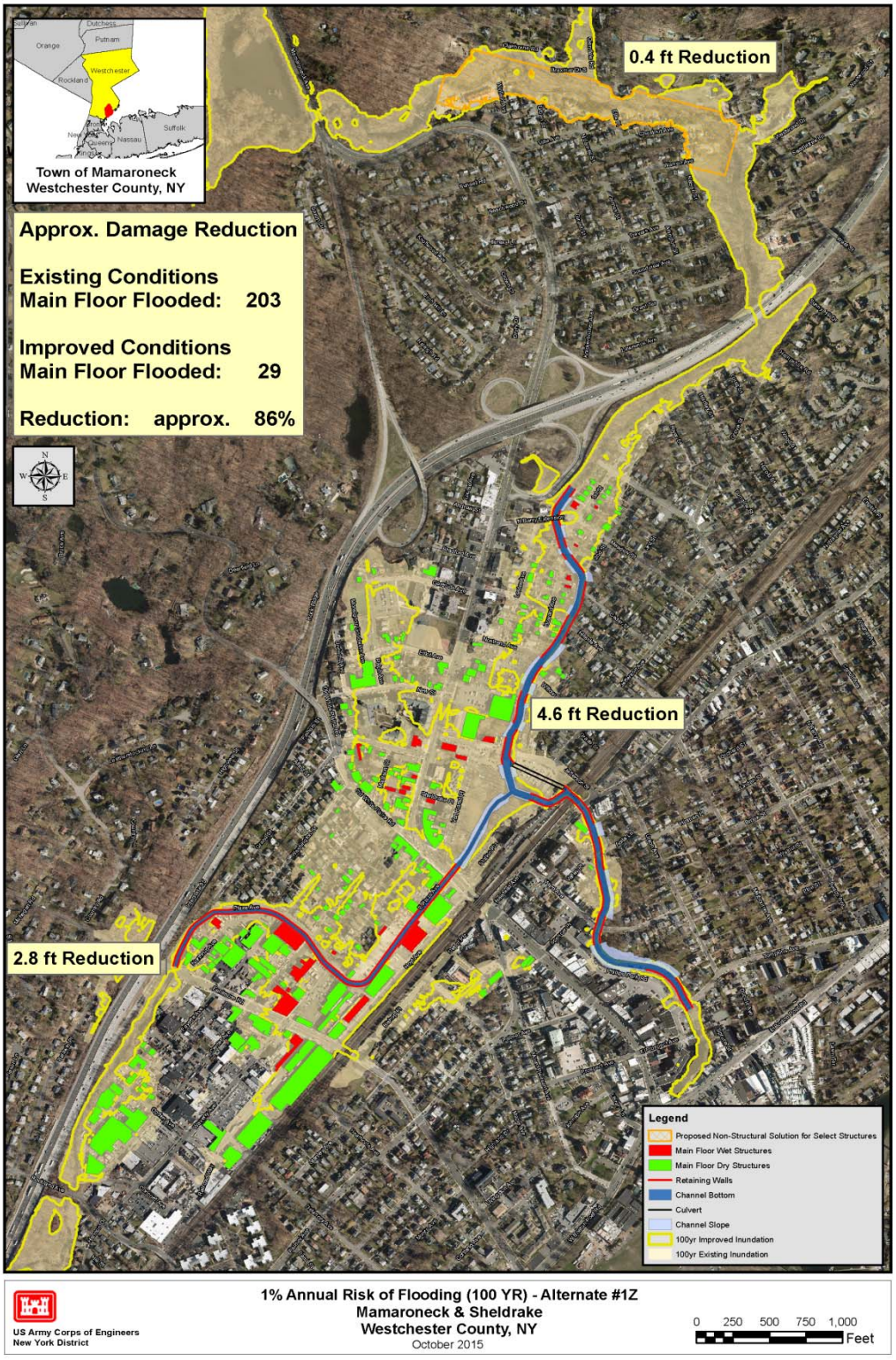

Figure 55: Residual risk associated with Alternative 1Z at the 1% Annual Risk of Flooding (100-yr event)

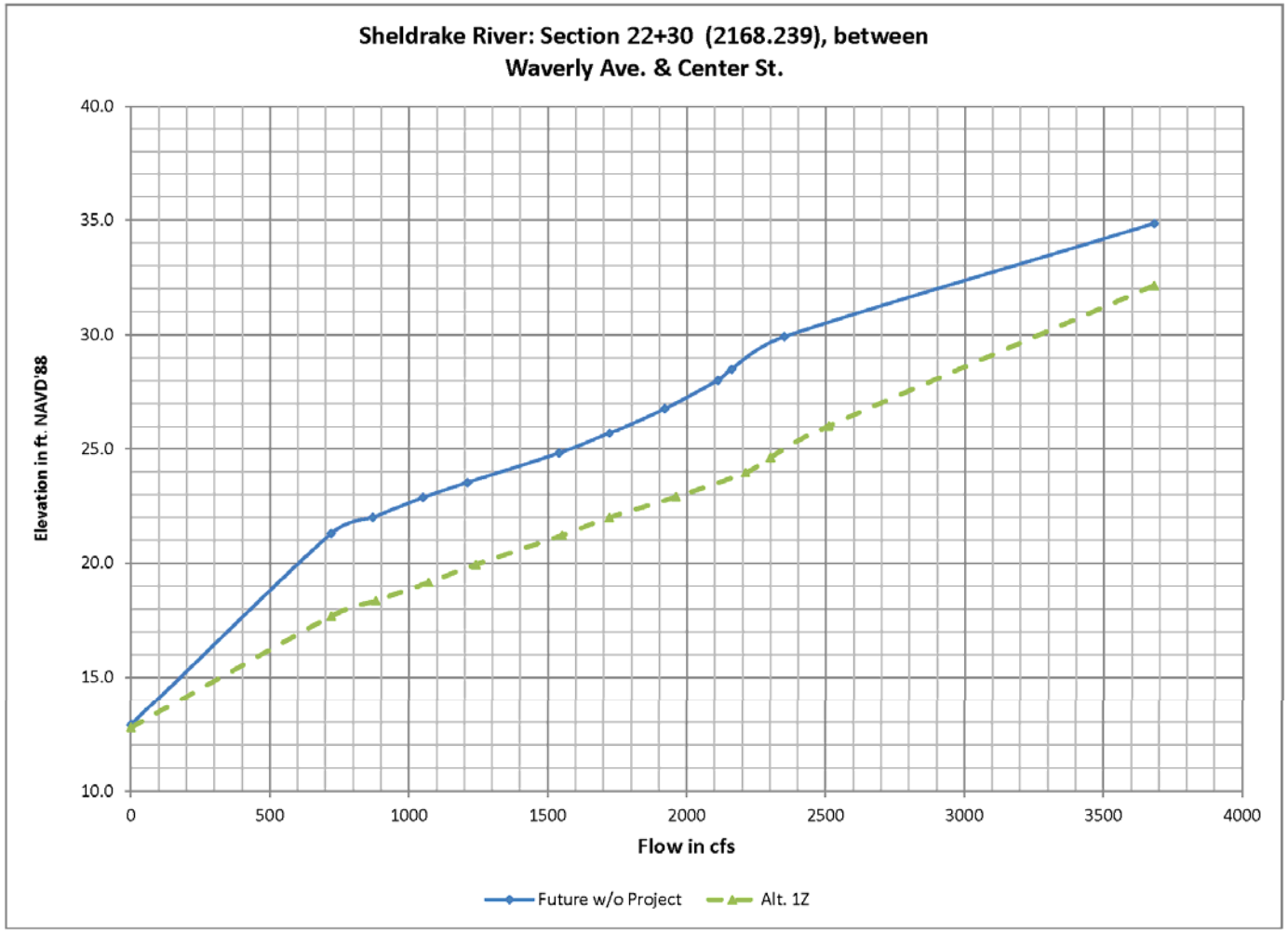

Figure 56: Rating Curve near Waverly Ave. at Sheldrake River

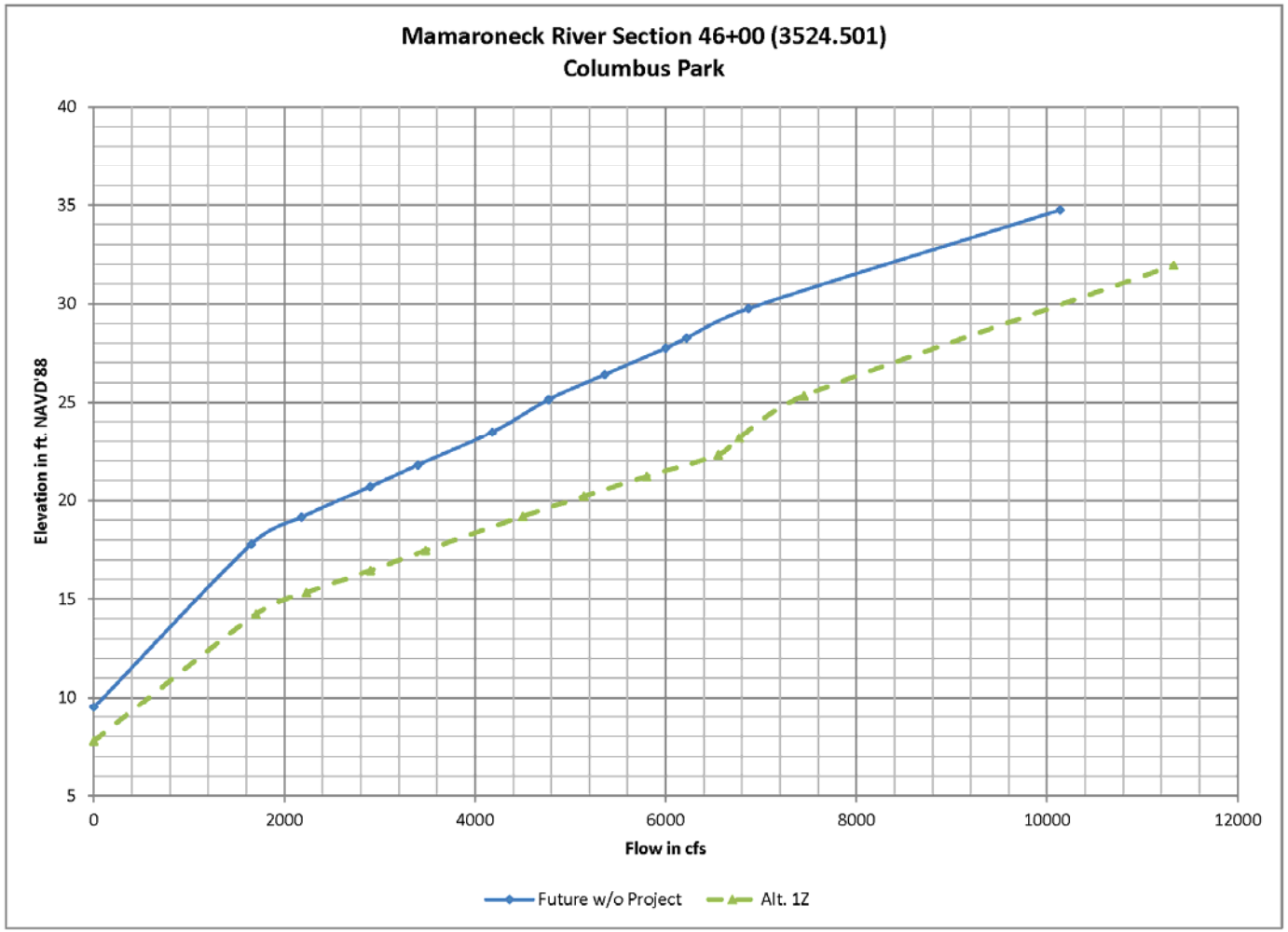

Figure 57: Rating Curve near Columbus Park at Mamaroneck and Sheldrake Confluence

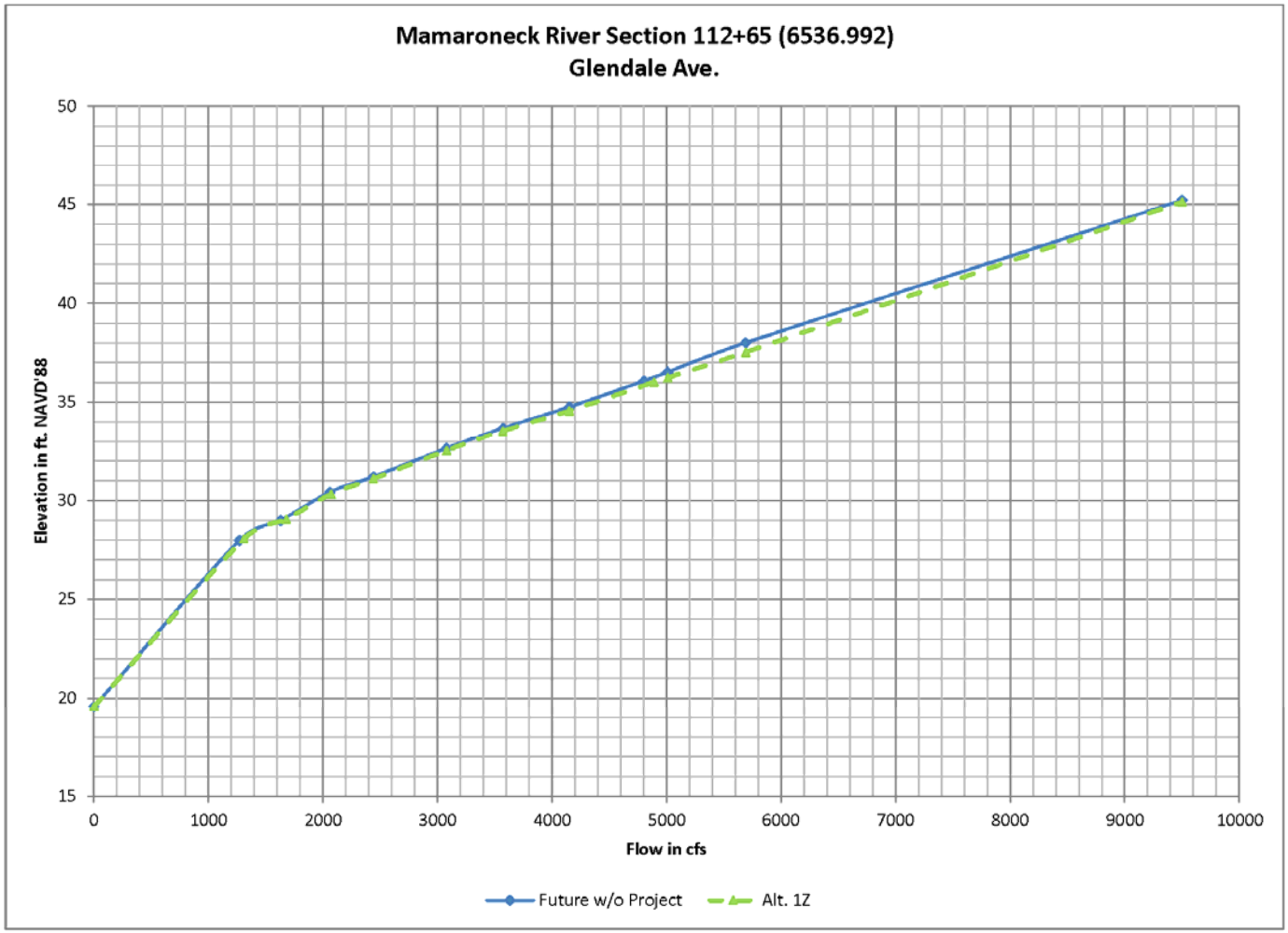

Figure 58: Rating Curve near Glendale Ave. at Mamaroneck River

4.7 Sedimentation

A rough sediment trend assessment was conducted. The soils and channel bottom for the project area above the Rail Road consists of silty sands. Below Halstead Ave. there are large cobbles and bedrock in the channel especially where the channel bottom gets steeper. There are small reservoirs upstream of the project area on both rivers which have been filling in and have historically been cleared of sediment. Both the Village Manager and the Village Engineer for the Village of Mamaroneck were contacted concerning erosion and shoaling in the river. The local officials noted that shoals have formed under one bay of the North Barry Ave. Bridge on the inside of a bend. They noted that after the April 2007 storm they removed some sediment from under Fenimore Rd., the Thruway, North Barry Ave. and the Anita Ln./Valley Place bridges. They also noted that the river banks in Columbus Park tend to erode.

An invert comparison between the cross-sections surveyed for the 1989 GDM and the cross-sections surveyed for this GRR was made. The comparison indicated that there was a small but general scouring trend for the Mamaroneck River. Based on the information collected, this small trend seems reasonable. The results of the last 20 years were extrapolated 50 years into the future and the future unimproved model inverts were lowered to reflect the small erosion trend. Since the improved conditions will include channel and bank erosion protection, none of the improved conditions models were modified to include an erosion trend.

It is anticipated that the reservoirs will continue to prevent some sediment from entering the project area. Large events will cause some bank and bottom erosion in or near the project area but much of that sediment will pass through the project area and be deposited in the harbor. Sediment transport is not expected to be a significant concern for this project.

4.8 Operation and Maintenance

Since this project consists of channel improvements and a culvert it is important that the hydraulic capacity of the channel and culvert be maintained over time. Access to the project must be maintained for inspection and maintenance purposes. The project and areas immediately upstream and downstream should be inspected annually. Channel improvements provide passive flood risk management assistance and do not require any operation. However, removal of debris, particularly from bridges during and after a storm, should be performed.

To maintain the hydraulic capacity of this project, shoals, debris, encroachments and heavy vegetation should be removed from the channel. Shoaling and debris accumulation can be expected after a significant flood especially under and around bridges, the inside of bends (North Barry Ave) and at the confluence of the Mamaroneck and Sheldrake Rivers. The channel cross-section should be maintained to the original design invert and bottom width as shown in the contract plans. The amount of sediment removal required should be slightly less than the historic volumes of sediment removed because the riprap and other erosion protection measures will reduce the amount of erosion experienced in the project area. Vegetation along the side slopes of the channel should be cut once a year in the late spring. There should be no woody vegetation on the low half of the slope. Small bushes and shrubs are permitted above mid-slope and trees are permitted at the top of slope only. Vegetation along the access and maintenance paths will need to be cleared several times a year. Riprap erosion protection should be inspected and any broken or displaced stones should be repaired or replaced. Dumping of snow or grass clippings into the channel is not permitted.

The culvert under the Station Plaza parking lot should be inspected yearly for cracks, movement and sediment accumulation. Large sized sediment or significant volumes of sediment should be removed.

Channel retaining walls and the culvert should be inspected yearly for cracks and movement such as sliding, rotation and tilting. All vegetation should be removed from the walls and drainage openings.

No improvements or changes shall be made over, under or through this project without prior determination by the District Engineer that the requested improvements or changes will not adversely affect the function of the improved channel and culvert.

Summary:

- No changes without district approval.
- No encroachments are permitted.
- Remove shoaling and debris.
- No dumping of snow or lawn clippings.
- Cut vegetation.
- Trees and shrubs are permitted near the top of bank only.
- Maintain access paths for inspection and maintenance.
- Inspect and maintain the riprap, walls and culvert.
- Keep the culvert clear.