# FINAL FEASIBILITY REPORT
# ON
# FLOOD DAMAGE REDUCTION
# AND ECOSYSTEM RESTORATION
# FEASIBILITY STUDY
# WOODBRIDGE RIVER BASIN,
# MIDDLESEX COUNTY, NEW JERSEY

**APPENDIX A**

HYDROLOGY and HYDRAULICS

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# WOODBRIDGE RIVER BASIN, NEW JERSEY

# FLOOD CONTROL AND ECOSYSTEM RESTORATION FEASIBILITY STUDY

## INTRODUCTION

### Description of Study Area and Vicinity

#### General

A1. The Woodbridge River Basin, located in northeastern New Jersey, is approximately 5 miles in length from its headwaters in the northeastern corner of Woodbridge near the Carteret/Rahway Township line to its mouth at the Arthur Kill along the boundary of Woodbridge and Perth Amboy. The drainage area is approximately 10.5 square miles (6,700acres), with its major tributaries being Heards Brook, Wedgewood Brook, and Spa Spring. The Woodbridge River Basin study area is shown in Figure 1. The Woodbridge River is tidally controlled from its mouth upstream to the Ideal Mobile Home Park area along Rahway Avenue. Fluvial conditions prevail above this area.

A2. Early in the feasibility phase, meetings and site visits were held with NJDEP, county and local governments, and area residents to determine the extent of flooding in the Woodbridge River basin. It was determined from this coordination, along with an evaluation of the economic data, that there are no widespread flooding problems in the Woodbridge River watershed downstream of the New Jersey Turnpike Bridge because most of the structures are above the 100-year floodplain. Consequently, the study area was modified, focusing on river reaches above the New Jersey Turnpike, specifically flood-prone areas within the Crampton Avenue neighborhood and the Ideal Mobile Home Park along Rahway Avenue.

## Problem Identification

A3. Storms have caused severe flooding along the Woodbridge River. Flooding upstream of the Ideal Mobile Home Park area is fluvial. Flood damages downstream of the Ideal Mobile Home Park area are tidally-dominated with additional damages associated with basin runoff. The communities repeatedly affected by the tidal surges are the Crampton Avenue neighborhood and the Ideal Mobile Home Park area along Rahway Avenue. There are approximately 299 structures in the 100-year floodplains of these communities. Tidal flooding typically occurs during hurricanes and northeasters when sustained onshore winds push storm surges inland up tidal channels. In addition to damages resulting from tidal inundation, tidal surges often block existing storm water drainage outlets, indirectly resulting in additional flood damages.

## Authorization

A4. The Woodbridge River, New Jersey, Flood Control and Ecosystem Restoration Study are being carried out under the Corps of Engineers' General Investigations (GI) Program. The study was authorized by a resolution of the Committee on Transportation and Infrastructure of the U.S. House of Representatives, dated May 7, 1997. The resolution states that:

"Resolved by the Committee on Transportation and Infrastructure of the United States House of Representatives, that, the Secretary of the Army review the report of the Chief of Engineers on the Rahway River, New Jersey, published as House Document 67, 89th Congress, the report of the Chief of Engineers on Arthur Kill and Kill Van Kull Basin, authorized by Section 206 of the Flood Control Act of 1958, and other pertinent reports, with a view to determine whether any modifications of the recommendations contained therein are advisable at the present time, in the interest of water resources development, including flood control, ecosystem restoration and protection and other allied purposes, within the Township of Woodbridge and vicinity and the Woodbridge River Basin, New Jersey."

A5. Under this study authorization, a reconnaissance report was completed in May 1999. This study concluded that there was at least one potential project in the Federal interest. The report recommended that 100-year level of flood protection for the Township of Woodbridge is to include a tide gate with an associated pump station, levees, floodwalls with an associated pump station, and floodproofing at the confluence of Heards Brook and Woodbridge River. Based on

these findings and recommendations, the Corps and the State of New Jersey entered into an agreement to perform a cost-shared multipurpose feasibility study of the Woodbridge River.

## Prior Studies By Others

A6. The Woodbridge River Basin has been subject to flooding and has been the subject of one prior study. The report is considered applicable to this study and has been compiled, reviewed, and utilized as appropriate. The following is a listing and description of this one prior report:

Flood Mitigation Planning Study, Woodbridge River and Tributaries, South Branch Rahway River and Tributaries, Pumpkin Patch Brook. May 1997. Killam Associates & Consulting Engineers.

This report investigates 17 floodprone areas along the Woodbridge River and tributaries, South Branch Rahway River and tributaries, and Pumpkin Patch Brook, a Robinson's Branch tributary. Both short and long-term solutions are proposed, many dealing with non-structural and interior drainage alternatives. There are, however, several structural solutions proposed in the report, which entail channel improvements, detention basins, diversion tunnels, levees and floodwalls. No estimate of economic benefits is provided in the report, and no projects have been constructed as a result of the study.

## Study Participants and Coordination

A7. The New York District is responsible for conducting the overall Feasibility Study in cooperation with the non-Federal sponsor, NJDEP. The feasibility study and implementation of the recommended project continues to receive strong support from the Township of Woodbridge. They are committed to working with the Corps to address flooding problems and opportunities for ecosystem restoration along the Woodbridge River.

## Historical Flooding

A8. The Woodbridge River study area is prone to imminent and severe flooding. Significant flood damages occurred during the northeaster of October 1996. Tidal backwater flooding from the Arthur Kill resulted in severe damage to residential properties in the Crampton Avenue

neighborhood and the Ideal Mobile Home Park areas. Damages from this storm were estimated to be in excess of $600,000.

# EXISTING CONDITIONS

## Hydrology

A9. The Woodbridge River is a tributary of the Arthur Kill. It has a drainage area of 10.51 square miles at its mouth and enters the Arthur Kill at the boundary of the City of Perth Amboy and the Township of Woodbridge, New Jersey. The Woodbridge River watershed, with its subbasins, is shown on Figure 1.

A10. The Woodbridge River basin is ungaged. However, it is likely that tidal flooding has been experienced at the damage centers within the basin from tidal surges due to Nor'easters or tropical cyclones during the following events: November 1950, September 1960, March 1962, September 1985, December 1992, 13 March 1993, and October 1996.

A11. The Woodbridge River basin was divided into eight sub-basins ranging in size from 0.28 to 2.18 square miles for analysis and HEC-HMS modeling purposes. Sub-basin Clark unit hydrograph parameters $t_c$ and R were computed from the physical parameters slope, drainage area and percent impervious area using regression equations contained in the report: Special Projects Memo No. 469, Hydrologic-Hydraulic Simulation, Rahway River Basin, New Jersey (The Hydrologic Engineering Center, USACE, November 1976). They resulted from a regional analysis relating $t_c$ and R with various physiographic characteristics of a drainage basin. The equations are:

$t_c = 8.29\ (1.0 + 0.03I)^{-1.28}\ (DA/S)^{0.28}$

$R = 1.85\ t_c$

Where:

- DA = drainage area in square miles

- S = watercourse slope, in feet per mile, defined as the average slope of the watercourse between points 10 and 85 percent of the distance upstream from the runoff site to the watershed boundary
- I = index of impervious cover in percent of total land area
- $t_c$ = time in hours from the end of a burst of rainfall excess to the inflection point on the recession limb of the resulting direct runoff hydrograph (Clark method)
- R = discharge at the inflection point on the recession limb of the direct runoff hydrograph divided by the slope of the recession limb at that point, in hours (Clark method)

A12. Sub-basin physical parameters were determined from the USGS quad sheets Perth Amboy, NJ and NY and Arthur Kill NY and NJ. Sub-basin physical parameters and resultant Clark unitgraph parameters $t_c$ and R are summarized in Table 1.

A13. Infiltration losses throughout the Woodbridge River basin were modeled in program HEC-HMS with the SCS curve number option. Sub-basin SCS curve numbers were computed using soils and land use data from the SCS Soils Survey for Middlesex County, New Jersey and Table 2-2 of SCS Technical Release No. 55, Urban Hydrology For Small Watersheds. Since the Woodbridge River basin is ungaged. Thus it was determined that the SCS loss option, with unitgraph parameters from proven regression equations, was the best approach for hydrologic modeling of the basin. Sub-basin SCS curve numbers are given in Table 1.

A14. Information on baseflow (the sustained or "fair-weather" runoff of prior precipitation that was stored temporarily in the watershed, plus the delayed subsurface runoff from the current storm) was also included as part of the HEC-HMS model input used for the Woodbridge River basin. The "recession constant" and "threshold flow" baseflow parameters developed from the gaged basin, Elizabeth River at Ursino Lake in Elizabeth, New Jersey were adopted for the ungaged Woodbridge River basin. It was judged the most physically likely basin to produce baseflow close to that of the Woodbridge River. It is a nearby, moderately sloped and heavily urbanized and storm-sewered basin like the Woodbridge River. The "initial flow" baseflow parameter used for the Woodbridge River basin was a "rule-of-thumb" average value. The definitions and adopted values of the recession baseflow parameters used are:

- **Initial Flow**: The initial flow at the beginning of the simulation = 2 cfs per square mile
- **Recession Constant**: The rate of baseflow decay (the ratio of baseflow now to the flow one day earlier) = 0.296225
- **Threshold Flow**: The point on the hydrograph where baseflow replaces overland flow as the source of flow from the subbasin. The threshold is given as a ratio of the peak flow immediately preceding the initiation of baseflow = 0.03 times the peak flow (cfs)

A15. Muskingum routing parameters K (travel time) and X (Storage Coefficient) were developed for four reaches to channel route hydrographs through the Woodbridge River basin. Routing parameters are given in Table 2.

A16. Specific-frequency hypothetical rainfall was applied to the HEC-HMS model to compute the required hypothetical discharge hydrographs. The hypothetical rainfall is given in Table 3 and had previously been developed for the nearby Rahway River basin.

A17. Selected existing conditions peak discharges (cfs) throughout the Woodbridge River basin are shown in Table 4. Existing conditions hydrographs at the inflow to the proposed tide gate structure are shown on Figures 2 to 6.

## Hydraulics

A18. For discussion purposes, the project area is broken down into the following four reaches: three reaches for the Woodbridge River upstream from the New Jersey Turnpike and one reach for Heards Brook. The physical parameters for the four reaches were taken from the report called "Flood Insurance Study of the Township of Woodbridge" (Flood Insurance Study done by FEMA in March 1983). The locations of these reaches are shown in Figure 7. The description of each reach is given below:

### Woodbridge River

A19. REACH #1: Downstream of the Port Reading Avenue bridge to upstream of the New Jersey Turnpike - The slope of this reach is approximately 0.0005 feet per foot. On the west side are a residential area and a fill site. On the east side are a rest-stop for the New Jersey Turnpike

and an intertidal-fluvial marsh. The total length of this reach is approximately 4,200 feet. The Heards Brook confluence is on the west side of this reach.

A20. REACH #2: Downstream of the railroad bridge to upstream of the Port Reading Avenue bridge - The slope of this reach is approximately 0.0004 feet per foot. On the west side are a residential area and newly built apartment complexes. On the east side are more intertidal-fluvial marshes. The total length of this reach is approximately 5,100 feet. The Wedgewood Brook confluence is on the west side of this reach and the Turtle Brook confluence is on the east side.

A21. REACH #3: Downstream of the Omar Avenue bridge to upstream of the railroad bridge - The slope of this reach is approximately 0.0007 feet per foot. On the west side is a residential area. On the east side are newly built apartment complexes and an intertidal-fluvial marsh. The total length of this reach is approximately 5,280 feet.

## Heards Brook

A22. REACH #4: Downstream end of a culvert near Rahway Avenue to the mouth of Heards Brook (confluence with the Woodbridge River) - The slope of this reach is approximately 0.002 feet per foot. The banks of this reach are mostly an intertidal-fluvial marsh and residential areas. The total length of this reach is approximately 2,330 feet.

A23. The Woodbridge River up to the Ideal Mobile Home Park and Heard's Brook up to the culvert is considered a tidal estuary. In order to define the existing conditions floodplain, a stage-frequency curve is needed for this tidal estuary. Stage frequency data for Perth Amboy near the mouth of the Raritan River and the Arthur Kill near the mouth of the Woodbridge River are published in "Total Stillwater Frequency-Elevations Implementation and Results - New York City Flood Insurance Study Report No.7", dated November 1981 by CDM (Camp Dresser & McKee). More recent stage frequency data for Perth Amboy was developed from the draft FIMP (Fire Island to Montauk Point) results in 2004. This data was then extrapolated to the mouth of the Woodbridge River using the relationships from the CDM data. It is noted that the 100 year elevation from the extrapolated data (11.85 ft, NGVD) is in general agreement with the NJDEP's

accepted 100 year elevation of 12.2 ft, NGVD. For this study, the existing conditions water levels in the Woodbridge River estuary are conservatively assumed to be equal to those defined by the stage frequency data at the mouth (confluence with the Arthur Kill). The stage-frequency curve representing the existing conditions in the Woodbridge River is shown in Figure 8.

### Without Project Conditions

A24. The without project condition was determined by projecting conditions in the study area over a 50-year period of analysis (2003-2053). It is expected that the current level of development within the floodplain is essentially stable with moderate increases anticipated over the life of the project. In the absence of Federal action, flooding problems in the study area are expected to continue and perhaps be exacerbated by increased damage potential in the floodplains of the areas of the Crampton Avenue neighborhood and the Ideal Mobile Home Park along the Woodbridge River, based upon increases in the values of structures and contents and by sea level rise.

## DEVELOPMENT OF ALTERNATIVES

### Screened Alternatives

A25. Several alternatives were explained in the early phases of this study. All plans considered at this stage of analysis provided for a 100-year level of protection in order to maintain consistency. These alternatives included non-structural protection (flood proofing and/or flood raising or a flood warning system), structural protection (floodwall and/or levees, tide gate structure, dredging, etc.), and a “no-action” alternative, if there is no federal action in the project area. The following paragraphs give a brief description of the no-action, non-structural, and structural alternatives for the project area.

### No Action Alternative

A26. This plan involves no additional Federal action to provide flood damage reduction. The no action alternative would avoid environmental and other impacts associated with implementation of additional plans for flood damage reduction. However, this plan fails to meet any of the study objectives. The result would be the continuation and potential exacerbation of

flooding problems in the study area. This alternative represents the default condition if a Federal project is not recommended and provides a reference for evaluation of without-project future conditions.

## Non-Structural Alternatives

*Flood Warning System.*

A27. In situations where a structural or non-structural flood damage reduction project is not feasible, a flood warning system may provide some relief to those located within an area subject to flood damages. Even in areas that can claim benefits from a completed project, a flood warning system can afford residents advance warning of what is to come and allow them time to make appropriate preparations. While a flood warning system does not prevent flooding and does not reduce damage to property that is left in the path of floodwaters, it can provide an aid in reducing property loss and increasing the safety of individuals. With the use of a flood warning system, property, such as motor vehicles, can be relocated to higher ground in time to prevent damage from rising waters. In addition, moveable items can be taken to higher floors within structures, where they will not be impacted. Finally, residents will have time to leave the area, if necessary, for their own safety.

A28. Elaborate flood warning systems can be designed and implemented for a particular location. However, this is not being considered for the Woodbridge River basin since other Federal agencies, such as the United States Geological Society and The National Weather Service, already provide satisfactory information sources that can address this situation. By using these available resources, an adequate flood warning system can be provided to the residents of Woodbridge.

*Floodproofing and floodraising.*

A29. This non-structural alternative involves floodproofing or raising approximately 110 structures in the Crampton Avenue neighborhood and approximately 189 structures in the Ideal Mobile Home Park area.

## Structural Alternatives

*Dredging.*

A30. As a structural alternative, dredging the Woodbridge River at selected locations was discussed. However, an increase in flood damage reduction along the Woodbridge River is not achievable by dredging the stream channel to a larger cross-section. The Woodbridge River is a tidal estuary, affected by the tides that proceed upstream from the Arthur Kill. The flood damage that is sustained in the identified damage centers along the Woodbridge River is predominately caused by this tidal flooding. As the tide moves upstream from the Atlantic Ocean, through Raritan Bay, Arthur Kill and into the Woodbridge River, it is capable of filling up any channel or area at an elevation lower than the level the particular tide will ultimately reach. Any surge that is added to the astronomical tide, based on a current storm condition, likewise, is of such volume as not to be reduced in elevation by an enlarged channel. Whatever size cross-section is provided will always fill completely to the elevation of the tide for the entire length of the channel system. Any increase in channel size provided to accommodate upland runoff will be filled with the tide prior to the runoff reaching the channel provided that the rainfall event is occurring on a high tide. If the rainfall event occurs on a low tide, then ample channel capacity will already be available and little damage should be realized since flood damage predominately is caused by the high tides.

*Floodwall and/or Levee segments in the Crampton Avenue and Ideal Mobile Home Park Areas.*

A31. As a structural alternative, a levee segment was proposed around the Crampton Avenue neighborhood. This improvement would be built to an elevation of 12 feet, NGVD and extend above the ground surface between 4 and 10 feet, depending on the local ground elevation The elevation of the floodwall and/or levee segment is only based on the 100-year tide condition. The elevation of the floodwall and/or levee segment would most likely be higher if risk and uncertainty is incorporated. The total length would be about 4560 feet. Also, there would be road raising of a section of Port Reading Avenue to an elevation of 12 feet, NGVD to prevent the tide from circumventing the floodwall and/or levee system and flooding developed areas. The location of the alignment of this floodwall and/or levee segment alternative is shown in Figure 9. A below grade submersible pump, with a capacity of 100 cfs, would also be required with this alternative to remove any interior runoff that collects along the landward side of the proposed protection during periods of high tidal stages. During periods of low tidal stages, the interior

runoff could be drained from the protected side of the improvement by gravity, using drainage structures spaced every 500 feet along the levee segment.

A32. Another part of this structural alternative includes a proposed floodwall and/or levee segment around the Ideal Mobile Home Park on Rahway Avenue. The maximum alignment of the floodwall and/or levee system is shown in Figure 10. The elevation of this segment would also be 12 feet, NGVD and extend above the ground surface between 4 and 10 feet, depending on the local ground elevation. The elevation of the floodwall and/or levee segment is only based on the 100-year tide condition. The elevation of the floodwall and/or levee segment would most likely be higher if risk and uncertainty is incorporated. The total length of this proposed floodwall and/or levee segment is about 2000 feet. In order to remove interior runoff on the protected side of the improvement during high tidal events, a 40 cfs capacity below grade submersible pump is required. During periods of low tidal stages, interior runoff can be drained by gravity through drainage structures spaced every 500 feet along the floodwall and/or levee segment.

*Tide Gate and levee system.*

A33. A proposed tide gate and levee system alternative includes of a tide gate with sluice gates and a levee system located just upstream from the Woodbridge Avenue bridge and perpendicular to the Woodbridge River (see Figure 13). This alternative would provide 100-year interior flood protection for most of the study area. The tide gate and levee system would tie into high ground west and east of the Woodbridge River. Under non-flood conditions, the tide gate would remain open allowing the flushing of the Woodbridge River. During abnormally high flood stages, the gate would be closed. With the gate closed, the normal Woodbridge River discharges would be impounded. Numerical modeling of this system will be explained in more detail below.

A34. An economic analysis, based on the benefit-to-cost ratio (BCR), was done for all alternatives stated above, to determine which alternatives would be screened out from further study and which alternatives would continue to be studied. Economic analyses of the non-structural and floodwall and/or levee alternatives each yielded a BCR of clearly less than 1.0, and were subsequently screened out from further study. In addition, dredging the Woodbridge

River would not provide any flood protection for the Woodbridge area and was eliminated from further study. Therefore, the most promising flood control solution for further, detailed study was deemed to be an appropriately sized tide gate structure with sluice gates and a levee system.

# IMPROVED HYDRAULICS CONDITIONS

## Tide Gate Analysis.

**A**35. The tide gate alternative was chosen for preliminary analysis. Using the existing data available to the NY District from Woodbridge Township, FEMA, and NJDEP, a rough analysis was done for the proposed tide gate structure at the downstream side of the NJ Turnpike Bridge over the Woodbridge River. The method behind the analysis was to do routing calculations with the tide gate structure and levee system forming the LOP (Line-of-Protection). This improvement will significantly reduce the severe tidal impacts of the Arthur Kill on the Woodbridge River damage centers by preventing storm surges from entering the protected area. The routing calculations are usually done by the computer program HEC-IFH (Hydrologic Engineering Center - Interior Flood Hydrology). In this case HEC-IFH could not be used for the following reasons: 1) Under normal tide conditions, the tide gate would remain open, and flow is allowed through the tide gate in both directions. HEC-IFH does not allow backflow into its routing calculations and 2) There is no option to regulate (open and close) the tide gate during the storm event to control the interior water surface elevation.

A36. An alternative to calculate the flow through the tide gate was to use a spreadsheet that not only does routing calculations, but also includes the ability to model the tide gate operation. This spreadsheet was developed while analyzing a similar tide gate structure for the "South River, Raritan River Basin: Hurricane & Storm Damage Reduction and Ecosystem Restoration – Draft Integrated Feasibility Report & Environmental Impact Statement," dated April 2002.

A37. This preliminary analysis was based on the following assumptions: 1) Storage available upstream of the tide gate was calculated using March 1962 topographic mapping. Available 2002 aerials combined with site visits indicated development at various locations that were not shown on the topographic mapping. This analysis includes estimates of the lost storage due to

the development of new apartment complexes along the Woodbridge River; 2) Additional storage can be used for the tide gate structure if the Mitigation Site is going to be developed as part of the Arthur Kill Project. An increase of storage will in turn lower the water surface elevation behind the tide gate structure, though this would be expected to be very small; and 3) It was assumed that no further development will occur for present and future conditions with the tide gate structure and levee system. If future development occurs, it will take away from the natural storage that is needed for the Woodbridge area.

A38. Flooding in the project area is primarily caused by a combination of storm surge and astronomical tides. Even though this is a tidal estuary, the methodology used to determine water surface elevations on the protected area behind the levee and tide gate system is based on interior drainage principles.

A39. The spreadsheet analysis is based upon basic continuity principles, where inflow minus outflow equals the change in storage. Outflow through the gates is computed by either of two methodologies, depending upon whether the gate is submerged or not. If the gate is submerged, flow is computed using the orifice equation. If the gate is not submerged, flow is computed by the submerged weir equation. Both methods allow flow in either direction depending upon the head difference between interior and exterior water levels. Gate operations, e.g. opening and closing the gates, can be programmed into the analysis. The spreadsheet has the capability of operating the gates based upon several different parameters. These parameters are the exterior and interior water levels at the tide gate. The gates can be operated (opened and closed): 1) at predetermined tide elevations, 2) at head difference between interior and exterior water levels (e.g. gates could be opened when the exterior levels are below (e.g. 0.5 ft. below) interior levels), and 3) at a specific time before and after a storm event (e.g. closing the gate at a low tide when a storm event is forecasted. This will result in the maximum storage on the landward side of the tide gate).

A40. The analyses were conducted for five interior storm events (2-, 10-, 50-, 100- and 500-year return periods). In order to develop a stage frequency relationship, the interior events were routed against exterior tidal marigrams. The exterior tidal marigrams that were used for this

study area are shown in Figure 11. Recent interior drainage studies conducted at other project sites (i.e. South River), have shown that there is only a limited correlation or dependence between interior runoff events and tidal flooding events. As shown in Table 5 (interior water surface elevations (WSEL's)), the interior storm events were routed against a 2-year exterior tide and a 2-year interior runoff event was routed against four (2-, 10-, 50-, 100- and 500-year return period) exterior events (the line of protection was designed to protect the damage centers from an event up to a 50-year storm surge) to establish the most likely interior stage-frequency curve. Due to the relatively small drainage area in the vicinity of the project area, the relative timing of precipitation was assumed to be coincidental with peak exterior tidal surges. The analyses were performed for only the present condition.

A41. Uncertainty was incorporated into the analyses by routing the interior storm events against a normal exterior tidal condition (the lower bound), and routing the interior events against a 10-year external tide and the 10-year interior event against the four (2-, 10-, 50-, and 100-year return periods) exterior tidal conditions (the upper bound) as shown in Table 5 (Interior WSEL's).

## DETAILS OF SELECTED PLAN

A42. The results from the tide gate analysis concluded that the tide gates and levee, without a pump station, is a hydraulically feasible option for a flood control alternative. Initially, the line of protection was designed to provide a 100-year level of protection, and included the tide gates and a levee system 4140 ft in length. This alternative proved not to be cost effective. However, the line of protection needed to provide a 50-year level of protection would be reduced significantly in size and cost due to the lower closure elevation. Two 50-year level structural alignments with tide gage structure were produced. One structural alignment provides protection for a sewage pumping station operated by the Middlesex County Utilities Authority, shown on the aerial photography (taken in the summer of 2002) between the NJ Turnpike, Woodbridge Avenue and the Woodbridge River. The second structural alignment does not provide protection to the sewage pumping station. Alignment A is a levee system with the tide gate structure, which does

not protect the pumping station, has a length of 1,074 ft. Alignment A is shown in figure 14. Alignment B is a levee/floodwall system with the tide gate structure, which does protect the pumping station, has a length of 924 ft. Alignment B is shown in figure 15. Alignment B was determined to be a more cost effective improvement. A detailed description of the recommended tide gate and levee system is provided below.

## 50-Year Level of Protection

### Tide Gate Structure & Operation Description.

A43. The proposed tide gate structure is a pile supported, stand-alone structure that does not need the New Jersey Turnpike Bridge for stability or support. Based on the existing width of the Woodbridge River, ten (10) sluice gates housed in the concrete structure will be required. Each gate opening will be approximately 60” X 60” (5’ X 5’). The gates will be electrically operated with a manual backup.

A44. The tide gate chamber is constructed with steel-reinforced concrete, and rests on a timber pile foundation. Total length of the tide gate structure is approximately 60 to 70 feet. The top width is approximately 8 feet. The top elevation is approximately 12 feet, NGVD (50-year tide elevation plus risk and uncertainty). Concrete head and wing walls connect the structure to the earthen levee on either side. The ten 5’ X 5’ stainless steel slide gates are equipped with both electrical and backup manual operation to control flow. On both the protected side and unprotected sides, bar screens cover the flow openings to prevent passage of large debris from the Woodbridge River into the chamber.

A45. A staff gage and warning system are included in the tide gate structure. When the exterior water level is forecasted to be higher than normal high tide, the gates will be closed at low tide before the storm event. The gates are reopened on the falling tide when the exterior water level drops more then 0.5 ft below the interior level, allowing the interior to drain naturally, thereby maximizing available storage behind the line of protection. The gates are again closed when the rising tide exceeds interior levels. This operating sequence will prevent storm

tides from flooding the interior areas while at the same time allow the interior area to drain naturally whenever conditions permit.

A46. The sluice gates will remain open during normal tidal elevations to allow passage of saline tidewater into marsh areas and drainage of rainfall runoff. The gates will be contained in a concrete chamber, supported on a pile foundation consisting of seventy 12” diameter piles, 30 ft. in length, placed 5 ft. on center. Manhole covers and a ladder will be provided for access and maintenance. Standard 3-rail fence will be placed on top of the concrete gate structure for safety.

A47. Several locations in the area of the proposed tide gate structure have been identified on the available topographic mapping as representing possible culverts under the New Jersey Turnpike. These culverts would need the installation of flap gates on their downstream ends to prevent the tide from backing up and causing flooding upstream of the line of protection.

**Levee System Description.**

A48. In addition to the tide gate structure, a levee system must be included to provide a continuous line of protection. Most of the tide gate structure with levee layout was developed using topographic mapping from March 1962. However, aerial photography taken for this project in the summer of 2002 shows differences in the location of structures and the addition of new structures. This impacted the alignment of the tide gate structure and levee system. In addition, it was determined that there has been significant regrading in the immediate project area since 1962, including the realignment of small streams along the New Jersey Turnpike. Much of this resulted from redesign and construction of improvements to the adjacent New Jersey Turnpike rest area to the north of the Woodbridge River. Without the availability of updated topographic mapping of the site, the proposed alignment of the project retains a degree of uncertainty.

A49. This levee/floodwall system will be designed with a 50-year level of protection. Alignment B, that does protect the facility, is approximately 924 feet in length. The alignment of this levee/floodwall is shown in Figure 15.

## IMPACTS OF PROPOSED FLOOD CONTROL ALTERNATIVE (WATER SURFACE ELEVATIONS)

A50. The results of the improved conditions analysis for Alignment B are shown in the form of a stage frequency curve in Figure 16. When compared to the existing conditions stage frequency curve, the reduction in WSEL for any particular event is apparent. For example, the levee and tide gate improvements for a 10-year event would result in a 1.3 ft reduction in WSEL behind the levee. The magnitude of the reduction in WSEL for all events is also shown in Table 5.

A51. The existing and improved conditions inundation limits for the 50-year event are shown in Figure 17. As can be seen from the mapping, the reduction in WSEL results in less structures being flooded by the 50-year event. Other structures at lower elevations may still be subject to flooding, but to a much lower extent.

## ALTERNATIVE 6A

A52. Following the July 2006 draft report, the Township of Woodbridge submitted a proposed Alternative Plan prepared for them by Najarian Associates, which has been labeled Alternative 6A. The Najarian Report is located in the Main Report in Section 9 – "Pertinent Correspondence & Public Involvement."

The Corps analysis of Alternative 6A is discussed in Section 2.5.6 - "Determining Water Surface Elevations along the Woodbridge River", located in the Main Report, in addition to directly below in paragraph A53.

A53. The Corps has reviewed the report prepared by Najarian Associates for the Township of Woodbridge dated February 2006, entitled: "Review of Advance Draft Feasibility Report:

'Woodbridge River Basin, New Jersey Flood Damage Reduction and Ecosystem Restoration', New York District, U.S. Army Corps of Engineers (November 2005)". The report prepared by Najarian Associates was submitted to the Corps in September 2006. One of the preliminary recommendations in this report was to enhance the Corps Alternative Plan 6 to come up with a more cost-effective levee design and to increase the level of protection up to the 100-year storm event. Najarian Assoc. came up with new water levels for the 50-year and 100-year by including tide gage records which they had from early January 1999 that the Corps did not have access to. Based on this additional information, Najarian Assoc. was able to modify the design of Alternative 6 to protect against a storm water elevation which was different than the one developed by the Corps. Najarian provided the following FEMA Flood Insurance Study for Woodbridge Township estimated water surface elevations:

a) 100-yr 9.7 ft. NGVD and 50-yr 8.8 ft. NGVD near Crampton Avenue (river mile 2.4)
b) 100-yr 9.8 ft. NGVD and 50-yr 9.0 ft. NGVD near Ideal Mobile Home Park (river mile 3.2)
c) 100-yr average 9.8 ft. NGVD and 50-yr 8.9 ft. NGVD (river mile 2.8)

For this additional alternative (Alternative 6A) developed by Najarian Assoc., it was assumed that the 50-year and 100-year water levels were in agreement with the Camp, Dresser, and Mckee (CDM) report. The CDM report developed predictions of total still water frequency elevations as part of their New York City Flood Insurance Study (Report No. 7), which included analysis node 41 in the Arthur Kill located approximately 1/2 mile north of the mouth of the Woodbridge River. It was the Corps' assumption that the stage-frequency relationship there reflects the water surface elevations directly at the Woodbridge River's mouth, resulting in the following surface water elevations:

100-yr 8.6 ft. NGVD and 50-yr 8.0 ft. NGVD at the Woodbridge River Mouth

It can be further assumed that the FEMA elevations included effects of local channel constrictions and wind fetch present at River Mile 2.8. These channel and wind effects elevate the water level at river mile 2.8 by approximately 12%. Thus taking the CDM stage frequency

relationship at Node 41, and elevating it by 12%, this results in the stage frequency relationship at River Mile 2.8 as follows:

| Storm Event in years | Water Surface Elevation (ft, NGVD) |
|---|---|
| 2 | 6.1 |
| 5 | 6.9 |
| 10 | 7.4 |
| 25 | 8.1 |
| 50 | 8.9 |
| 100 | 9.7 |
| 200 | 10.4 |
| 500 | 11.5 |

The stage-frequency curve that is given above in table form was generated with the information from the CDM study, and it was needed for input for economics to do their benefits and costs analysis. The 50- and 100-year storm event elevations taken from the previous analysis at node 41 is in agreement with the data produced by Najarian.

The stage-frequency table given above was submitted to Economics to do their analysis. The results of the economics can be found in the Main Report and Economics Appendix.

# FINAL FEASIBILITY REPORT
# ON
# FLOOD DAMAGE REDUCTION
# AND ECOSYSTEM RESTORATION
# FEASIBILITY STUDY
# WOODBRIDGE RIVER BASIN,
# MIDDLESEX COUNTY, NEW JERSEY

## **APPENDIX A – CHARTS & FIGURES**

HYDROLOGY and HYDRAULICS

| Table 1: Woodbridge River Sub-basin Physical Parameters and Clark Unitgraph Parameters | | | | | | |
|---|---|---|---|---|---|---|
| Sub-basin Name | DA ($mi^2$) | S (ft/mi) | I (%) | tc (hrs) | R (hrs) | SCS Curve Number |
| Carteret | 1.94 | 8.5 | 37.5 | 0.75 | 1.39 | 86 |
| Avenel | 1.97 | 2.8 | 48.5 | 0.78 | 1.46 | 88 |
| New Jersey Turnpike | 0.82 | 21.41 | 28.2 | 0.60 | 1.11 | 81 |
| Port Reading | 0.28 | 3.5 | 23.8 | 0.86 | 1.60 | 84 |
| Heard's Brook | 2.18 | 64.9 | 46.7 | 0.35 | 0.64 | 93 |
| Sewaren | 0.95 | 3.8 | 37.5 | 0.76 | 1.42 | 90 |
| Spa Spring | 1.91 | 27.0 | 37.5 | 0.54 | 1.00 | 91 |
| Arthur Kill | 0.46 | 1.7 | 41.0 | 0.72 | 1.33 | 91 |

| Table 2: Muskingum Routing Parameters | | |
|---|---|---|
| Reach Name | Muskingum K (hrs) | Muskingum X |
| Wood 1: (Railroad Bridge to Port Reading Avenue) | 2.22 | 0.2 |
| Wood 2: (Port Reading Avenue to NJ Turnpike Bridge) | 1.00 | |
| Wood 3: (NJ Turnpike Bridge to Spa Spring Confluence) | 0.75 | |
| Wood 4: (Spa Spring Confluence to Confluence with Arthur Kill) | 1.01 | |

| Table 3: Hypothetical Rainfall Data for Woodbridge River Drainage Basin | | | | | | | |
|---|---|---|---|---|---|---|---|
| DURATION | | | RAINFALL DEPTH (in) FOR EACH HYPOTHETICAL EVENT | | | | |
| Minutes | Hours | Days | 2-year | 10-year | 50-year | 100-year | 500-year |
| 5 | | | 0.43 | 0.56 | 0.71 | 0.77 | 0.90 |
| 15 | | | 0.80 | 1.13 | 1.50 | 1.64 | 2.00 |
| 60 | 1 | | 1.45 | 2.15 | 2.83 | 3.10 | 3.75 |
| 120 | 2 | | 1.75 | 2.70 | 3.48 | 3.80 | 4.75 |
| 180 | 3 | | 2.00 | 2.90 | 3.90 | 4.30 | 5.30 |
| 360 | 6 | | 2.40 | 3.60 | 4.75 | 5.20 | 6.45 |
| 720 | 12 | | 2.85 | 4.30 | 5.70 | 6.30 | 7.80 |
| 1440 | 24 | 1 | 3.35 | 5.10 | 6.70 | 7.35 | 9.10 |

| Table 4: Selected Existing Conditions Peak Discharges | | | | | |
|---|---|---|---|---|---|
| Sub-basin | 2-year Peak Discharge (cfs) | 10-year Peak Discharge (cfs) | 50-year Peak Discharge (cfs) | 100-year Peak Discharge (cfs) | 500-year Peak Discharge (cfs) |
| Carteret | 668 | 1274 | 1771 | 1963 | 2463 |
| Avenel | 718 | 1315 | 1802 | 1991 | 2479 |
| New Jersey Turnpike | 258 | 539 | 779 | 873 | 1115 |
| Port Reading | 74 | 153 | 218 | 243 | 310 |
| Heard's Brook | 1457 | 2530 | 3419 | 3758 | 4616 |
| Sewaren | 356 | 653 | 893 | 986 | 1226 |
| Spa Spring | 909 | 1645 | 2250 | 2482 | 3073 |
| Arthur Kill | 183 | 331 | 452 | 499 | 619 |
| Inflow to Tide Gate Structure | | | | | |
| New Jersey Turnpike Bridge | 1690 | 3009 | 4144 | 4586 | 5707 |

**Table 5:**
**Interior Water Surface Elevations of Tide Gate Structure with Sluice Gate (ft., NGVD29)**

| Interior Flow | Exterior Stage | Time Condition | Peak Interior WSEL | Peak Exterior Stage | Interior Flow | Exterior Stage | Time Condition | Peak Interior WSEL | Peak Exterior Stage | Max WSEL | Risk |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 yr | Normal | Present | 4.00 | 3.20 | | | | | | 4.00 | Low |
| 10 yr | Normal | Present | 4.78 | 3.20 | | | | | | 4.78 | Low |
| 50 yr | Normal | Present | 5.92 | 3.20 | | | | | | 5.92 | Low |
| 100 yr | Normal | Present | 6.24 | 3.20 | | | | | | 6.24 | Low |
| 500 yr | Normal | Present | 7.10 | 3.20 | | | | | | 7.10 | Low |
| 2 yr | 2 yr | Present | 5.19 | 5.08 | 2 yr | 2 yr | Present | 5.19 | 5.08 | 5.19 | Likely |
| 10 yr | 2 yr | Present | 6.14 | 5.08 | 2 yr | 10 yr | Present | 6.27 | 7.58 | 6.27 | Likely |
| 50 yr | 2 yr | Present | 6.91 | 5.08 | 2 yr | 50 yr | Present | 6.97 | 10.67 | 6.97 | Likely |
| 100 yr | 2 yr | Present | 7.22 | 5.08 | 2 yr | 100 yr | Present | 7.24 | 11.85 | 7.24 | Likely |
| 500 yr | 2 yr | Present | 8.11 | 5.08 | | | | | | | |
| 2 yr | 10 yr | Present | 6.27 | 7.58 | 10 yr | 2 yr | Present | 6.14 | 5.08 | 6.27 | High |
| 10 yr | 10 yr | Present | 7.37 | 7.58 | 10 yr | 10 yr | Present | 7.37 | 7.58 | 7.37 | High |
| 50 yr | 10 yr | Present | 8.22 | 7.58 | 10 yr | 50 yr | Present | 8.38 | 10.67 | 8.38 | High |
| 100 yr | 10 yr | Present | 8.51 | 7.58 | 10 yr | 100 yr | Present | 8.68 | 11.85 | 8.68 | High |
| 500 yr | 10 yr | Present | 9.46 | 7.58 | | | | | | | |

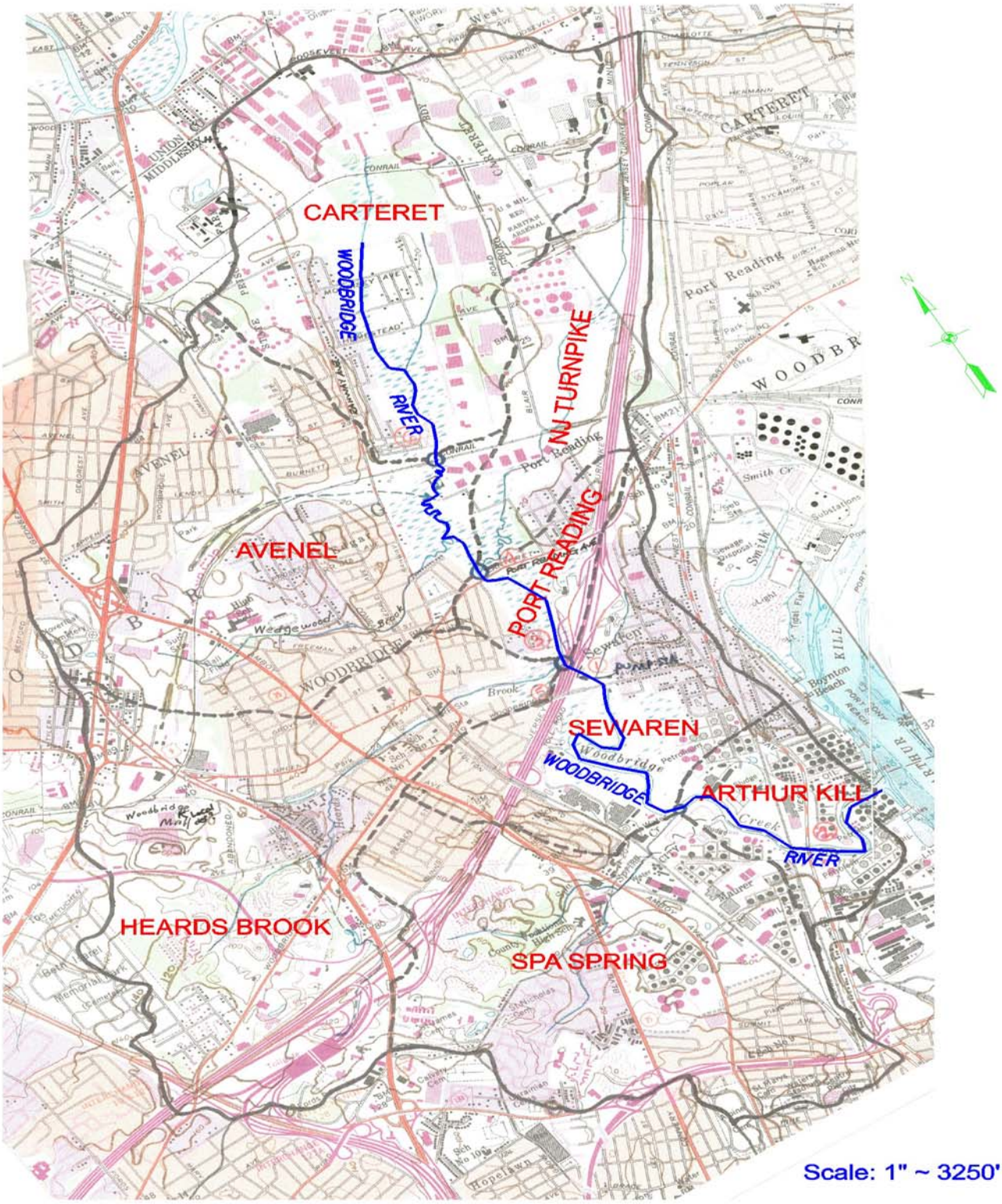



Figure 1: Woodbridge River Watershed Map and Sub-Basin Delineation

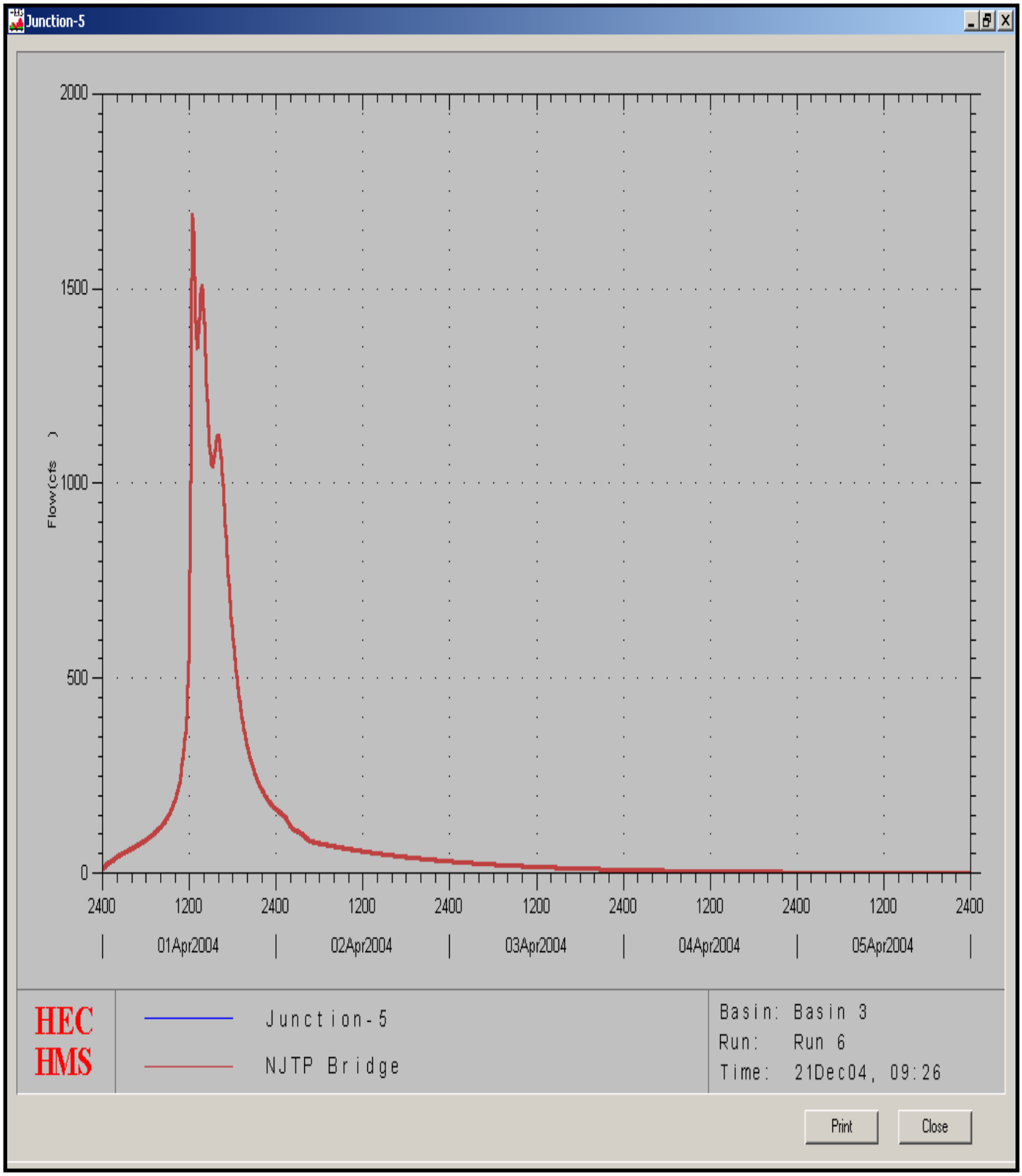



Figure 2: Hydrograph of inflow to tide gate structure (2-year flood)

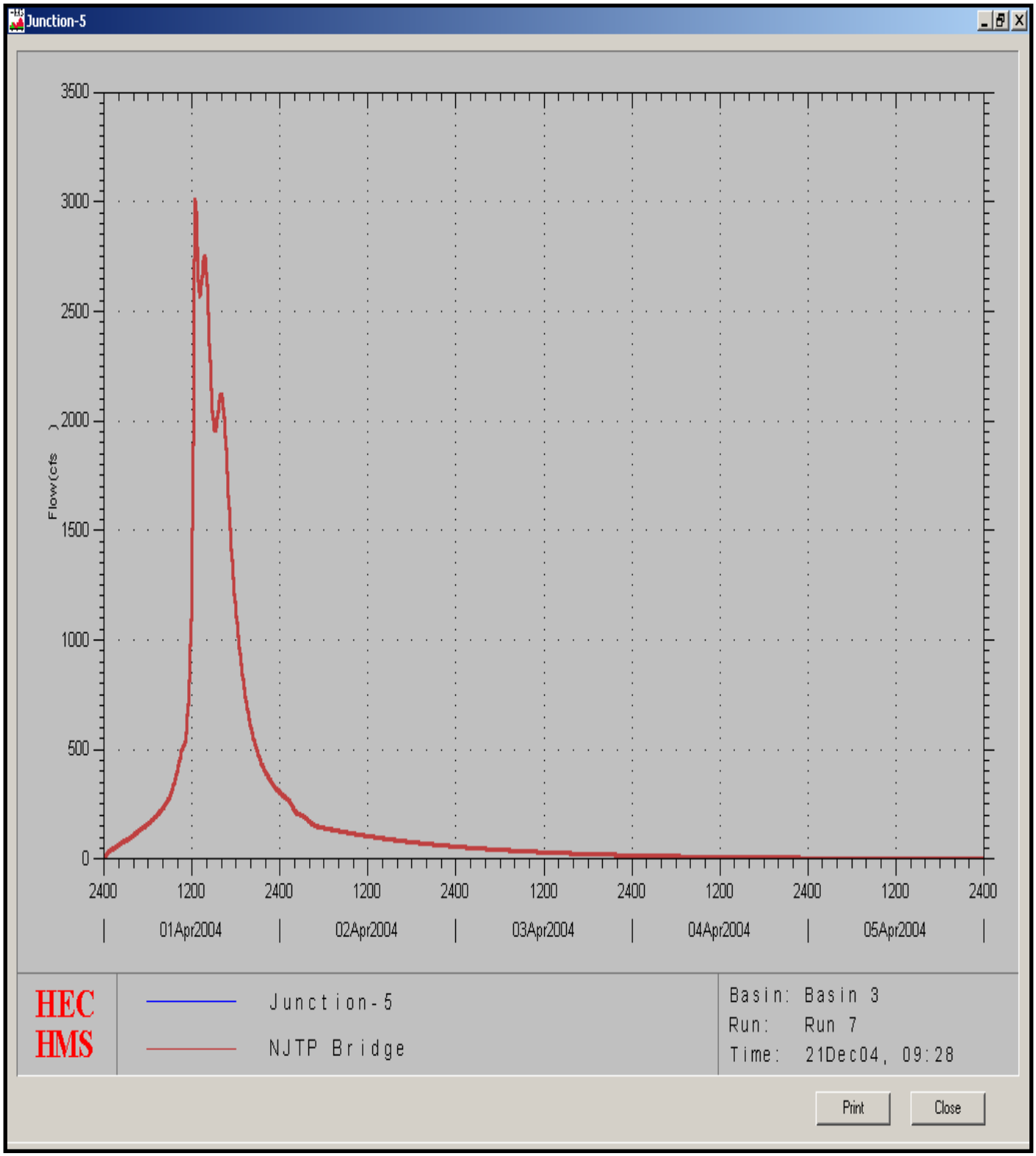



Figure 3: Hydrograph of inflow to tide gate structure (10-year flood)

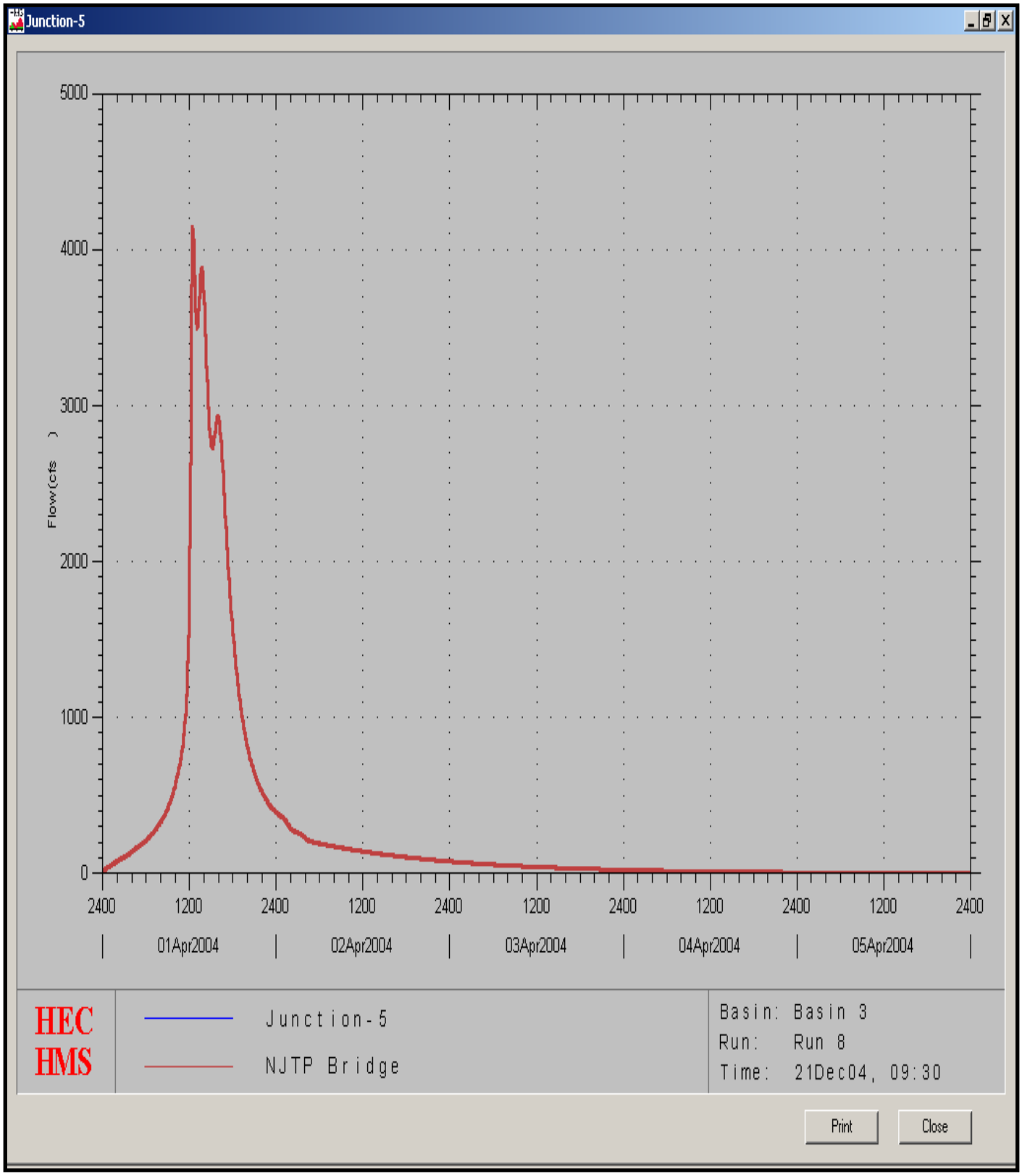



Figure 4: Hydrograph of inflow to tide gate structure (50-year flood)

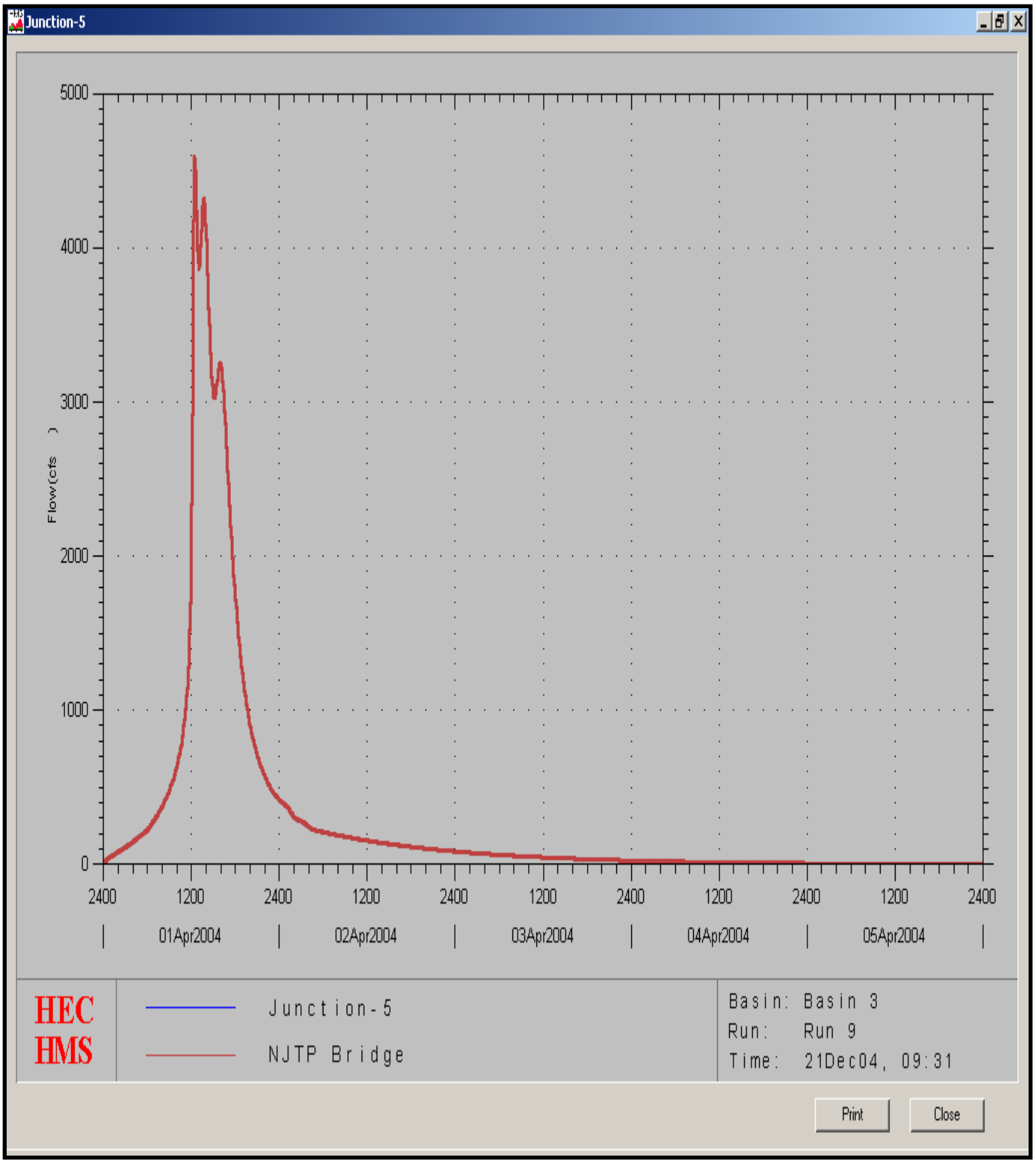



Figure 5: Hydrograph of inflow to tide gate structure (100-year flood)

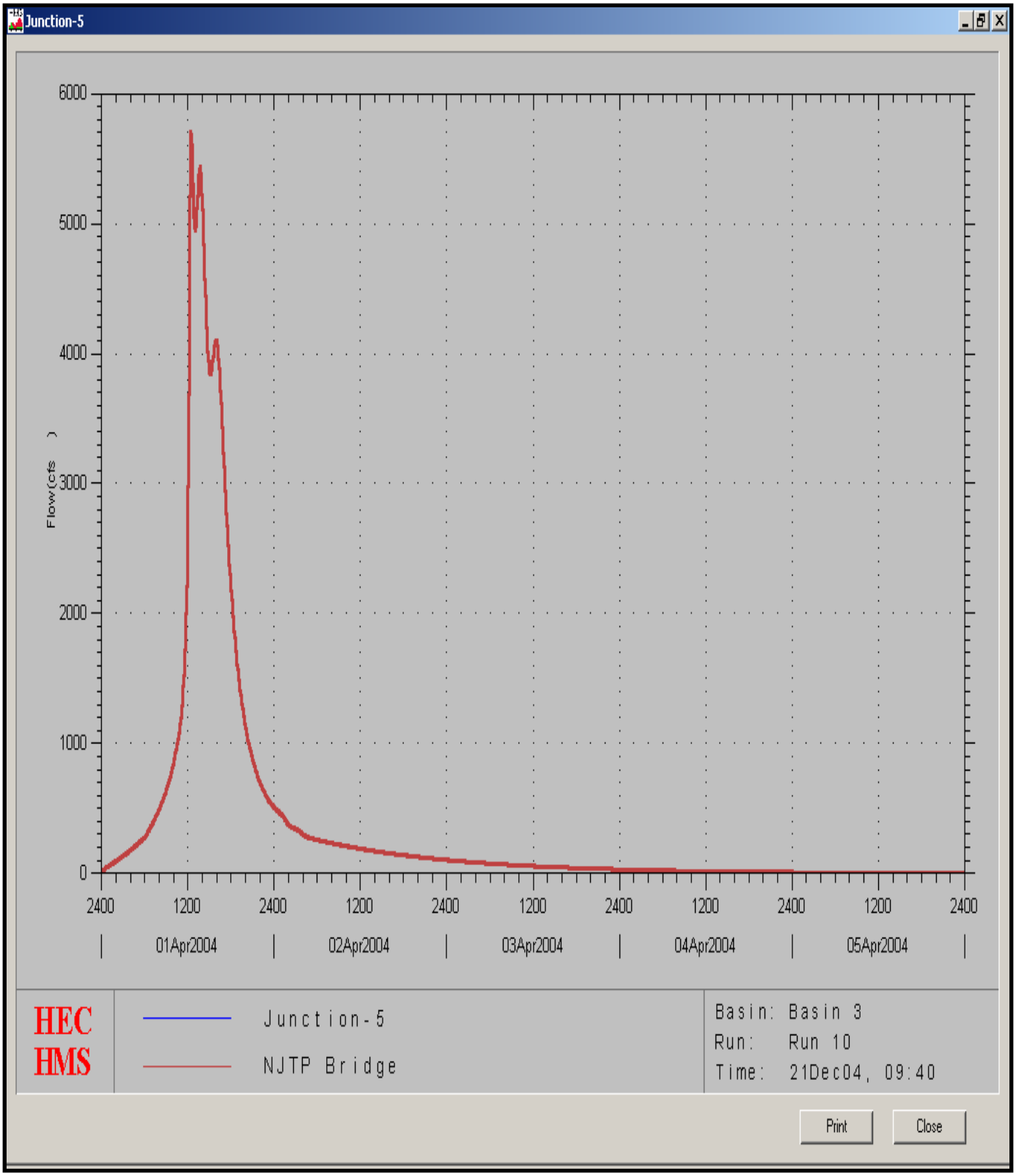



Figure 6: Hydrograph of inflow to tide gate structure (500-year flood)

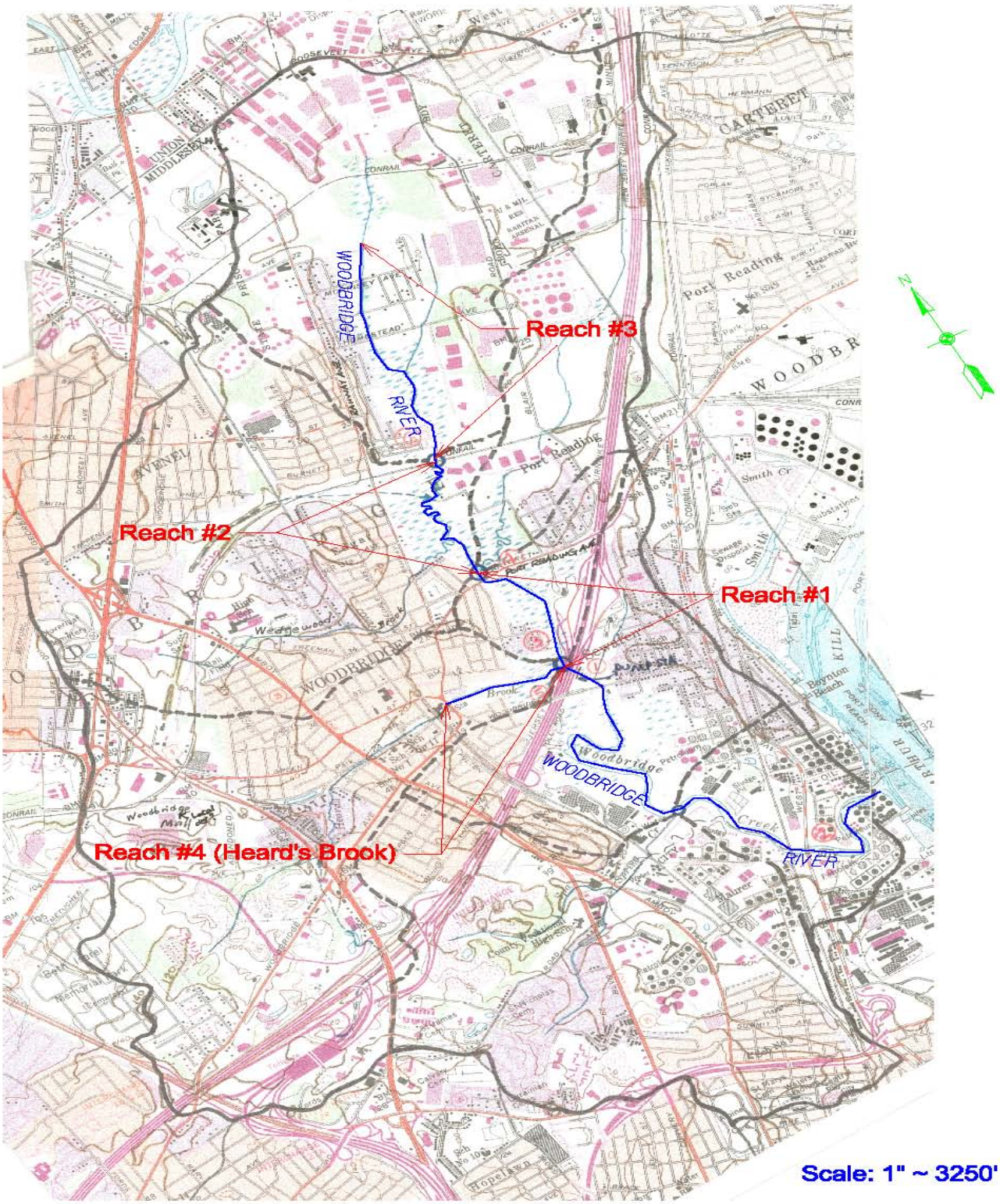



Figure 7: Location of Reaches along Woodbridge River and Heard's Brook

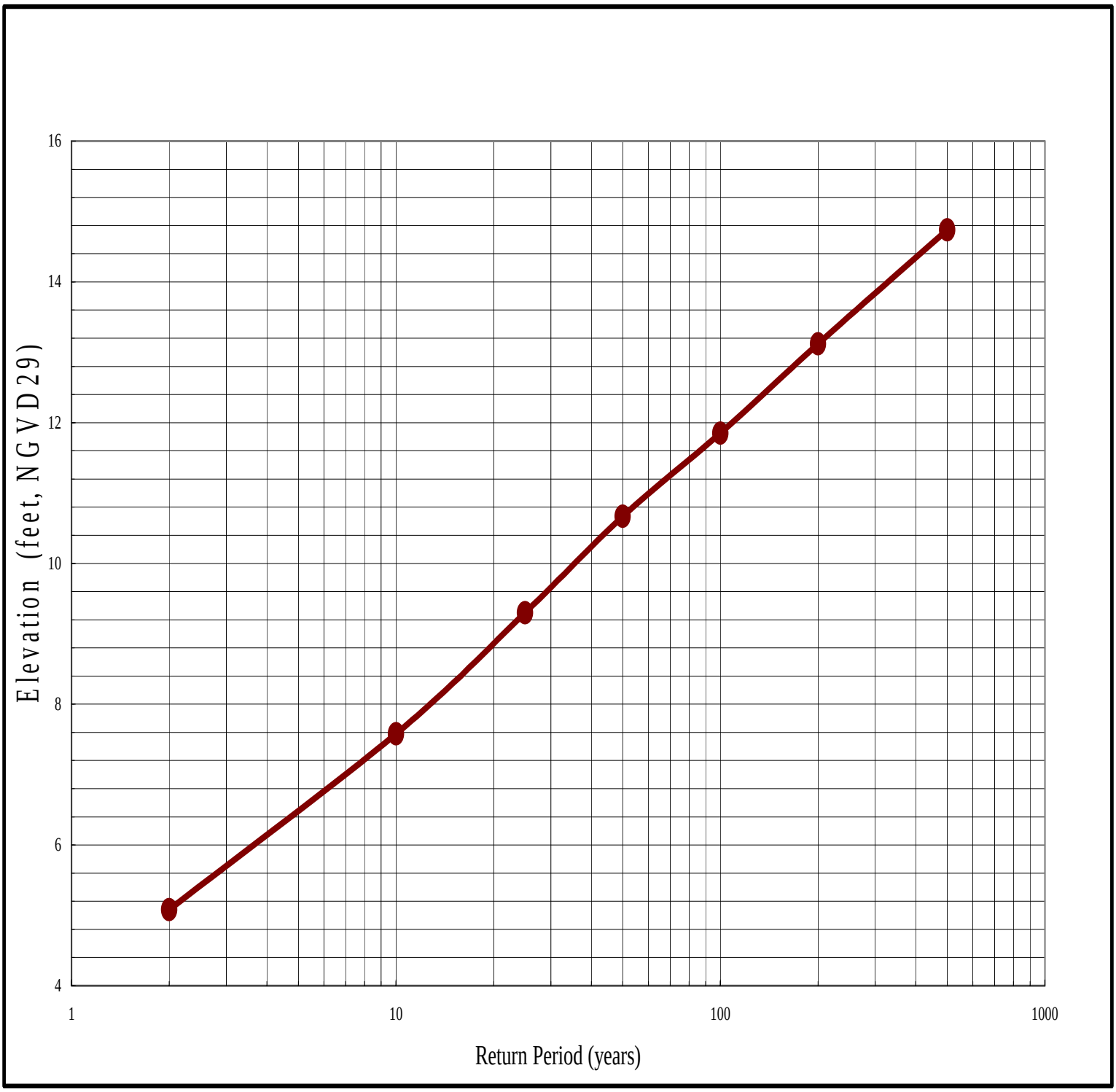



Figure 8: Existing Conditions Stage Frequency Curve for the Woodbridge River at its mouth at Arthur Kill

Figure 9: Floodwall and/or Levee Segment (100-year Level of Protection at Crampton Avenue Neighborhood)

Figure 10: Floodwall and/or Levee Segment (100-year Level of Protection at Ideal Mobile Home Park Along Rahway Avenue)

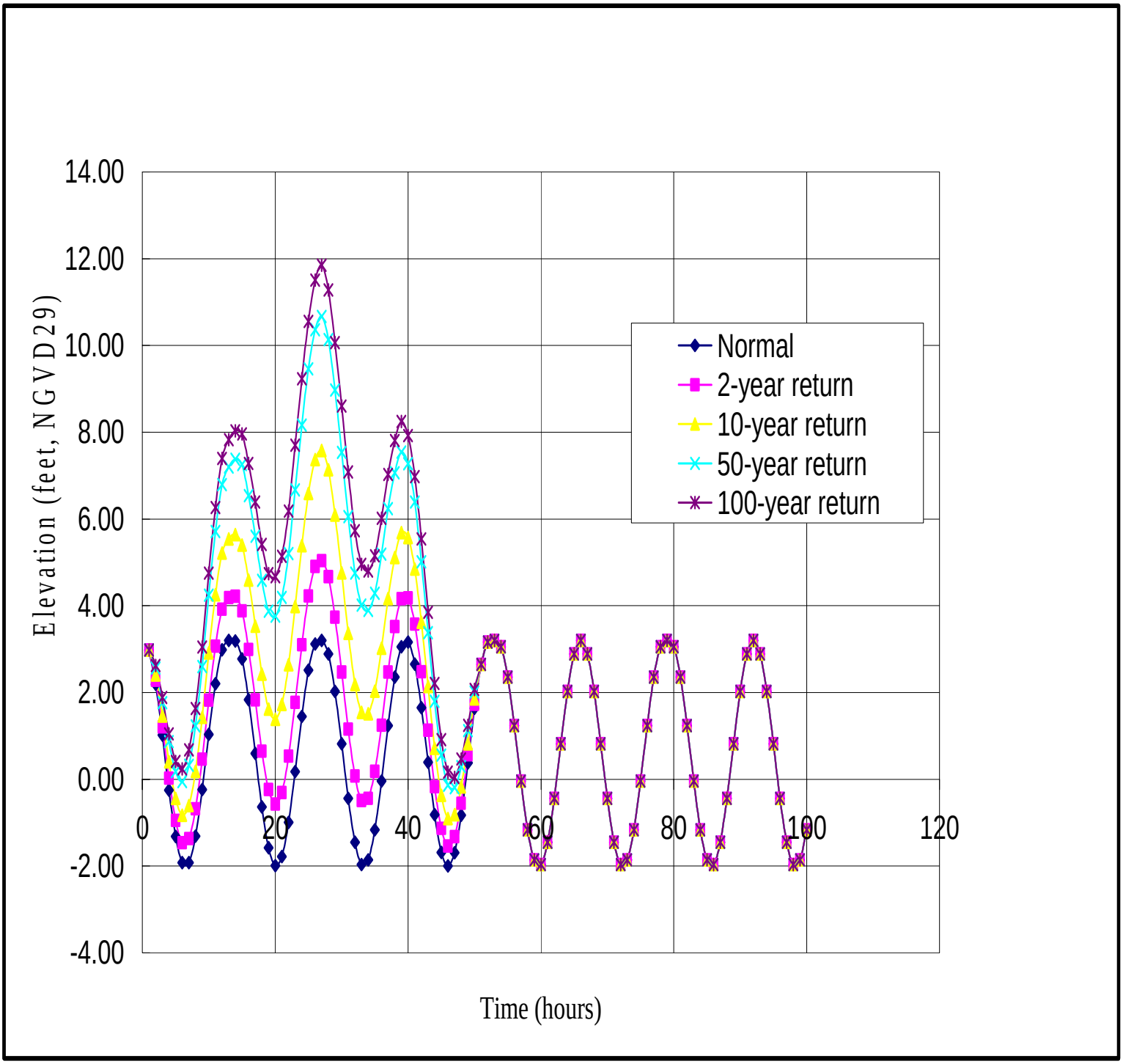



Figure 11: Exterior Tidal Marigram for all storm events (Estimated marigram at Woodbridge River & Arthur Kill Confluence using Sandy Hook NOAA Station (ID: 8531680))

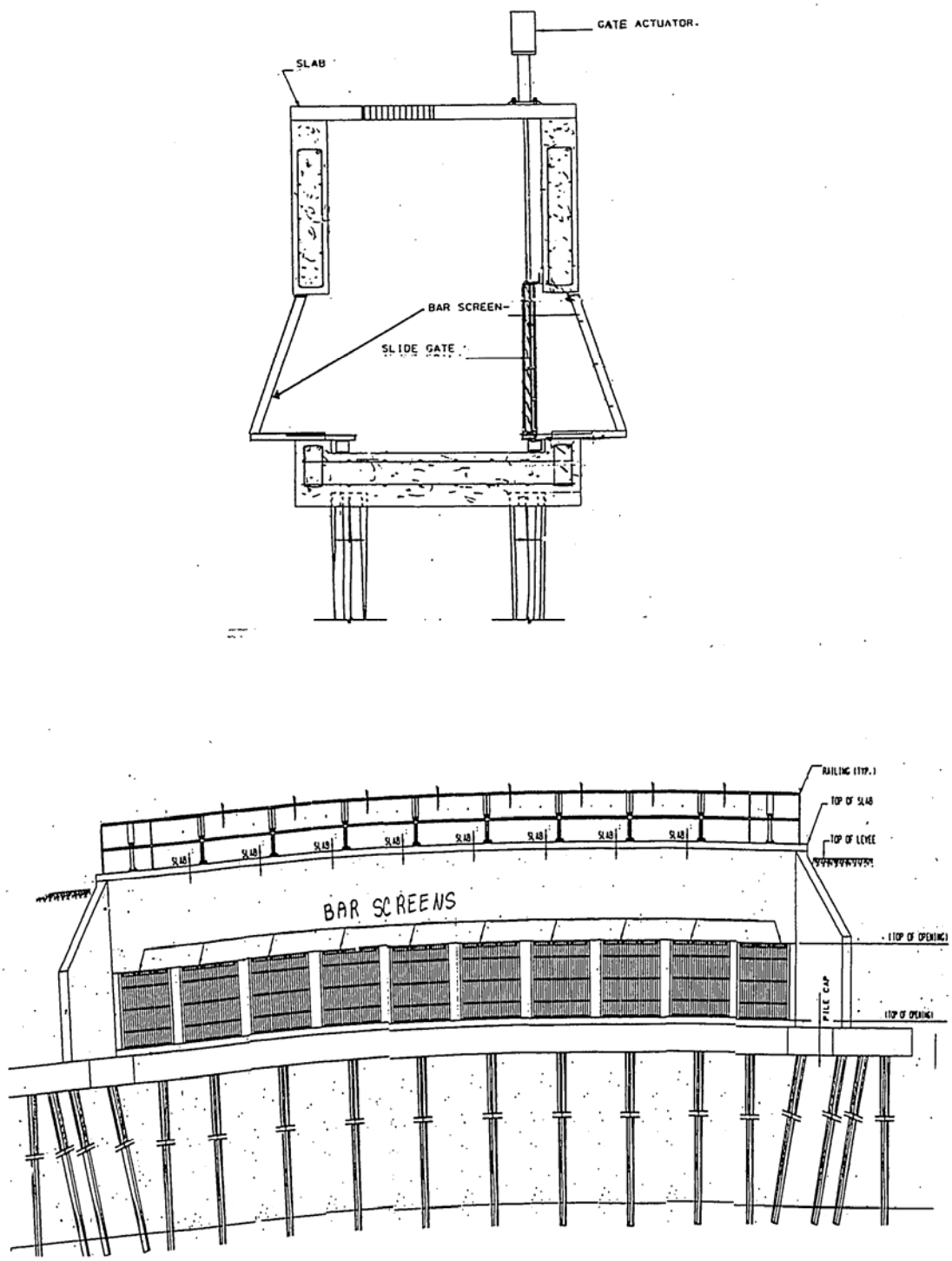



Figure 12: Side and Front View of the Proposed Tide Gate Structure (Not to Scale)

Figure 13: Tide Gate Structure and Sluice Gate (100-year Level of Protection)

Figure 14: Alignment A – Levee System and Tide Gate Structure (50-year Level of Protection)

Figure 15: Alignment B – Levee System and Tide Gate Structure (50-year Level of Protection)

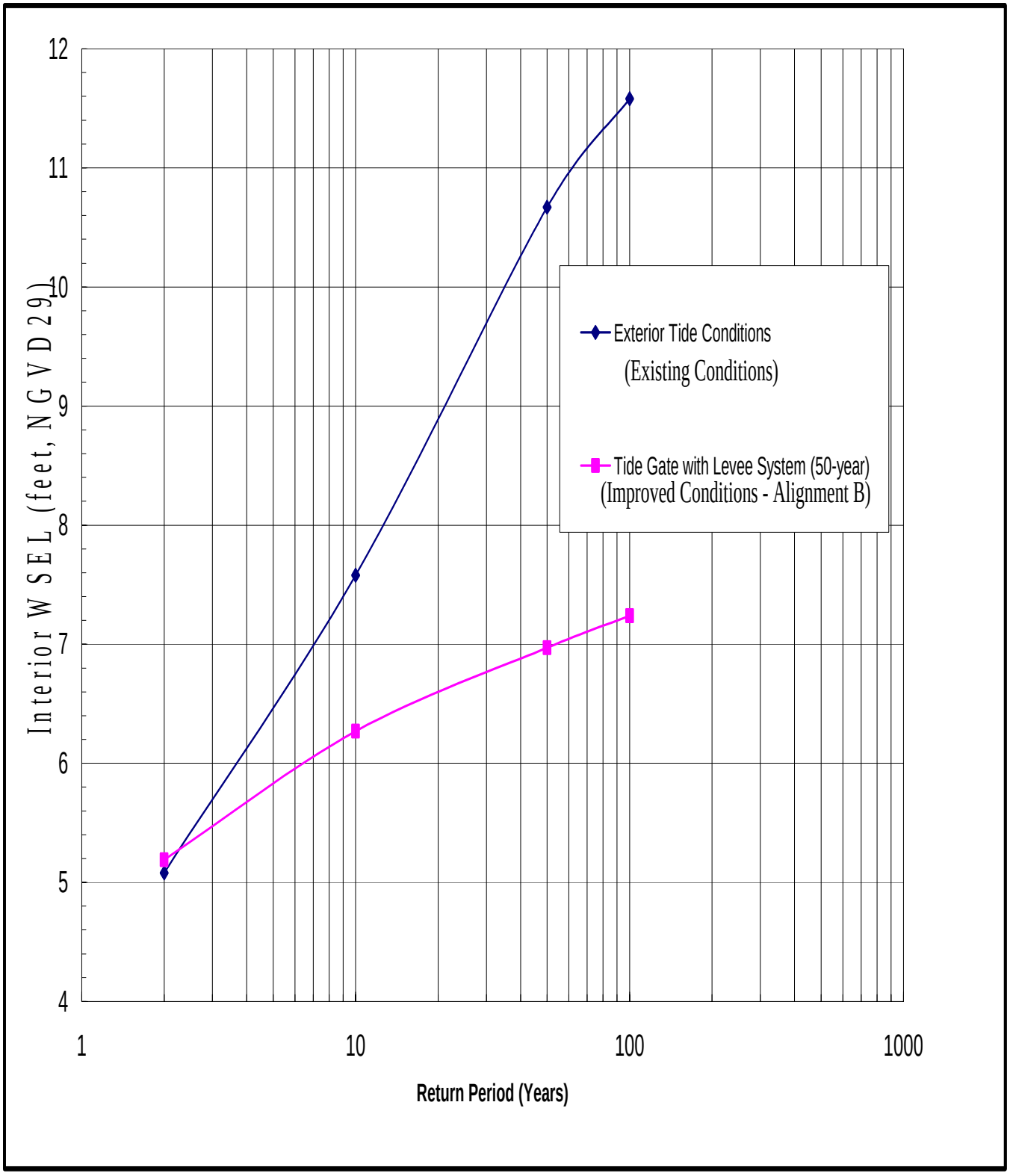



Figure 16: Stage Frequency Curve for existing and improved conditions at the upstream side of the New Jersey Turnpike Bridge

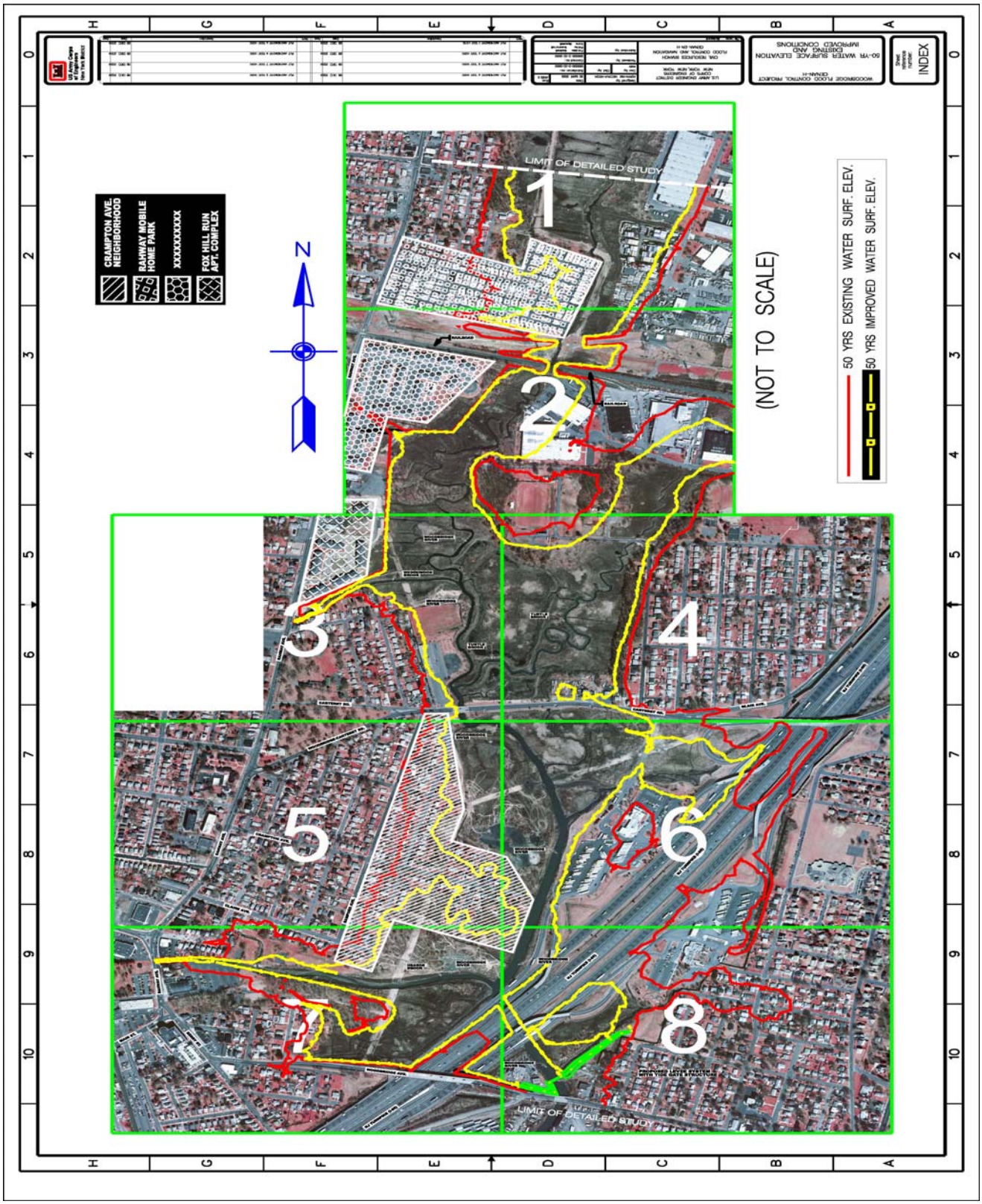
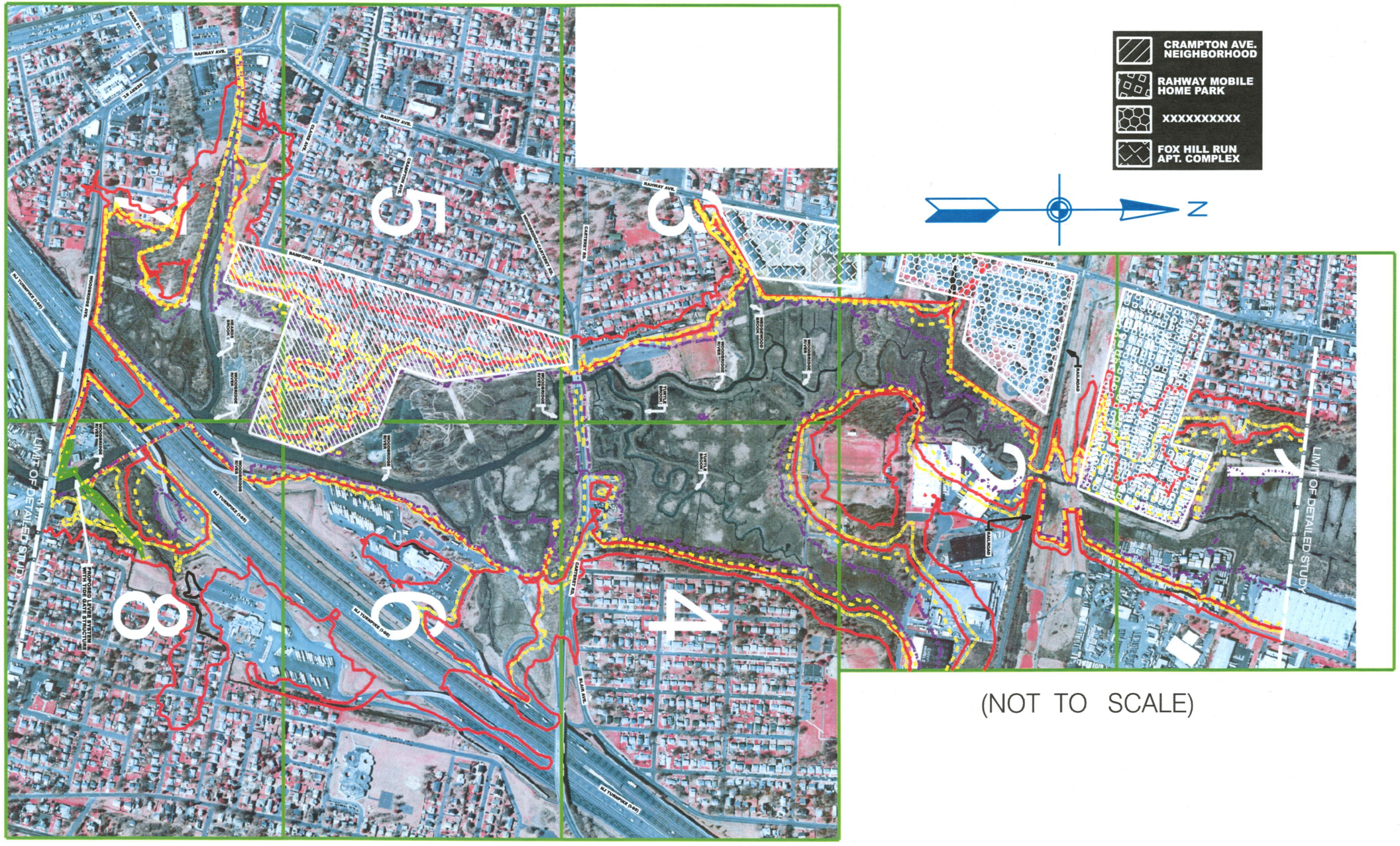



Figure 17: Inundation limits for existing and improved conditions upstream of the New Jersey Turnpike Bridge




(NOT TO SCALE)

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WOODBRIDGE FLOOD CONTROL PROJECT
CENAN-H

2-YR, 10-YR AND 50-YR
EXISTING AND
IMPROVED CONDITIONS

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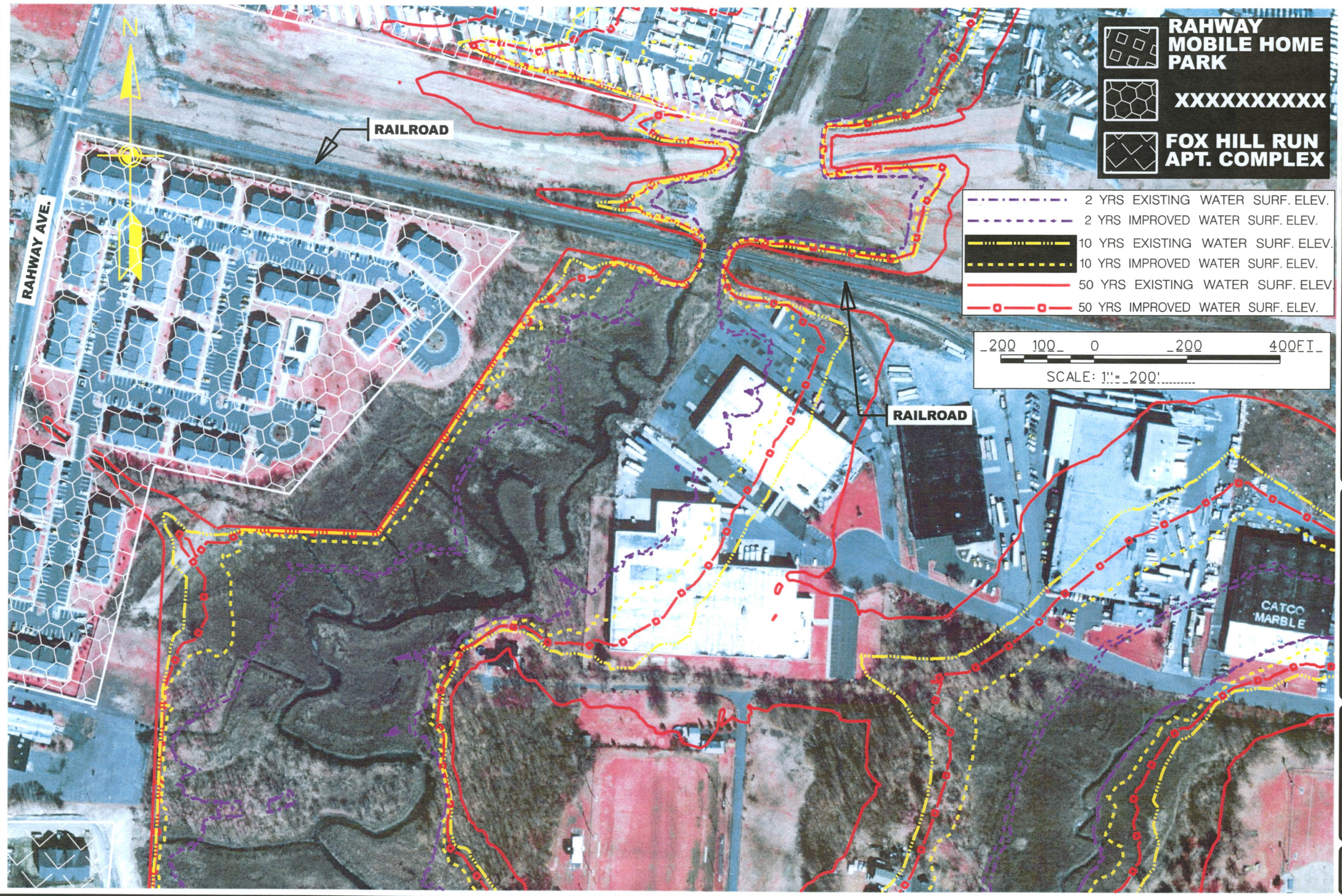


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Date: 30 MAR 2005
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Contract No.: DACW51-00-D-0003
Prepared by: Rhumb Line
Submitted by: Rhumb Line

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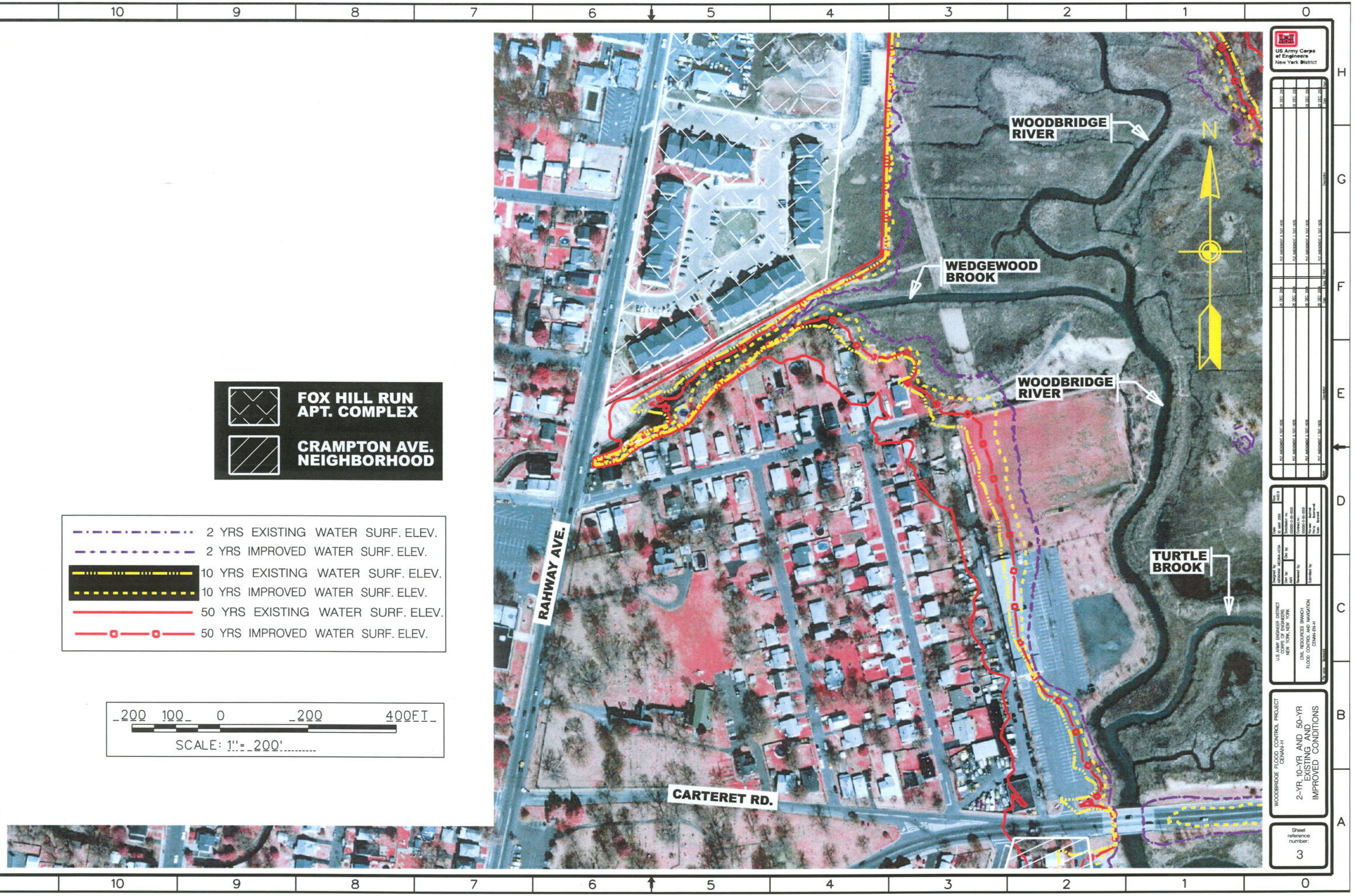


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Designed by: Michael Mahaka-Kotva
Date: April 2005
Solicitation no.: 0000-0-00-0000
Contract no.: 0000-0-00-0000
Drawn by: AM
Checked by:
Submitted by:
Approved by: Michael Mahaka-Kotva
Approved by: Bruce Bachmann

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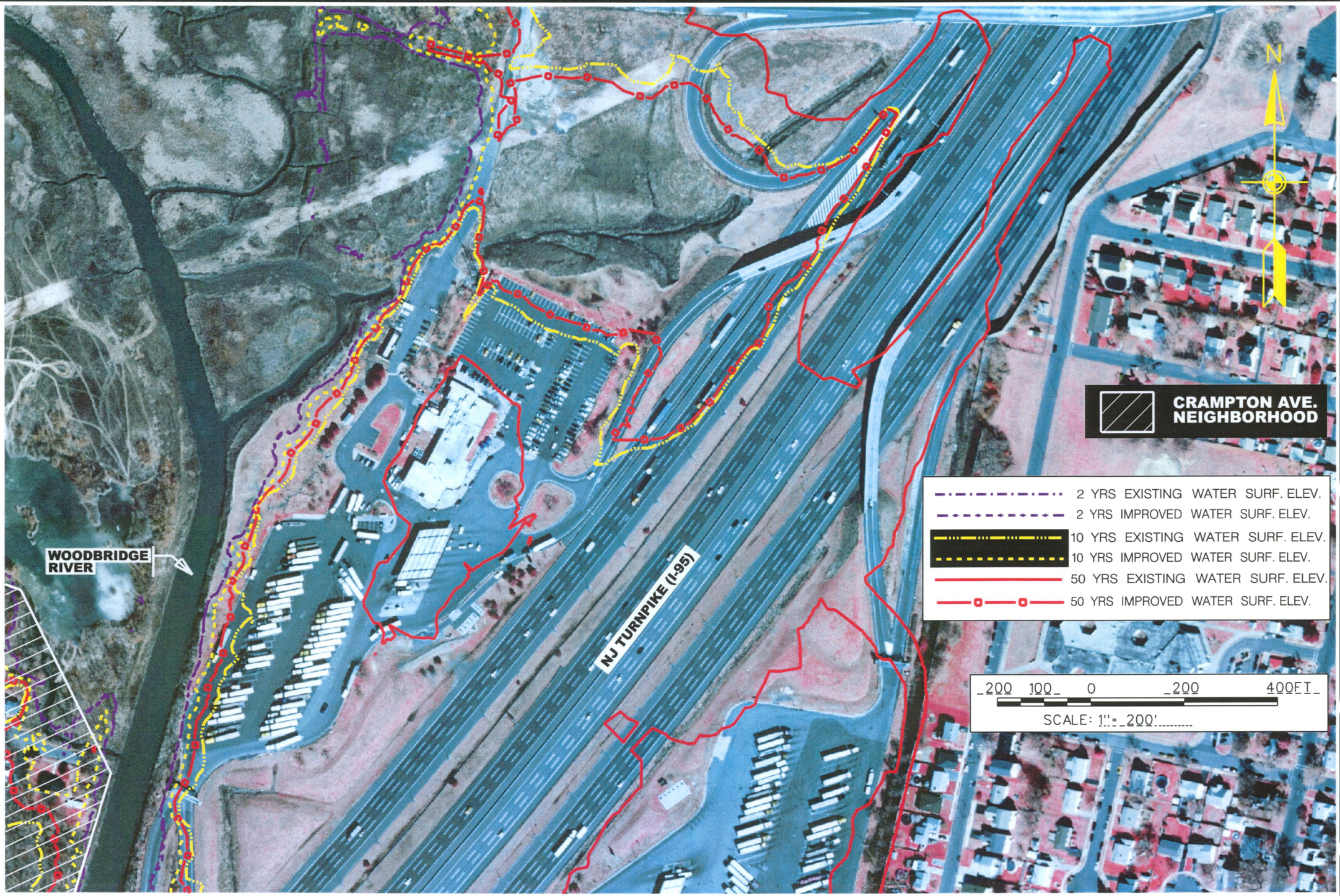


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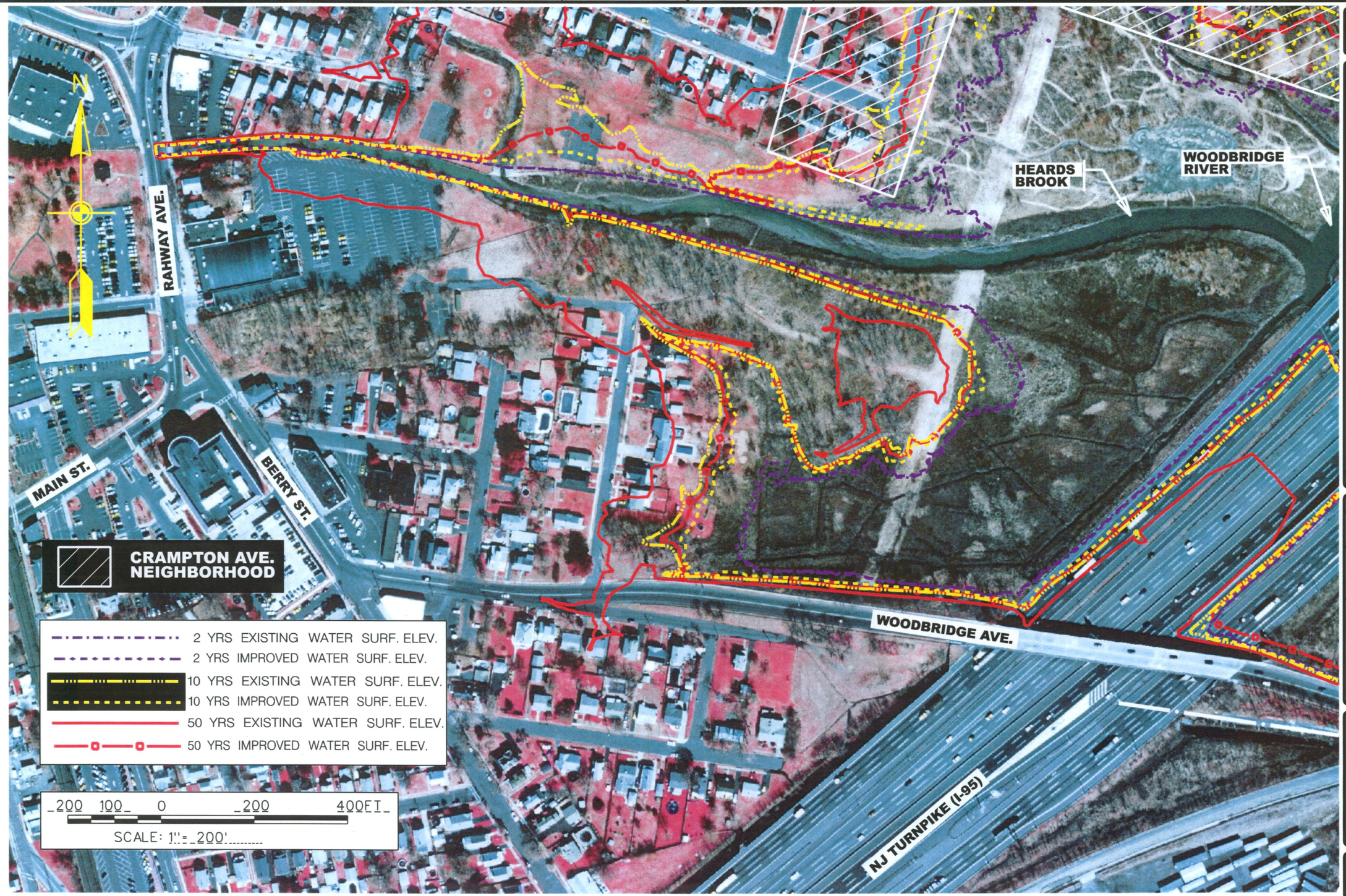


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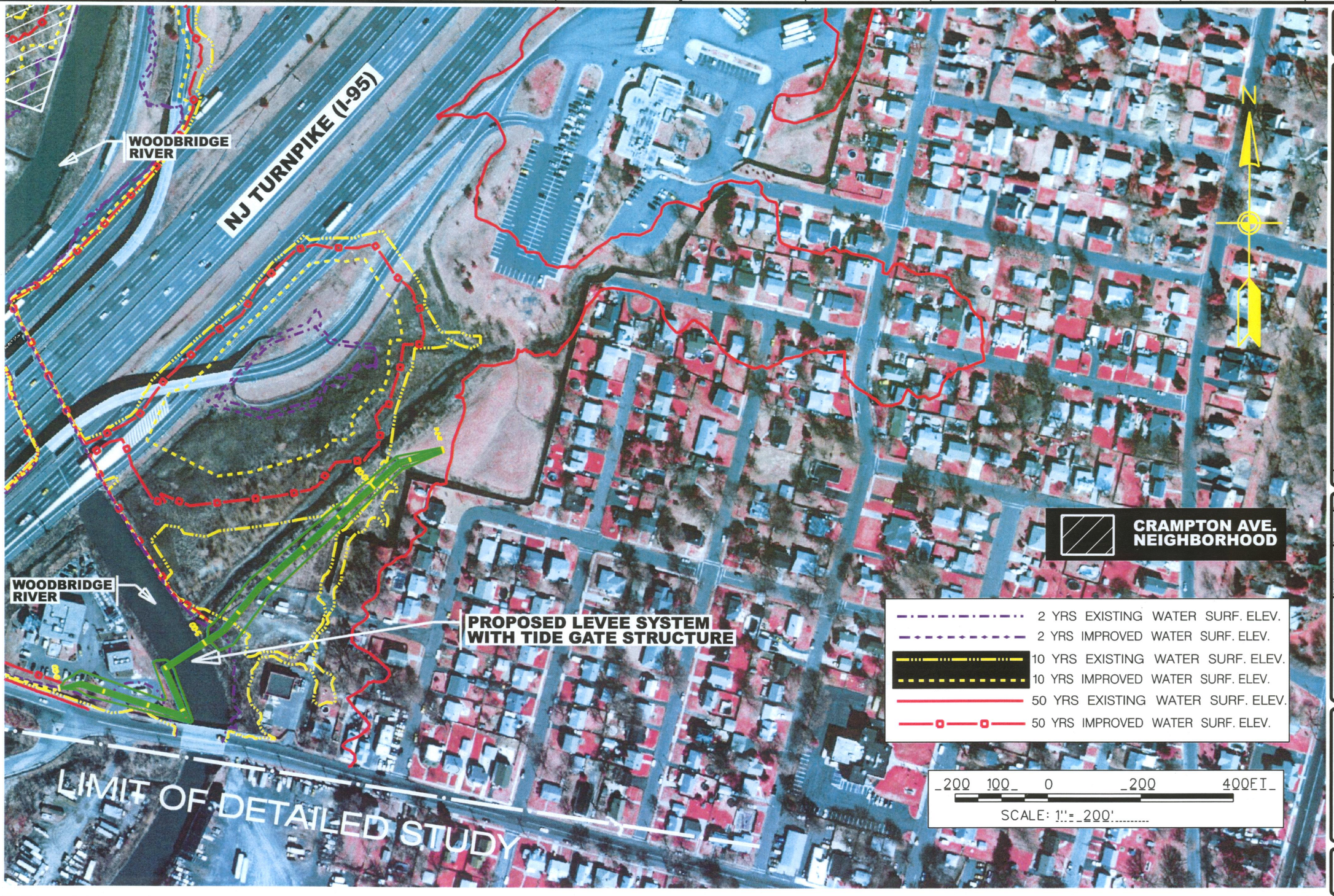


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2-YR, 10-YR AND 50-YR EXISTING AND IMPROVED CONDITIONS

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Submitted by: USERNAME

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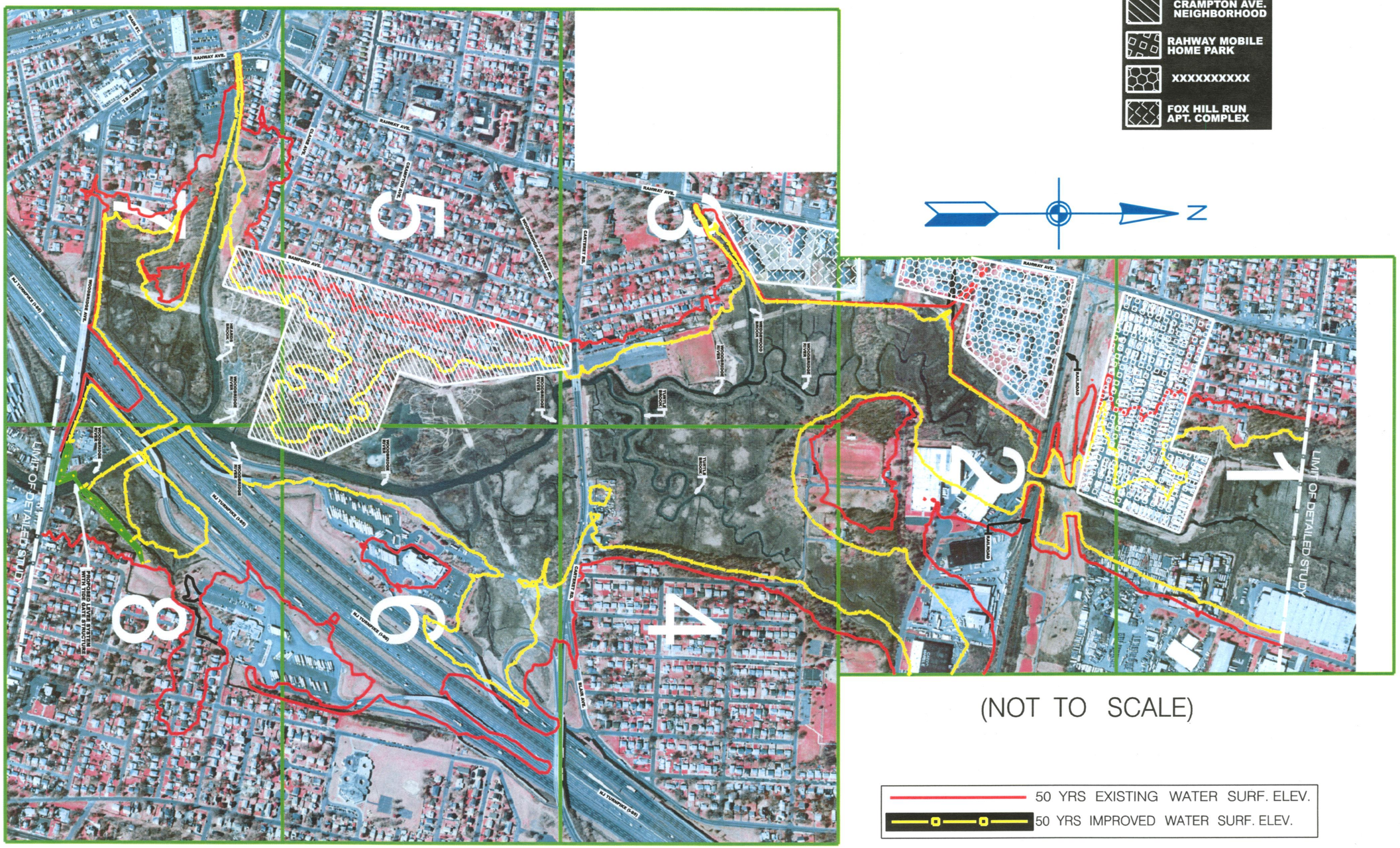


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50-YR WATER SURFACE ELEVATION
EXISTING AND
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NEW YORK, NEW YORK

CIVIL RESOURCE BRANCH
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Designed by: Abhishek MEDIA-4/15/4
Date: 18 MAR 2005
Drawn by: Dwg No:
Reviewed by:
Submitted by:
Contract no.: 02/03-0-0-0000
Approved by: Date: Approved

Sheet reference number: 2

FOX HILL RUN APT. COMPLEX

CRAMPTON AVE. NEIGHBORHOOD

50 YRS EXISTING WATER SURF. ELEV.

50 YRS IMPROVED WATER SURF. ELEV.

200 100 0 200 400FT

SCALE: 1" = 200'

US Army Corps of Engineers New York District

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50-YR WATER SURFACE ELEVATION EXISTING AND IMPROVED CONDITIONS

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New York District

| Rev. | Date | Description | Drawn | Chk'd | Appd. |
|---|---|---|---|---|---|
| 1 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 2 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 3 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 4 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 5 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 6 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 7 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |
| 8 | 28 DEC 2005 | PLY AMENDMENT & TEXT UPDATE | | | |

Designed by: Hydrologic HEC-RAS-UTDA
Drawn by:
Checked by:
Reviewed by:
Submitted by:
Date: 18 MAR 2005
Scale: NTS
Solicitation no.: DACW51-01-B-0009
Contract no.: W912DQ-04-C-0020
Plot date:
Approved by:
Date:
Revised:

U.S. ARMY ENGINEER DISTRICT
CONTRACTOR: DAVID BERG
NEW YORK, NEW YORK

CIVIL RESOURCES BRANCH
FLOOD CONTROL AND NAVIGATION
CENAN-CE-H

WOODBRIDGE FLOOD CONTROL PROJECT
CENAN-H

50-YR WATER SURFACE ELEVATION
EXISTING AND
IMPROVED CONDITIONS

Sheet reference number:
5

US Army Corps of Engineers
New York District

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

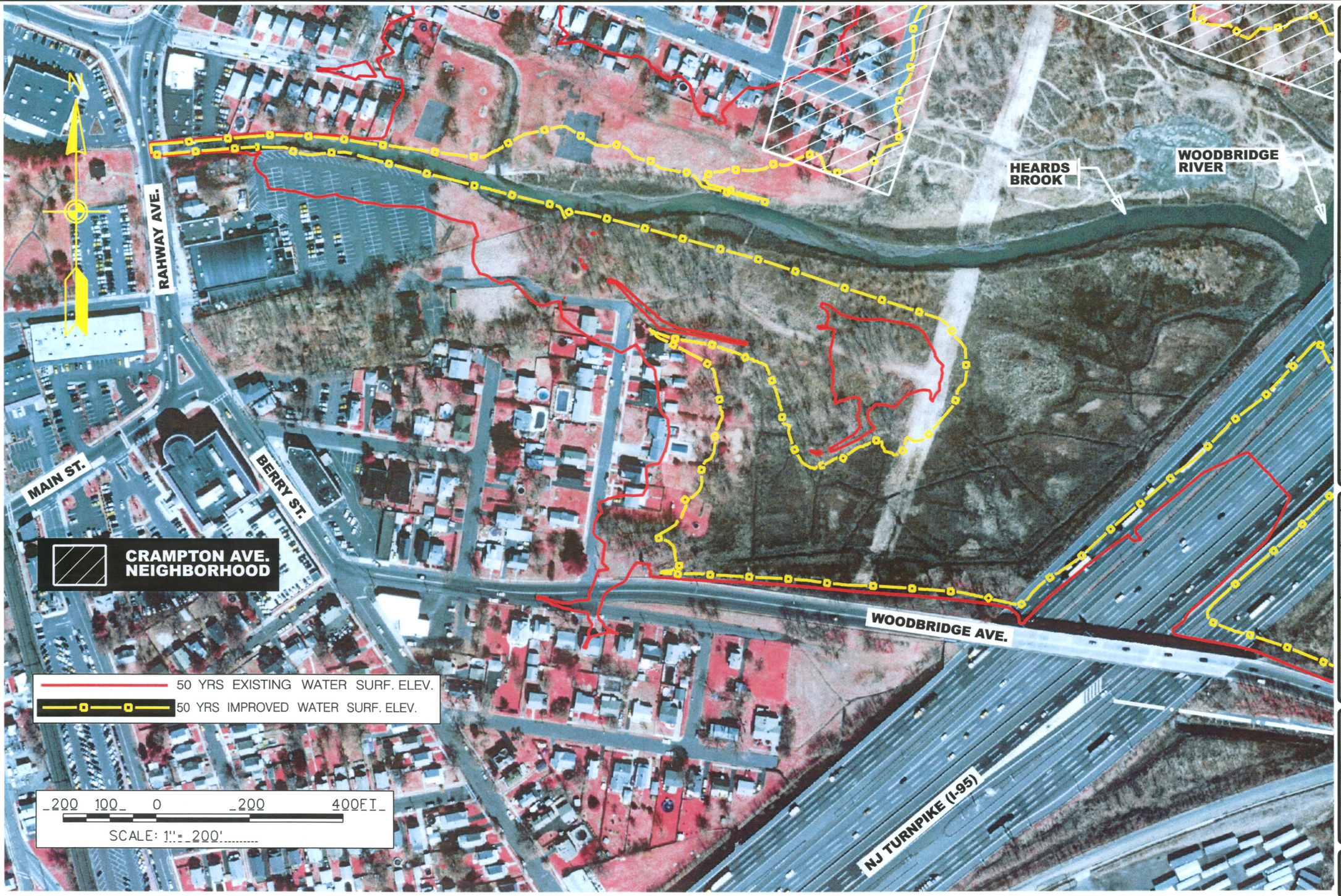
CIVIL RESOURCES BRANCH
FLOOD CONTROL AND NAVIGATION
CIVIL-PN-H

WOODBRIDGE FLOOD CONTROL PROJECT
CIVIL-H-1

50-YR WATER SURFACE ELEVATION
EXISTING AND
IMPROVED CONDITIONS

Sheet reference number: 6

Woodbridge Flood Plain(Ex&Imp)-Fin 05/17/2005 09:58:40 AM




US Army Corps of Engineers
New York District

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CORPS OF ENGINEERS
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CIVIL RESOURCES BRANCH
FLOOD CONTROL AND NAVIGATION
CENAN-EN-H

WOODBRIDGE FLOOD CONTROL PROJECT
CENAN-H

50-YR WATER SURFACE ELEVATION
EXISTING AND
IMPROVED CONDITIONS

Sheet reference number:
7

Woodbridge Flood Plain(Ex&Imp)-Fin 05/17/2005 09:57:49 AM

US Army Corps of Engineers
New York District

U.S. ARMY ENGINEER DISTRICT
JACOB K. JAVITS FEDERAL BLDG.
NEW YORK, NEW YORK

CIVIL RESOURCES BRANCH
FLOOD CONTROL AND NAVIGATION
CENAN-EN-H

WOODBRIDGE FLOOD CONTROL PROJECT
CENAN-H

50-YR WATER SURFACE ELEVATION
EXISTING AND
IMPROVED CONDITIONS

Sheet reference number:
8

Woodbridge Flood Plain(Ex&Imp)-Fin 05/17/2005 09:56:39 AM