Suffolk County Dept. of Public Works

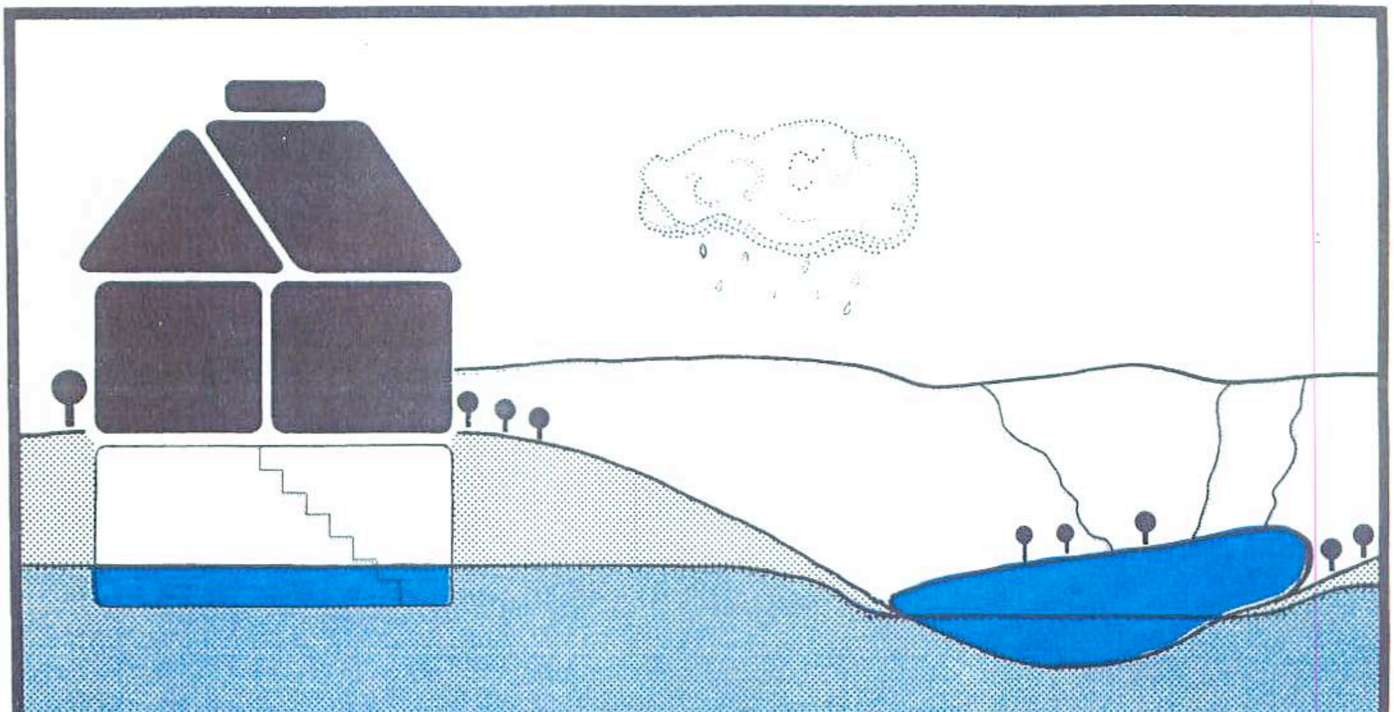
Drainage Improvements

including

Groundwater Relief Phase I-Feasibility Study

Capital Project No. 5013

Volume 2 - Solutions to Flooding in the Vicinity of The Northeast Branch of the Nissequogue River



March 1980

Holzmacher, McLendon and Murrell, P.C. / H2M Corp.
Consulting Engineers, Planners and Environmental Scientists
Melville, N.Y. Farmingdale, N.Y. Riverhead, N.Y. Newton, N.J.

William S. Matsunaye, Jr., P.E. Consulting Civil Engineer; Medford, N.Y.

Suffolk County Dept. of Public Works

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Phase I - Feasibility Study

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Volume 2 - Solutions to Flooding in the Vicinity of
The Northeast Branch of the Nissequogue River

Area of Study

Hamlets of Smithtown, Hauppauge,
Nesconset, Ronkonkoma,
Village of the Branch and
Environs in the Towns of Islip,
Smithtown and Brookhaven.

County Executive
Peter F. Cohalan

Commissioner, Department of Public Works
R.M. Kammerer, P.E., L.S.

Chief Engineer, Department of Public Works
A. Barton Cass, P.E.

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Associated Engineers

March 31, 1980

Commissioner Rudolph M. Kammerer, P.E., L.S.
Department of Public Works
County of Suffolk
Yaphank Avenue
Yaphank, New York 11980

Dear Commissioner Kammerer:

We are pleased to transmit herewith Volume 2 (SOLUTIONS TO FLOODING IN THE VICINITY OF THE NORTHEAST BRANCH OF THE NISSEQUOGUE RIVER) of our Report on the FEASIBILITY OF DRAINAGE IMPROVEMENTS, INCLUDING GROUNDWATER RELIEF, in the Hamlets of Smithtown, Hauppauge, Nesconset, Ronkonkoma, Village of the Branch and Environs in the Towns of Islip, Smithtown and Brookhaven, designated as Capital Project No. 5013. This is in accordance with our contract with the Suffolk County Department of Public Works.

Volume 2 addresses the flooding problems which exist in the vicinity of the Northeast Branch of the Nissequogue River. Based on the data base, that was compiled in Volume 1 of this study, the feasibilities associated with the various alternatives which exist for mitigating the flooding problems in this area are assessed. Following this analysis, a specific set of measures is proposed for providing relief of the flooding problems. An evaluation of these proposed measures with respect to implementation, environmental impact, potential for improvement, and cost is then undertaken.

Our firms employed the services of D. Dan (Rabinowitz) Raviv, Ph.D., Consulting Hydrogeologist during numerous phases of groundwater testing and hydrologic analyses which were performed in conjunction with this Report. In particular, his services were utilized in calculating the transmissivity, storativity and other aquifer properties influencing the behavior of the groundwater regime in the vicinity of the Northeast Branch of the Nissequogue River. These properties were then utilized in determining the effectiveness of various solutions which were considered for lowering the water table in this area.

The urgency of the circumstances surrounding these flooding problems is paramount. Even as the enclosed volume was in final draft form, three inches of rain falling during March 21st and 22nd, 1980, posed serious hazard and inconvenience to area residents.

Roadways, such as sections of Townline Rd. and Mt. Pleasant Rd., were near impassible during the storms. Water was measured as backed up over 1.5 feet in the Northeast Branch and its headwater ponds, holding large amounts of water in storage for an extended period of time, and forcing a rapid rise in area groundwater levels. Forty-eight hours after the storm subsided, groundwater levels in the HMM/WSM observation wells located in residential areas along the Northeast Branch were measured at more than one foot higher than those levels which preceded the storm. Several homeowners were forced to begin pumping their basements, as they had done throughout the Winter and Spring of 1979.

It is our hope that the proposed actions described in this volume will be implemented as soon as possible. Only with quick, positive action can the flooding problems which plague this area be alleviated.

Volume 3 will be concerned with identifying potential solutions to the flooding in the vicinity of Lake Ronkonkoma. The completion of Volume 3 is awaiting an environmental review of the recommended construction.

Very truly yours,

HOLZMACHER, McLENDON & MURRELL, P.C./
WILLIAM S. MATSUNAYE, JR., P.E.

Norman E. Murrell, P.E.

William S. Matsunaye, Jr., P.E.

WHS/kc
Encl.

cc: A. Barton Cass, P.E., Chief Engineer

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS

DRAINAGE IMPROVEMENTS INCLUDING GROUNDWATER RELIEF PHASE I - FEASIBILITY STUDY

VOLUME 2 - SOLUTIONS TO FLOODING IN THE VICINITY OF THE NORTHEAST BRANCH OF THE NISSEQUOGUE RIVER

MARCH 1980

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COUNTY OF SUFFOLK

DEPARTMENT OF PUBLIC WORKS

PHASE "1" - FEASIBILITY STUDY

OF

DRAINAGE IMPROVEMENTS, INCLUDING GROUNDWATER RELIEF

IN THE VICINITY OF THE NORTHEAST BRANCH

OF THE NISSEQUOGUE RIVER

MARCH 1980

18. RECOMMENDATIONS - NORTHEAST BRANCH

Our inspections, investigations, tests, and evaluations of the various localities situated along the Northeast Branch of the Nissequogue River indicate an urgent need for stormwater and groundwater drainage improvements in order that serious flooding problems which exist therein will be relieved.

Briefly, our recommendations to alleviate these recurring flooding problems throughout the area are as follows:

(1) Immediate initiation of a second phase for the design of a new drainage system for the entire Northeast Branch area, including a renovation of the existing main channel, its tributaries, and many surrounding secondary systems.

(2) Following design of the new channel, immediate commencement of a third phase for construction and renovation necessary for providing flooding problem relief. (An order for implementation and coordination of the various parts of the design and construction phases appears in Section 28, "PRIORITY RECOMMENDATIONS FOR IMPLEMENTATION").

(3) Establishment of a scheduled maintenance program for all ponds, streams, and drainage structures which comprise the drainage basin of the Northeast Branch.

The flooding problems which plague this area can be systematically eliminated as the proposed measures contained in this study are implemented. These measures have been chosen as the most feasible and cost-effective measures available for providing relief to this area. With a firm commitment by the various municipalities and area residents involved, it can be less than a year before the flooding problems which exist in the vicinity of the Northeast Branch are either very nearly or completely eliminated.

19. SUMMARY OF FINDINGS - NORTHEAST BRANCH

The Northeast Branch of the Nissequogue River has been identified as the major cause of the flooding problems which plague residents in the stream's vicinity. Information gathered in Volume 1 of this study clearly indicated a deteriorated existing channel condition, an ever increasing drainage burden, and a direct relationship between stormwater flow capacities, base flow capacities, and surrounding groundwater levels.

The surrounding groundwater table has reached very high levels during the past two years. Basement, property, and even some roadway flooding conditions in the area have been a direct result of these high groundwater levels. The Branch is the means by which groundwater is naturally removed from most of this area. In its present condition, the Northeast Branch has been determined to be inadequate both as a storm drainage system and as an outlet for groundwater discharge from the area. Poor use of available gradient, channel siltation, inadequately sized culverts, heavy debris, poor alignment, stream bank erosion, lack of maintenance, nearby development, and overall abuse have rendered the stream incapable of handling the storm and base flows necessary for flood prevention.

The need to compensate for the stream's inadequacies is paramount if relief is to be provided for flood-prone areas. Such compensation can be provided by a number of means, including upgrading the stream's existing condition, as well as the installation of supplemental systems, for the purposes of providing adequate stormwater runoff disposal and groundwater discharge for this area.

20. STORMWATER RUNOFF ANALYSIS

FIELD OBSERVATIONS

Field observations during moderate to heavy rainfall have indicated that the Northeast Branch (as it exists) does not function adequately as a stormwater disposal system.

In particular, areas upstream from N.Y.S. Rt. 347 continually recharge the groundwater regime, due to extensive and prolonged ponding behind the twin 36-inch diameter culvert located at Rt. 347.

The most severe case of prolonged ponding was in March 1979, due to exceptional amounts of precipitation. However, as recently as March 22, 1980, ponding occurred between Rt. 347 and Bow Dr. (submerging the steel footbridge at Bow Dr.), due to a relatively low intensity storm.

GENERAL METHODOLOGY

In order to analytically determine just how inadequate the current storm drainage capacity of the Northeast Branch is, a complete analysis of the Northeast Branch's stormwater drainage basin was performed. Various storm conditions were simulated, in order to determine the resultant flooding in the vicinity of the Northeast Branch.

The following section of this study outlines the methodology used to determine the existing inadequacies, and outlines the rationale employed to determine design criteria that we recommend should be used to convert the existing Northeast Branch into a capable stormwater drainage collection and disposal network.

HYDROLOGIC METHODS

The rational method is perhaps the most commonly used technique for estimating peak runoff rates resulting from rainstorms. This method, like all other methods used to determine watershed runoff rates, is not an exact analysis; but rather a series of logical assumptions that are applied with good engineering judgement in order to determine the effects of a given storm.

An alternate method to the rational method is the hydrograph method. Whenever large (greater than 200 mi^2) watersheds are being analyzed, the hydrograph method is the preferred method due to its extraordinary reliability. However, the data required to use the hydrograph method is usually not available when analyzing small watersheds (such as that of the Northeast Branch).

In this study, peak runoff rates were determined by using concepts from both the rational and hydrograph methods. The basic concept used was that of the rational method, but many characteristics of the hydrograph method are also utilized due to their known reliability. The resulting technique, sometimes known as the "modified rational method", produces very good results, especially for small drainage basins such as that of the Northeast Branch (less than two square miles).

RATIONAL METHOD

The first calculation to be performed in an analysis of this kind is the peak inflow to each reach of the stream. In this instance, the word reach can be taken to mean "that section of channel that is being studied". To calculate peak inflows, the rational method was used.

In the rational method, stormwater runoff is related to rainfall intensity by the formula:

$$Qp = CiA$$

where, Qp = peak runoff rate in cubic feet per second
C = runoff coefficient depending on the various ground conditions of the drainage area
i = the average rainfall intensity in inches per hour (in./hr.)
A = drainage area in acres

Once the peak flow has been calculated, the other characteristics of the watershed must be accounted for in order to present an accurate representation of resulting conditions during a given rainstorm. Factors that must be considered are surface storage, basin size, length of channels, and calculations of stream flow velocities in order to determine relative timing characteristics of the floodwave as it travels through the watershed. In order to quantify these various factors, techniques are borrowed from the hydrograph and storage routing methods. Open channel flow analysis is also used to simulate the flooding condition during the design storm.

The resulting method is a compilation of several reliable techniques, and produces results that can be used with a high degree of confidence.

RAINFALL INTENSITY

The determination of rainfall intensity is based upon a number of factors, including time of concentration, design frequency period, and the rainfall intensity-duration characteristics.

Basic data for rainfall intensity-duration-frequency are derived from gauge measurement of rainfall. Results of these measurements are presented in the U.S. Weather Bureau Technical Paper No. 40. The data presented in this paper have been utilized to develop intensity duration curves (see Figure 44) for the Long Island area.

This curve has three parameters:

(1) Frequency - (years)

(2) Rainfall intensity - (inches per hour)

(3) Time or duration - (minutes)

The time on the intensity-duration-frequency curve is the time of concentration. The time of concentration may be defined as the time for rainfall runoff from the most remote point in the drainage area under study to reach a point under consideration.

Time of concentration calculations, for example, might involve overland flow + open channel flow + pipe flow. The time of concentration in this instance, therefore, would be the sum of the three times of flows.

RUNOFF COEFFICIENT

The runoff coefficient, C, is the variable in the rational method that is the most difficult to quantify precisely. The fact that a single value is used as a runoff coefficient in the formula implies that there is a fixed ratio of rainfall to runoff for a given drainage area. In actuality, the runoff coefficient for a given area varies with seasonal and climatological conditions, and may also also change throughout the duration of a single rainfall.

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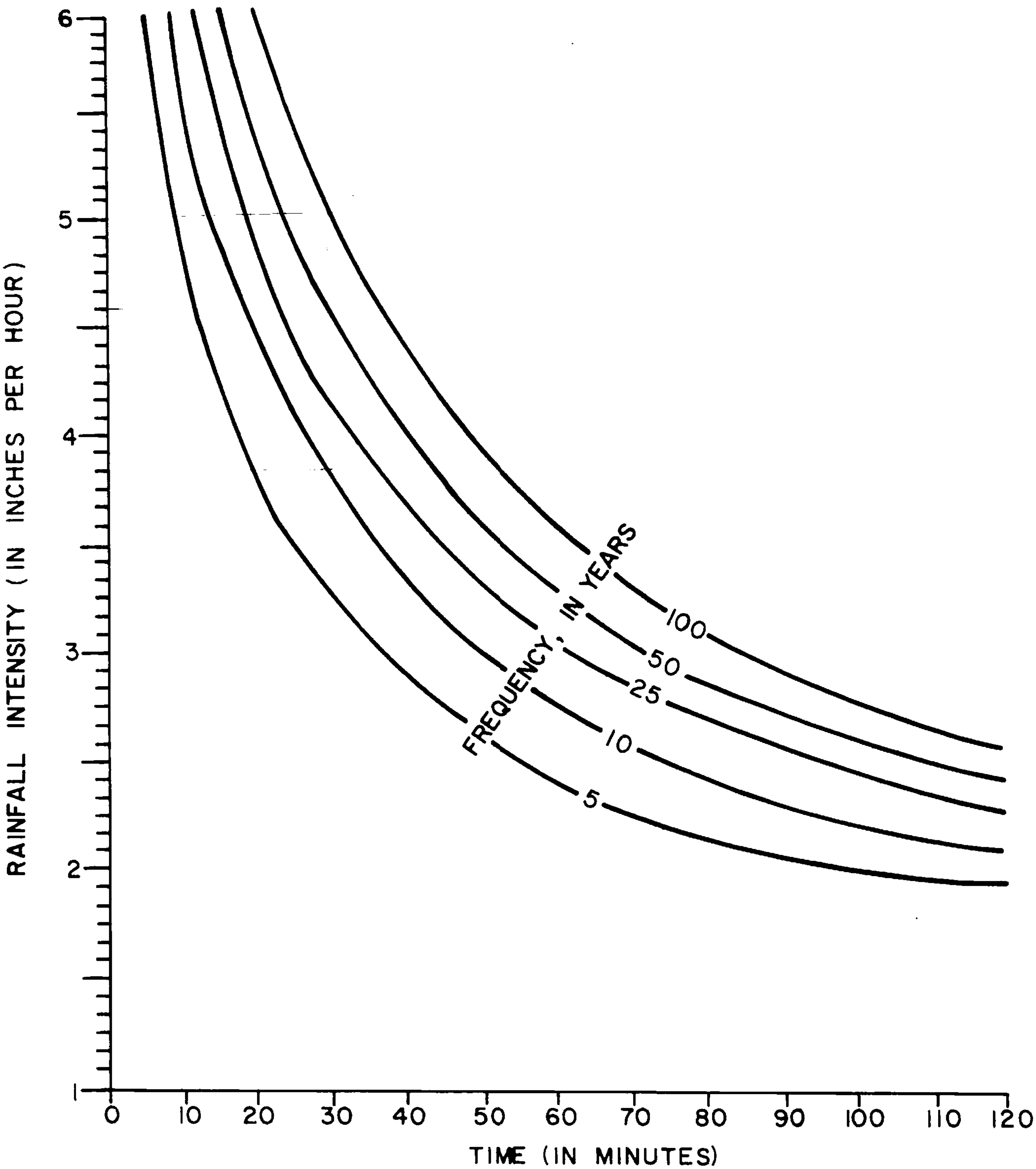

INTENSITY - DURATION CURVES

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The use of average coefficients for various surface types, which are assumed not to vary throughout the duration of a storm, is common practice. Generally, satisfactory results are obtained with these overall coefficients, but it is important to understand the individual components that effect the runoff coefficient, so that good judgement can be applied when adjusting runoff coefficients for other than average conditions.

Some of the components that affect the overall runoff coefficient are:

(1) <u>Interception by vegetation</u> - Interception is not usually significant in urban areas but can be up to 0.5 inch in forest areas, depending on the type of ground cover.

(2) <u>Infiltration into permeable soils</u> - The ability of a soil to absorb rain and percolate it deeper into the subsurface varies noticeably with antecedent conditions to a given storm. For example, if a specific storm has been preceded by other storms within the last week, the subsurface very possibly could be saturated to the point that all rainfall is forced to run overland rather than percolate and recharge the groundwater regime.

A similar situation occurs when it rains during cold periods and the upper layers of the ground are frozen. In this case runoff approaches 100 percent due to the fact that rain cannot penetrate frozen ground.

Lastly, in the extreme case when rainfall occurs on frozen ground covered by several inches of snow, it is possible for the runoff coefficient to exceed 100 percent as the snow melt accompanies the rainfall runoff.

(3) Retention in surface depressions - Initially, rainfall fills the depressions present in all surfaces. However, again due to antecedent conditions, surface depressions may or may not be filled prior to the start of a storm. Correspondingly, the same drainage area might conceivably have more storage capability on one given day than on another.

Table 13 is a breakdown of average composite runoff coefficients commonly accepted for various types of land use. These coefficients assume unfrozen ground conditions.

When detailed information is available concerning the character of the surface (pavement, soil, etc.), it is often desirable to calculate a composite runoff coefficient based upon the amounts of the various surface types. Table 14 lists the individual runoff coefficients commonly applied to various ground surfaces.

DESIGN FREQUENCY DETERMINATION

No matter what capacity a stormwater system is designed for, there still remains the possibility that a more extreme storm will occur, thereby pushing the system beyond its planned limit.

For this reason, all floodworks projects are planned and designed on the basis of a frequency period. For any given frequency period, it is assumed that there is a storm of a certain magnitude that will be equaled or exceeded at least once during that frequency period. In other words, a 50 year storm

TABLE 13

AVERAGE COMPOSITE RUNOFF COEFFICIENTS

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, Cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad Yard	0.20 to 0.35
Unimproved	0.05 to 0.30

TABLE 14

INDIVIDUAL RUNOFF COEFFICIENTS

FOR VARIOUS TYPES OF LAND SURFACES

CHARACTER OF SURFACE	RUNOFF COEFFICIENTS
Pavement	
Asphalt	0.70 to 0.95
Concrete	0.80 to 0.95
Roofs	0.75 to 0.95
Lawns, Sandy Soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, Heavy Soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

is one that is expected to be equaled or exceeded once every 50 years (on the average).

The main concern when designing a stormwater drainage network is to determine the probability of occurrence for a predetermined flood magnitude being exceeded. To do this, standard statistical techniques can be utilized to derive a "Flood Risk Factor".

From statistics, the probability of exceeding a storm with a design frequency of T, within a period of n years is:

$$\text{probability} = 1 - (1-1/T)^n$$

where the probability could also be thought of as a "Flood Risk Factor" (FRF). For example, the FRF for a 10 year storm within a two year period is:

$$\text{FRF} = 1 - (1-1/10)^2 \times 100\% = 19\%$$

This means that there is a 19 percent chance that a storm with a 10 year design frequency will be equaled or exceeded within any two year period.

Using the above equation a "Flood Risk Factor" can be calculated for any frequency period T, and period of years, n. This information can then be plotted graphically to show visibly what effects the various parameters have on the FRF. Figure 45 represents such a plot.

Different projects are designed for different frequency storms depending upon a number of factors. Using good judgement, a designer can decide upon an acceptable "Flood Risk Factor" and then use the matching frequency (sometimes referred to as "return") period.

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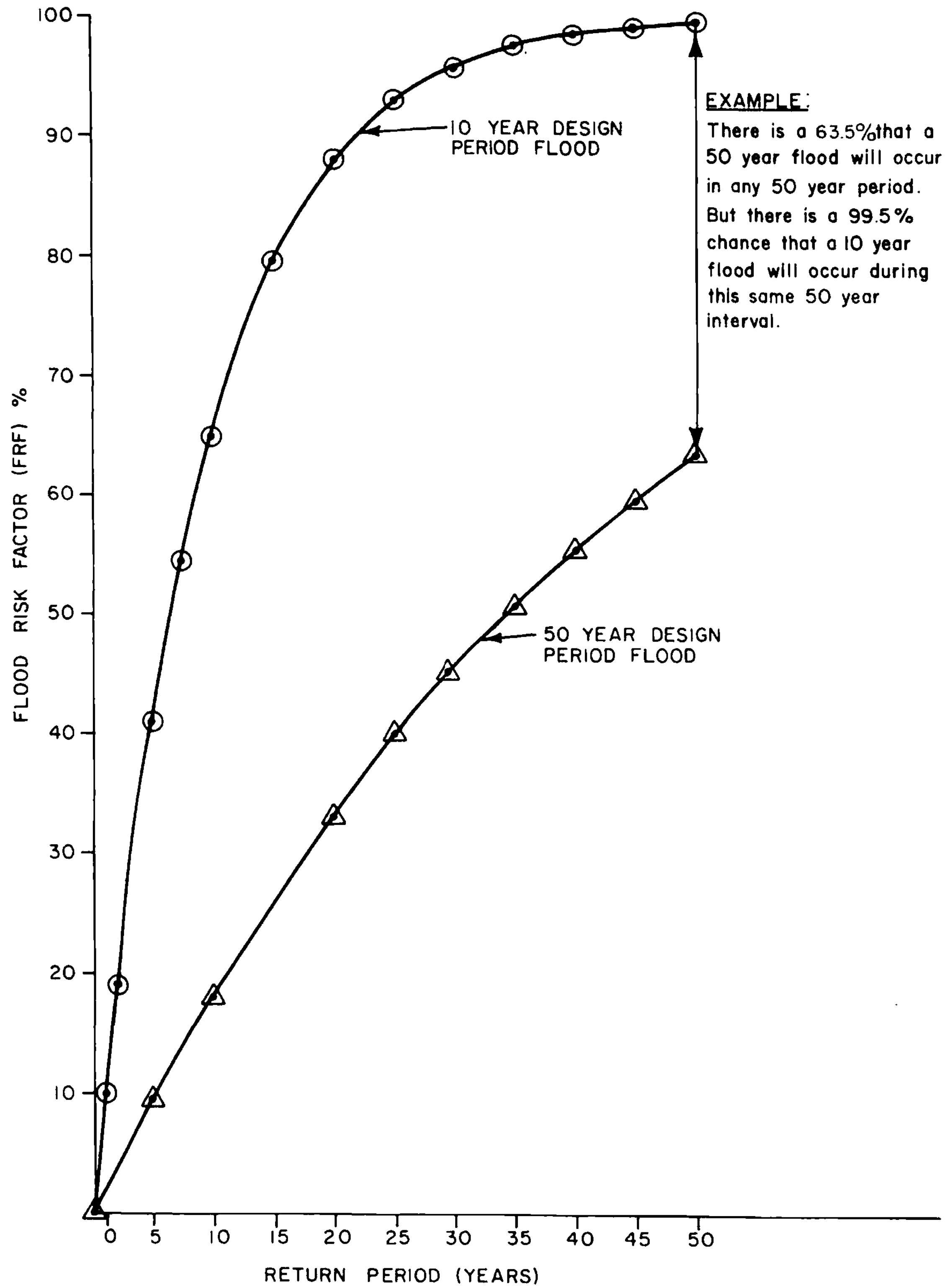

FLOOD RISK FACTORS

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When determining a safe FRF, various things must be considered, such as:

(a) What is the potential danger to life and property if the proposed system is overloaded beyond its design capability?

(b) What is the potential for minor or major inconvenience if the proposed system is overloaded beyond its design capability?

(c) Since costs are directly proportional to design frequency, are there economic restraints that will perhaps limit the flood risk factor to a predetermined value?

(d) A cost/benefit ratio for various options must be kept in mind to insure maximum benefit per dollar spent on storm drainage improvements.

DESIGN CRITERIA FOR THE NORTHEAST BRANCH

The Northeast Branch drainage network consists of several components. The first of these is the individual drainage systems for the various subdivisions adjacent to The Branch. These tributary systems collect the stormwater runoff from the rooftops, lawns, driveways and streets, and pipe it either directly to the stream or to an open channel that leads to the stream. Additionally there is the stream itself, which must have adequate capacity to carry stormwater that is being delivered to it by the various tributary systems. Finally, whenever the stream passes under a roadway, it must do so by passing through a culvert, which must have adequate capacity to pass the flows delivered to it by the stream.

Culverts

The New York State Dept. of Transportation (NYSDOT) indicates that culverts should have adequate capacities so that they are capable of passing the peak flow from a storm with a 50 year design frequency, without backing up enough to cause any upstream flooding.

Sometimes a culvert is purposely sized to accept a flow that is smaller than the design flow. At the instant that the flow reaching the culvert exceeds the culvert capacity, water begins ponding behind the culvert until it has developed sufficient height (or head) to force its way through the culvert. The elevation of the ponded surface of water on the upstream side of the culvert is known as the headwater elevation and effectively increases the capacity of the culvert. Headwater is usually controlled so that there is a very small probability that it will rise to within one foot of the roadway it is passing under. In order to account for the additional capacity due to headwater depth the State of California has set forth the criteria that a culvert must have the capacity to pass the peak discharge of a 10 year storm without surcharging (no headwater allowed above the crown of the culvert). The culvert must also be capable of passing a 100 year discharge with a limited or safe headwater depth. We intend to use a combination of the California design practice and the criteria set forth by NYSDOT. All culverted sections of The Branch will be designed to carry a 10 year flood discharge without surcharge (as per California), and a 50 year flood with limited surcharge (as per New York).

<u>50 Year Storm Patterns</u>

One clarification is appropriate at this point. The term "50 year storm" does not imply that there is exactly one intensity-duration combination that results in a 50 year recurrence interval. For example, two storms might have a 50 year recurrence interval; yet storm A could have a duration of two hours with an average intensity of 1.70 inches per hour, while storm B lasts for 30 minutes with an average intensity of 4.30 inches per hour. The end result of storm A would be 3.40 inches of rain over a two hour period. The result of storm B would be 2.15 inches of rain over a 30 minute period. Both of these storms would have a 50 year recurrence interval (in this study area), yet the resulting runoff patterns from both storms would be vastly different, resulting in different peak flows and different timing characteristics.

<u>Storm Intensity-Durations</u>

In general, a storm of short duration and high intensity will cause a greater peak flow at an earlier time, but will return to base flow conditions very rapidly. It is customary practice to simulate the desired design frequency storm with varying intensity-duration combinations, in order to determine which combination yields the maximum storm flow.

There is one significant difference between the normal design condition and the existing condition throughout the stormwater drainage basin of the Northeast Branch.

The existing storm drainage systems tributary to the Northeast Branch from the various surrounding subdivisions are all designed to handle 5 to 10 year storms of long duration. It is standard practice in subdivision drainage design to assume that the temporary back-up of a storm system due to a high intensity, short duration storm is a reasonable trade-off as opposed to the relatively high costs that would be incurred in constructing a system that can instantaneously accept the flows from such storms. This is a significant factor with regard to sizing the main channel of the Northeast Branch.

The preceding discussion indicates that it would be more logical to design the stream channel to have adequate capacity to carry a longer duration storm, rather than a short duration storm. This conclusion is based upon the fact that a high intensity short duration storm would undoubtedly cuase back-ups of the existing subdivision storm drainage systems such that excess storm flows would have to reach the Northeast Branch in an attenuated flow pattern. In other words, rather than having a situation occur where rain falls, flows overland to a collection system and immediately enters the system to be carried to its final destination; the various sytems tributary to the Northeast Branch are designed so that when an individual catch basin backs up, the excess stormwater either finds an alternate route to its final destination (via slower overland flow through yards, etc.), or it simply ponds in the street until the storm flows have subsided to a point that excess water can enter the system. In effect, the roadway gutters act as retention facilities.

In either case, travel time to the stream is increased, and peak flows to the stream are decreased. In effect, a short duration-high intensity storm is transformed into a longer duration-less intense storm due to the limited capabilities of the drainage network tributary to the Northeast Branch.

120 Minute Duration Storm

With this in mind, we have decided that it would be most practical to design the Northeast Branch stream channel and the various culverts (N.Y.S. Rt. 347, Branch Dr. and Terrace Lane) to handle a 120 minute duration storm.

As previously stated, the culverts will be sized so that they can pass a 10 year storm without surcharge and a 50 year storm with a predetermined safe headwater depth.

An adaptation of the same criteria will be used to size the stream channel. The reconstructed channel will be capable of passing a 10 year storm with a depth of flow that provides at least a one foot freeboard or safety factor to the top of the channel side slope; and a 50 year storm will theoretically cause the channel to flow full. Referring to the formula on page 20.10 and Figure 45, it can be seen that for any one year period there is a 98 percent probability that the proposed system will be able to accommodate the expected rainfall. In our opinion, this is an excellent factor of safety, since it is economically not feasible to design a drainage system with a 100 percent flood free risk factor. It should be pointed out here that although there is a 2 percent chance that flow in the reconstructed channel of the Northeast Branch would over top its banks in any given year, this excess flow could be directed so that overflow takes place in a predetermined location that would cause only minimal street flooding in residential areas. For example, the stream channel can be designed to allow flows in excess of the 50 year storm to over top the channel banks in selected wetland areas. The point chosen for such an overflow would act as a temporary storage area until such time as the stormwater can re-enter the channel.

By allowing flows in excess of the 50 year storm to be temporarily stored via controlled flooding, the "Flood Risk Factor" actually approaches zero percent for storms with frequencies greater than 50 years.

STORMWATER FLOWS

Using techniques mentioned in the Hydrologic Methods section and Design Frequency section, we have analytically simulated the effects of both a 10 and 50 year design frequency, 120 minute duration rainfall. These analyses have provided us with complete simulations of theoretical flows at any point in the drainage basin, throughout the duration of given design storms. If flow vs. time at a given point is plotted graphically, the resulting figure is known as an inflow hydrograph. Figures 46 through 49 are inflow hydrographs to the various culverts for both the 10 and 50 year design storms. Peak flows are extrapolated from the hydrographs and are used as design values. The peak flows to the individual culverts have also been listed in tabular form (see Table 15).

Naturally, the peak flow to an individual culvert not only determines the size of that culvert, but also serves as the design criteria for the reach of The Branch directly upstream.

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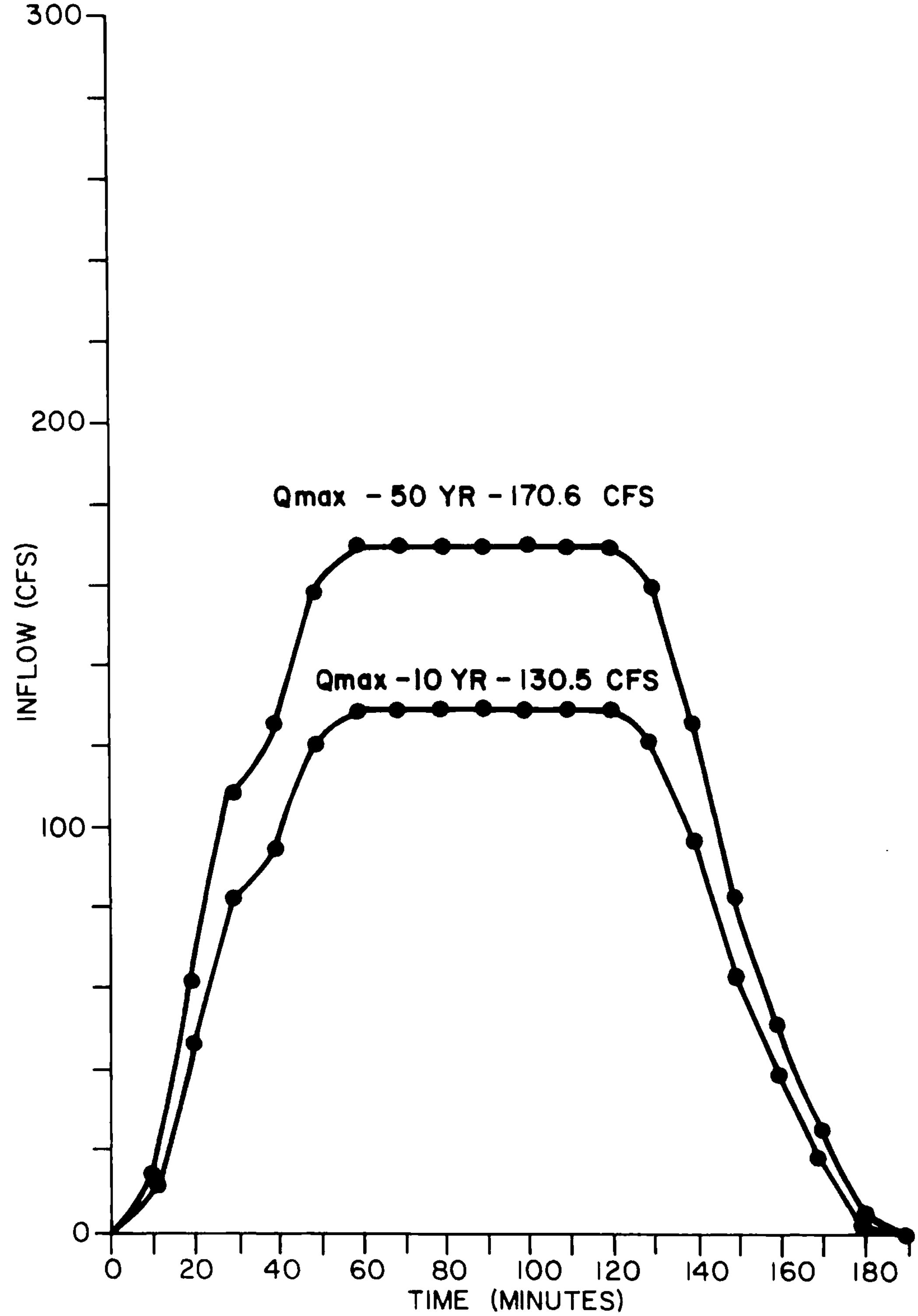
INFLOW HYDROGRAPHS TO ROUTE 347 CULVERT
FOR THE NORTHEAST BRANCH



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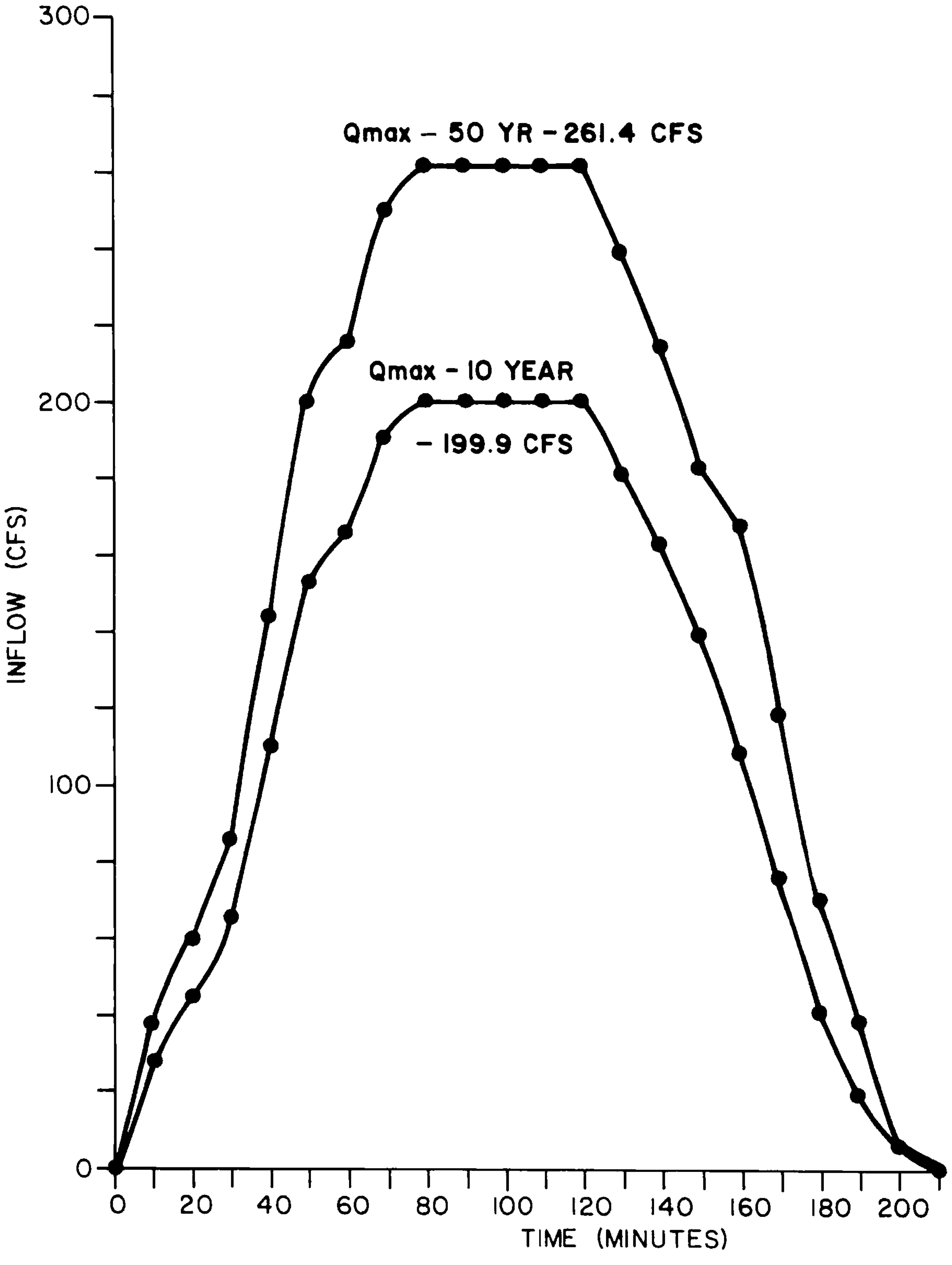
INFLOW HYDROGRAPHS TO BRANCH DRIVE CULVERT
FOR THE NORTHEAST BRANCH



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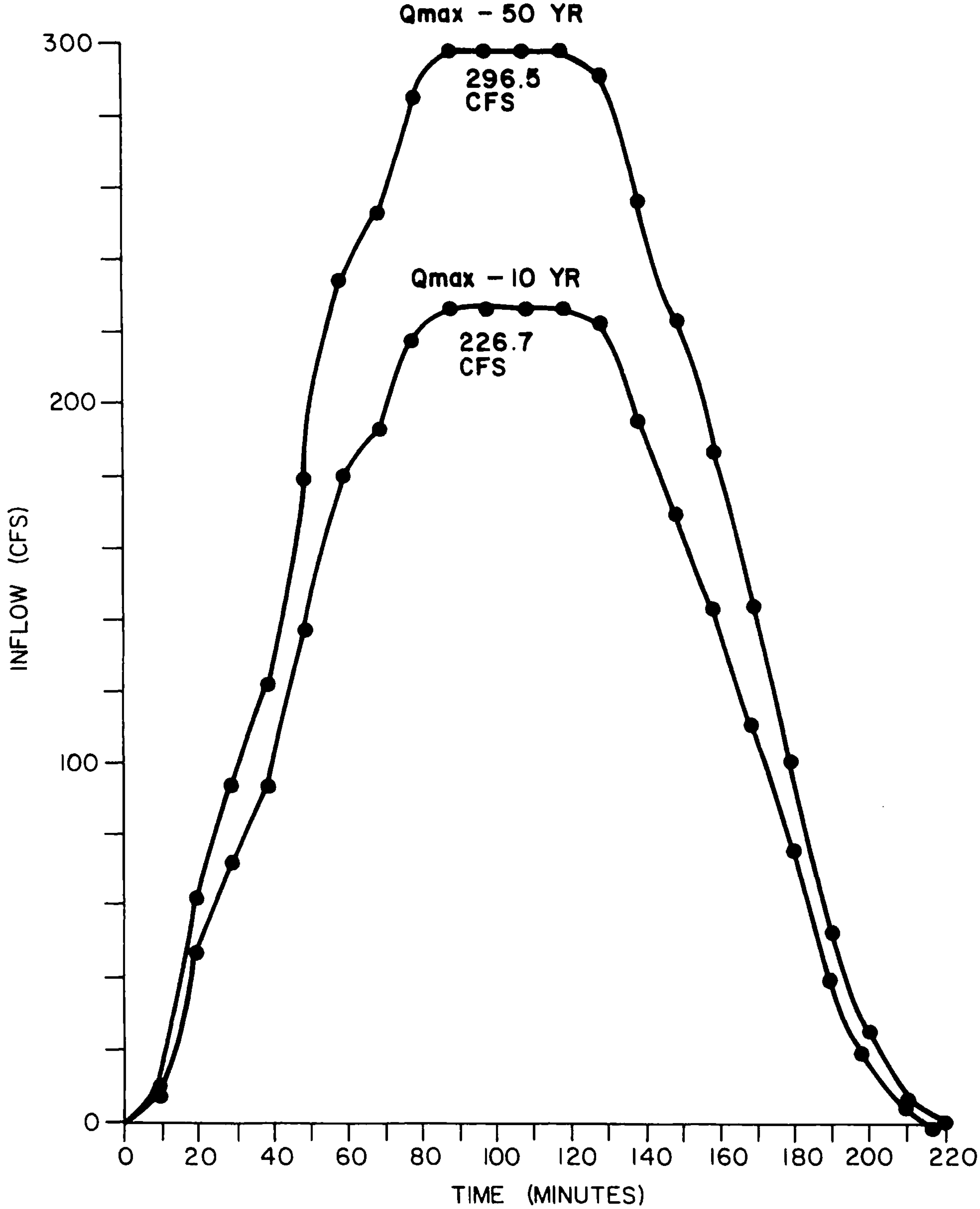
SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

INFLOW HYDROGRAPHS TO TERRACE LANE CULVERT
FOR THE NORTHEAST BRANCH



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INFLOW HYDROGRAPHS TO ROUTE 111 CULVERT
FOR THE NORTHEAST BRANCH

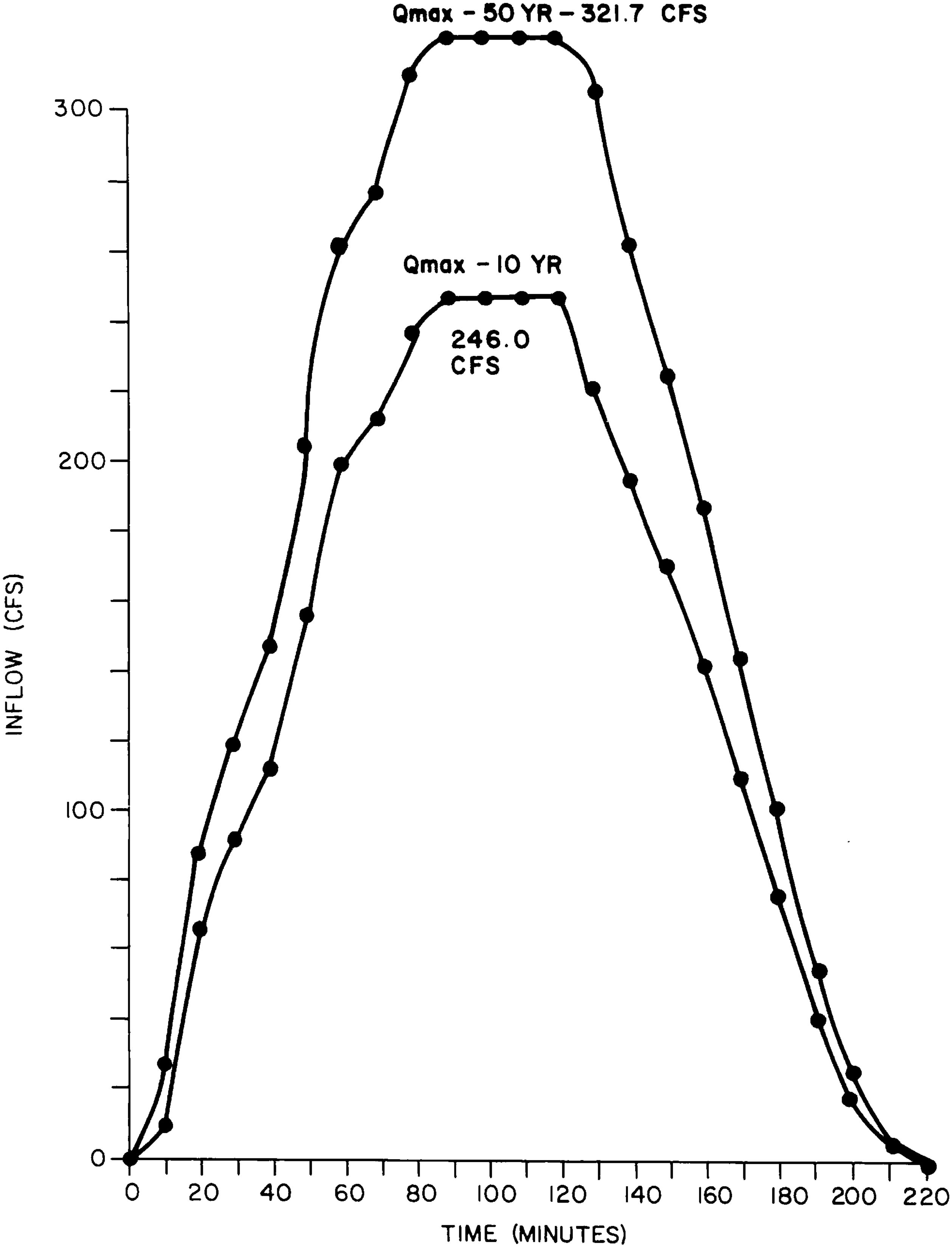

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TABLE 15

PEAK FLOWS AT CULVERTS
ON THE NORTHEAST BRANCH

	FLOW (CUBIC FEET PER SECOND)	
LOCATION	10 YEAR* (CFS)	50 YEAR** (CFS)
ROUTE 347	130.5	170.6
BRANCH DR.	199.9	261.4
TERRACE LA.	226.7	296.5
ROUTE 111	246.0	321.7

*BASED ON A 10 YEAR STORM, 120 MIN. DURATION
**BASED ON A 50 YEAR STORM, 120 MIN. DURATION

Comparison of the required design capacity to the existing capacity of the Northeast Branch, conclusively shows that major renovation is necessary to relieve the threat of major flooding.

The existing channel capacity ranges from below 10 cfs to approximately 30 cfs, while design criteria indicated herein dictate channel capacities ranging between 170.6 and 321.7 cfs (see Table 15).

Culvert capacities must also be increased to preclude the possibility of prolonged back-up and upstream storage that is presently occurring. Table 16 lists the location of existing culverts, their present unsubmerged capacities, and the required unsubmerged capacities for adequate passage of flows associated with a 10 year design storm.

TABLE 16

COMPARISON OF REQUIRED

UNSUBMERGED CULVERT CAPACITIES TO

EXISTING CAPACITIES

ON THE NORTHEAST BRANCH

	*10 YEAR DESIGN FLOW (CFS)	EXISTING UNSUBMERGED CAPACITY
Route 347	130.5	80 cfs
Branch Drive	199.9	120 cfs
Terrace Lane	226.7	120 cfs
Route 111	246.0	645.4 cfs

* required unsubmerged capacity

21. GROUNDWATER HYDROLOGIC ANALYSIS

WATER TABLE CONFIGURATION

Water table contour maps for an area which includes Lake Ronkonkoma, the Northeast Branch, Miller's Pond, New Mill Pond, and Townline Rd., were compiled and published periodically by the U.S.G.S. and Suffolk County agencies. A compilation of these maps is presented in Figures 13 to 23, Vol. 1. The periods included are 1904, 1951, March and May 1959, 1966, April 1968, Spring 1971, March 1974, March 1977, March 1978 and March 1979. In addition, two maps were compiled as part of this study (Figures 24 and 27, Vol. 1) for the periods of November 20, 1979 and December 30, 1979. The November 20th contour map is based on water level measurements made during this study utilizing existing wells. The December 30th contour map includes water level measurements in the new HMM/WSM observation wells installed for the purpose of this study. Most of the new wells are along the Northeast Branch between Townline Rd. and Miller's Pond with two wells located north and northwest of the Pond. The purpose of the new wells was to better define the local groundwater regime within the problem area.

In viewing and comparing the set of water table contour maps, the following exceptions should be emphasized:

(1) Water table elevations are based on measurements utilizing wells penetrating different levels of the aquifer.

(2) The water table contours are of different contour intervals depending on data availability (i.e. 2 feet, 5 feet and 10 feet).

(3) Water level measurements were conducted in different seasons of the year.

(4) The additional refinement of water level contours presented for December 30, 1979 (Figure 27, Vol. 1) is based on new shallow wells and surveyed elevations of open water-bodies (recharge basins, impoundments and swamp areas).

The purpose of the water table contour maps evaluation is two-fold: (a) to document the history of water table fluctuations in a region which includes the problem area; and (b) to relate the local groundwater regime, along the Northeast Branch, to the regional water table configuration. In addition, the water table contour maps provide a direct means by which generalized groundwater flow direction and locations of high recharge areas can be delineated and hydraulic gradients calculated.

Comparison of water table contours (excluding the 1904 period) indicates the 1966 measurements as the lowest recorded water table elevations. Water table elevations in 1966 ranged from between 4 and 10 feet lower than those measured in 1977, 1978 and 1979. Regardless of the water table elevations, however, the general regional water table configuration remained the same throughout the period of record (1904-1979). A high water table (mound or recharge area) exists near Lake Grove with general groundwater flow in all directions. Consistently, a very flat water table exists in the area encompassing Terry Rd., the Northeast Branch above Miller's Pond, and Mt. Pleasant Rd. This area is located southwest of the groundwater mound. The area of flat water table resembles a configuration of a terrace.

On the average, hydraulic gradient changes from about 0.15 to 0.20 percent in the area of influence of the mound to about 0.05 percent in the flat area. The additional data obtained during the drilling program of this study and the water level monitoring of December 30, 1979 indicates local hydraulic gradients created by groundwater discharge into the Northeast Branch (Figure 27, Vol. 1).

The published water table contour maps also indicated the position of the regional groundwater divide for the years 1904, 1959, 1968 and 1971. Allowing for subjective interpretation by the various investigators, the groundwater divide for these periods remained practically unchanged. The additional water level data incorporated in the December 30, 1979 water table contour map (Figure 27, Vol. 1) introduced a local refinement to the groundwater divide. The departure from the previously reported groundwater divide is due primarily to the interaction between the existing ponds B-5, B-6 and B-7; recharge basins R-1, R-2 and R-3; and the local groundwater flow pattern as measured in the new observation wells W-1, W-2, W-3 and W-10 (Figure 27, Vol. 1). Neglecting these additional observation points the interpreted position of the groundwater divide would have remained unchanged.

INTERPRETATION OF PUMP AND RECOVERY TESTS

An aquifer test was conducted utilizing wells No. 8 and 9 (Figure 36, Vol. 1). The data was analyzed by several methods in order to determine the local aquifer parameters of transmissivity, T; hydraulic conductivity, K; and storativity, S. The purpose of this test was to determine expected well yields and the magnitudes of these hydrologic parameters. These values were used to estimate expected ranges in well yield, to estimate required well and underdrain spacing, to compare between measured and calculated base flow, and to calculate the affect of lowering the stream bed on the adjacent water table.

Analyses of the original test data are presented in Figures 37 through 40 (Vol. 1) and are summarized in Table 11 (Vol. 1). For the two wells, the range of calculated values of T is given, along with the average T value of about 6,000 gpd/ft. An average value of S equal to 0.0004 is given for the observation well. The test data was analyzed both by the straight line method, for pumping and recovery, and the Theis nonsteady method (Vol. 1) for the purpose of calculating S. The reason for applying the Theis method was to double check the relatively small S value of 0.0004 calculated by the Jacob method (Figure 37, Vol. 1). This value is generally more characteristic of a confined aquifer. Well logs, surface geology and the rapid response of water levels to rainfall near the Northeast Branch suggest that the local aquifer should be classified as a water table aquifer. S values for such an aquifer are usually between 0.01 and 0.2.

From well logs obtained during this study (Figures 31, 32 and 33, Vol. 1) the saturated thickness of the local aquifer was determined to be larger than 20 feet along the Northeast Branch downstream to Miller's Pond. North of Miller's Pond clay was encountered at shallow depths. On the regional scale clay layers exist at about 40 to 60 feet below the surface. For the purpose of calculations, the saturated thickness was assumed to be at about 40 feet below the ground surface.

If a correction factor is applied to the calculated T value (Figures 37 and 38, Vol. 1) to account for water table conditions, the resulting S will be about 0.002 as compared to 0.0004. The location of test wells No. 8 and 9 is probably in a fill material which locally creates confined conditions. This uncertainty in the results prompted parametric analysis of the various calculations to be presented below.

The pumping rate during the test decreased from about 34 gpm, after 10 minutes of pumping, to about 23 gpm at the end of the pumping period (data is tabulated in Appendix C, Vol. 1). The maximum drawdown in the observation well located 20 feet from the pumped well (Figure 36, Vol. 1) was about 3.10 feet between 42 and 70 minutes at a pumping rate of about 30 gpm. During the period between 70 minutes and the end of the test (360 minutes) drawdown in the observation well recovered to about 2.70 feet (or about 13 percent of the maximum drawdown). Based on the one test, a single explanation to the observed recovery in water levels is not possible. The following are possible causes: pump efficiency, recharge boundary (i.e. the Northeast Branch), vertical leakage, or delayed effect of partial penetration of the well. The penetration factor at the pumped well is about 0.17 to 0.25 (ratio of screen length to saturated thickness of aquifer).

PARAMETRIC (SENSITIVITY) ANALYSIS

As part of this study analytical calculations were made with respect to the effect that different dewatering or relief schemes may have on lowering groundwater elevations. Basic groundwater flow equations were solved for both steady and nonsteady state conditions.

The equations used for calculating well spacing, underdrain spacing and depth, stream bed lowering and the corresponding changes in water table configuration were subjected to parametric analyses.

The parametric analyses were used as judgement guides in satisfying limited geohydrologic data, natural variability in subsurface conditions, and the assumptions used in the solution of the groundwater flow equations.

Parametric analyses were conducted by subjecting the mathematical relationships to variations in aquifer coefficients to qualitatively assess degrees of sensitivity. Included in this analysis were variations in transmissivity, T, specific yield, S, (or storage coefficient), and their ratio, T/S.

The pumping and recovery tests, conducted at well sites W-8 and W-9 (Figure 36, Vol. 1), resulted in an average transmissivity of 6,000 gpd/ft and storage coefficient of 0.0004. The pumped and observation wells were completed to about 15 feet total depth with screen length of 10 feet. The strata tested is typical of the area along the Northeast Branch, which is fill over Upper Glacial material.

The time drawdown curves (Figures 37 and 38, Vol. 1) indicate leaky conditions or a nearby recharge boundary. As mentioned, it should also be recognized that an S value of about 0.0004 is typical of a confined aquifer and not the Upper Glacial aquifer. This low value is probably due to local conditions associated with the characteristics of the fill material.

Parametric analyses were carried out to determine the range in calculated drawdown as a function of variations in the T/S ratio, the calculated water table elevation as a function of the constant head boundary condition, and the calculated base flow resulting from lowering of the Northeast Branch as a function of variations in T/S ratio.

As an example of these analyses, Figure 50 was prepared to illustrate the possible range in the calculated drawdown due to local changes of T and S. The shaded area, between the two transmissivity lines, indicates the range in the calculated lowering of groundwater levels at a point 200 feet away from a well pumping at 20 gpm for a period of 30 days. The storage coefficient was varied between values of 0.0004 and 0.1.

22. STORMWATER MANAGEMENT ALTERNATIVES

The existing stormwater drainage network operative in the vicinity of the Northeast Branch consists of several individual positive drainage systems which utilize the stream and its surrounding wetlands for outfall and disposal. A plan of the existing storm drainage basin of the Northeast Branch was presented as Plate VI in Volume 1 of this study. The present system has two major shortcomings. It is presently not capable of handling heavy storm flows (10 year return period) without backing up. Additionally, tributary systems to the main stream are inefficient, and allow significant recharge of the water table during precipitation events.

Stormwater disposal in the study area must be improved. There exist several alternatives for obtaining such improvement. Some methods are better suited to this area than others. However, each alternative must be evaluated in view of the existing groundwater flooding problems as well as the storm drainage problems in this area.

GROUNDWATER DISCHARGE OF STORMWATER

Heavy stream flows which accompany storms are the result of surface runoff which is rapidly concentrated at the stream via positive drainage systems. The directing of this stormwater to a recharge basin (from which the water could infiltrate back into the groundwater regime) rather than the stream would lower peak storm flows. However, it must be noted that in areas surrounding

The Northeast Branch, groundwater levels are high. Utilization of recharge basins for the disposal of stormwater would be ill-advised, since the operating efficiency of the basin would be hampered by the high water table, and area groundwater flooding problems would be aggrevated by recharge. Our study indicates that two recharge basins in the study area are not functional. One, at Bow Dr., has overflowed, has required pumping, and has aggrevated basement flooding in nearby homes. Another, located off the east end of Brilner Dr., was suspected of causing basement flooding, and has since been modified by excavation of its eastern wall so water could pass through the basin, directly to the Northeast Branch. These two basins are located as shown on Figure 51.

RETENTION OF STORMWATER

Reduction of peak flows can be accomplished by retention of runoff. Presently, Miller's Pond and New Mill Pond are acting as retention facilities. Water is held by these bodies of water for esthetic and recreational reasons. The fact that they serve to diminish peak flows is probably coincidental.

In addition to these two ponds, retention is presently taking place in the wetland ponds which comprise the headwaters of The Branch. This is having an intensifying effect on the water table flooding problems in this area, as these bodies of water are causing recharge of the groundwater table, instead of allowing groundwater to drain away from the area.

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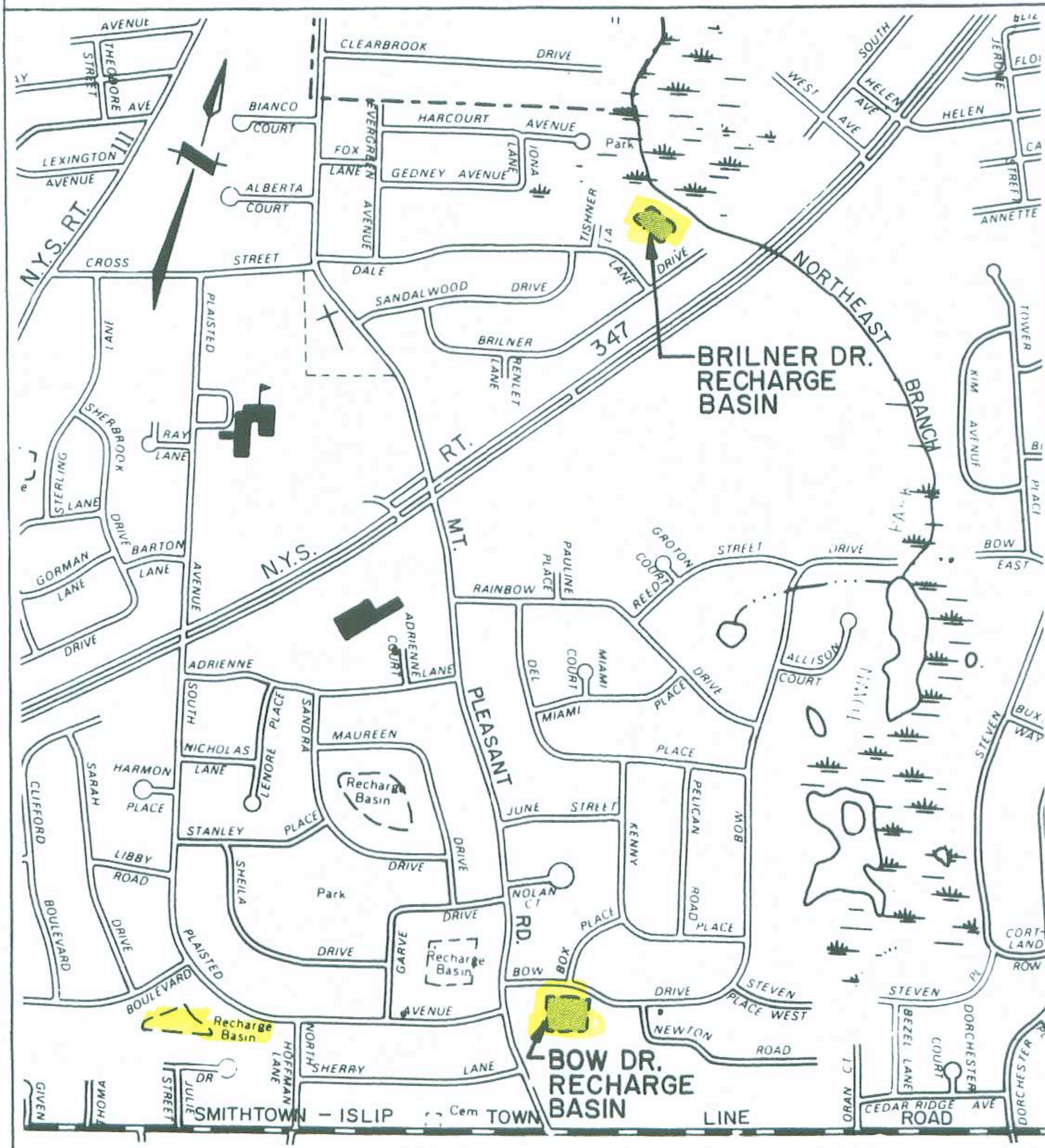

LOCATION OF MALFUNCTIONING RECHARGE BASINS

AT BOW DRIVE AND BRILNER DRIVE

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Construction of new retention facilities along the stream to relieve peak flows would pose groundwater problems, as do the headwater ponds. While recharge may be prevented by lining such a facility, it should be noted that the costs involved in obtaining land for and constructing a lined retention basin (or basins) which would have appreciable effects in diminishing peak flows would be prohibitive.

DETENTION OF STORMWATER

As was previously explained, peak stormwater flows are the result of the rapid concentration of storm runoff in the stream via positive drainage systems. Detention refers to a process by which runoff is held until after peak flow periods have past. This process is presently occurring in The Branch storm drainage basin. For example, the culvert at N.Y.S. Rt. 347 backs-up during severe storms. This back-up prevents inundation of downstream areas. As flow subsides, the water which has backed up behind the culvert begins to drain at a rate below the capacity of the downstream channel. Unfortunately, while the channel (upstream) is backed up, runoff remains ponded throughout the upstream channel, ponds, and wetlands, causing recharge of the water table, and greatly intensifying the water table rise associated with the storm.

Detention is also occurring in many of the main stream's tributaries, which carry water from storm system outfall pipes (located in adjacent wetlands) to the main channel.

The present condition of many of these channels is such that heavy flows are dispersed throughout the wetlands prior to arrival at the main channel. In some cases, no channels exist, and in others, obstructions and vegetation pose serious obstacles. Recharge, rather than runoff, is occurring. Peak flows are diminished at the risk of causing basement and roadway flooding. These existing conditions indicate that utilization of detention facilities in the vicinity of the Northeast Branch would be impractical in view of the existing groundwater problems.

REHABILITATION OF PONDS

It has been determined by bathymetric survey (see Plate V, Vol. 1) and bottom sampling that Miller's Pond has a thick, organic mud covering most of its bottom. This material is generally quite impervious, and prevents water from seeping out of the Pond. If the bottom of the Miller's Pond were dredged clean of this mud layer, significantly increased recharge of the water table could take place, and conceivably, stormwater could be held there as if it were a recharge basin. Recharge from the Pond is undesirable, as increases in surrounding water table levels may result. Furthermore, dredging Miller's Pond would seriously disturb Pond biota with the destruction of the bottom habitat.

In the case of the headwater ponds, rehabilitation of the ponds would allow for more rapid movement of groundwater to the surface water regime. Presently, these ponds are plugged with relatively impervious organic material, preventing such movement. It should be noted that dredging and cleaning of these ponds to allow for better drainage of groundwater would only be effective if this water could be removed from the ponds by adequate surface flow. This would require significant channel improvements in conjunction with pond rehabilitation.

REMOVAL OF MILLER'S POND

Miller's Pond has actually been created by damming a section of channel along the Northeast Branch (more than 100 years ago). It represents the terminus of the first section of the Northeast Branch channel. Removal of the Pond would effectively increase available gradient through this first section of channel, and increased storm flow capacities would be realized.

Removal of the Pond, however, would pose serious problems. It is the center of a preserved park area. Esthetically, it is invaluable to area residents. Additionally, it serves as a retention facility, protecting the downstream section of channel from inundation.

CHANGING WATER LEVEL OF MILLER'S POND

The present water level at Miller's Pond (36.86 feet msl, Dec. 30, 1979) is acceptable. Raising the level of the Pond would seriously disrupt flow from the N.Y.S. Rt. 111 culvert to the Pond, causing water to stand or flow more slowly from the culvert due to lack of gradient. The culvert invert on its downstream side is 37.25 feet msl.

If, on the other hand, the Pond level is lowered, increased available gradient would be realized. The gradient could only be utilized to aid storm flow in upstream reaches if the Rt. 111 culvert is replaced at a new elevation. The existing culvert is comprised of two 7 foot by 4 foot reinforced concrete box sections which extend for 160 feet beneath N.Y.S. Rt. 111.

As was indicated in the bathymetric survey presented in Volume 1, lowering Miller's Pond by as little as one foot would severely diminish the Pond's surface area, and further, would uncover an organic mud bottom which would become its new shoreline.

Creating additional available gradient by lowering the water level of Miller's Pond would be a three-fold problem. First, the water level would be lowered. Additionally, the N.Y.S. Rt. 111 culvert (and other culverts) would have to be replaced at a lower elevation. Finally, the Pond (particularly newly revealed shore areas), would have to be dreged clean of dangerous soft muck.

CHANNEL MODIFICATIONS FOR STORMWATER DISPOSAL

The poor condition of the Northeast Branch has already been noted. A significant increase in capacity will be realized if the channel is simply cleared of debris. Presently, the debris which occupies the channel has blocked as much as two-thirds of the flow area through certain reaches.

Realignment of the channel would prove quite rehabilitative. Increased capacities could be obtained if sharp bends in the stream were removed. Furthermore, erosion along the outer banks of existing bends would be controlled, and undermining of vegetation (including large trees which are falling into the stream) would cease.

Regrading the channel would also provide increased upstream flow capacities. Under existing conditions, the gradient upstream of Branch Dr. is very flat, while gradients downstream of Branch Dr. are significantly steeper. Storm flow analysis of the Northeast Branch has shown that high peak flows occur in the upstream reaches due to extensive positive drainage. These upstream flows could be more adequately handled if the gradient were increased. Creation of a uniform gradient along the channel from the headwaters to Miller's Pond would significantly improve overall storm capacities along the stream. At the present time, culverts along the stream are fixing the channel gradients. Regrading the channel would require replacement of these culverts at new elevations.

One remaining modification would be to widen the channel. Flow capacities could be greatly increased if a wider channel is utilized. Such a measure would require significant excavation, and more important, extensive restabilization of the new channel banks after construction.

Cleaning, realigning, regrading, and widening of the channel are clearly measures which will increase storm flow capacities. It is important to note that these measures will have no adverse effects on groundwater levels, as all of these modifications will serve to remove stormwater, and preclude recharge.

LINING THE CHANNEL

Lining the channel is an alternative for increasing capacities. A smooth concrete channel, for instance, would move large volumes of water much more rapidly than would an earthen channel. Flooding of the banks could be prevented by building steep, tall concrete walls along the channel.

This alternative would be very costly, and would pose serious problems to groundwater discharge into the stream from the surrounding water table. A concrete channel would seriously hamper groundwater drainage along the stream. Therefore, lining of any extensive section of the Northeast Branch is not a viable alternative.

RENOVATION OF TRIBUTARY STORM CHANNELS

Many of the positive drainage systems which service residential areas adjacent to the Northeast Branch have outfalls that drain toward the main channel via small tributary channels. As previously mentioned, these tributary channels are often blocked, clogged, or even non-existent. In particular, the areas surrounding the headwater ponds, and areas along the east side of the Northeast Branch from Annette Ave. to Hallock Ave. are served by these tributary systems. They can be seen on Plate III of Volume 1 of this study.

Clearing, realigning, or even setting pipe in these channels would significantly increase the efficiency of the existing stormwater systems. Presently, water backs-up, and disperses along these channels. Improvements would allow stormwater to move from outfall locations to the main channel and out of the area very quickly, allowing minimal recharge. This would be a desirable improvement, in view of existing groundwater flooding conditions which occur in these areas.

It should be noted that improvement of these channels would increase peak flows in the main stream by allowing more rapid and intense concentration of storm runoff at the streams. Therefore, such improvements, while very desirable, can only be considered in conjunction with improvements which would serve to increase stormwater capacities in the main channel.

IMPROVEMENT OF EXISTING STORMWATER COLLECTION SYSTEMS

There exist a number of locations in the vicinity of the Northeast Branch where existing stormwater collection systems have proven inadequate. The resulting roadway flooding at these locations poses many problems. Initially, there is inconvenience and hazard to motorists. Water which floods some of these roadways eventually spills onto properties, inundating non-roadway areas. These waters, which are retained on the street and on nearby properties, often recharge into the ground, which is unacceptable at some locations due to existing groundwater flooding problems.

The roadways along which problems exist in collection systems are Townline Rd. (C.R. 76), between Mt. Pleasant Rd. and Terry Rd., Terry Rd. (C.R. 16) north of Smithtown Blvd., Mt. Pleasant Rd. north of N.Y.S. Rt. 347, roadways in the vicinity of North, South, Hallock and Larson Aves., and roadways in the vicinity of Princeton, Cornell and New York Aves. Conditions at each location vary.

Townline Rd. has a very limited and incomplete positive drainage system. Flooding could be alleviated if such a system was constructed. Outfall for runoff could be piped to either a recharge basin or to the Northeast Branch. The proximity of the water table is prohibitive with regard to utilization of a recharge basin, and additional burdens which runoff would pose to the Northeast Branch are not presently acceptable. Construction of a positive drainage system for Townline Rd. with outfall to the Northeast Branch can only be considered in conjunction with improvements in channel storm capacities.

Under present conditions, Terry Rd., north of Smithtown Blvd., receives large amounts of stormwater runoff from steep slopes and roadways along its east side. During heavy rain, for example, the roadway floods and water rushes across Terry Rd. at Rhoda Ave. and into the low-lying residential area to the west. Similar conditions exist at the Larson Ave. intersection. Runoff to these areas is compounding their stormwater and groundwater problems. Ideally, Terry Rd. should dispose of its runoff in-situ. Installation of leaching pools, as well as regrading to prohibit flow from entering neighborhoods along the west side of the road, could help relieve flooding of Terry Rd. as well as groundwater recharge and flooding in adjacent neighborhoods. Severe flooding which used to close

Terry Rd. at its northern end during heavy rains has been relieved by installation of leaching facilities, indicating that such methods are workable.

Positive piping of runoff to the Northeast Branch is another alternative for Terry Rd. However, as with Townline Rd. the channel must be modified for increased stormwater flow capacity if it is to be used for outfall from Terry Rd.

Mt. Pleasant Rd. (north of N.Y.S. Rt. 347), and areas near North, South, Hallock, Larson, Princeton, Cornell and New York Aves. all represent roadways where adequate positive drainage systems are not presently operative. In many cases, water runs off of streets and onto properties. Princeton Ave. is a location where this problem is acute.

These streets do have varying amounts of facilities. Many appear clogged and non-functional. Curbing is absent in most cases. The need for an inventory and clean-up of existing structures is indicated, followed by a study to determine where facilities would be necessary to relieve the remaining problems. As with Townline and Terry Rds., direction of runoff to the Northeast Branch is an alternative that can be considered only if the stormwater capacity of the Northeast Branch is increased.

23. ALTERNATIVES FOR DEPRESSING THE LOCAL WATER TABLE

The hydrologic parameters which we have determined to exist in the vicinity of the Northeast Branch of the Nissquogue River indicate that a variety of feasible options are available for depressing the water table. These alternatives can be summarized as follows:

(1) Installation of underdrain networks which lower the water table by utilizing hydraulic gradient to carry groundwater to the stream (the Northeast Branch).

(2) Installation of well point dewatering systems throughout the flooding problem area for removal of groundwater and its conveyance to the stream.

(3) A general clean-up of The Branch for purposes of augmenting storm and base flow capacities.

(4) Complete renovation of the Northeast Branch, including lowering the channel to increase hydraulic gradient to the stream, and thereby lower area groundwater levels.

(5) Utilization of existing public water supply wells, in conjunction with additional high yield wells, to lower the local water table.

The feasibility of obtaining relief through implementation of any one or a combination of these alternatives is evaluated based on existing data, local drilling, the pump test which we conducted, and various hydrologic calculations. While this data base is sufficient for preliminary projections, it must be noted that actual design of underdrain, or pumped well systems could not take place without a considerable amount of additional pump testing of the various flooding problem areas. Such testing would be necessitated by the existence of various surface and aquifer materials encountered throughout the study area, and the need for hydrologic parametric analysis of each prior to determining exact underdrain spacings or well point depth and spacing.

For purposes of discussing and analyzing each alternative, the data base of Volume 1 of this study was carefully utilized in calculating results. Certain assumptions were made, as indicated in each section. Where assumptions were made, ranges of values were often used to represent a range of possible resultants. Certain data have been subjected to sensitivity analyses as previously discussed, so as to define a range of error which could be expected in various calculations. In all cases, estimates or results have been conservative in arriving at conclusions regarding degree of relief.

THEORY AND BASIC ASSUMPTIONS

Groundwater and surface water are not separate and independent units of the hydrologic system, but are rather closely inter-related. Withdrawal of groundwater from a water table aquifer near a stream will produce a time-delayed decrease in stream flow. The water table, in turn, responds to fluctuations in stream flow. The interdependence of surface water and groundwater is not limited to the case of a natural stream and the surrounding geologic deposits. Interactions with groundwater are also evident in the flow phenomena of canals, drainage ditches, recharge basins, lakes and reservoirs.

The purpose of this hydrologic study is to evaluate the feasibility of lowering the water table along the Northeast Branch of the Nissequogue River, in order to relieve flooding conditions which are associated with high water table conditions. Hydrologic evaluations were carried out by analyzing the local and regional movement of groundwater, the flow of surface water, and the interaction between them.

Three alternatives for water table lowering were evaluated by conducting geohydrologic calculations. These were: (a) well spacing and interference of wells to lower water table elevations; (b) placement (depth and spacing) of underdrains to intercept groundwater flow; and (c) lowering the Northeast Branch invert elevation (by regrading the main channel) to affect a lowering of surrounding water table elevations.

With certain limitations, and subject to certain stated hydrologic assumptions, the calculations of well spacing, underdrain depth, and underdrain spacing were conducted utilizing conventional standard procedures. However, changes of water table configuration as a result of lowering the stream channel are subject to a more restrictive set of assumptions, as will be discussed later.

UNDERDRAINS

Underdrains collect and redirect portions of natural groundwater flow. An underdrain normal to the direction of groundwater flow (interception drain), with the appropriate gradient, will result in flow approximately parallel to water table contours or normal to groundwater flow. By intercepting and diverting portions of groundwater flow, the water table in the area down gradient from the drain will be lowered. The effective distance from the drain, within which the water table is lowered down gradient from the drain, is directly proportional to the depth of the drain. The upgradient influence of an interception drain is very small.

An underdrain system oriented parallel to the direction of groundwater flow (relief drain) with the appropriate spacing, will cause the water table to be lowered to drain elevation at the drain interface, and will lower water table elevations between drains by some measure less than at the drain interface. The underdrain is constructed with perforated pipe and backfill having a hydraulic conductivity much higher than the adjacent aquifer material. The difference between the relief and interception type underdains is shown schematically in Figure 52.

(A) Design Discharge - The capacity of underdrains must be equal to the groundwater flow intercepted. The flow (Q) of groundwater which is intercepted and for which the underdrains must be designed equals the average groundwater velocity (V) times the

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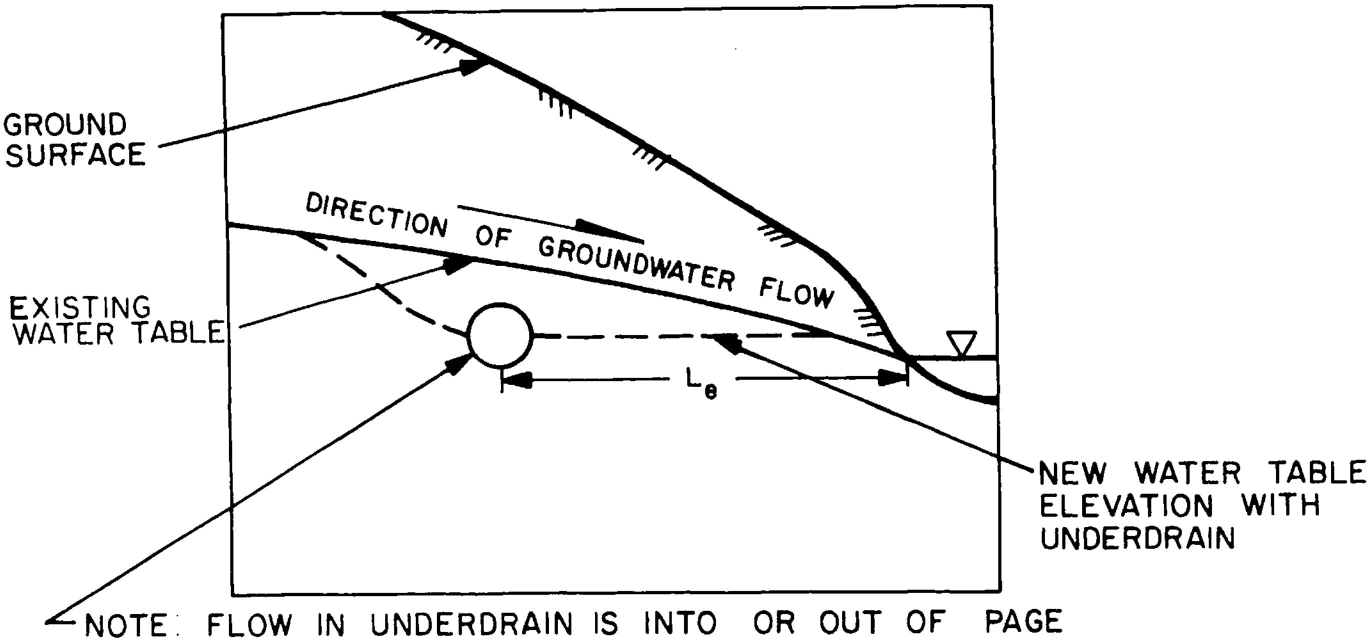
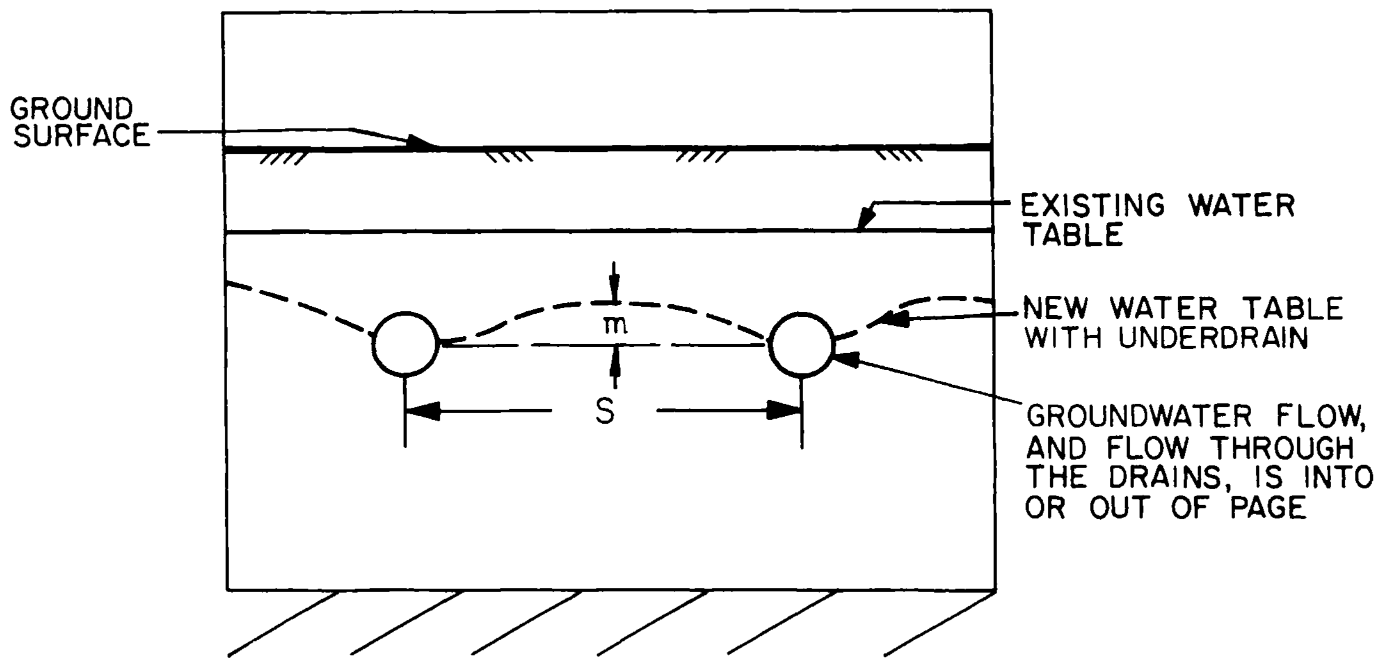


INTERCEPTION DRAIN



RELIEF DRAIN

UNDERDRAIN SYSTEMS

NOT TO SCALE

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cross-sectional area (A) of the portion of the aquifer which is intersected below the water table:

$$Q = VA \; . \; . \; . \; (1)$$

Darcy's law states:

$$V = K \, dh/ds \; . \; . \; . \; (2)$$

where, V = velocity of groundwater flow
K = hydraulic conductivity (permeability)
dh/ds = hydraulic gradient (natural)

The cross-sectional area of the aquifer intersected by an under-drain equals the effective depth of the drain times the length of the drain:

$$A = d_e \, L \; . \; . \; . \; (3)$$

where, d_e = vertical distance from the bottom of the drain to the undisturbed water table elevation

Therefore, combining equations (1), (2) and (3), the equation of the design discharge of an underdrain is:

$$Q = K \, d_e \, L \, dh/ds \; . \; . \; . \; (4)$$

where, Q = design discharge of a drain (gpd)
K = hydraulic conductivity (gpd/ft^2)
d_e = average effective drain depth (feet)
L = length of drain (feet)
dh/ds = natural hydraulic gradient of water table

(B) Analysis - The influence down stream from an interception drain (Le) can be determined from equation (4) above with the addition of two terms:

$$Le = \frac{K \, dh/ds}{q} (d_e - dw + W) \; . \; . \; . \; (5)$$

where, Le, K, dh/ds and d_e are previously defined
q = drainage coefficient (ft/day)
dw = the desired minimum depth to water table after drainage at a distance Le (feet)
W = the depth to water table before drainage at distance Le from the drain (feet)

W is assumed to be the same as the depth to the water table at the location of the drain itself.

The spacing between two relief drains (S) is calculated from the following equation:

$$S = \sqrt{\frac{4K\ (m^2 + 2am)}{q}} \quad . \; . \; . \; (6)$$

where, S = drain spacing (feet)
K = hydraulic conductivity (ft/day)
a = depth below drain to impermeable barrier (feet)
q = drainage coefficient or the drainability by gravity (ft/day)
m = vertical distance between water table, after drawdown, and the mid-point connecting the two drains (feet)

The effective distance of influence for interception drains and the spacing of relief drains were calculated utilizing known and assumed hydrologic conditions near the Northeast Branch.

(C) Effective Distance of Influence for Interception Drains - Using a range of numerical values such as K = 200 to 400 gpd/ft, a hydraulic gradient of about 0.003 to 0.007, an effective drain depth of 4 to 6 feet, a depth to water table at the effective influence distance of 2 to 4 feet, and a drainage coefficient of about 40 inches per year, the resulting distance of influence was calculated to be between 80 and 160 feet. This value is linearly proportional to the natural hydraulic gradient and inversely proportional to the drainage coefficient (q). The value for q is an unknown and was selected to be between 40 and 60 inches per year. A two-fold increase in the natural hydraulic gradient will double the effective influence of the interception drain.

Existing conditions within the problem area are such that the natural groundwater gradient is very low. As described earlier in this volume, the Northeast Branch drains a relatively flat area of water table. Therefore, unless gradient is increased (i.e. the channel is lowered), interception drains can provide very little relief.

(D) Spacings of Relief Drains - Spacings of relief drains parallel to the groundwater flow direction, were calculated for the following assumptions: a clay layer exists at about 40 feet below the surface; K = 200 to 400 gpd/ft^2(property of fill and aquifer materials); q = 44 to 88 inches per year; and a vertical distance, m, between 1 and 2 feet. The resulting spacing was calculated to be between 900 and 1,350 feet. Note that relief drains collect water and carry it by gravity for outfall to the stream. The effective drawdown of the water table (m) is a function of how deep the drains can be placed. Drains cannot be placed lower than the surface of water in the stream. The flatness of the present hydraulic gradient to the Northeast Branch is a limiting factor. Lowering of the stream would extensively enhance relief drain operation in this area.

The foregoing discussion indicates that the relief drains are best utilized for lowering the water table for short distances along the direction of the groundwater flow. In order to depress the water table elevation down gradient of a particular location, interception drains are far more effective. In either case, available hydraulic gradient is an important factor for efficient operation and maximum results.

RELIEF WELLS (WELL POINTS)

This alternative was evaluated for application in two ways:

(1) For utilization in relieving flooding problems at all locations within the Northeast Branch drainage area; and

(2) For utilization in limited, local areas such as the area in the vicinity of Adrienne Lane.

The first situation (relief in the entire Northeast Branch area) was eliminated based on the extensive number of wells required (and the related construction costs), the length of the connecting manifold (requiring extensive excavation), high operation and maintenance costs, and the fact that the need for a major clean-up of the Northeast Branch is not eliminated.

The second situation (localized relief in the vicinity of Adrienne Lane) is evaluated herein, as utilization of small yield wells is feasible for localized problem areas. Adrienne Lane serves as an example of such an area. As seen from the December 30, 1979 water table contour map (Figure 27, Vol. 1), the local groundwater divide separates the Adrienne Lane problem area from flow toward the Northeast Branch. Although modification to the Northeast Branch stream bed would change the local water table configuration, the problem area near Adrienne Lane will remain outside the Northeast Branch area of influence.

A. Method of Analysis - A preliminary feasibility study of potential relief wells has been made for the problem area near Adrienne Lane. The study determined the capacities, spacings and resultant drawdowns based on data described below.

Estimates of well spacing have been based on theoretical calculations utilizing the nonsteady state solution for pumped wells and the principal of superposition. The time dependent solution is justified by the fact that the proposed system of relief wells will not be operating on a continuous basis, but rather during periods of groundwater recharge (rain, snow melt, etc.).

The following assumptions were made:

(a) Dewatering is mostly from the Upper Glacial aquifer;

(b) Average depth of wells is about 15 to 20 feet;

(c) Water levels will be drawn down a minimum of three feet from their present elevations;

(d) T equals 6,000 gpd/ft (for the partially penetrating wells), S equals 0.04 to 0.1 (water table condition);

(e) A minimum pumping rate of 10 gpm per well is used, and drawdown is projected at the end of a 30 day pumping period.

Aquifer testing indicated that confined conditions probably exist in areas such as this (fill material present). However, for the purpose of conservative analysis, semi-confined and water table conditions were assumed.

B. <u>Analysis</u> - A well field layout is shown on Figure 53. A grid network of 15 wells, with approximate distance of 400 and 200 feet between adjacent wells, was calculated to be sufficient. Composite water table drawdown resulting from the simultaneous pumpage of the 15 wells was calculated at the center points of each grid unit. The calculated drawdowns were corrected to reflect water table conditions assuming saturated thickness of 40 feet. The results are tabulated below (see Figure 53 for grid point location):

COMPOSITE DRAWDOWN (FEET)[1]

GRID POINT	Q=30 gpm S=0.04	Q=20 gpm S=0.1	Q=10 gpm S=0.1
a	12	7	3.3
c	14	8	3.8
e	13	7	3.3
g	14	8	3.6

A pumping rate of 10 gpm per well, under water table conditions (S=0.1), will cause a water table drop of about three feet at the center points of the indicated grid system. Smaller S values should enhance larger water table drops. In the case of S values higher than 0.1 additional wells would be required to create the desired water table drawdown of about three feet.

Clearly, the utilization of low yield wells will provide necessary relief in the vicinity of Adrienne Lane. This example also serves to indicate that such a system could be successfully utilized in other localized areas.

GENERAL CLEAN-UP OF THE NORTHEAST BRANCH

Hydrologically, a clean-up of the Northeast Branch would assuredly have positive effects on water table flooding problems. Groundwater discharge to the stream and out of the area would be

(1) For all cases: T=6,000 gpd/ft, t=30 days and drawdown corrected for water table conditions.

Present hydraulic gradient to the stream, particularly in upstream reaches, is very slight. Groundwater entering the stream, according to Darcy's equation, is largely a function of this gradient. The flatness of the hydraulic gradient which persists in this area will severely limit the results which could be gained by channel cleaning.

Additionally, cleaning alone will not successfully increase storm flows. Channel dimensions, culvert size, and sharp channel bends will leave the existing channel with a less than adequate storm flow capacity, and allow for storage (back-up) of stormwaters, recharge of the water table, and serious groundwater flooding conditions with moderate and heavy rain events.

The alternative for cleaning up the Northeast Branch is an important one. It must be incorporated into any plan which is implemented for providing flooding problem relief. However, by itself, it will not be sufficient to provide appreciable relief from groundwater flooding problems in the vicinity of the Northeast Branch.

LOWERING OF THE NORTHEAST BRANCH STREAM BED

The purpose of the hydrologic evaluation of this alternative was to: (a) determine the amount of stream bed lowering necessary in order to lower the water table between two and three feet at a distance of up to 1,000 feet away from the stream; (b) to determine the increase of rate of flow resulting from water table lowering; and (c) conduct parametric analysis to evaluate the numerical range for the projected results.

The viability of this alternative was evaluated by utilizing the following data:

(a) Water table contour map of December 30, 1979 (Figure 27, Vol. 1).

(b) Water table cross-sections normal to the Northeast Branch at four locations.

(c) Results of stream flow gauging; and

(d) Results of aquifer pumping and recovery tests near the Northeast Branch.

In addition, the following assumptions were made:

(a) A clay layer or a layer of lower hydraulic conductivity exists at a depth of about 40 feet below the Northeast Branch and adjacent areas;

(b) The local groundwater divide is considered a constant head boundary;

(c) The Northeast Branch is maintained by a steady base flow due to groundwater discharge; and

(d) Recharge rate to the water table from precipitation in the study area is about 20 inches per year.

A. Method of Analysis - The equations describing the movement of water in a stream aquifer system are fairly simple. The solution of these equations, however, is difficult to obtain using known conventional analytical techniques because of complex boundary conditions. For this reason, when sufficient field data is available to describe the stream aquifer system, and its boundary and initial conditions can be defined, a numerical technique can be set-up for handling the solution.

The numerical technique is a model capable of duplicating an actual physical process with reasonable accuracy. In the case of the Northeast Branch, two conditions are believed to be most prevalent in the stream aquifer system under consideration: (a) discharge from the aquifer into the stream (base flow); and (b) seepage from the stream into the aquifer at times of high flow (bank storage). Both of these flows are taking place with the stream and aquifer hydraulically connected. Equations for nonsteady (time dependent) flow for these conditions should provide accurate representation of flow conditions. However, nonsteady flow across the stream boundary is caused primarily by fluctuation in the depth of water in the stream, and these fluctuations ordinarily occur over durations of time that are relatively short compared to the time increment used in groundwater modeling (i.e. finite difference model). Actual measured stream flow during periods of base flow (following a period of no precipitation) were compared with calculated flows using a steady state flow equation. The agreement between the measured and calculated flows was very close. For the purpose of calculating stream base flow, the hydraulic gradients were measured from the water table contour map of December 30, 1979 (Figure 27, Vol. 1) and the aquifer hydraulic conductivity and stream effective thickness from the average transmissivity (Table 11, Vol. 1).

The condition of seepage from the aquifer into the stream is illustrated in Figure 54. Flow is considered from point (a), some distance parallel to the direction of flow from the stream

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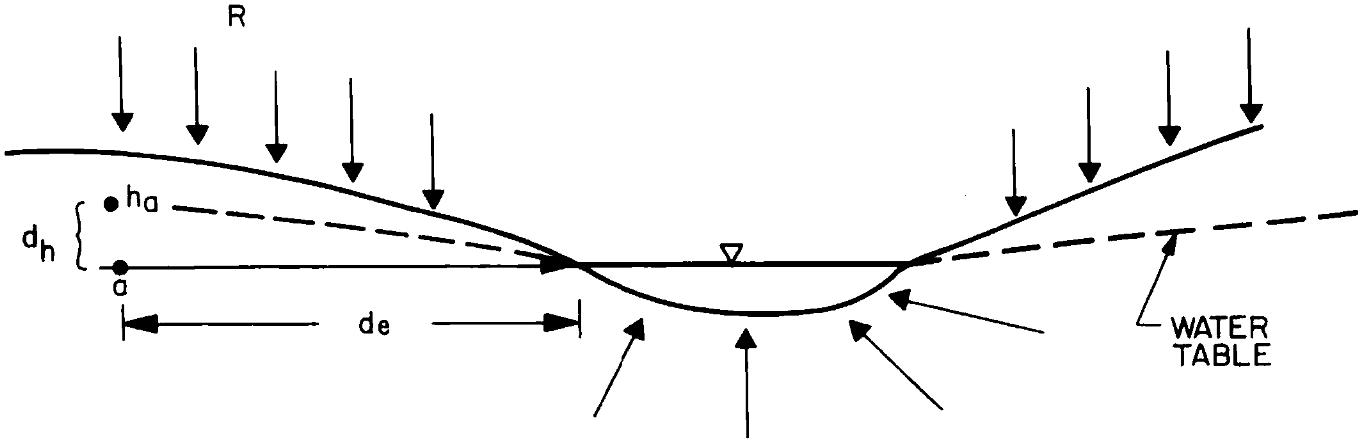
SEEPAGE FROM AQUIFIER INTO STREAM



$v = K \frac{dh}{de}$ - seepage velocity

$q = K\bar{h} \frac{dh}{de}$ - specific discharge

$q = Rde$ - specific discharge equals recharge

where, v is seepage velocity to stream.

q is the discharge of groundwater into the stream per unit length of stream (gpd/ft).

R is recharge (feet/day).

$K\bar{h}$ is the transmissivity (T) for saturated conditions in the direction of flow (gpd/ft).

K is the hydraulic conductivity (permeability) for saturated conditions (gpd/ft^2).

dh/de is the hydraulic gradient where dh is the difference in total head between point "a" and the stream, and de is the distance from point "a" to the stream.

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boundary (i.e. groundwater divide), to the stream. The relationship for seepage discharge into the stream from point (a) can be expressed by Darcy's law.

Darcy's flow equation can be integrated between the boundary conditions of constant heads (h_1 = water table elevation at the groundwater divide and h_2 = stream flow elevation), and over the length of flow (L between the groundwater divide and the stream). The resulting equation is the relationship between water table elevation, distance from the groundwater divide, recharge rate and the hydraulic conductivity. By knowing the constant head elevations along a flow line, normal to the local groundwater elevations, an approximate profile can be calculated. By integrating the specific discharge equation the amount of base flow can be calculated.

The steady state groundwater profile can also be determined by twice integrating the one-dimensional Laplace equation between two constant water table elevations. No explicit recharge term is included. However, the constant head boundary conditions are maintained by two fully penetrating streams (or other waterbodies). The high head boundary condition can be replaced by a constant head which coincides with the groundwater divide without introducing large error to the computations. The lowering of the stream channel is mathematically indicated by the sudden change of the constant head elevation in the stream.

At four locations along the Northeast Branch, cross-sections were constructed using the December 30, 1979 water table contour map (Figure 27, Vol. 1). The orientation of these cross-sections was approximately parallel to the groundwater flow (see Figure 55). When possible, the cross-sections were extended between groundwater divides. The steady state equations for water table flow were solved for conditions with and without recharge. The solutions to these equations allow the calculation of the water table elevation as a function of distance from a constant source (or head) boundary. The solutions are:

(with recharge) $$h_1^2 - h_2^2 = \frac{RL^2}{K} \quad . \; . \; . \; (7)$$

and,

(without recharge) $$Z^2 = h_1^2 - \frac{(h_1^2 - h_2^2)}{L}\, x \quad . \; . \; . \; (8)$$

where, h_1 = water table head (above datum) at a constant source or groundwater divide

h_2 = water table head at the stream (equals stream elevation)

R = recharge rate

L = total distance of flow between h_1 and h_2 measured in the direction of flow

K = hydraulic conductivity of the aquifer between h_1 and h_2

Z = water table elevation at any point x between h_1 and h_2. x is measured in the direction of flow

Groundwater discharge to the stream was calculated for both the steady state (base flow) and nonsteady state conditions. The nonsteady state discharge is needed in order to calculate the water table time-discharge in response to lowering water levels

FIGURE 55

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

LEGEND

WATER TABLE CONTOURS DEC. 30, 1979

INFERRED CONTOURS

WATER TABLE CROSS SECTIONS

MAIN LONG ISLAND GROUNDWATER DIVIDE

2000 0 2000

SCALE: 1" = 2000'

CONTOUR INTERVAL 2'
DATUM M.S.L.

ORIENTATION OF WATER TABLE CROSS SECTIONS THRU THE NORTHEAST BRANCH

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WILLIAM S MATSUNAYE, JR. PE MEDFORD, N.Y. (516) 475-1018 in the stream. The specific discharges (discharge per unit length of the stream), q, are:

(steady state) $$q = \frac{K}{2L}\left(h_1^2 - h_2^2\right) \quad . . . (9)$$

and,

(nonsteady state) $$q(x,t)\Big|_{x=0} \cong \frac{T(\Delta h)}{\sqrt{\pi (T/S) t}} \quad . . . (10)$$

where, q = specific discharge (gpd/ft of stream)

Δh = the change in stream bed elevation or water level (feet)

The complete nonsteady state discharge equation, for a water table aquifer, includes an exponential term which equals unity at x=0 (the interface with the stream). Equation (10) is an approximation, solved at x=0, eliminating the need for the exponential.

The use of equations (7) through (10) is subject to the stated assumptions and conditions. In order to check the applicability and accuracy of these equations in calculating the approximate resultant water table profiles which will follow the lowering of the stream bed, they were used in conjunction with existing water table profiles. Water table elevations at various distances from the constant head boundary were calculated and compared with the measured elevation (Figure 27, Vol. 1). The recharge rate was assumed to be about 20 inches per year, constant head boundaries are assumed or as dictated by groundwater divides, and a hydraulic conductivity K = 600 gpd/ft^2 was utilized. The results indicated the following:

(a) Reconstruction of the existing water table profile using the equation without the recharge term gave better results than the equation with the recharge term.

(b) The best agreement between the two equations in calculating existing water table elevations was in areas of relatively low hydraulic gradient.

(c) Calculated annual average base flow was found to to be in a very good agreement with stream gauging during September 1979.

B. <u>Analysis</u> - Water table profiles which would result from the lowering of the stream bed were calculated and plotted on the cross-sections which were oriented as shown on Figure 55. The results are shown in Figures 56 through 59.

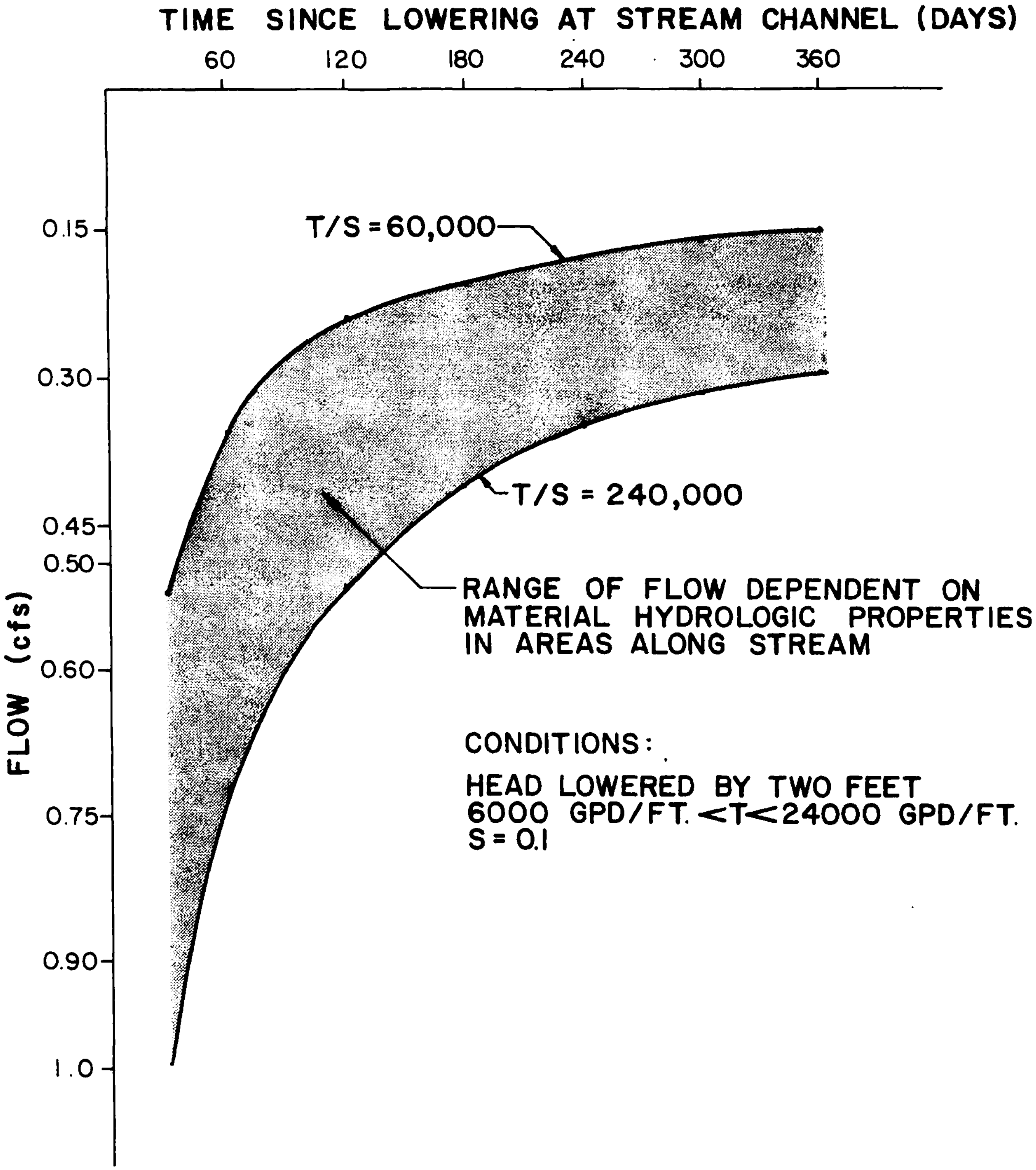
Following the lowering of the stream bed, the water table adjacent to the stream will adjust itself to the lower head by discharging a certain quantity of water into the Northeast Branch. The discharge can be calculated using equation (10). This relationship was calculated for water table conditions of S=0.1 and two transmissivities T_1=6,000 gpd/ft and T_2=24,000 gpd/ft (see Figure 60).

PUBLIC WATER SUPPLY WELLS

Three Suffolk County Water Authority (SCWA) well fields exist near the Northeast Branch study area. These are: (1) Pierson St. Station in Nesconset; (2) Liberty St. Station in Hauppauge; and (3) New York Ave. in Smithtown. A total of six high capacity wells are in operation (yields range from about 900 to 1,500 gpm

FIGURE 60

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



TIME VS. GROUNDWATER DISCHARGE

TO THE NORTHEAST BRANCH
FOLLOWING LOWERING OF CHANNEL

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each). The total depth of these wells ranges from about 183 feet (Well No. 1 at Liberty St.) to about 600 feet (Well No. 4 at New York Ave. Station). Average specific capacity of the three well fields is approximately 60, 50 and 25 gpm/ft for Liberty St., Pierson St. and New York Ave. Stations, respectively. Pumping of each well in these pump stations has been reported to average between 1,000 and 1,500 hours per year for the period 1973 to 1978. Utilizing the rated capacities for each well, this amounts to more than 2.5 million gallons per day.

A cursory analysis of the radius of influence of these well fields indicates no impact on the problem areas near or along the Northeast Branch. Furthermore, since the total depth of the wells is relatively large it is probable that production is primarily from the Magothy aquifer. If any of these well fields is having an appreciable effect on groundwater levels in the study area, visible effects should have been noticed near the Liberty St. well field, which is the shallowest in total depth, and the nearest to the golf course ponds south of Townline Rd. However, no evidence exists to support any effect by this pumping station on the high water table which exists below the golf course, and it has maintained the levels of water in the golf course ponds. It should be noted that our groundwater contour map (see Figure 27, Vol. 1) did not show any water table depressions in the vicinities of these wells.

This strongly suggests that high capacity wells, in order to effectively lower the water table in the study area, would have to remove water from surface glacial material. The yield of a high capacity well is the combination of relatively high hydraulic conductivity and large aquifer saturated thickness. These conditions do not exist along the Northeast Branch.

Furthermore, utilization of this water in south shore areas as had been suggested, would be difficult. No transmission facilities with diameters greater than 12 inches exist for exportation of water from this area to the south shore areas of Long Island. Additionally, water table quality in this area is poor, due to high groundwater table prcximity with cesspools and residential area pollutants, such as detergents and fertilizers. Utilization of this water would require a significant amount of purification treatment.

In view of the foregoing factors, a quantitative discussion and calculations for high yield well use as a method of gaining relief to flooding problems in the vicinity of the Northeast Branch has not been attempted. Such calculations would require testing of the aquifer interface between the Magothy and Upper Glacial aquifers. Rather than conduct such a costly test (construct several test production wells at depths exceeding 100 feet), we have concluded that utilization of high yield water supply wells for relief is not feasible based on data from existing facilities, and our own field tests and observations.

24. PROPOSED SOLUTIONS TO FLOODING PROBLEMS IN THE VICINITY OF THE NORTHEAST BRANCH

The preceding sections of this volume have delineated and described a variety of means available for gaining relief from stormwater and groundwater flooding problems which exist in this area. By carefully choosing and combining the most appropriate measures from the stormwater and groundwater alternatives, a comprehensive plan can be devised whereby each corrective measure provides relief for both groups of flooding problems, without further aggrevating existing conditions. Compatibility of the selected alternatives is of paramount importance.

Existing flooding problems, as well as available optional solutions strongly indicate that modifications to the Northeast Branch of the Nissequogue River will be necessary if relief is to be obtained throughout the area. Additionally, other steps will be necessary in dealing with more localized problems within the area.

REHABILITATE THE NORTHEAST BRANCH

The most important element in a flood relief plan for the vicinity of the Northeast Branch is a renovation of the Northeast Branch to a point where it is capable of performing its natural functions. The two primary functions of the Northeast Branch are:

(1) To intercept and collect groundwater and transport this flow downstream, thereby allowing the groundwater table to establish itself at a lower equilibrium position.

(2) To collect stormwater runoff and transport it downstream, via an adequate positive flow network that will prevent stormwater ponding in upstream reaches for unnaturally long periods of time, thereby minimizing groundwater recharge during heavy rainfall periods.

Modifications to the Northeast Branch must accomplish two major goals. Base flow must be increased so that the subsequent drawdown of the surrounding water table is maximized. Storm flow capacities must be upgraded to accommodate maximum storm flows without jeopardizing surrounding areas. The combined effects of lowering, regrading, widening, realigning, and stabilizing and maintaining the channel will accomplish these necessary improvements.

A. Lowering - lowering the channel will appreciably increase its ability to draw groundwater levels down in surrounding areas. The resultant drawdown has been calculated through four cross-sections which transect the stream through flooding problem areas. The change in groundwater elevation which will result from the amount of lowering shown in these cross-sections will end most of the basement flooding problems along the stream. These cross-sections, and the calculated result of channel lowering, were shown on Figures 55 through 59. Note that 2 to 3 feet of relief are gained in most problem areas, which is more than what is necessary to dry most basements, where flooding has generally been between 2 and 16 inches.

The extent of the channel lowering will be from the upstream side of the N.Y.S. Route 111 culvert to Bow Dr. In addition, the headwaters region south of Bow Dr. will require channelization, thereby extending the limits of vertical alignment from just south of N.Y.S. Route 111 to Steven Place.

B. Regrading - lowering of the channel will necessitate regrading. Regrading and lowering will involve removal and replacement of culverts at N.Y.S. Rt. 347, Branch Dr. and Terrace Lane. Removal and replacement of culverts is compatible with storm drainage improvements, which will also dictate replacement of culverts for purposes of achieving greater capacities.

Our storm flow calculations indicate that available gradient along the stream would be best utilized if held constant from the headwaters to Rt. 111. We have plotted a proposed new profile (see Plate IX) for the improvement of the stream, which maintains a constant channel gradient of 0.043 percent, and utilizes a gradient of 0.3 percent through all culverts upstream of Rt. 111. This profile, which lowers channel bottom elevations along the stream by as much as three feet, provides inverts which coincide with those shown on the cross-sections which appeared in Figures 56 through 59. Therefore, a high degree of groundwater flooding relief would be provided by implementation of this profile improvement.

Regrading of the channel, as described herein, will not appreciably increase stormwater capacities along the stream. However, lowering invert elevations along the channel will yield higher available gradients to the stream from tributary collection systems, thereby enhancing the efficiency with which these systems can transport stormwater runoff to the Northeast Branch.

C. Widening - while lowering and regrading of the channel will have the positive effect of lowering the surrounding water table, such changes will have other ramifications which must be considered. For instance, a new lower channel invert in the headwaters region will decrease the available gradient between Miller's Pond and the headwaters. There is no feasible way of avoiding this decrease in gradient, however, since elevations at both Miller's Pond and the headwaters are controlled by important factors. Miller's Pond, for instance, cannot be lowered in conjunction with the stream channel in order to maintain a higher gradient. This fact is demonstrated graphically by our bathymetric survey of Miller's Pond, which shows that any lowering of Miller's Pond will be accompanied by a large corresponding loss in Pond surface area; which is undesirable (see Plate V, Vol. 1). Therefore, it is recommended that Miller's Pond be maintained at its present level of 36.9± feet.

Additionally, as previously mentioned, it has been determined that the headwaters must be lowered to elevation 43.0 in order to have the desired effect on the surrounding groundwater table. Practically speaking, the gradient between Miller's Pond and the headwaters is fixed, and must be dealt with accordingly.

There is only one means by which a decrease in the channel gradient through the Northeast Branch can be accomplished without a decrease in flow capability. In conjunction with the channel lowering, various reaches of the channel will require widening in order to maintain the capacity necessary to carry the calculated design stormwater flows. The peak flows calculated for each reach are presented in Table 15, earlier in this volume.

The proposed widths and channel cross-sections which will be necessary to obtain sufficient capacities along the stream have been calculated, and are shown in Figure 61. Capacities for these trapezoidal channels are shown graphically in Figure 62. The cross-sections show trapezoidal channels with two on one side slopes (two feet horizontal to one foot vertical). The channel widths depicted in the cross-sections, in conjunction with the profile shown on Plate IX, will provide adequate storm drainage capacity throughout the stream channel, while relieving groundwater flooding problems in adjacent areas.

D. Realignment - stream realignment will be necessary at certain points. In its present condition, the stream contains a number of sharp meanders. Such bends pose two problems. First, sharp meanders retard flow, and allow debris to collect and block the channel. In addition, severe erosion takes place along the outside bank at these bends allowing the channel to migrate sideways into still sharper bends with time. During this migration, large trees become undermined, eventually falling across and blocking the channel.

Creation of an efficient channel is a necessary condition if adequate stormwater and groundwater drainage is to be provided. If the sharp bends which presently exist in the stream remain, an even wider channel would be necessary to compensate for the resulting inefficiencies. Furthermore, maintenance of a sharply meandering channel is extremely difficult, as stabilization is stifled by erosive forces. Plates X and XI depict the proposed channel realignment on the aerial photos.

E. Stabilize and Maintain - the channel which we have suggested herein will provide a great deal of relief. However, if the channel is to remain effective in relieving flooding problems in the vicinity of the Northeast Branch it must be stabilized and maintained. Stabilization will largely be vegetative. Flat gradient, large channel area, and absence of sharp bends will limit the erosion which high velocity flows might cause.

A maintenance program must be adopted. Present channel conditions indicate that garbage, waste, and debris periodically fill the channel. In addition, the fall season provides large amounts of leaves, branches, and other vegetation which clogs culverts, or degenerates into an organic silt deposition. Yearly maintenance should include a thorough cleaning after the fall season. In this way, cleaning follows the period during which debris collection in the stream is the greatest, and precedes a season during which storm flows are generally most critical.

For purposes of reducing human impaction on the stream bed, footbridges should be provided as deemed necessary. Presently, a temporary footbridge at Bow Dr. exists, but is dangerous and not aligned with existing sidewalks. Upon completion of the new channel this bridge will be inadequate and should be replaced. Another bridge, located at N.Y.S. Rt. 111, should be maintained for pedestrian crossing. Local residents should be polled to determine if other crossings will be necessary. Such bridges should be safe for pedestrian crossing, and should be chosen to esthetically blend with the natural stream surroundings (see Figure 63).

Each of the aforementioned channel improvements are necessary and equally important if restoration of an acceptable groundwater profile and flow regime is to be accomplished, and stormwaters are to be adequately and safely handled. Compromises will only serve to detract from the benefits and relief which these modifications would bring.

PROVIDE NEW CULVERTS

In order to carry the design flows outlined earlier in this volume, within a safe headwater limit, the culverts located at N.Y.S. Rt. 347, Branch Dr., and Terrace Lane must be upgraded or replaced completely by new structures.

There are two locations where it might be possible to replace the existing culvert at the new channel invert and simply add more pipes to obtain the required capacity. Specifically, these are the Branch Dr. and Terrace Lane culverts. It must be noted, however, that these culverts are constructed of corrugated metal pipe, which might reveal itself to be too deteriorated to be moved and reused cost-effectively.

The twin 36 inch diameter reinforced concrete pipes at N.Y.S. Rt. 347 cannot be reused under any conditions, due to their inadequate capacity.

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

PROPOSED FOOT BRIDGE TO BE UTILIZED

AS STREAM CROSSING

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RESTORE THE STORMWATER TRIBUTARY SYSTEM

The majority of the positive storm drainage systems which serve areas near the Northeast Branch have outfalls into small open channels, which in turn have outfalls to the main channel. Virtually all of these channels are clogged to the point of being non-functional. Water which enters these channels is often dispersed throughout the area, without finding its way to the main channel. Recharge of the area groundwater occurs. In light of present groundwater flooding problems, this is an unacceptable condition.

The modifications which we have suggested herein for the main channel will lend additional available gradient to these channels. Clearing, cleaning and maintaining of these channels, as well as regrading where necessary, will complete the network which is necessary for all stormwater considerations.

We have indicated the locations of these tributary channels on Plates X and XI. The systems which feed these tributaries appear on Plate VI, Vol. 1.

Each channel will have to be field surveyed and sized for proper design flows. In some cases, it may be necessary to extend pipe from existing outfall to a location at the main channel to provide adequate flow capacity. The need for major excavation or rerouting of these channels, however, is not anticipated. The goal in this case is to clean and restore existing natural, efficient drainageways wherever possible. Piping and excavation will be considered only where stormwater dispersion and the resultant groundwater recharge cannot be controlled.

The result of this tributary clean-up will be a more efficient, and more rapid concentration of stormwater at the main channel for transport from the area. This will preclude recharge of the water table. The main channel modifications which we outlined earlier will provide the increased capacity which will be needed to handle increased peak storm flows resulting from tributary improvements outlined herein.

PROVIDE CLEAR PASSAGE BETWEEN N.Y.S. RT. 111 AND MILLER'S POND

The channel which extends downstream of N.Y.S. Rt. 111 opens to a network of small meandering streams which eventually become Miller's Pond. These meandering streams are debris laden, and therefore, not functioning efficiently.

These streams will have to be cleaned, cleared, and graded. Construction of a single deep, wide channel, however, will not be necessary. While stormwater must be able to readily pass through this area to Miller's Pond (to prevent back-ups past N.Y.S. Rt. 111), dispersion of water throughout this span of channel will not be a problem, as flooding of roadways and basements will not result. It is imperative that flow through the existing channels be maximized by adequate cleaning of present debris and periodic maintenance.

IMPROVE FLOW FROM MILLER'S POND TO NEW MILL POND

The improvements which are indicated for the Northeast Branch from the headwaters to Miller's Pond will result in increased storm flows arriving at Miller's Pond. The new spillway, which is planned for Miller's Pond will have the necessary capacity to handle these flows. The downstream channel will require a field survey to determine what improvements will be necessary in maintaining a safe flow capacity. Preliminary studies show that the average gradient to New Mill Pond is sufficient, and channel widths are generally sufficient. The culvert at Brooksite Dr. is adequate. However, the channel condition indicates a need for clean-up and some grading. One location in particular (see Plate IX) downstream of Brooksite Dr., contains a mounded section of channel behind which water backs-up.

VERIFY OPERATION OF POND AND CULVERT AT BOW DR. AND REED ST.

A pond, which lies between Rainbow Dr., Bow Dr. and Reed St., serves as the outfall for positive storm drainage systems from surrounding streets. The water collects in the Pond and then moves to the Northeast Branch via a channel which is culverted beneath Bow Dr.

Lowering of the Northeast Branch will lower groundwater levels around and at the Pond. The expected water table level at the Pond in response to channel lowering is 45.5 feet above mean sea level. The exact elevation of the culvert, its condition, and the Pond depth and bottom composition must be such that water levels in the Pond can adjust to the new water table level. Existing maps show the existing culvert invert at 45.4 feet above mean sea level. Therefore, if the culvert is completely clear, water can move rapidly out of the Pond during and after precipitation, returning the Pond to groundwater level. In addition, if the Pond is shallow with a silty mud bottom (as anticipated), groundwater recharge will be held to an acceptable minimum.

It should be noted that if the Pond were incapable of properly draining to the new water table level (i.e., culvert blocked or improperly aligned), a water table mound condition might occur at the Pond, posing possible groundwater flood problems to adjacent homes.

INSTALL PERMANENT WELL POINTS FOR DEWATERING IN THE VICINITY OF ADRIENNE LANE

Severe groundwater basement flooding conditions exist in the area surrounding Adrienne Lane, Adrienne Ct., and the northern ends of Lenore Pl. and Sandra Dr. This area lies in a topographic depression, which reportedly was once a pond prior to filling for the construction of the adjacent shopping center.

The proposed lowering of the Northeast Branch will provide limited, if any relief for these residences, as they are quite distant from the stream, and separated from the stream by the main groundwater divide. The problem in this area is due to low land surface elevation and high water table conditions.

Presently, the area drains to a recharge basin pump station located in its northwestern corner. Water from this basin is pumped to the Northeast Branch when basin levels rise. This pumping has not provided sufficient relief to residents, and flooding has persisted.

The limited extent of this area, as well as its acute problem, dictate the installation of a permanent dewatering system connected to the existing pump station and outfall to the Northeast Branch for purposes of gaining flood relief. The layout of the streets, and the location of the existing pump station, will readily lend themselves to a 15 point grid of well points for dewatering, whereby adequate relief will be provided. This grid was shown in Figure 53, earlier in this volume.

Our calculations showed that, utilizing this grid, and 20 to 25 foot deep well points, a pumpage rate of only 20 gpm will be necessary to assure dewatering of the wet basements and properties at this location.

Figure 53 also shows grid points for which drawdowns of the water table resulting from implementation of this system were calculated. These values are in excess of those found to be needed during our field survey of these homes.

A separate pump and well system (see Figure 53, Auxiliary System) may be necessary as a means of controlling movement of water from the Sandra Dr. recharge basin to the dewatered area. This system would recirculate water from the basin to the ground and back to the basin, setting up a divide between the basin and the main grid system. The need for this auxiliary system, as well as final design of the entire well point system, will be determined after installation of a four point grid for testing of the groundwater regime at this location. Our initial investigations (see pump test at Bow Dr., Vol. 1) indicate that the dewatering system proposed herein will be adequate.

INSTALL GRAVITY PIPE OVERFLOW CONTROL FOR THE BOW DR./ MT. PLEASANT RD. RECHARGE BASIN

Severe basement flooding has occurred in homes surrounding this recharge basin. The land surface elevation at the basin is only 50 feet above mean sea level, representing a topographic depression in the area. As in surrounding areas, high water table conditions exist. However, they are compounded around the basin by low land surface elevations and hydrologic mounding which naturally occurs around a recharge basin. Water levels in the basin have climbed as high as 49.5 feet.

Lowering the invert of the Northeast Branch, as we have proposed, will not have a direct affect on water levels in the vicinity surrounding the recharge basin, as the main groundwater divide separates the recharge basin from the stream. Furthermore, the basin will always be a source of recharge to the water table, as it is a collection point for stormwater runoff. As a matter of fact, the area drained by the basin is the only area between Mt. Pleasant Rd. and the Northeast Branch headwater ponds which does not drain directly to the stream.

Lowering the channel will have an indirect effect on water levels at the basin. Such lowering will provide the gradient between the basin and the stream which would be necessary for installation of a gravity overflow pipe system from the basin to the stream. Such an overflow would maintain low water levels in the basin, minimize recharge at that location, and control the hydrologic mound which has been responsible for basement flooding in the adjacent area.

Figure 64 shows the plan for a gravity overflow from the basin to the proposed channel. The overflow inlet invert will be set at 45 feet above mean sea level. Our calculations show that maintenance of non-storm water levels at 45 feet above mean sea level in the basin will provide relief from hydrologic mounding and groundwater flooding in the vicinity of the recharge basin.

IMPROVE ROADWAY STORM DRAINAGE SYSTEMS

(A) Townline Rd. - Townline Rd. between Mt. Pleasant Rd. and Terry Rd. presently floods during heavy rainfall. No adequate, complete positive drainage system exists for this roadway. Flood waters are a road hazard, as well as a source of flooding on adjacent properties and in area homes.

The proposed modifications to the Northeast Branch would make installation of a positive drainage system for this roadway feasible. Recharge to surrounding areas could be minimized, as runoff could be directed to the headwaters of the Northeast Branch. The proposed stream would have capacities which would accept the Townline Rd. stormwater contribution. A proposed drainage system appears on Figure 65.

(B) Mt. Pleasant Rd. - Drainage improvements are also necessary along Mt. Pleasant Rd., between N.Y.S. Rt. 347 and N.Y.S. Rt. 111. Our study indicates, however, that such improvements could be minimized if existing facilities are simply cleaned, cleared, and rendered operative. Furthermore, curbing along certain gutter areas where property flooding takes place is advisable. Initial relief activity at this location, however, should be an inventory and clean-up of existing facilities.

(C) Terry Rd. - As we have indicated, the section of Terry Rd. which extends north of Smithtown Blvd. is in need of drainage facilities. Present conditions allow stormwater to run across Terry Rd. and into low-lying residential areas along its west side. In addition, the road floods periodically, and is unpassable at times. Installation of a comprehensive drainage system for this roadway is indicated. Leaching pools along the roadway are advised. Additionally, drainage could be directed in part to the Northeast Branch after improvements are completed.

(D) Residential Streets - The residential roadways on which flooding problems have been noted (North Ave., South Ave., Hallock Ave., Princeton Ave., Cornell Ave., etc.) are all in need of positive drainage facilities. In most cases, curbing and leaching pools should be installed to provide relief from property flooding.

25. RELATIVE MERITS OF PROPOSED SOLUTIONS

ENVIRONMENTAL ASSESSMENT

The existing environmental conditions described in Volume 1 for the Northeast Branch of the Nissequogue River present two crucial considerations for implementing management of this drainage system. First, groundwater throughout the area needs to be drained for effective lowering of the water table and alleviation of basement flooding problems in outlying developments. Secondly, flooding and wetland expansion due to the combined effects of rapid collection and storage of stormwater runoff at the Northeast Branch coupled with the regional water table rise must be eliminated and prevented from reoccurring.

The engineering phase of this volume presents five manageent alternatives as follows:

(1) Use of existing water supply wells and construction of new high capacity wells for large volume pumpage of groundwater to lower the local water table elevation.

(2) Underdraining problem areas to export flood waters.

(3) Relief (well points) or low-yield well constructions for low volume groundwater pumpage to lower the water table elevation.

(4) Clean up the Northeast Branch, thereby providing unimpeded storm flow to reduce overland flooding/wetland expansion.

(5) Excavate to lower the stream channel and change its hydraulic gradient, and redesign all culverts (except at Rt. 111) to improve stream flow into Miller's Pond and increase maximum channel capacity.

Proposed Action- Channel Lowering

Alternative number (5) has been selected as the primary management plan based on its overall merits compared to the other potential solutions. A detailed discussion of the proposed action is presented in earlier sections of this volume. Essentially, this plan calls for changing the hydraulic gradient to the channel by widening and deepening the segments between Steven Place north to Rt. 111. Average post-excavation stream depth will be 5.5 feet (headwaters) to 4.0 feet (south of Rt. 111), channel bottom width will be 15.0 feet (south of Rt. 347), 30.0 feet (Rt. 347 to Terrace Lane) or 35.0 feet (Terrace Lane to Rt. 111), while channel top width or the necessary right-of-ways will be 40.0 feet (south of Rt. 347), 50.0 feet (Rt. 347 to point of change from cut to fill) or 60.0 feet (south of Rt. 111).

As the Northeast Branch receives constant contributions from local groundwater supplies, representing the actual height of the water table, the proposed action can achieve significant, benefits as a multi-purpose watershed management plan. Stream channel cross-sections 1 through 4 (Figures 56 through 59) depict the expected change in groundwater gradient input to the Northeast Branch system. The various depth-of-excavtion lines presented provide information on water table lowering in outlying areas. A substantial drop in local water table elevation is anticipated within a quarter-mile (horizontally outward) from the stream centerline, by excavation to the depths presented earlier. Working in unison with increased stream bed depth, channel widening enables greater quantities of ground and surface runoff waters to enter the drainage system and work their way to their ultimate point of discharge (Long Island Sound) without overflowing stream banks during peak flow periods.

Environmental Advantages

Increased maximum stream capacity coupled with a drop in water table height creates a number of advantages from an environmental/public health standpoint:

(a) Relief of basement flooding problems due to water table lowering improves the quality of life in the residential areas affected;

(b) Outlying developed areas will be protected from stormwater inundation during future peak flow periods;

(c) The majority of construction related impacts will be confined to undeveloped areas, thereby supporting the economic and social feasibility of the proposed action;

(d) Right-of-way acquisitions necessary are held to a minimum; privately owned properties will not be significantly, if at all, impacted;

(e) Both short and long-term resource consumption will be held to an absolute minimum (i.e. money, fuel, water); as once excavated, the drainage system requires minimal periodic maintenance of culverts and stormwater outfall points;

(f) During base flow periods there will be no unnatural stream inpacts. Channel excavation will only influence peak flow handling capabilities;

(g) Improved drainage of outlying areas will allow wetland expansion boundaries to recede back to their natural limits thereby retrieving previously flooded upland forested habitats;

(h) Established recreational use of the drainage system (i.e. Greenbelt Nature Trail) will be improved in terms of both esthetics and public health and safety due to clean-up efforts and flood water recession.

Environmental Disadvantages

Although the above benefits can be realized through implementation of the proposed action, certain disadvantages, in the form of natural environmental constraints do exist:

(aa) Channel excavation will irretrievably commit a significant amount of New York State regulated fresh water wetland resources. Activities of this type are subject to New York State Environmental Conservation Law - Article 24 rules and regulations governing all fresh water wetland modifications;

(bb) Channel excavation will significantly alter the existing stream ecosystem. Extensive habitat modification, sedimentation and stream bed alteration will result. This activity is subject to New York State Environmental Law - Article 15, Title 5 (protection of waters) rules and regulations governing disturbances of a stream bed;

(cc) Minimal loss of wetland wildlife habitat;

(dd) The potential to impact wetlands outside of the necesary limit of clearing for excavation. Substantial drainage improvements in outlying areas may reduce available waters supporting contiguous wetlands;

(ee) The limit of clearing outside of the earlier presented right-of-way needs (channel top widths) is 5.0 feet on each stream bank. Provisions must be made for disposal of excavated spoil and cleared vegetation. Post-excavation, these denuded areas will have to be stabilized with acceptable vegetative ground cover so as not to be subject to wind and water erosive forces.

Based on these environmental constraints, stream alteration to such a degree and the significance of potential impacts associated with the proposed action will necessitate filing of an Environmental Impact Statement (EIS). Rules and regulations governing activities of this type and magnitude are those of the New York State Environmental Quality Review Act (6NYCRR-Article VIII).

Mitigating Measures for Proposed Action

Impact mitigations with regard to constraints (aa) and (cc) are possible; physical loss of wetlands acreage/habitat is unavoidable. Constraints (bb) and (ee) do lend themselves to amelioration. Sedimentation impacts downstream can be minimized by employing traps in the form of staked hay bales, sediment basins, stormwater outfall settling basins, or other appropriate collection devices. Upon project completion, trapped alluvium can be mechanically removed from the collectors and disposed of according to construction specifications.

The potential impact to wetlands outside the clearing limits, constraint (dd), are undetermined at this time. Problems of this nature will be investigated in the EIS once initiated. In the event that impacts are defined, appropriate management recommendations will be formulated.

An extremely important consideration is the need for stream bank and channel stabilization post-excavation. The most environmentally compatible means of accomplishing this task is by vegetating the stream channel and upper banks. Stabilization is desirable for two primary reasons: (1) as a soil conservation/erosion measure; and (2) to optimize the utilization of lands along the watercourse in vegetated form as an evapotranspirative agent. Acceptable plant species for this purpose include grasses such as Bermudagrass/Kleingrass mixtures, and shrubs such as Purple-osier willow. Both of these plant groups are recommended by the U.S. Soil Conservation Service as stream bank protection/permanent channel vegetation species. In the EIS phase of this study a detailed analysis of existing vegetation will be performed. Naturally occurring species will be evaluated for stabilization purposes, as this would be a more desirable means of control.

The effects of plant evapotranspiration on water table dynamics and precipitant recharge to groundwater are appreciable on Long Island. Maintenance of vegetation on exposed and/or non-constructed areas is then very beneficial in support of the principal objective of this project: to lower the height of the local water table. The more transpiring vegetation, the greater the need for supplemental water to meet plant requirements. Surface evaporation is thereby augmented, effectively drawing down shallow groundwater supplies. Peak evapotranspiration occurs primarily in late spring, summer and early fall during the growing season. Establishing cover by the start of this season will ensure both rapid rooting of cover plants and resultant substrate stability prior to periods of peak stormwater flow.

Adjunctory management recommendations include: (A) requiring screens as a preliminary treatment measure on stormwater discharge pipes outfalling into the Northeast Branch to prevent large debris from clogging the stream channel and impeding flow; (B) periodically maintaining these discharge pipes for unrestricted outfall; and (C) rip-rap steep slopes at culvert locations where needed (greater than 2:1 grades) up to overpassing roadways where vegetative cover is difficult to establish and maintain.

Excavation activities overall will be managed from an environmental as well as engineering standpoint. Clearing activities and equipment access and operation will be supervised to retain as much existing cover in an undisturbed state as practicable. Spoil disposal and site restoration will be performed in a competent and environmentally compatible manner.

Alternatives To The Proposed Action

The remaining management alternatives outlined previously, although feasible, are either less effective and/or more resource consumptive on both short and long-term basis.

Cleaning the Northeast Branch

Alternative (4), cleaning up the Northeast Branch to enhance stream flow, is the most obvious necessary action as described in Volume 1 under "EXISTING CONDITIONS". Effective implementation of alternatives (1) through (3) commonly require extensive channel cleaning to improve drainage and volumetric capacity.

Stream channel conditions must improve for efficient, unrestricted flow of surface waters through this drainage system. A massive clean-up effort would serve this purpose, however, it would not ameliorate the crucial problems of basement flooding and associated public health and safety hazards, wetland expansion, and impaired recreational usage. In its present condition the Northeast Branch simply cannot handle peak flows resulting from both natural surface runoff and stormwater discharge from adjacent residential sectors.

Large Volume Pumpage

Large volume pumpage as proposed in alternative (1) has merit, but the overall disadvantages of this action makes it an unacceptable solution. Relief of basement flooding and long-term maintenance of safe and healthy conditions is possible, thereby meeting project objectives. This action would preclude any potentially significant adverse environmental impacts to regional fresh water wetlands as well. However, the shortcomings of this type action include:

1.(a) In order to effectively lower the water table, groundwater must be withdrawn from glacial aquifer supplies.

Disposal of large volumes of glacial water is then the problem. The present conditions of the Northeast Branch prohibit stream discharge of this pumpage without a minimum supportive action as described for alternative (4). The Northeast Branch cannot handle elevated base flow conditions and be expected to eliminate problems other than basement flooding during peak flow periods.

1.(b) The unpotable nature of glacial water supplies in the project vicinity preclude its use for drinking water export to south shore areas in need of supplement. Therefore, pumpage export is unreasonable on a cost-effective basis due to transportation costs and the need for treatment prior to consumption.

1.(c) Total system energy requirements on a long-term basis are non-conservative. Although the system requires little maintenance due to the small number of wells involved, energy consumption in operation of high capacity pumps is great. Continuous pumping will be required in the event that water table heights return to problem levels between periodic lowerings. Energy needs will then be constant.

1.(d) An important physical constraint is the potential for local subsidence of geologic substrata due to high volume pumping. The zone of influence (water table lowering and cones of depression) surrounding these wells could result in foundation settling in the heavily developed sectors to be managed by this means.

1.(e) High volume pumpage of this type is regulated by the New York State Department of Environmental Conservation under

Article 15, Title 15 of the New York State Environmental Conservation Law (6NYCRR) governing water supplies and Long Island wells. An EIS may be required additionally, if significant impacts are anticipated from this action.

1.(f) The large zone of influence described in item 1.(d) previous, has the potential for dewatering adjacent areas outside the project boundaries.

1.(g) Permits from both the State Conservation Department and the Suffolk County Health Service will be necessary with regard to pumpage disposal and regional quality of life, and, as well as clean-up operations.

1.(h) Pumpage outfalling to the Long Island Sound or Great South Bay represents an irretrievable loss of water resources.

Relief Wells

Alternative (3), the use of relief wells or low yield wells achieves the same results as alternative (1). Basement flooding will be alleviated and the integrity of wetlands along the Northeast Branch will be preserved. Disadvantages of this action include:

3.(a) Construction of the large number wells and associated connecting pipes required will significantly impact all residential sectors involved due to excavation needs. Impeded traffic, noise and potential loss of private properties comprise the major impacts.

3.(b) Total energy consumption will be great considering the number of wells, connecting pipes, electrical needs, pumpage export, and man-power and maintenance in all respects.

Continuous pumpage and energy needs will be warranted if acceptable reduced groundwater levels cannot be maintained.

3.(c) As with alternative (1), pumpage disposal is a problem. The limiting factor is the ability of the Northeast Branch to meet peak flow needs without flood ramifications. Discharging low volume waters into the Northeast Branch may result in severe flooding in the event of 10 or 50 year storms.

3.(d) Low volume discharge to the Northeast Branch will mean continued unnatural elevated base flow conditions. Stream scouring that results could impact downstream segments and/or Miller's Pond through accelerated uncontrolled sedimentation.

3.(e) Without stream channel cleaning this action will be ineffectual, accentuating overland flooding and wetland expansion into residential sectors and upland forests in outlying areas.

3.(f) The same State and County permits will be required as for alternative (1). Rules and regulations governing this type action fall under New York State Environmental Conservation Law - Article 15, Title 15 water supply and Long Island wells.

Underdrains

The final feasible solution, underdraining (alternative (2), can achieve relief of basement flooding with minor disturbance of local fresh water wetlands. Effective implementation of this action, however, is predicated on the existence of a sufficiently high hydraulic gradient to drain away excess groundwater to areas of lesser concentration or to surface water outlets.

Ultimate discharge of drainage waters would be into the Northeast Branch system, as retrieval and export are impracticable with this technology. The same peak flow constraints hold for this alternative as for those previously mentioned.

In order to effectively drain the project area a substantial network of drains must be placed as the benefits of a single drain are very localized. The impacts to private residences and the need for obtaining rights-of-way would then be great. Upon construction, the effectiveness of the drains as influenced by groundwater behavior, stream conditions, and prevailing atmospheric factors will be difficult to assess. Construction impacts to both the social and natural environments could then be considerable. The flatness of the project area water table requires excavation to appreciable depths to achieve the necessary hydraulic gradient for draining. Wetland and stream bed disturbance would then occur, its magnitude dictated by the existing conditions of specific problem areas. Overall construction related impacts will be similar to those described for alternative (3), (item a).

Permit requirements will combine the needs of alternative (5) (fresh water wetlands and protection of waters) and (1) and (3) (water supply and Long Island wells). Additionally an EIS may be required under Article VIII of the New York State Environmental Quality Review Act (6NYCRR, Part 617).

IMPLEMENTATION

The nature of the flooding problems which presently plague areas in the vicinity of the Northeast Branch of the Nissequogue River are such that gross inconvenience, hazard, and illness might result from postponement of relief measures. The rapidity with which the implementation of rectifying measures can take place is very important. A degree of urgency exists.

Of the various alternatives which exist for providing relief in flooding problem areas, we feel that the measures which we have proposed represent those which can be most readily, and easily implemented. If adopted, the channel improvements which we have proposed need only be designed, bid, construction initiated and completed with only minor land acquisition. The need for additional large scale hydrologic testing would be eliminated. Time consuming design of pump, transmission and dewatering facilities would be unnecessary. Acquisition of funding could be less time consumptive, as decisively lesser funding is required for implementation of our proposed measures versus well and underdrain alternatives.

The single most time consumptive aspect of the high yield well, well point and underdrain alternatives is additional testing. Particularly in the vicinity of the Northeast Branch, where large amounts of widely varying types of fill have been utilized in development, hydrologic testing would have to take place in each individual area. Design of well point grids, underdrain patterns, or high yield well grids, could only take place if all soil and substrata types in each area were tested for hydrologic properties.

As a result, implementation of any of these options would have to come in two phases. Initially, test and observation wells would have to be designed, bid, and installed throughout all areas. These wells would then have to be utilized in aquifer testing. The second phase, would be the design of wells (or underdrains), pumps, and transmission facilities for each individual area. Bidding and construction would follow. Necessary storm drainage improvements, including stream improvements at the Northeast Branch, could take place throughout this two phase process.

Clearly, the measures which we have proposed represent the best alternative for quick relief. While time of implementation should not in and of itself be the determining factor as to which alternative is utilized, it most assuredly is a prime consideration.

POTENTIAL IMPROVEMENTS

The methods which we have suggested herein for dealing with flooding problems in the vicinity of the Northeast Branch of the Nissequogue River could potentially solve nearly all flooding problems in the area caused by groundwater. While various factors, including continued development, very localized special conditions, or poor maintenance, could cause relief to be slightly less than 100 percent, we are confident that our recommendations do represent the best alternative for relieving of the vast majority of existing flooding problems.

It must be pointed our that, technically, a surer, more complete (100%) solution is available. A dense network of dewatering well points, header pipe, and pump stations throughout the entire vicinity of the Northeast Branch would assure dryness. The water table could conceivably be unnaturally held at any chosen depth in all areas. This alternative, however, would require constant pumping (utilizing resources and manpower for operation and maintenance), years to implement (at least three years for testing and construction), excavation in nearly every street (and many properties) in flooding problem areas, and far greater costs than the alternative we have chosen. While the potential for improvement associated with well points is higher (100%), we feel that this potential is not significantly higher than the potential of that which we have recommended, and certainly does not warrant utilization of well points for relief throughout the study area.

Potential for improvement associated with utilization of high yield wells throughout the study area is similar to that associated with well points. Distance between these wells, however, would allow a slightly greater possibility for the non-solution of very localized problems between these high intensity wells.

Installation of an underdrain network throughout the study area represents the alternative which carries the least potential for improvement. Results in most areas would be nill, unless the existing channel were lowered to increase hydraulic gradient in conjunction with underdrain installation. Even with channel lowering, the amount of improvement would be subject to localized special soil or hydrologic characteristics. The dense underdrain network which would be needed would pose similar problems to that of the well point network (large scale excavation, testing, and high construction time and cost).

A routine clean-up of the Northeast Branch would provide marginal storm and groundwater flooding relief. However, by itself, this alternative will only reduce the degree of flooding, rather than end flooding conditions. Potential for long-term, complete flooding relief in most affected areas along the Northeast Branch due to a simple, routine cleaning, is poor.

In assessing the potential improvement associated with the measures which we have proposed, it is important to realize just what will be gained. We are certain that implementation of the measures which we have proposed will result in an adequate, highly efficient storm drainage system for the entire Northeast Branch area. Stormwater recharge of the water table will be kept to an absolute minimum. In addition, the vast majority of all groundwater flooding problems will be terminated, both through efficient stormwater removal, and maximization of groundwater discharge from the area.

While we cannot presently foresee any areas in which complete relief will not be gained, we must be prepared for this possibility. Localized conditions, such as those operative at Adrienne Lane, or a small clay bed, or an impervious fill area, could conceivably allow a few flooding problems to persist. In the event of such an occurrence, utilization of a small underdrain or well point system might be necessitated. Our proposed measures do not preclude utilization of such systems. To the contrary, our proposed measures include all channel modifications which would be necessary if these alternatives were exercised.

As a result, our proposed measures carry with them not only the guarantee of a vast amount of flooding problem relief, but additionally, the potential for 100 percent solution of all the flooding problems which exist in the vicinity of the Northeast Branch.

RELATIVE COSTS

In order to make the decision as to which solution yields the maximum benefit per dollar spent; a relative comparison of costs for each option must be made.

The five alternative solutions are:

(1) Use of existing water supply wells and/or construction of new high capacity wells.

(2) Underdrain system.

(3) Well point (low yield) system.

(4) Routine clean-up of the Northeast Branch.

(5) Major renovation of the Northeast Branch, including widening, horizontal and vertical realignment, and culvert replacement.

Alternates (1) and (3) (high capacity wells and well point system, respectively) would undoubtedly be the most expensive. In addition to the large expenditures for construction of either of these alternatives, alternative (4) would have to be implemented in conjunction with them, for stormwater and efficient groundwater transportation purposes.

An underdrain system is not a feasible solution to the groundwater situation unless it is part of a plan that includes alternative (5) (major renovation of the Northeast Branch). This is because an underdrain system requires the hydraulic gradient that only can be provided by lowering the stream channel.

A routine clean-up of the Northeast Branch (alternate (4) would not provide sufficient lowering of the water table without implementation of either alternative (1) or (3). Additionally, a routine clean-up of the Northeast Branch would not transform the Northeast Branch into an adequate storm drainage channel. This would mean that upstream ponding could still occur during moderate and heavy rain events, thereby defeating the purpose of the entire project.

The most feasible of the five alternatives is the option calling for complete renovation of the Northeast Branch. This is the only alternative that can stand on its own. In other words, once alternative (5) is implemented, no other alternate need be carried out in conjunction with it (other than isolated problem areas such as Adrienne Lane).

The fact that alternate (5) can be carried out on its own, makes it by far the most economical as well as cost-effective solution.

LAND ACQUISITION

Each alternative for alleviating the flooding problems in the vicinity of the Northeast Branch requires some taking of land; either for easement purposes, or outright purchasing of land for permanent use.

The intent of this section is to make a relative comparison regarding land acquisition for each of the alternates.

For instance, construction of recharge (or retention) basins to relieve flooding would obviously require many times more land than simply realigning and widening the Northeast Branch.

Other methods of groundwater flooding relief mentioned in this report are well point and underdrain systems.

Underdrain systems as well as well point systems rely upon critical spacing in order to have their desired effect on the groundwater table. This would necessarily mean running underdrains or header pipes through privately owned residential properties, causing substantial inconvenience to residents during the construction period, and high restoration costs to return the land to its prior condition.

We believe that complete rehabilitation of the Northeast Branch, will by far cause the least amount of inconvenience to local residents during construction; and in fact requires the minimum amount of land acquisition of all the options.

It must be noted here that even if an alternate other than complete stream channel rehabilitation were chosen, the channel would still require realignment. Therefore, certain lands adjacent to the stream must be obtained so that the stream may be properly aligned, regardless of which alternate is chosen. By choosing the alternate in which construction will take place primarily within the existing stream channel right-of-way, extraneous land purchases and easement acquisitions can be avoided.

As stated previously, land acquisition required to widen, deepen and realign the main channel is relatively small. For any point upstream from the Incorporated Village of the Branch, all lands required for channel widening or realignment are publicly owned by either the Town of Smithtown or Suffolk County. Obtaining permission to utilize these lands should present no major problem.

Figure 66 has been prepared to show which lands within the Incorporated Village of the Branch must be acquired. From this figure it can be seen that on the average, a 5 foot wide strip of land is necessary from the backside of each lot. This will increase the present 50 foot right-of-way to the required 60 foot right-of-way.

At points of curvature, more land will be required from the properties on the inside of the curve, while lots on the outside of the curve will remain untouched.

The total amount of land required from private owners for proper stream alignment and widening within the Incorporated Village of the Branch, is approximately 0.5 acre. This, of course, is a preliminary quantity which would be refined during the actual acquisition survey.

This amount of land acquisition is minimal when compared to the land necessary to construct a series of recharge or retention basins. The relative inconvenience to local residents is negligible since most improvements will be on public lands as compared to the inconvenience of installing an underdrain or well point system on many private properties.

26. INITIAL ESTIMATE OF COST AND FUNDING

GENERAL

Precise construction costs are difficult to estimate due to wide fluctuations in the local economic climate. Variations may occur depending upon the time of year, current labor and material costs, the needs of various local contractors for work, and the determination of more exacting quantities after comprehensive surveying and mapping of the project area.

PROJECT COST SUMMARY

The initial estimate of cost for realigning, regrading and widening the Northeast Branch, in conjunction with several satellite improvement projects, such as the Bow Dr. recharge basin overflow control, Adrienne Lane well point relief system, and Townline Rd. drainage, is estimated at $3,570,000., including engineering fees, legal fees, bonding costs and contingencies, but excluding land acquisition. This cost estimate is summarized in Table 17. Detailed cost estimates appear in Appendix G.

FUNDING

The initial estimate of project construction cost of $3,570,000. need not be completely borne by the County of Suffolk. Monies are available through various governmental agencies.

TABLE 17

SUMMARY OF INITIAL ESTIMATES OF PROJECT COSTS

MARCH 1980

ITEM	ESTIMATED COST
Main Channel Reconstruction	$ 1,800,000.
N.Y.S. Rt. 347 Culvert Replacement	240,000.
Branch Dr. Culvert Replacement	80,000.
Terrace Lane Culvert Replacement	90,000.
Renovating Tributary Channels to the Northeast Branch	55,000.
Channel Restoration from N.Y.S. Rt. 111 to New Mill Pond	35,000.
Overflow from Bow Dr. Recharge Basin to Headwaters	160,000.
Adrienne Lane Well Point System	290,000.
Reconstruction of Townline Rd. Including Drainage Improvements	820,000.
TOTAL PROJECT COST	$ 3,570,000.

For example, $835,000. in Federal Housing & Urban Development (HUD) community development funds were recently made available to the Town of Smithtown. A significant amount of this money could easily be utilized to relieve flooding problems, particularly in low to moderate income areas (as per HUD) such as those in the vicinity of Hallock, North and South Avenues (renovating tributary channels to the Northeast Branch to provide flood relief in this residential area is part of this project).

The New York State Dept. of Environmental Conservation (NYSDEC), in January of 1980, was considering an expenditure of $75,000. to study the water table and drainage situation in the vicinity of the Northeast Branch. The money was not expended, as this study (funded by Suffolk County) precluded the need for a NYSDEC study. Such monies should now be considered for expenditure in the design and construction phases of the proposed relief measures.

Town Highway Department monies represent another possible source of funding. The cost of many acitivies, including large scale pumpage of various recharge basins, as well as projects such as the modification of the Brilner Dr. recharge basin (reported in October of 1979 to be as much as $10,000. by the Smithtown Highway Department) are presently being borne by the Smithtown Highway Department. An end to such expenditures would be realized upon completion of the proposed project, allowing for the possibility of designating certain Town Highway funds to be used for the project.

Modification of the N.Y.S. Rt. 347 culvert will be necessary, indicating that the possibility of securing some NYSDOT funding exists. In addition, availability of Federal Highway Administration funds, other HUD funds (such as Federal flood insurance administration monies), and Town, County, State and Federal environmental conservation funding should be investigated.

The cost of not implementing flooding relief measures is another consideration. Assessed values of many of the flooded homes have already been lowered to reflect a drop in home value caused by the flooded basement conditions. This re-evaluation of flooded homes is continuing. Such a trend will result in appreciable yearly losses in revenues from property taxes.

27. ESTIMATED BONDING AND DEBT SERVICE

GENERAL

The initial estimate of project cost totals $3,570,000., as outlined in the previous section, and detailed in Appendix G. This figure represents total construction costs, including contingencies, engineering fees, legal fees and bonding costs, but excluding land acquisition.

The estimated bonding cost presented herein reflects the initial estimate of cost of the project. It should be noted that this cost could be considerably diminished if funding from other sources (as mentioned in the previous section) is secured.

In addition to estimating bonding and debt service for the entire project, we have also estimated a bonding and debt service for renovation of the Northeast Branch, including culvert replacement, but excluding the Adrienne Lane well point system, Townline Rd. drainage improvements, and the Bow Dr. recharge basin overflow control. For this case, total construction costs, including contingencies, engineering fees, legal fees and bonding costs, but excluding land acquisition would be $2,300,000.

ESTIMATED BONDING

A 30 year bond issue at 7 percent interest has been assumed in estimating annual costs. Presently, bonds have been sold at a rate of 6 to 6-1/2 percent. However, during recent months, instability in the national economic situation indicates the possibility of a 7 percent rate being in effect at the time when the bond issue is finally floated.

Based on construction starting no later than spring of 1981, bond anticipation notes will be used to pay construction costs until the bond issue is floated.

Assuming the bond issue is sold not sooner than late 1981, the first principal and interest payment should be included in the 1982 budget. The principal payments will be increased systematically to prevent excessively high debt service in the first few years when interest payments are high. In accordance with the local finance law, the maximum principal payment is not more than one and one-half times the smallest principal payment in any year. Table 18 indicates possible principal payment increments for a $3,570,000. bond issue. Table 19 indicates possible principal payment increments for a $2,300,000. bond issue.

DEBT SERVICE FOR PROPOSED PROJECT

Annual debt service for the proposed $3,570,000. bond issue is presented in Table 20. Based on the principal payments as detailed in Table 18, a 7 percent interest rate, and a generally declining debt service, the maximum payment (principal plus interest) occurring in the first year is $349,000.

Annual debt service has also been calculated for a $2,300,000. bond issue and is presented in Table 21. Based on payments as detailed in Table 19, a 7 percent interest rate and a generally declining debt service, the maximum payment (principal plus interest) occurring in the first year for the $2,300,000. bond issue would be $221,000.

TABLE 18

PROPOSED $3,570,000 - 30 YEAR BOND ISSUE

ANNUAL PRINCIPAL PAYMENT INCREMENTS

YEARS	ANNUAL PRINICPAL PAYMENT	TOTAL
First 6	$100,000	$ 600,000
Next 7	110,000	770,000
Next 6	120,000	720,000
Next 6	130,000	780,000
Next 5	140,000	700,000
30		$3,570,000 BOND ISSUE

TABLE 19

$2,300,000 - 30 YEAR BOND ISSUE

ANNUAL PRINCIPAL INCREMENTS

YEARS	ANNUAL PRINCIPAL PAYMENT	TOTAL
First 6	$60,000	$ 360,000
Next 7	70,000	490,000
Next 8	80,000	640,000
Next 9	90,000	810,000
30		$2,300,000 BOND ISSUE

TABLE 20

PROPOSED $3,570,000 BOND ISSUE

ANNUAL DEBT SERVICE

YEAR	REMAINING BOND ISSUE	INT. (7%)	ANNUAL PRINC.	ANNUAL DEBT SERVICE
1980		BAN*		
1981		BAN*		
1982	$3,570,000	$249,900	$100,000	$349,900
1983	3,470,000	242,900	100,000	342,900
1984	3,370,000	235,900	100,000	335,900
1985	3,270,000	228,900	100,000	328,900
1986	3,170,000	221,900	100,000	321,900
1987	3,070,000	214,900	100,000	314,900
1988	2,970,000	207,900	110,000	317,900
1989	2,860,000	200,200	110,000	310,200
1990	2,750,000	192,500	110,000	302,500
1991	2,640,000	184,800	110,000	294,800
1992	2,530,000	177,100	110,000	287,100
1993	2,420,000	169,400	110,000	279,400
1994	2,310,000	161,700	110,000	271,700
1995	2,200,000	154,000	120,000	274,000
1996	2,080,000	145,600	120,000	265,600
1997	1,960,000	137,200	120,000	257,200
1998	1,840,000	128,800	120,000	248,800
1999	1,720,000	120,400	120,000	240,400
2000	1,600,000	112,000	120,000	232,000
2001	1,480,000	103,600	130,000	233,600
2002	1,350,000	94,500	130,000	224,500
2003	1,220,000	85,400	130,000	215,400
2004	1,090,000	76,300	130,000	206,300
2005	960,000	67,200	130,000	197,200
2006	830,000	58,100	130,000	188,100
2007	700,000	49,000	140,000	189,000
2008	560,000	39,200	140,000	179,200
2009	420,000	29,400	140,000	169,400
2010	280,000	19,600	140,000	159,600
2011	140,000	9,800	140,000	149,800
	TOTAL:	$4,118,100	$3,570,000	$7,688,100

* BAN - Bond Anticipation Note Interest under Bond Issue

TABLE 21

$2,300,000 BOND ISSUE

ANNUAL DEBT SERVICE

YEAR	REMAINING BOND ISSUE	INT. (7%)	ANNUAL PRINC.	ANNUAL DEBT SERVICE
1980		BAN*		
1981		BAN*		
1982	$2,300,000	$161,000	$ 60,000	$221,000
1983	2,240,000	156,800	60,000	216,800
1984	2,180,000	152,600	60,000	212,600
1985	2,120,000	148,400	60,000	208,400
1986	2,060,000	144,200	60,000	204,200
1987	2,000,000	140,000	60,000	200,000
1988	1,940,000	135,800	70,000	205,800
1989	1,870,000	130,900	70,000	200,900
1990	1,800,000	126,000	70,000	196,000
1991	1,730,000	121,100	70,000	191,100
1992	1,660,000	116,200	70,000	186,200
1993	1,590,000	111,300	70,000	181,300
1994	1,520,000	106,400	70,000	176,400
1995	1,450,000	101,500	80,000	181,500
1996	1,370,000	95,900	80,000	175,900
1997	1,290,000	90,300	80,000	170,300
1998	1,210,000	84,700	80,000	164,700
1999	1,130,000	79,100	80,000	159,100
2000	1,050,000	73,500	80,000	153,500
2001	970,000	67,900	80,000	147,900
2002	890,000	62,300	80,000	142,300
2003	810,000	56,700	90,000	146,700
2004	720,000	50,400	90,000	140,400
2005	630,000	44,100	90,000	134,100
2006	540,000	37,800	90,000	127,800
2007	450,000	31,500	90,000	121,500
2008	360,000	25,200	90,000	115,200
2009	270,000	18,900	90,000	108,900
2010	180,000	12,600	90,000	102,600
2011	90,000	6,300	90,000	96,300
	TOTAL	$2,689,400	$2,300,000	$4,989,400

* BAN - Bond Anticipation Note Interest under Bond Issue

PRIORITY RECOMMENDATIONS FOR IMPLEMENTATION

Two factors govern the determination of priorities for implementation of measures which will relieve flooding problems in the vicinity of the Northeast Branch of the Nissequogue River. The first of these is time. Each day that passes prior to gaining relief in the flooding problem areas carries the threat of more hardship, health hazard, inconvenience, and destruction of property in these areas. The second factor is the inter-relationship of measures. Certain measures will not provide relief until other measures, upon which the first are dependent, are completed. Our priority recommendations are outlined as follows:

(A) The first priority, and most important of all measures, will be the decision which must be made toward a firm commitment regarding implementation of the measures proposed herein for providing relief to flooding problem areas through adequate funding.

(B) Immediately following this commitment, the land acquisition and design phase should be entered. Following a complete topographic survey of the Northeast Branch and its tributaries, and a soil boring program, design of the new channel, along with culvert and tributary improvements, should be undertaken.

(C) At the same time the minor land acquisition required should commence immediately after the necessary surveys are completed.

(D) The construction and renovation phase for the channel, its tributaries, and the culverts should be entered as soon as is possible following design and land acquisition, as herein lies the majority of the relief which will be gained during the project.

(E) The project design phase should continue as the channel construction phase is entered, with design of secondary systems such as Townline Rd. drainage, Adrienne Lane well points, and Bow Dr. recharge basin overflow being completed.

(F) The construction phase for each of these secondary systems should be entered as soon as the main channel renovations are completed to a degree whereby outfall from the completed secondary systems could be accepted by the main channel.

(G) A periodic maintenance schedule for all systems should be designed and implemented for purposes of propagating the high degree of flooding problem relief which will be obtained upon completion of the construction phase.

In our opinion, implementation of the measures which we have proposed, in the order of priority which we have outlined herein, will provide the most rapid relief possible for residents in the vicinity of the Northeast Branch of the Nissequogue River.

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APPENDIX G

INITIAL ESTIMATES OF PROJECT COSTS

APPENDIX G (CONT'D.)

The following nine cost estimates entitled, "INITIAL ESTIMATE OF COST", have been prepared as guidelines for comparisons of costs for the various relief measures recommended in this study.

They precede, but do not supplant, the usual "Preliminary Estimate of Cost" or "Engineer's Estimate" which is prepared only after complete and comprehensive surveying and mapping of the project area, with the project being designed therefrom and construction plans being completed therefor.

The surveying, design and preparation of plans would follow the acceptance of the recommendations contained in this study and to the extent determined by the County.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST FOR THE NORTHEAST BRANCH MAIN CHANNEL RECONSTRUCTION

ITEM	ESTIMATED COST
Clearing & Grubbing	$ 80,000.
Dewatering & Stream Diversion	30,000.
Channel Excavation (Unclassified) Bow Dr. to N.Y.S. Rt. 111	490,000.
Channel Excavation in Head-waters (Unclassified) South of Bow Dr.	220,000.*
Construction & Removal of Temporary Roads	30,000.
Rip-rap (Slope Protection-Change in Stream Alignment)	180,000.
Topsoil	120,000.
Seeding (Stream Bank Stabilization)	40,000.
Planting	80,000.
Footbridge at Bow Dr.	10,000.
Subtotal Construction Cost	$1,280,000.
Development Cost	260,000.
Contingency Cost	260,000.
TOTAL COST.	$1,800,000.

*Further detailed soils investigation may indicate the feasibility of utilizing a portable dredge, resulting in a significantly lower cost for this item.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST
FOR
N.Y.S. RT. 347 CULVERT REPLACEMENT

ITEM	ESTIMATED COST
Remove & Dispose of Existing Culvert	$ 10,000.
Remove & Replace Metal Guard Rails	5,000.
Pipe - 48" RCP	24,000.
Dewatering	10,000.
Stream Diversion	11,000.
Concrete Headwalls	25,000.
Traffic Maintenance & Protection	50,000.
Detour & Temporary Paving	21,000.
Pavement Restoration	8,000.
Resodding	6,000.
Pavement Striping	1,000.
Subtotal Construction Cost	$ 171,000.
Development Cost	34,500.
Contingency Cost	34,500.
TOTAL COST	$ 240,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST

FOR

BRANCH DR. CULVERT REPLACEMENT

ITEM	ESTIMATED COST
Remove & Dispose of Existing Culvert	$ 5,000.
Remove & Replace Wooden Guard Rails	1,000.
Pipe - 43"x68" ERCP	16,000.
Dewatering & Stream Diversion	7,000.
Concrete Headwalls	20,000.
Traffic Maintenance & Protection	2,000.
Concrete Curb	500.
Pavement Restoration	2,000.
Resodding	3,500.
Subtotal Construction Cost	$ 57,000.
Development Cost	11,500.
Contingency Cost	11,500.
TOTAL COST	$ 80,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST
FOR
TERRACE LANE CULVERT REPLACEMENT

ITEM	ESTIMATED COST
Remove & Dispose of Existing Culvert	$ 5,000.
Remove & Replace Wooden Guard Rails	1,000.
Pipe - 38"x60" ERCP	20,000.
Dewatering & Stream Diversion	8,000.
Concrete Headwalls	22,000.
Traffic Maintenance & Protection	2,000.
Concrete Curb	500.
Pavement Restoration	2,000.
Resodding	3,500.
Subtotal Construction Cost	$ 64,000.
Development Cost	13,000.
Contingency Cost	13,000.
TOTAL COST	$ 90,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST FOR RENOVATING TRIBUTARY CHANNELS TO THE NORTHEAST BRANCH

ITEM	ESTIMATED COST
Channel Excavation	$ 27,000.
Slope Protection	6,000.
Clearing & Grubbing	6,000.
Subtotal Construction Cost	$ 39,000.
Development Cost	8,000.
Contingency Cost	8,000.
TOTAL COST.	$ 55,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST
FOR
CHANNEL RESTORATION FROM N.Y.S. RT. 111
TO NEW MILL POND

ITEM	ESTIMATED COST
Channel Cleaning	$ 25,000.
Subtotal Construction Cost	$ 25,000.
Development Cost	5,000.
Contingency Cost	5,000.
TOTAL COST	$ 35,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST FOR OVERFLOW FROM BOW DR. R/C BASIN TO HEADWATERS OF THE NORTHEAST BRANCH

ITEM	ESTIMATED COST
Clearing & Grubbing	$ 1,000.
Pipe - 18" RCP	45,000.
Dewatering	4,000.
Catch Basin	2,000.
Manholes	14,000.
Concrete Overflow Structure	3,000.
Concrete 18" Flared End Section	500.
Replace Fence (R/C Basin)	500.
Rip-rap	500.
Bulkhead Existing Pipes	500.
Concrete Curb Replacement	3,500.
Pavement Restoration	35,000.
Resodding	1,000.
Unsuitable Excavation	2,500.
Select Backfill	1,000.
Subtotal Construction Cost	$ 114,000.
Development Cost	23,000.
Contingency Cost	23,000.
TOTAL COST.	$ 160,000.

APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST

FOR

ADRIENNE LANE WELL POINT SYSTEM

ITEM	ESTIMATED COST
Well Pumps & Appurtenances	$ 40,000.
2" dia. Wells	22,000.
Main Discharge Pump	9,000.
Pipe - 6" DIP	79,000.
Pavement Restoration	53,000.
Resodding	4,000.
Subtotal Construction Cost	$ 207,000.
Development Cost	41,500.
Contingency Cost	41,500.
TOTAL COST.	$ 290,000.

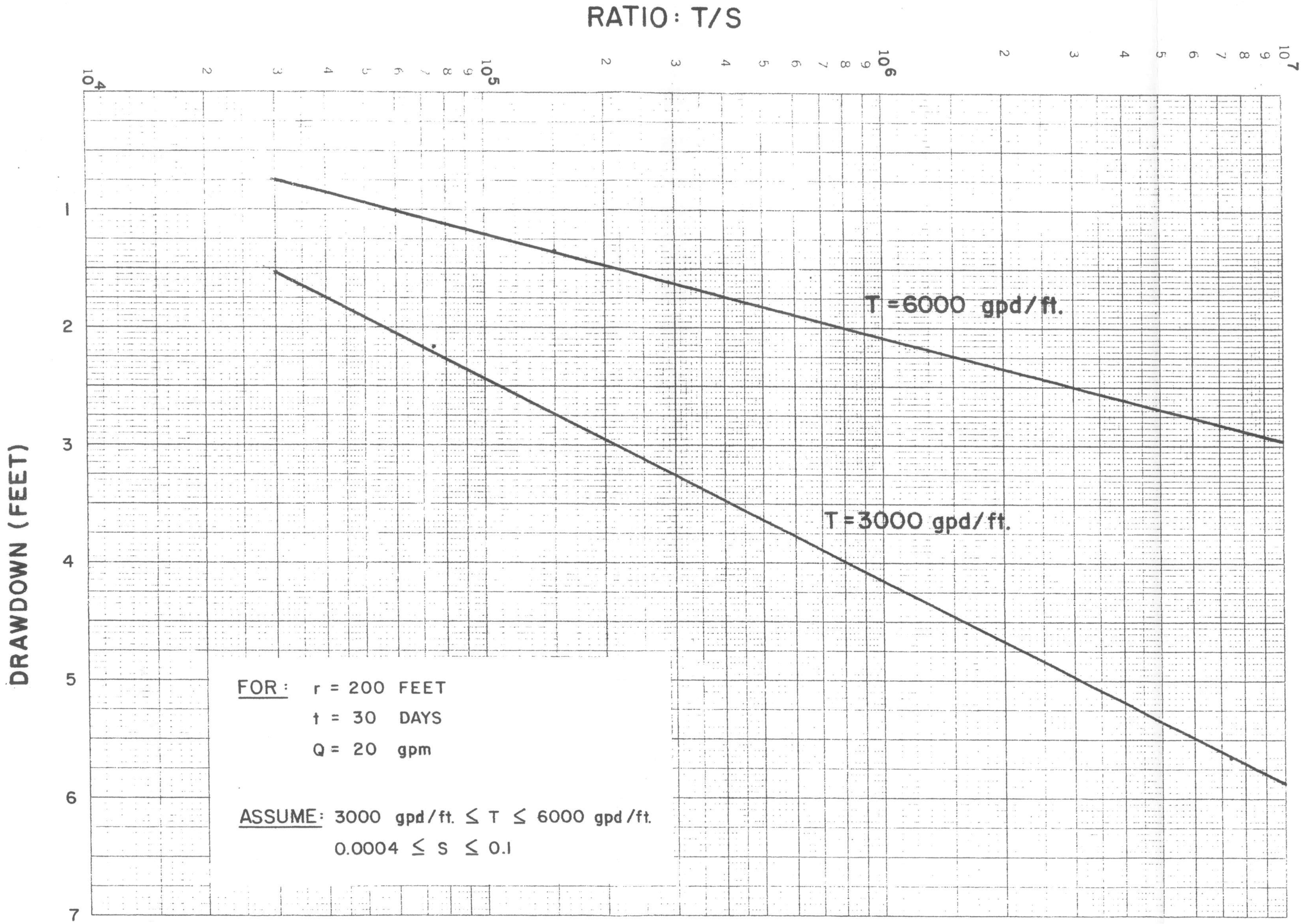
APPENDIX G (CONT'D.)

INITIAL ESTIMATE OF COST
FOR
RECONSTRUCTION OF TOWNLINE RD.
INCLUDING DRAINAGE IMPROVEMENTS

ITEM	ESTIMATED COST
Pipe - 18" RCP	$ 23,000.
24" RCP	25,000.
36" RCP	104,000.
Catch Basins	26,000.
Manholes	9,000.
Concrete Headwall	6,000.
Rip-rap	1,000.
Traffic Maintenance & Protection	22,000.
Concrete Curb	56,000.
Asphalt Concrete (Wearing Base Course)	300,000.
Grading	14,000.
Subtotal Construction Cost	$ 586,000.
Development Cost	117,000.
Contingency Cost	117,000.
TOTAL COST	$ 820,000.

FIGURE 50

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



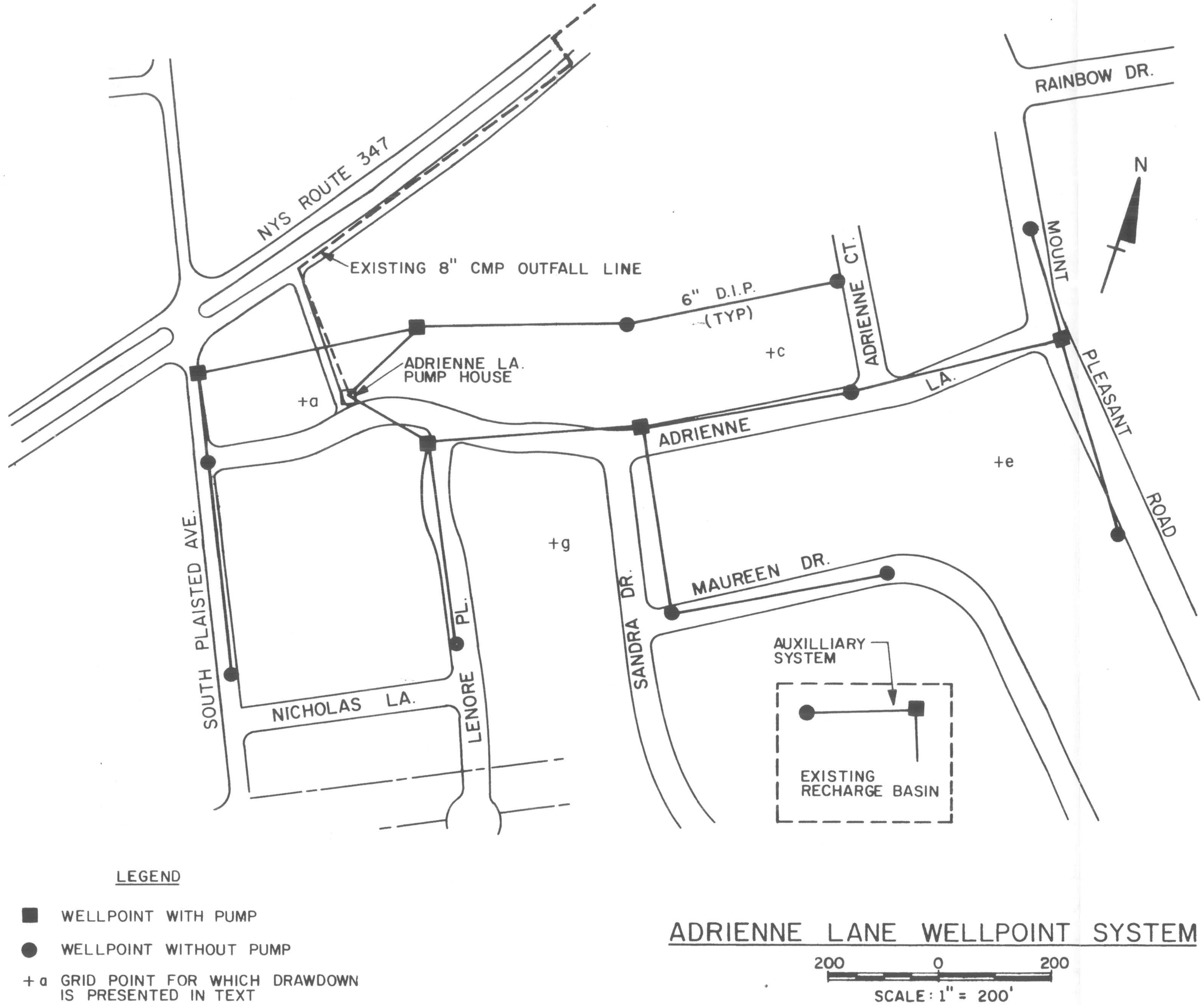
SENSITIVITY ANALYSIS

(FOR SENSITIVITY OF DRAWDOWN TO T/S RATIO)

HOLZMACHER, McLENDON & MURRELL, P.C.
WILLIAM S MATSUNAYE, JR, PE
MELVILLE, N.Y. (516) 752-9060
MEDFORD, N.Y. (516) 475-1018

FIGURE 53

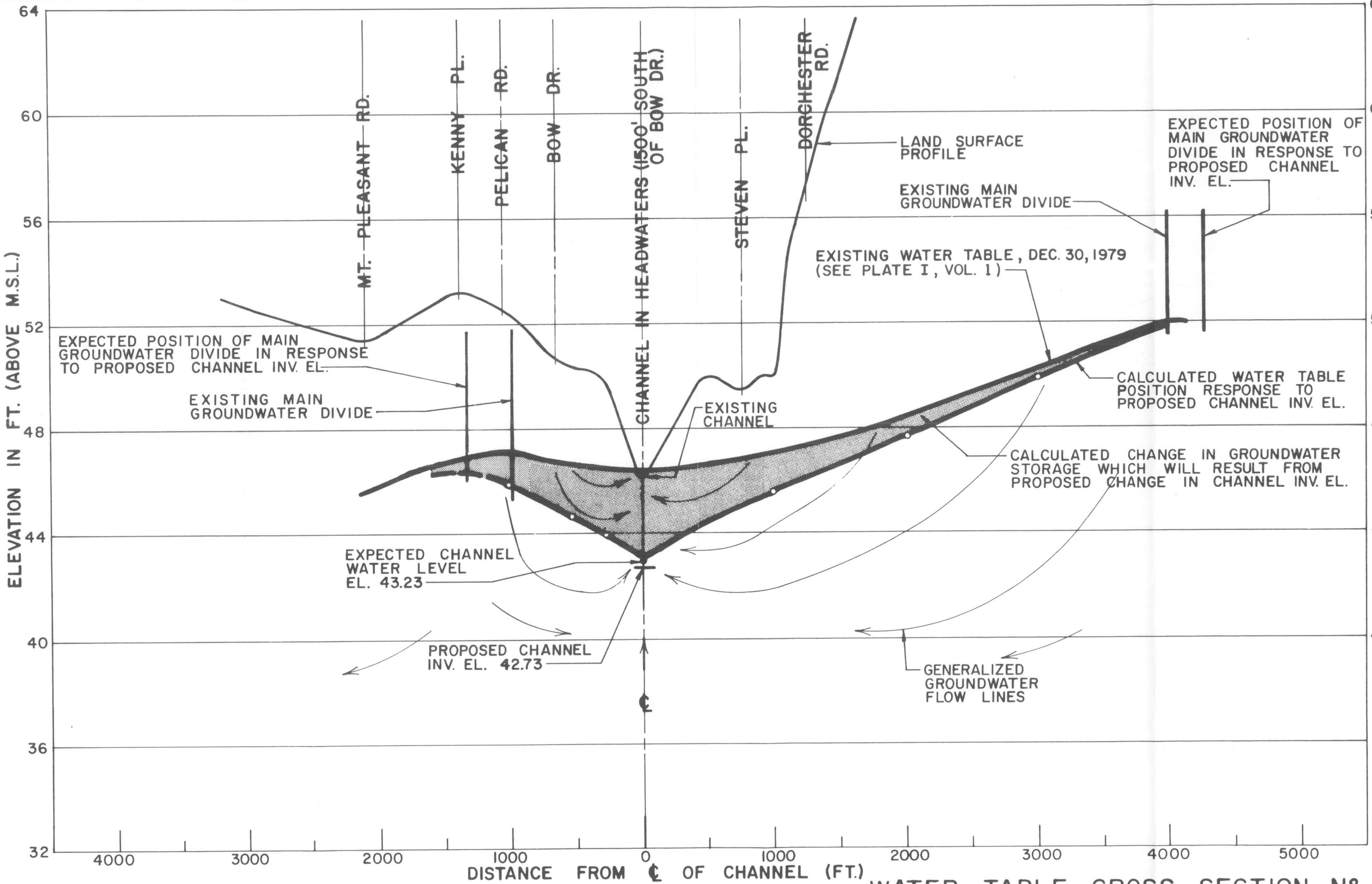
SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



ADRIENNE LANE WELLPOINT SYSTEM

HOLZMACHER, McLENDON & MURRELL, P.C. MELVILLE, N.Y. (516) 752-9060
WILLIAM S MATSUNAYE JR PE MEDFORD N.Y. (516) 475 1018

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

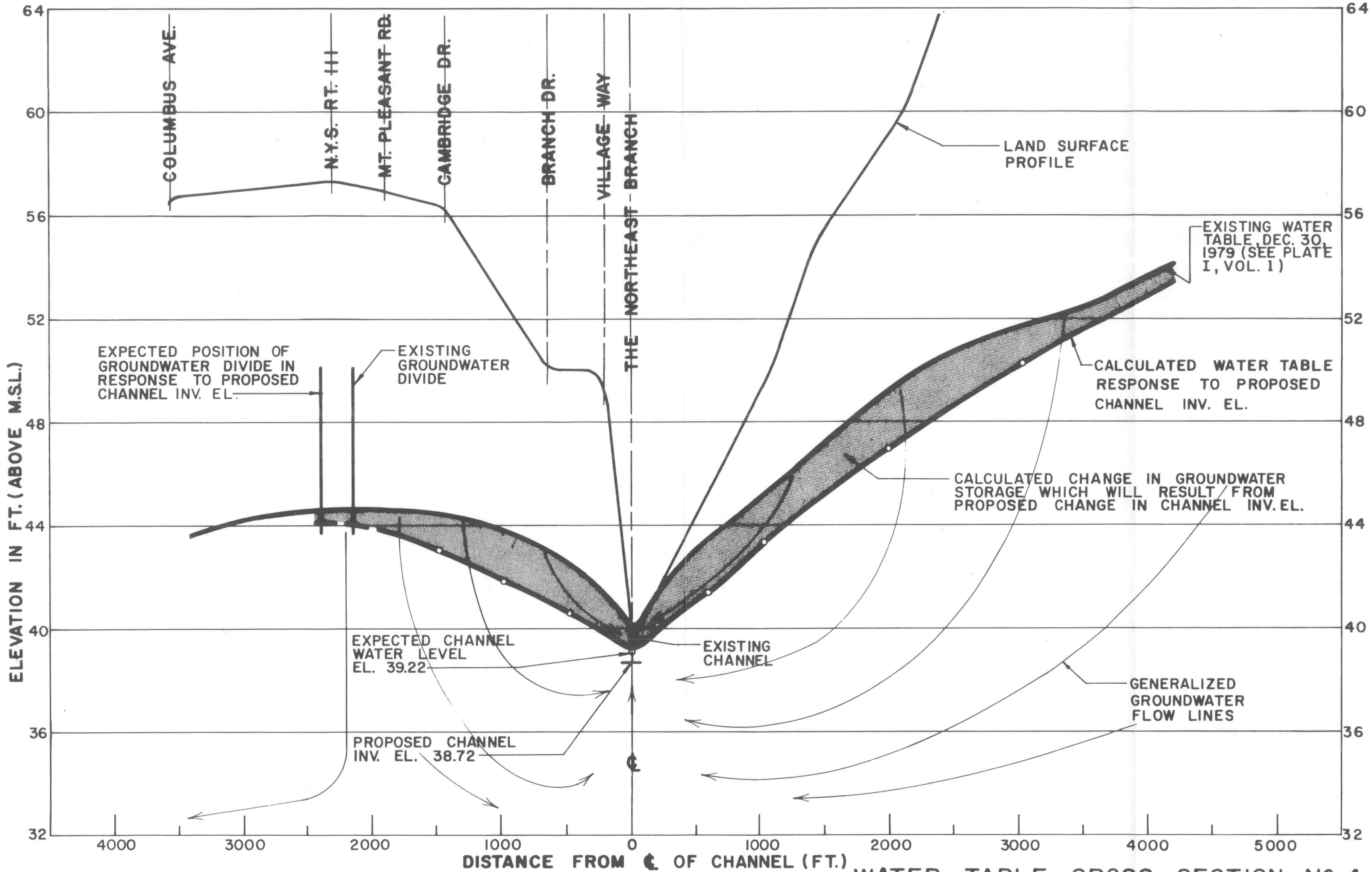


WATER TABLE CROSS SECTION Nº 1

CHANGES IN GROUNDWATER ELEVATIONS
DUE TO LOWERING OF THE CHANNEL OF THE NORTHEAST BRANCH

HOLZMACHER, McLENDON & MURRELL, P.C. MELVILLE, N.Y. (516) 752-9060
WILLIAM S MATSUNAYE JR PE MEDFORD NY (516) 475-1018

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



WATER TABLE CROSS SECTION Nº 4

CHANGES IN GROUNDWATER ELEVATIONS

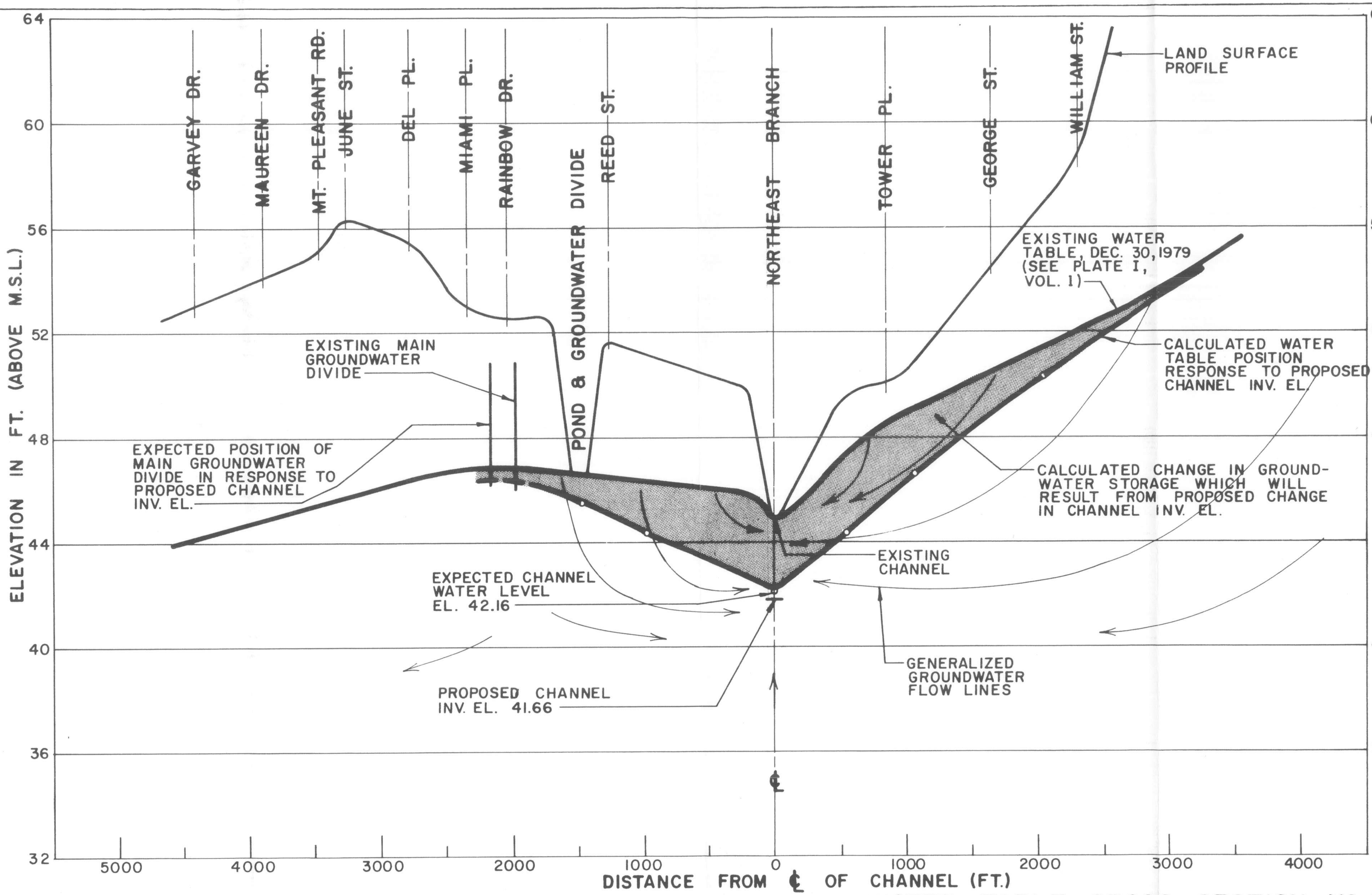
DUE TO LOWERING OF THE CHANNEL OF THE NORTHEAST BRANCH

HOLZMACHER, McLENDON & MURRELL, P.C. — MELVILLE, N.Y. (516) 752-9060 ■

WILLIAM S MATSUNAYE JR PE — MEDFORD N.Y. (516) 475-1018 □

FIGURE 57

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

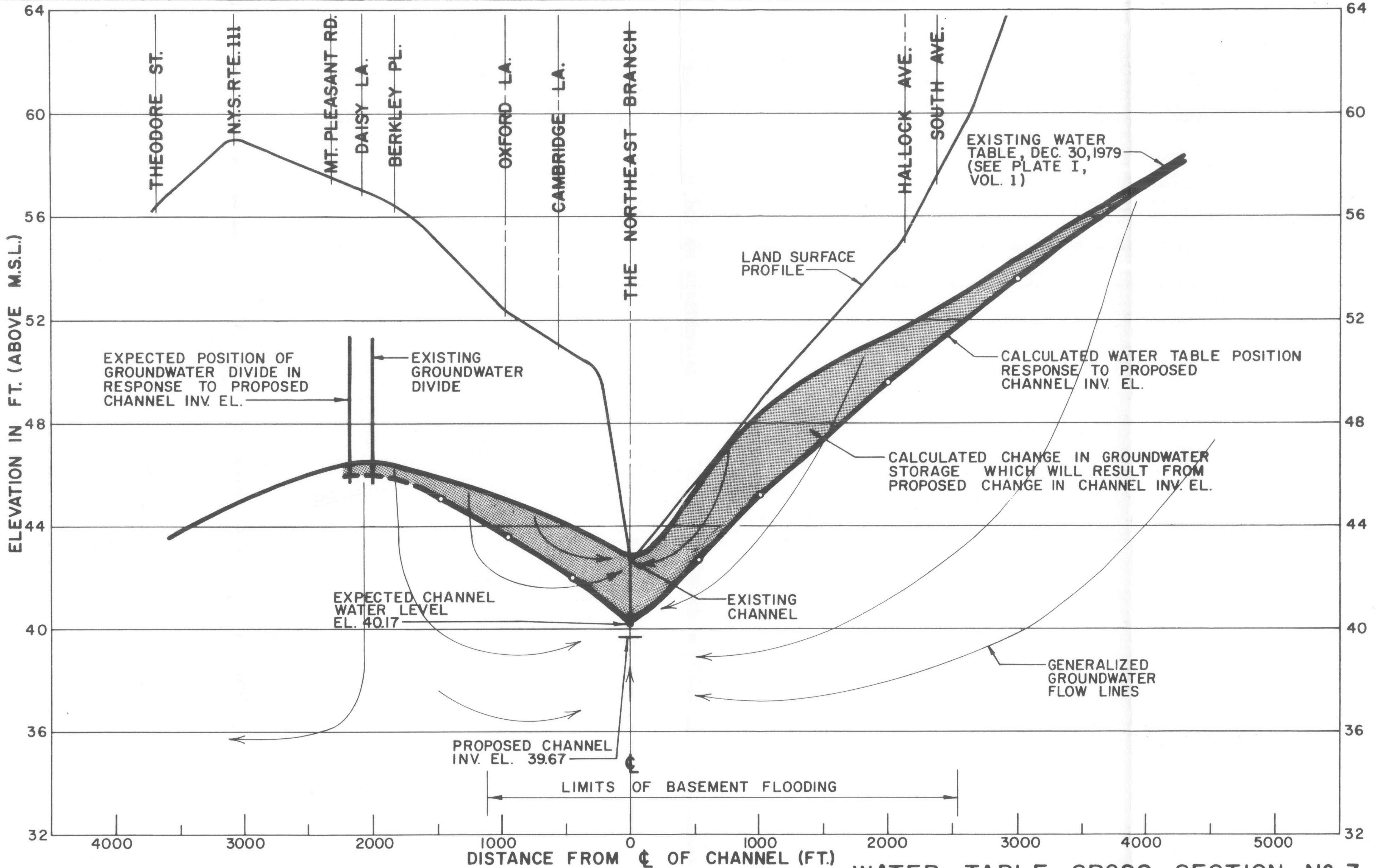


WATER TABLE CROSS SECTION Nº 2

CHANGES IN GROUNDWATER ELEVATIONS
DUE TO LOWERING OF THE CHANNEL OF THE NORTHEAST BRANCH

HOLZMACHER, McLENDON & MURRELL, P.C. MELVILLE, N.Y. (516) 752-9060
WILLIAM S MATSUNAYE JR PE MEDFORD NY (516) 475-1018

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



WATER TABLE CROSS SECTION № 3

CHANGES IN GROUNDWATER ELEVATIONS
DUE TO LOWERING OF THE CHANNEL OF THE NORTHEAST BRANCH

HOLZMACHER, McLENDON & MURRELL, P.C.
WILLIAM S MATSUNAYE JR PE
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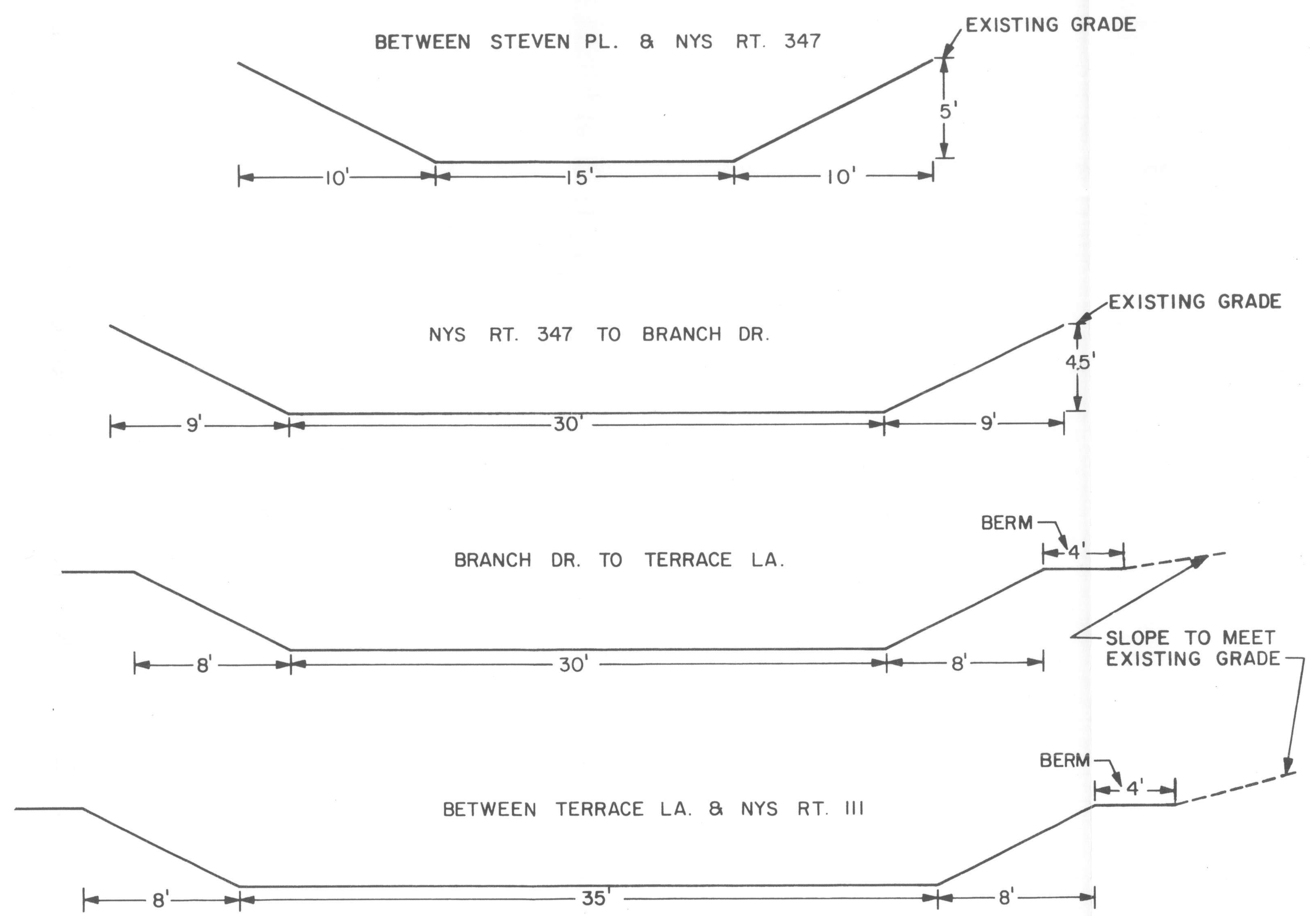
MEDFORD N.Y. (516) 475-1018

FIGURE 61

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

PROPOSED TYPICAL CHANNEL CROSS-SECTIONS ALONG THE NORTHEAST BRANCH



HORIZ. 1" = 6'
SCALE VERT. 1" = 6'

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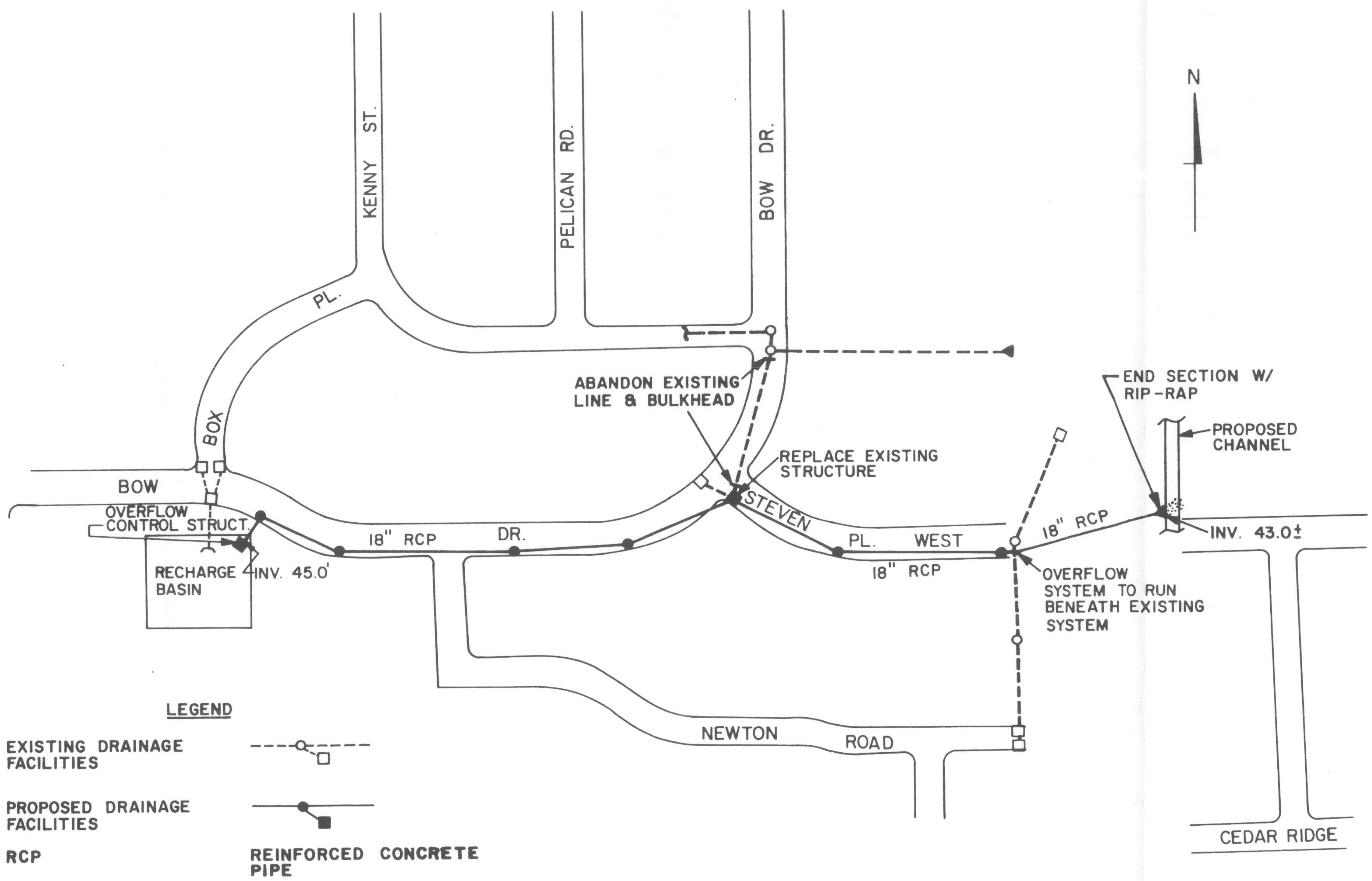
FIGURE 64

SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF

OVERFLOW RELIEF FOR BOW DR. RECHARGE BASIN



SCALE: 1" = 200'

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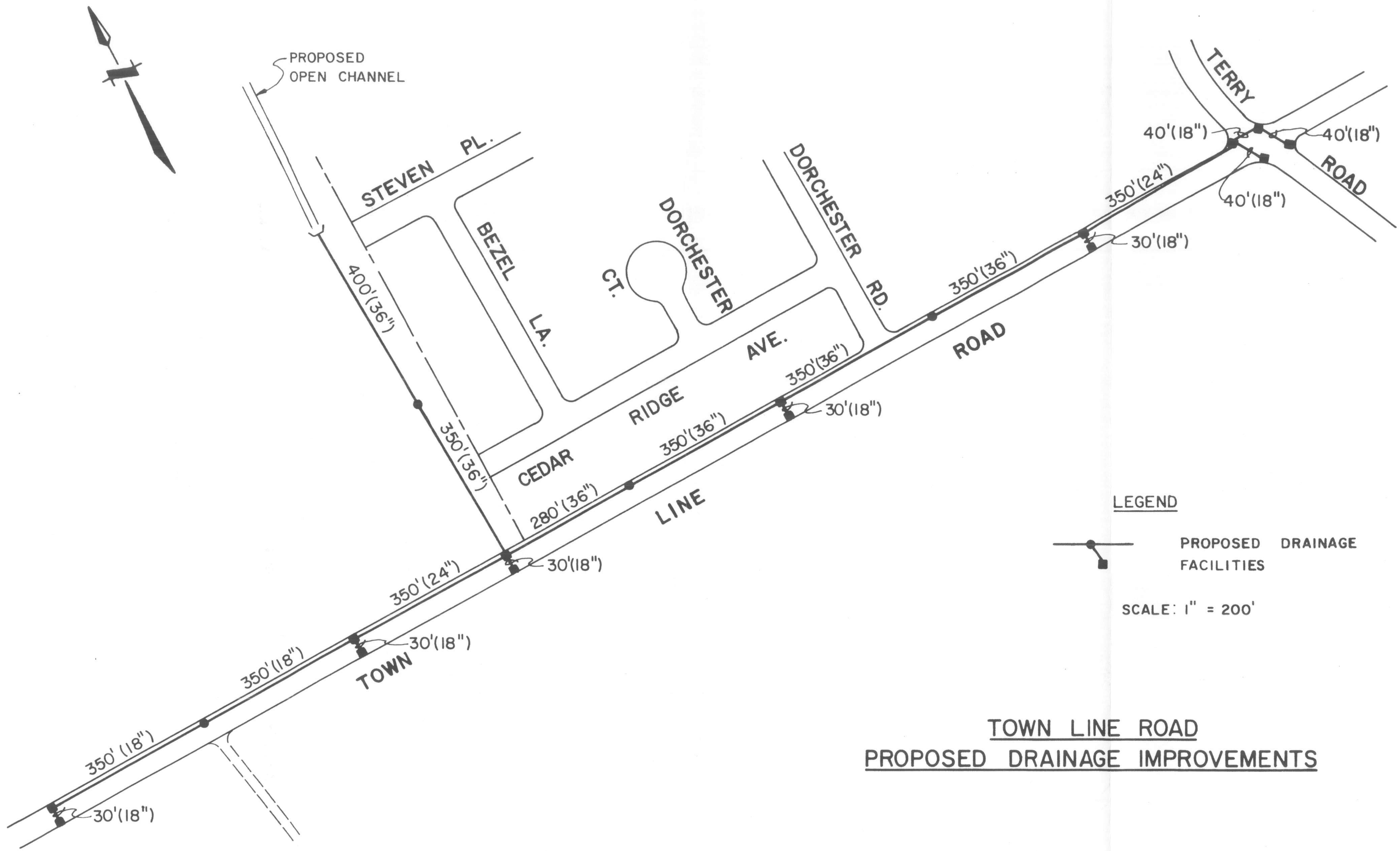
SUFFOLK COUNTY DEPT. OF PUBLIC WORKS - CAPITAL PROJECT NO. 5013 - GROUNDWATER RELIEF



TOWN LINE ROAD
PROPOSED DRAINAGE IMPROVEMENTS

HOLZMACHER, McLENDON & MURRELL, P.C.
WILLIAM S MATSUNAYE JR PE

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