Stormwater Management Detention Pond Design Within Floodplain Areas

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ABSTRACT

A unique approach to stormwater management for projects requiring mitigation of additional runoff caused by increases in paved surface areas is presented in this paper. Based on a design project developed for the General Foods Corporate Headquarters site in Rye, New York, a stormwater detention pond has been implemented within the floodplain of an adjacent water course. Encroachment of construction activities within a floodplain required the development of a detention pond that was capable of controlling excess runoff from adjacent areas while providing continued floodplain storage volume capacity. This methodology minimized the impacts of flooding on adjacent properties and provided suitable land areas for development in accordance with the intended use of the property. Occurrence of peak flooding along the watercourse did not coincide with peak stormwater runoff conditions from the smaller adjacent drainage area. By utilizing flood hydrograph principles and analyses that were developed by the Soil Conservation Service, U.S. Department of Agriculture, it was possible to develop a detention pond to provide a stormwater management phase and a flood control phase. Computerized analyses were compared for pre- and postdevelopment conditions using stormwater runoff and flood flow data on the basis of storms with return period frequencies of 10, 25, 50, and 100 years. By providing inlet pipes and outlet structures to control detention pond storage, peak flows from the pond to the watercourse and peak flood flows on the watercourse were reduced. The detention pond provides an aesthetic and effective method of mitigating flooding impacts that might have resulted from site development.

Continuing growth in urban areas coupled with increasing land values and decreasing availability of suitable development sites often causes federal, state, and local governments and private property owners to seek unique and innovative ways to pursue development. The use of detention ponds as a means of stormwater management and runoff control has become a widely accepted method for controlling stormwater runoff from highways, roadways, and new land use developments.

Construction of a detention pond within a floodplain provides a method for controlling adjacent site runoff while providing continued floodplain storage volume capacity. As a result, substantial benefit can be derived by adjacent property owners upstream and downstream of a development site by using such a detention pond to attenuate peak flood flows on the watercourse.

This approach to stormwater management is applicable to all projects that require mitigation of impacts that result from additional runoff caused by increases in paved surface areas. Transportation facilities, including new highways and interchanges; roadway widening and realignments; airport expansions; and construction of structures and parking facilities for intermodal transfer, maintenance, storage, and related facilities can all benefit from improved and alternative methods of runoff control and stormwater management.

The term "detention pond," as used in this discussion, refers to a man-made depression that will retain water year-round and provide for temporary storage and controlled discharge of excess water through an outlet structure. When a detention pond is to be constructed within the floodplain of a watercourse, additional measures must be developed to maintain the existing flood storage capacity of the floodplain while permitting increased storage capacity for detention and control of excess stormwater runoff from adjacent areas.

The General Foods Corporation has successfully implemented such a development program for their corporate headquarters building and surrounding access roads located in Rye, New York. Completion of this project required development of a stormwater detention pond approximately 6.3 acres in surface area, primarily located within the 100-yr floodplain of Blind Brook. Figure 1 shows the extent of encroachment of the 100-yr floodplain on the General Foods headquarters site before development. Design and construction of the detention pond had to be performed using the following criteria:

1. Control of postdevelopment rates of stormwater runoff tributary to the pond at or below predevelopment peak discharge rates to Blind Brook during the 10-, 25-, 50-, and 100-yr return period storms; and
2. Replacement of all floodplain storage volume that has been removed because of construction of buildings, embankments (berms), or other structures within the existing 100-yr floodplain limits.

To maintain preconstruction floodplain storage volume capacity within the site, it was necessary to divert a portion of the floodwaters from Blind Brook into the detention pond. Construction within the flood areas could not result in any measurable increase in water surface elevations along Blind Brook at any point upstream or downstream from the site at the Westchester Avenue box culvert and the Bowman Avenue Bridge.

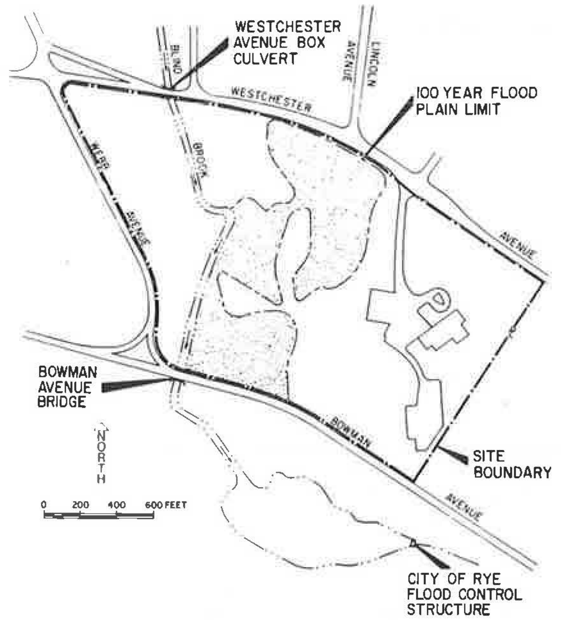

FIGURE 1 Predevelopment conditions.

Preliminary investigations indicated that the detention pond could be designed to provide a stormwater management phase and a flood control phase as an acceptable approach when flood hydrograph principles and analyses were considered. The goal of the first phase is to limit peak site runoff that results from increased impervious areas and to handle off-site runoff generated from tributary areas adjacent to the site. The detention pond location, shown in Figure 2, has a total drainage area of 89.2 acres. Because site flooding from Blind Brook resulted from a considerably larger drainage area of approximately 4,060 acres, hydrologic principles dictate that peak stormwater runoff conditions will not occur simultaneously. Peak runoff discharge from the immediate drainage area will occur before peak flood conditions within the Blind Brook floodplain.

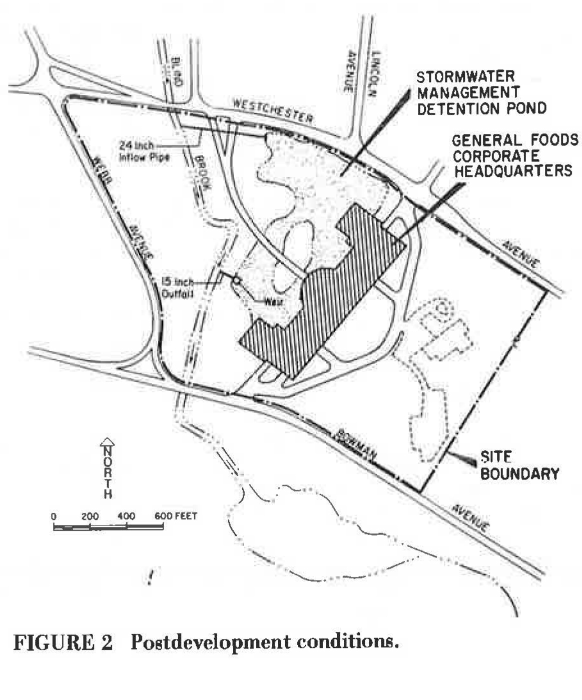

FIGURE 2 Postdevelopment conditions.

During the second phase of detention pond operation, partial diversion of flood water from Blind Brook to the pond is accomplished through an appropriately designed inflow pipe. The amount of flood water diversion from Blind Brook to the pond is based on the floodplain storage volumes that are lost as a result of the encroachment of development. Although these two phases can be considered essentially distinct, they must occur simultaneously during a design storm event.

The design approach is to provide a detention pond with an outlet structure that is sufficient to substantially reduce flood flows that result from the small watershed immediately adjacent to the site, and simultaneously to attain an established pond water surface elevation that corresponds to the storage volume to be returned to the floodplain. In this way, as inundation of the detention pond by Blind Brook floodwater occurs, sufficient detention pond storage volume is also readily available to meet the previously outlined criteria.

Verification of the feasibility of the foregoing approach was performed using conservative empirical approaches as outlined by the Soil Conservation Service (SCS), U.S. Department of Agriculture (1-3). These design approaches have their bases in the unit hydrograph theory. Flood routing (4) was extremely useful in providing reasonable estimates of expected peak flows and their time of occurrence. In addition, these methods provided good approximation of retardant pool storage, which was required within the detention pond, as well as the approximate size of the required outlet structure.

EXISTING AND FUTURE FLOOD ELEVATIONS

Water surface profiles for Blind Brook for pre- and postdevelopment conditions were prepared by using the Hydrologic Engineering Center (HEC-2) model (5). Input to the HEC-2 computer program consists of cross sections of typical channel segments, other parameters describing flood conditions such as channel roughness and expansion and contraction coefficients, bridge and culvert data, and peak flow data.

The analyses utilized cross-sectional data obtained from the Federal Insurance Administration (FIA) (6) study performed for the town of Rye in 1978. Cross sections of the channel within the site were determined from topographic contour mapping of the site to more accurately reflect channel conditions and more fully describe flood zone areas.

Input parameters generated by the FIA study to describe the Westchester Avenue culverts were utilized. The city of Rye flood control structure was described in data obtained through field surveys. To preserve the accuracy of the calibrated FIA model, the channel roughness and contraction and expansion coefficients determined by the FIA were utilized in the computer runs performed for this project.

Flood flows for the 10-, 50-, and 100-yr events used by the FIA were also utilized in this study. These peak flows are based on SCS flood routing for Blind Brook and were adjusted according to existing stream gauge information and frequency analyses. Starting water surface elevations downstream of the structure were also taken from the FIA study.

The 25-yr flood peak was approximated using a storm frequency (semilogarithmic) plot as shown in Figure 3. The initial elevation for the 25-yr condition was established using a semilogarithmic plot as shown in Figure 4.

Future conditions were modeled by simulating building encroachments on the existing stream channel cross sections and modifying the cross sections to allow for cut and fill. Flow was restricted to

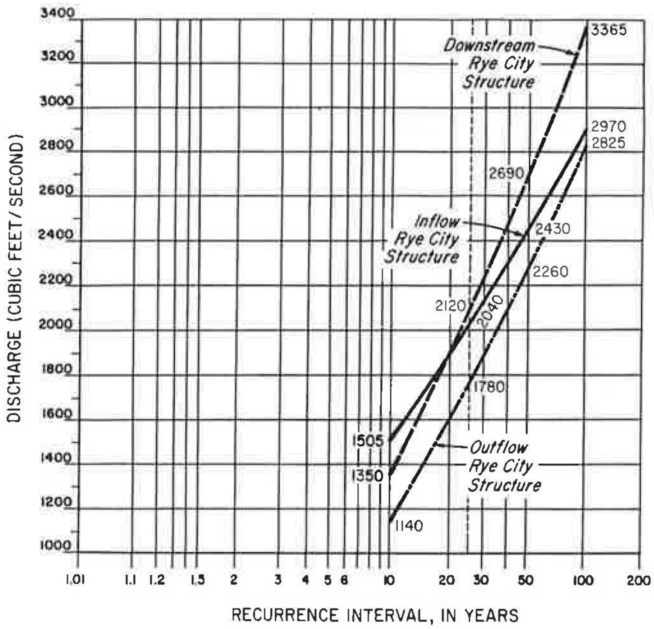

FIGURE 3 Blind Brook flood discharge.

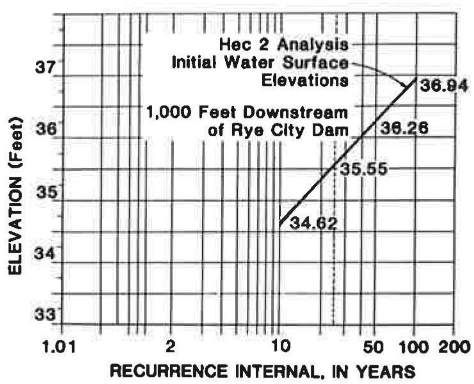

FIGURE 4 Blind Brook flood water surface elevations.

the channel section area outside the pond berm. Improvements in channel conveyance due to grading, grassing, and brush removal were reflected in the HEC-2 input by reduction of the channel roughness coefficient for improved overbank area.

Results of the HEC-2 analyses are summarized in Tables 1 and 2 for pre- and postdevelopment conditions, respectively. These results indicate that no increase in water surface elevations would occur upstream or downstream because of construction of the pond berm or the building.

These analyses verify subcritical flow conditions for this section of Blind Brook. Under subcritical flow, changes in channel conditions at any given location have no hydraulic effect on downstream water surface elevations. Because the detention pond will serve to substantially reduce discharge to Blind Brook and thereby reduce peak flows, conditions downstream of Bowman Avenue were projected to improve slightly after construction of the pond.

ENCROACHMENT VOLUMES

To determine storage volume requirements for the detention pond, several issues had to be considered.

TABLE 1 Blind Brook Water Surface Elevations: Predevelopment Conditions

	Elevation (ft)[a] by Storm Frequency (yr)			
Cross Section	100	50	25	10
1,000 ft downstream of Rye City Dam	36.94	36.26	35.55	34.62
Immediately downstream of Rye City Dam	48.65	47.81	46.74	45.02
Rye City Dam	61.80	61.17	60.54	59.51
120 ft upstream of Rye City Dam	61.96	61.28	60.56	59.46
Bowman Avenue	63.12	62.34	61.70	60.05
225 ft upstream of Bowman Avenue	63.49	62.74	62.12	60.97
1,050 ft upstream of Bowman Avenue	66.84	66.25	65.40	64.79
170 ft downstream of Westchester Avenue	69.26	68.71	68.27	67.50
Westchester Avenue	72.02	71.92	71.52	69.36
1,000 ft upstream of Westchester Avenue	78.96	78.59	77.02	76.00

[a]At mean sea level.

TABLE 2 Blind Brook Water Surface Elevations: Postdevelopment Conditions

	Elevation (ft)[a] by Storm Frequency (yr)			
Cross Section	100	50	25	10
1,000 ft downstream of Rye City Dam	36.94	36.26	35.55	34.62
Immediately downstream of Rye City Dam	48.65	47.81	46.74	45.02
Rye City Dam	61.80	61.17	60.54	59.51
120 ft upstream of Rye City Dam	61.97	61.28	60.56	59.46
Bowman Avenue	63.12	62.34	61.70	60.05
225 ft upstream of Bowman Avenue	63.46	62.71	62.10	60.95
1,050 ft upstream of Bowman Avenue	66.80	66.28	65.54	64.78
170 ft downstream of Westchester Avenue	69.26	68.70	68.24	67.47
Westchester Avenue	72.02	71.91	71.52	69.36
1,000 ft upstream of Westchester Avenue	78.76	78.59	77.02	76.00

[a]At mean sea level.

Pond storage volumes consisted of predevelopment flood plain volumes in overbank areas adjacent to the Blind Brook channel, floodplain volumes displaced by fill material, and additional volumes required because of increased runoff from additional impervious areas within the site. These required volumes are shown in Figure 5.

Volumes of floodplain storage capacity that are affected by design conditions were determined by using average end area calculations for the portion directly affected by proposed construction. These affected volumes included the region confined by the outside toe of the slope of the pond berm and the outside wall of the proposed building. On the basis of predevelopment conditions, floodplain storage volumes were affected by the office building and detention pond construction. A portion of these floodplain volumes was lost directly as a result of construction of earthen embankments and the building. Provisions for returning this volume back to the floodplain were made through excavation within the limits of the pond area. The affected floodplain and encroachment volumes are given in Table 3.

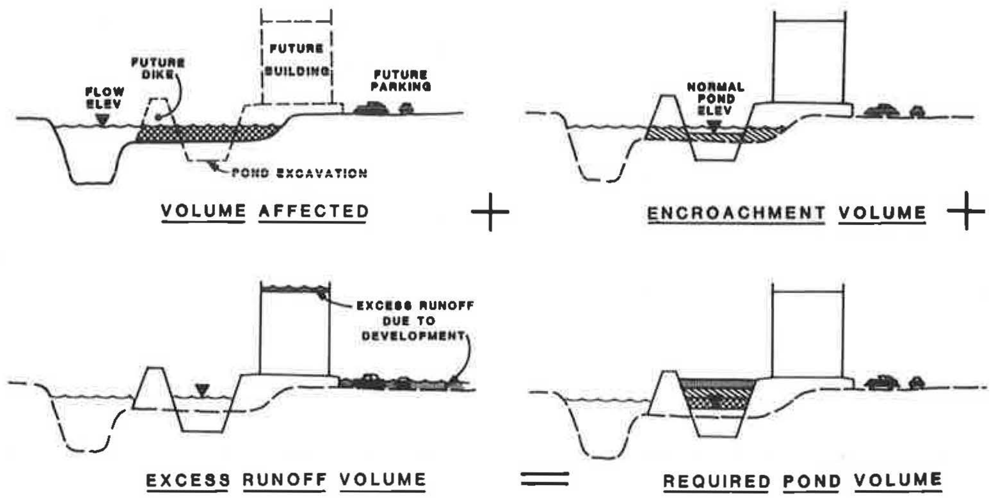

FIGURE 5 Detention pond storage volume.

TABLE 3 Affected Floodplain and Encroachment Volumes

Storm or Flood Frequency (yr)	Affected Flood-plain Volume (acre-ft)	Encroachment Volume (acre-ft)
10	10.6	1.4
25	24.0	3.0
50	25.9	4.0
100	26.8	5.6

In addition to flood storage volumes lost because of construction, there was a net increase in runoff volume caused by the increased impervious surface area on the site, which includes parking, road, roof, and pond. Provisions had to be made for storing this excess volume within the pond as well. Calculations of the estimated runoff volume were made with SCS curve numbers (CN) that are based on soil types and land use (see Table 4).

For estimating runoff volumes, the following equation was used to determine the depth of water runoff over the drainage area:

$$Q = (P - 0.2S)^2/(P + 0.8S) \qquad (1)$$

where

Q = depth of water discharge (in.),
P = precipitation (in.), and
S = (1,000/CN) - 10.

By using the above equation, the excess runoff volumes for 24-hr duration rainfall amounts over the 37-acre drainage area were estimated and these data are given in Table 5.

Combining flood storage volume losses that result from encroachment and additional runoff caused by development yields storage that must be effectively returned to the floodplain during the design floods. The required pond storage volumes necessary to ensure sufficient runoff detention are given in Table 6.

FLOOD ROUTING

Runoff hydrographs were generated for pre- and post-development site conditions that involve the use of methods outlined by the SCS (1,4). Runoff hydro-

TABLE 4 Retention Structure, Land Use, and Determination of Runoff Curve Number (RCN)

Conditions	Land Use (%)	RCN
Existing		
Undeveloped site (forest and floodplain)	24.2	60
Commercial	7.7	92
Residential (¼-acre lots)	68	75
Weighted curve number		72.6
Rounded weighted curve number		73
Future[a]		
Mixed commercial	24.2	83
Commercial	7.7	92
Residential (¼-acre lots)	68	75
Weighted curve number		78.3
Rounded weighted curve number		78

Note: area = 89.2 acres and soil class B is assumed.
[a]The calculated RCN for development is RCN = [0.64 (98)]/[0.36 (57)] = 83.

TABLE 5 Excess Runoff Volumes from 10-, 25-, 50-, and 100-yr Storms or Floods

Parameter	Storm or Flood Recurrence Frequency (yr) 100	50	25	10
Precipitation, P (in.)	7.2	6.5	5.8	5.0
Predevelopment Curve Number	60.0	60.0	60.0	60.0
Predevelopment Runoff, Q_E (in.)	2.8	2.3	1.8	1.3
Postdevelopment Curve Number	83.0	83.0	83.0	83.0
Postdevelopment Runoff, Q_F (in.)	5.2	4.6	3.9	3.2
Area, A (acres)	37.0	37.0	37.0	37.0
Excess Runoff Volume (acre-ft), $[(Q_F - Q_E)/12]$ A	7.4	7.1	6.5	5.9

TABLE 6 Storm Recurrence Frequency and Volumes

Frequency Occurrence (yr)	(1) Existing Volume (acre-ft)	(2) Total Encroachment Volume (acre-ft)	(1) + (2) Required Pond Volume (acre-ft)
100	26.8	13.0	39.8
50	25.9	11.1	37.0
25	24.0	9.5	33.5
10	10.6	7.3	18.0

graphs provide the engineer with a model of any flood of given duration and return period. The inflow hydrograph is used to perform flood routing through the pond outlet works and spillways and simultaneously allow for storage of inflow runoff volume.

Runoff curve numbers for pre- and postdevelopment conditions were determined on the basis of land use. Inflow hydrographs for 24-hr duration storms were generated using SCS computer program (7). Results of these calculations and computer analyses are given in Table 7.

TABLE 7 Pre- and Postdevelopment Peak Flows

Return Period (yr)	Predevelopment Peak Discharge (ft^3/sec)	Predevelopment Time to Peak (hr)	Postdevelopment Peak Discharge (ft^3/sec)	Postdevelopment Time to Peak (hr)
10	54	10.5	79	10.3
25	72	10.5	101	10.3
50	87	10.5	120	10.3
100	104	10.5	140	10.3

Detention pond design includes a 24-in. diameter reinforced-concrete inflow pipe, which allows water to flow into the pond, thus providing water recirculation through the pond during normal conditions. During flooding conditions, this pipe removes storm flow from Blind Brook, stores these volumes in the pond, and thereby provides the required storage volume that was calculated earlier.

Outlet works consist of a reinforced-concrete box riser with a 15-in.-diameter reinforced-concrete pipe conduit that discharges water to Blind Brook. The main function of the outlet structure is to maintain normal pond water surface elevation (60.0 ft). During flooding, the outlet structure balances discharge so that the inflows (from Blind Brook and direct runoff) are detained, which allows for required storage within the pond.

An emergency spillway weir was provided to control discharge to and from the pond during extremely severe flood events. This weir should only function for storms of approximate return periods of 100 yr or greater.

Storage and net discharges from the pond during a storm become a function of adjacent area runoff and variable pond and Blind Brook water surface elevations. To route the floods, it was necessary to solve for pond water surface elevation using an iterative approach for successive time intervals. An algorithm was developed to determine storage-discharge values for a given set of conditions along Blind Brook, a given discharge hydrograph to the pond.

The basis for numerical solution to the routing problem is the following basic volume-balance relationship for inflow, outflow, and storage:

$$\bar{I}\Delta t - \Delta S = \bar{O}\Delta t \tag{2}$$

where

$\bar{I} = (I_1 + I_2)/2$ = average outflow rate,
$\bar{O} = (O_1 + O_2)/2$ = average outflow rate,
$\Delta S = S_2 - S_1$ = change in storage volume, and
$\Delta t = t_2 - t_1$ = routing period (T).

Subscripts 1 and 2 denote the beginning and end of the routing period. Restating and rearranging the basic equation yields the following routing formula:

$$(I_1 - O_1) + (2S_1/\Delta t) \mp (2S_2/\Delta t) + (O_2 - I_2) \tag{3}$$

Solution of this volume-balance equation requires that it be reduced to a function of a single dependent variable. In this case, Hp, the pond elevation at the end of each successive time interval, can be used to define inflow and outflow discharge rates as well as pond storage volumes. This was accomplished by making a simplifying pressure flow assumption with regard to flow at the 15-in. riser conduit and 24-in. inflow pipe and describing the proposed pond storage/elevation relationships using a linear approximation for storage-volume variation between specific elevation intervals. Inserting standard relationships into the equation yields the following working equations for the solution:

$$f(Hp) = \{(2m_i Hp/\Delta t) + (2y_i Hp/\Delta t) \pm [Kr(Hp - Hr)^{0.5} \pm (Hp - Hw)^{0.5} - I_2 - A\} \tag{4}$$

$$df(Hp)/dHp = (2m_i/\Delta t) + 0.5\,Kr\,(Hp - Hr)^{0.5} + 0.5Kp\,(Hp - Hw)^{-0.5} \tag{5}$$

where

Hp = pond elevation;
Hw = Blind Brook headwater elevation at inflow pipe;
Hr = Blind Brook tailwater elevation at outflow riser conduit;
I_2 = direct runoff discharge to pond at end of time interval;
$A = I_1 - O_1 + (2S_1/\Delta t)$, a constant, calculated for start of specific time interval;
$Kr = C_d Ar(2g)^{0.5}$, a constant conveyance factor for calculating flow through the rise conduit;
$Kp = C_d Ap(2g)^{0.5}$, a constant conveyance factor for calculating flow through the inflow pipe;
Δt = time interval;
m_i = linear slope constant for storage-elelevation curve;
y_i = intercept constant for storage-elevation curve;
C_d = coefficients of discharge specific for each pipe, as a function of length, diameter, pipe material, and direction of flow determined by using King and Brater Handbook of Hydraulics (8);
Ar, Ap = pipe areas; and
g = 32.2 ft/sec² gravity constant.

It should be noted that the first two terms of the right-hand side of Equation 4 establish storage volume. The third and fourth terms describer riser and inflow pipe hydraulics, respectively.

The signs of the second and third components of Equation 4 are shown as being variable because the relative elevation of the pond water surface (Hp), inflow pipe (Hw), and riser conduit (Hr) will change over the course of the design storm event. Although flow through the inflow pipe will be directed into the pond for a significant portion of the storm event, flow reversal will eventually occur as Blind Brook floodwater recedes below the elevation of the pond water surface. For certain portions of some events, flow reversal into the pond occurs at the riser location because Blind Brook flood elevations briefly exceed pond elevations there. For this reason, it was necessary to establish a basis for performing the routing under the full range of flow conditions that could occur during a particular event. The four different cases that are possible are presented in Table 8. Sign conventions are applied on the basis of the direction of discharge.

TABLE 8 Sign Conventions for Routing Solution

	Flow Conditions		Sign Convention	
Case	At 24-in. Inflow Pipe	At 15-in. Riser Conduit	Kp	Kr
1	Hw > Hp	Hr < Hp	–	+
2	Hw > Hp	Hr > Hp	–	–
3	Hw < Hp	Hr < Hp	+	+
4	Hw < Hp	Hr > Hp	+	–

Thus, under Cases 2 and 3, net discharge must be negative or positive, respectively, because Case 2 involves all flow going into the pond whereas Case 3 involves all flow going out of the pond. Under Cases 1 and 4, the sign of the net discharge is dependent on the relative magnitude of flow between the pond and Blind Brook at each of the structures.

For numerical solution of the routing formula, the basic formula, used by Newton's method to generate the next approximation of the zero root is

$$Hp_{i+1} = Hp_i - f(Hp_i)/f'(Hp_i) \quad (6)$$

Successful convergence is accomplished when the absolute value of the numerical difference between successive approximations of Hp is smaller than a specified value, E. For this design analysis, convergence occurred when E was set at 10^{-4}.

Stage versus time relationships for the outlet and inlet structure points along Blind Brook were prepared for each design storm. These values were determined using the HEC-2 computer program for various flow discharges. Stage-discharge curves generated for each structure location were then correlated with runoff hydrographs for Blind Brook (7).

Stage-discharge curves used in determining Blind Brook elevations are shown in Figure 6. Hydrographs

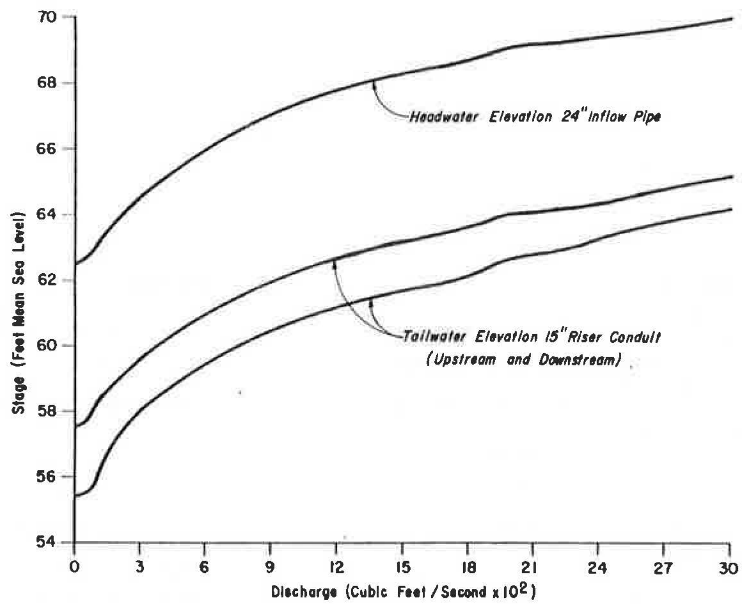

FIGURE 6 Blind Brook stage and discharge curves.

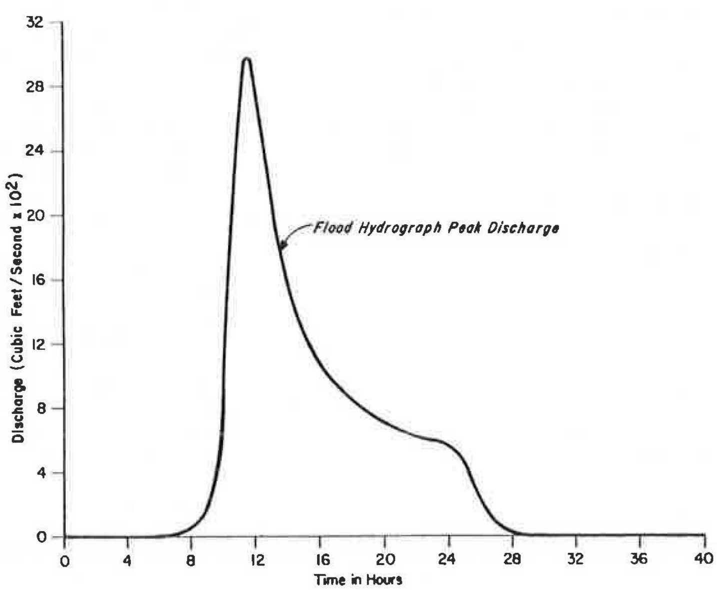

FIGURE 7 Blind Brook design hydrograph.

were used in developing Stage-time data used in flood routing, and the design hydrograph for the 100-yr storm in shown in Figure 7. The relationship of storage volume to water surface elevation for the pond, as used in the routing, is shown in Figure 8.

Flood routing data for the 100-yr storm event are given in Table 9, and are shown in Figure 9. Comparable flood routing data were also prepared for the 10-, 25-, and 50-yr storm events. Input for each time interval consisted of direct runoff discharged to the pond and the Blind Brook elevation at the inflow pipe and outlet conduit. Net discharges are expressed as the net flow through the combined system. Negative values indicate that inflow to the pond from Blind Brook exceeds loss from the pond through discharge. Generally, this reflects the condition when flow through the 24-in. diameter inflow pipe is greater than inflow passing out of the 15-in. diameter riser conduit. The case (1, 2, or 3) refers to the flow conditions described previously.

DISCUSSION OF RESULTS

The data that resulted from the routing indicated that construction of the detention pond would result in a dramatic mitigative effect of peak runoff from the proposed site. Peak flows would be reduced 77, 75, 71, and 66 percent for 100-, 50-, 25-, and 10-yr storms, respectively. Moreover, these peaks would occur at a point in time during the storm well after the time of peak flood occurrence in Blind Brook. At the expected time of peak, there is net negative discharge. In this manner, peak flow from the site would be further attenuated and discharge peaks greatly reduced.

Storage requirements have been met or exceeded for all conditions. During filling of the pond by diversion of flow from Blind Brook, peak flows in the stream are reduced, which further improves conditions downstream, in addition to the improvements already affected by site peak discharge attenuation.

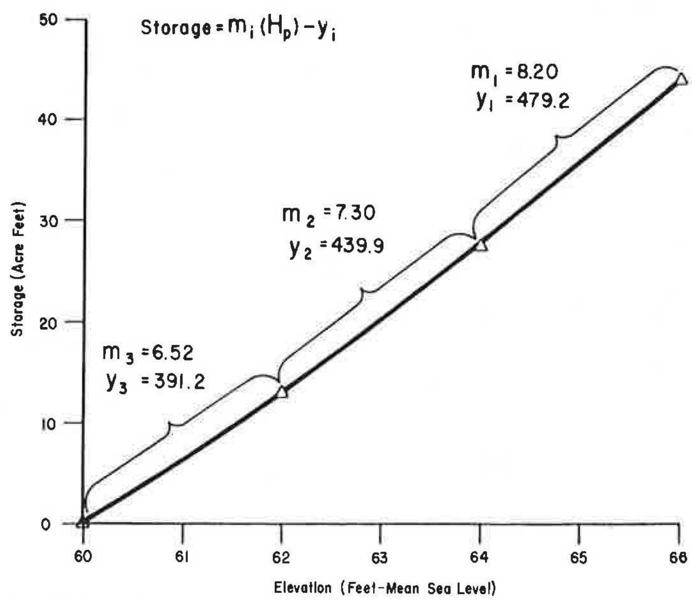

FIGURE 8 Detention pond storage versus elevation curve.

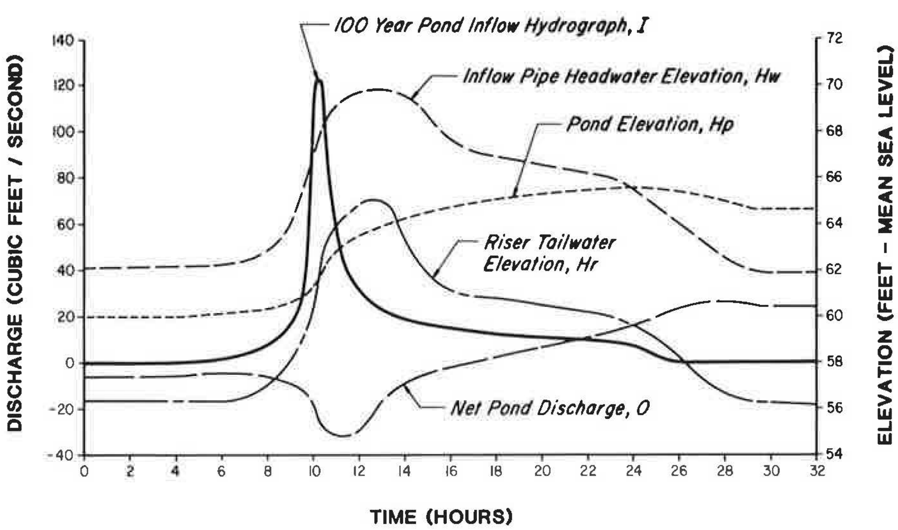

FIGURE 9 Detention pond flood routing—100-year storm.

TABLE 9 Detention Pond Flood Routing: 100-Yr Storm

Time (hr)	Runoff Discharge to Pond (ft^3/sec)	Headwater Elevation at Inflow Pipe (ft at mean sea level)	Tailwater Elevation at Riser Conduit (ft at mean sea level)	Case	Net Pond Discharge (ft^3/sec)	Pond Water Surface Elevation (ft at mean sea level)	Storage Volume (acre-ft)
4.3	1	62.1	56.5	1	-1.30	60.01	0.10
6.3	3	62.25	56.6	1	-1.53	60.10	0.65
8.3	12	62.9	57.3	1	-3.60	60.33	2.13
10.3	140	68.0	62.25	2	-26.43	61.86	12.13
11.3	46	69.0	64.2	2	-29.01	63.29	22.11
12.3	31	69.8	65.05	2	-28.09	64.01	27.66
14.3	21	68.2	62.35	1	-6.83	64.81	34.25
16.3	17	67.4	61.38	1	-0.83	65.27	38.01
18.3	14	66.8	60.90	1	3.44	65.55	40.31
20.3	12	66.5	60.55	1	6.28	65.72	41.67
22.3	11	66.1	60.25	1	8.88	65.69	41.46
24.3	8	65.4	59.70	3	18.77	65.55	40.32
26.3	0	64.0	57.75	3	26.82	65.14	36.94
28.3	0	62.2	56.5	3	31.71	64.53	31.93

The unique and innovative stormwater management system implemented for the General Foods site has effectively accomplished significant stormwater discharge and flood mitigation within all established policy and criteria. Since completion of this project in the spring of 1983, the detention pond is reported to have functioned well, in accordance with design parameters. General Foods has received correspondence from property owners downstream of their site, expressing appreciation for the observed mitigation of flooding along Blind Brook that resulted from the construction and operation of the detention pond.

Combining technology from established methods of stormwater management permitted development of a dynamic system to control increases in local stormwater runoff and provide additional flood control for a portion of the peak discharge on Blind Brook. In addition, the resultant detention pond is considered an aesthetically pleasing improvement to the site, which enhances the appearance of the General Foods corporate headquarters.

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