APPENDIX B

Section III
Of
" Elements of Urban Stormwater Design"

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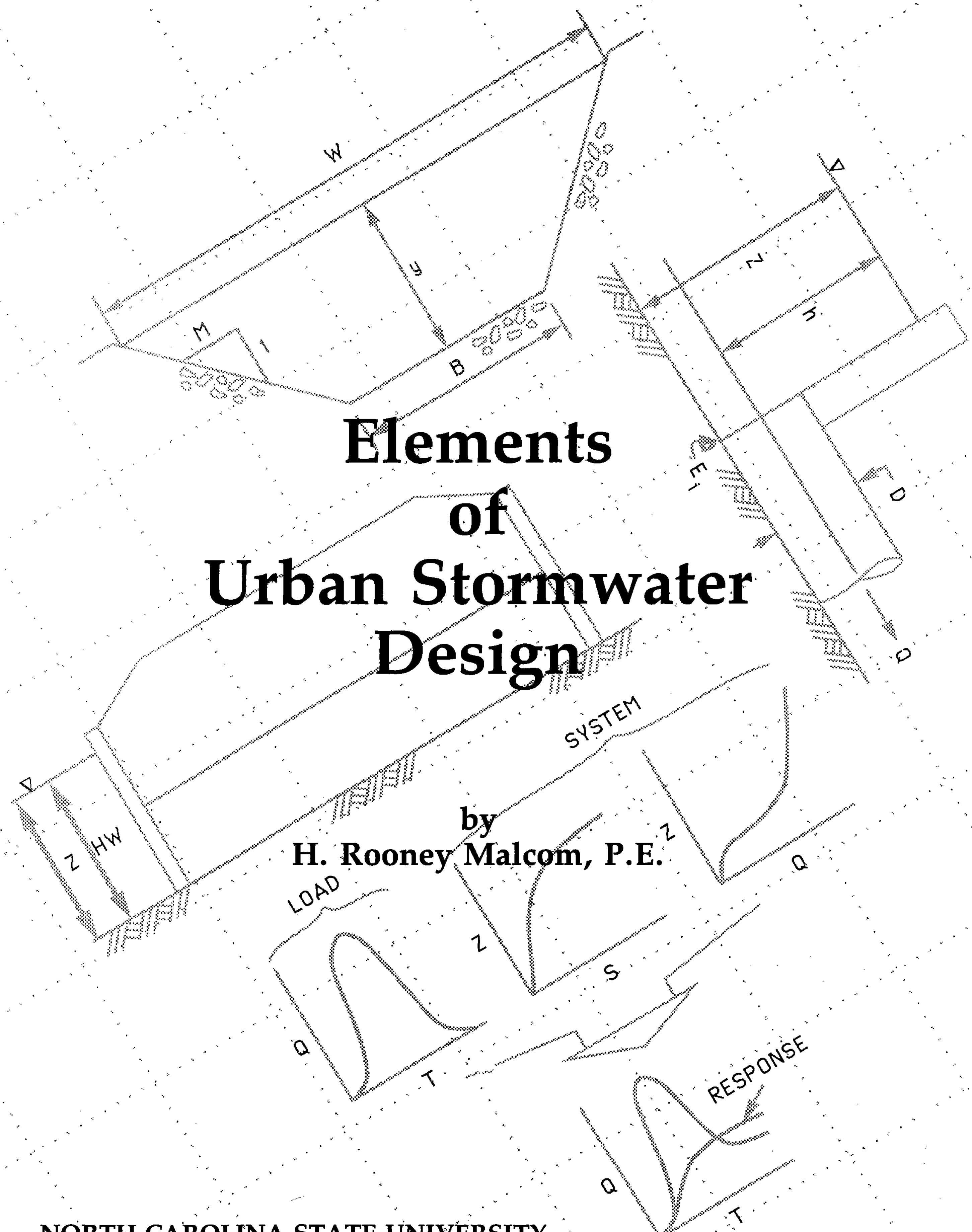

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SECTION III

STORMWATER IMPOUNDMENTS

GENERAL

Stormwater impoundments serve several purposes in urban watersheds. Impoundment types include flood-control reservoirs, stormwater detention ponds, aesthetic or recreational ponds, sediment-control basins, water-quality-control ponds, and even culverts. Multipurpose facilities are common, and there are many variations on the theme. Irrespective of purpose, impoundments are subjected to flood-wave loading. The elementary design of impoundments for the control or passage of flood waves is treated in this section.

The process of design and analysis of stormwater detention systems has emerged as a routine activity in stormwater management in recent years. Stormwater detention is essentially flood control at small scale. Flood water is stored temporarily in an impoundment and released such that the maximum release rate is reduced to some satisfactory level below that which would have been expected without the detention facility. The design objective in a typical stormwater detention regulation might require that the ten-year peak discharge from a site after development be no greater than the ten-year peak discharge from the same site prior to development. The design storm for detention varies from place to place, but the ten-year storm seems to predominate. Application of detention policy on a watershed basis requires careful consideration of whether to distribute control facilities widely at small scale or to develop larger scale flood-control reservoirs at strategic points in the stream system.

Facilities take several forms. The larger scale ponds usually involve placing flood storage above a normal pool. Smaller scale facilities may use normally dry ponds designed in conjunction with other uses: storage on parking lots, in parks, perhaps behind culverts. Rooftop storage, though sometimes used, seems to the author rarely justifiable.

The design process includes formulation of the inflow hydrograph, selection of the size and shape of the storage container, and selection of the type and size of the outlet device.

For an impoundment, there are virtually always two design storms to consider. One is the control storm for which the principal spillway is designed. The other is the emergency storm, a large storm for which an emergency spillway is designed to pass the excess discharge without overtopping the dam. In some facilities the principal spillway incorporates the emergency capacity.

The basis for analyzing detention facilities is the flood-routing algorithm. It derives from the continuity principle which states that, at every instant during the passage of a flood, the rate of change of storage in the reservoir is equal to the rate of inflow minus the rate of outflow. There are several procedures available for executing the routing algorithm. All involve a numerical solution of the differential equation:

$$\frac{ds}{dt} = I - O$$

where:

$\frac{ds}{dt}$ = Rate of change of storage with respect to time.

I = Rate of inflow.

O = Rate of outflow.

A detention basin or reservoir or impoundment consists of a storage volume and outlet devices which together cause the inflow hydrograph to be flattened as it comes through the basin. A hydrograph is a record (graph or table) of discharge versus time. In a flood, the discharge increases from some negligibly low rate to a peak, and then it falls away gradually to a small rate again. The area under the curve plotted as the hydrograph is equal to the volume of water constituting the flood. That same volume is preserved under the outflow hydrograph. In the usual detention impoundment, the volume of water in the flood is not reduced. The shape of the hydrograph is made longer and flatter such that the peak flow is satisfactorily reduced.

In view of this brief discussion, it may be inferred that all routing procedures, in order to model a reservoir, must involve three sets of source data on which to operate. One set represents the inflow hydrograph. Another represents the size and shape of the storage container. The third represents the hydraulics of the set of outlet devices. In the discussion below, the formulation of the source data is first treated, followed by a discussion of routing procedures. Design is discussed following analysis.

MATHEMATICAL MODELS

The principal mathematical models used in the design of stormwater impoundments are:

1. Hydrograph formulation procedures -- Several options exist for modeling the storm hydrograph representing the loading of the impoundment.

2. Weir equations -- used for modeling important elements of outlet devices that behave as weirs.

3. Orifice equation -- used for modeling elements of outlet devices judged to behave as orifices.

4. Energy balance -- used to model outlet behavior in systems having high tailwater.

5. Continuity principle -- the guiding principle for modeling the passage of a flood wave through a reservoir.

SOURCE DATA

Inflow Hydrograph

The formulation of the inflow hydrograph is the most difficult of the tasks comprising flood routing. Hydrograph formulation procedures involve a great deal of uncertainty. The objective is to obtain the hydrograph for say the ten-year storm at a given point of interest. But one can show, for example, that the ten-year storm is not a unique event and that there is a spectrum of storms that would qualify under statistical analysis as a ten-year storm. A ten-year storm can be one that is brief, producing a steep hydrograph of small volume, or it may be a storm of long duration, producing the same peak in a long hydrograph of large volume. If these storms have peaks equal to the discharge associated with the ten-year flood, they both qualify as such, but if one routes each of them through a given impoundment, the outflow hydrographs produced will be quite different. In the face of such uncertainty, hydrograph formulation procedures necessarily include arbitrary judgments, and the various procedures make different arbitrary judgments. So, adoption of a method becomes a matter of local consensus. Methods that prevail in one location find no favor in another.

Alternate Hydrograph Formulation Methods

Methods exist in large variety. Perhaps the most detailed are the computerized watershed models such as HEC1 of the Corps of Engineers and TR-20 of the Soil Conservation Service. For smaller watersheds, the methods of TR-55 are useful (SCS, 1986). Desktop methods, such as unit-hydrograph synthesis, are described in most hydrology texts. Many of these require a heavy investment of time and effort in field data gathering and data set preparation. Such precision may not be justified for small facilities and early feasibility studies.

Small-Watershed Hydrograph-Formulation Method

The author has proposed a method for use in routine design of small systems and for feasibility studies and site selection studies of larger watersheds (Malcom, et al, 1986). A variation of the method was adopted for use in design of facilities in the Houston area (Harris County, TX, 1984).

The method is based upon the observation that there are three important aspects of the hydrograph on which the design will depend. These are the peak discharge, the volume of water under the hydrograph, and the shape of the hydrograph. Separate decisions may be made regarding these, and the hydrograph will be determined. The necessary decisions, and the author's suggestions are:

1. Accept as a pattern function a step-function approximation to the SCS dimensionless unit hydrograph (see McCuen, 1982, for a listing and discussion). The step-function devised by the author is

For $0 \le t \le 1.25\,Tp$

$$Q = \frac{Qp}{2}\left[1 - \cos\left(\frac{\pi t}{Tp}\right)\right] \qquad \text{(III-1)}$$

For $t > 1.25\,Tp$

$$Q = 4.34\,Qp\,\exp\left[-1.30\left(\frac{t}{Tp}\right)\right] \qquad \text{(III-2)}$$

in which

Qp = Peak discharge of the design hydrograph

Tp = Time to peak of the design hydrograph, measured from the time of significant rise of the rising limb to the time at which the estimated peak occurs

t = Time of interest at which the discharge is to be estimated.

The argument of the cosine is in units of radians.

The volume of water under this hydrograph is, in consistent units,

$$Vol = 1.39\,Qp\,Tp \qquad \text{(III-3)}$$

From this, the appropriate time to peak may be estimated as

$$Tp = \frac{Vol}{1.39\ Qp} \qquad \text{(III-4)}$$

Having accepted the pattern function, estimates of peak discharge and hydrograph volume may be made to allow computation of the time to peak. Use of the peak discharge and time to peak in the step function will provide a hydrograph that peaks at the estimated peak discharge and possesses the estimated volume.

2. Estimate the peak discharge by any applicable means. For small systems, the author usually obtains acceptable results with the Rational Method (see Section I), but other methods of peak estimation may be used as appropriate. The idea is to decide at what discharge the hydrograph is to reach its peak.

3. Estimate the volume under the hydrograph by any applicable means. Here is the arbitrary judgment similar to that which is imbedded in any design hydrograph formulation technique. The author has found that hydrographs can be matched reasonably closely with the significant center portions of those of methods that use a 24-hour center-weighted design storm. It is usually satisfactory to include under the hydrograph the volume of runoff from the six-hour precipitation of the return period of interest. The depth of precipitation may be obtained from Exhibit 2, or similar. Runoff depth may be estimated from the Soil Conservation Service Curve Number Method, in which a Curve Number estimate is obtained for the soil type and cover conditions of the watershed (McCuen, 1982). The following equations yield an estimated depth of runoff:

$$S = \frac{1000}{CN} - 10 \qquad \text{(III-5)}$$

$$Q^* = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \qquad \text{(III-6)}$$

in which:

S = Ultimate soil storage (in)

P = Precipitation depth (in)

Q* = Runoff depth (in), the asterisk being added by the author to distinguish this use of Q, which is depth, from the usual use of Q, which is discharge.

Volume of runoff is found by multiplying runoff depth by watershed area.

Stage-Storage Function

The stage-storage function represents the most important aspects of the size and shape of the storage container in its influence on the shape of the outflow hydrograph. The stage-storage function may be presented as a graph of stage plotted versus storage. Stage is the elevation of the water surface in the reservoir, most conveniently referred to the lowest point or reservoir bottom. Storage is the volume of water held in the reservoir, most conveniently expressed in cubic feet, when water level is at the associated stage.

The stage storage function can be formulated either as a graph or as a mathematical expression.

In many cases, the detention storage container is a natural stream valley, a ravine or a draw that has evolved to its current topographic shape over time. The source of information for the stage-storage function is usually a topographic map.

A representative set of storage volumes can be computed by applying the average-end-area method vertically from contour to contour. A conventional stage-storage curve may thus be plotted. If one plots storage versus stage on log-log axes, the resulting line is usually remarkably straight, suggesting that the reservoir stage-storage function may be adequately represented by a power-curve fit of the form

$$S = Ks\, Z^{b} \qquad \text{(III-7)}$$

in which

S = Storage volume (cu ft)

Z = Stage (ft) referred to the bottom of the reservoir

The computation of such a function is perhaps best illustrated by an example. Figure III-1 shows the contours digitized from city topo for a location in piedmont North Carolina. The site is just upstream of a road crossing thought to be suitable as a detention basin. The computation is carried out in Table III-1.

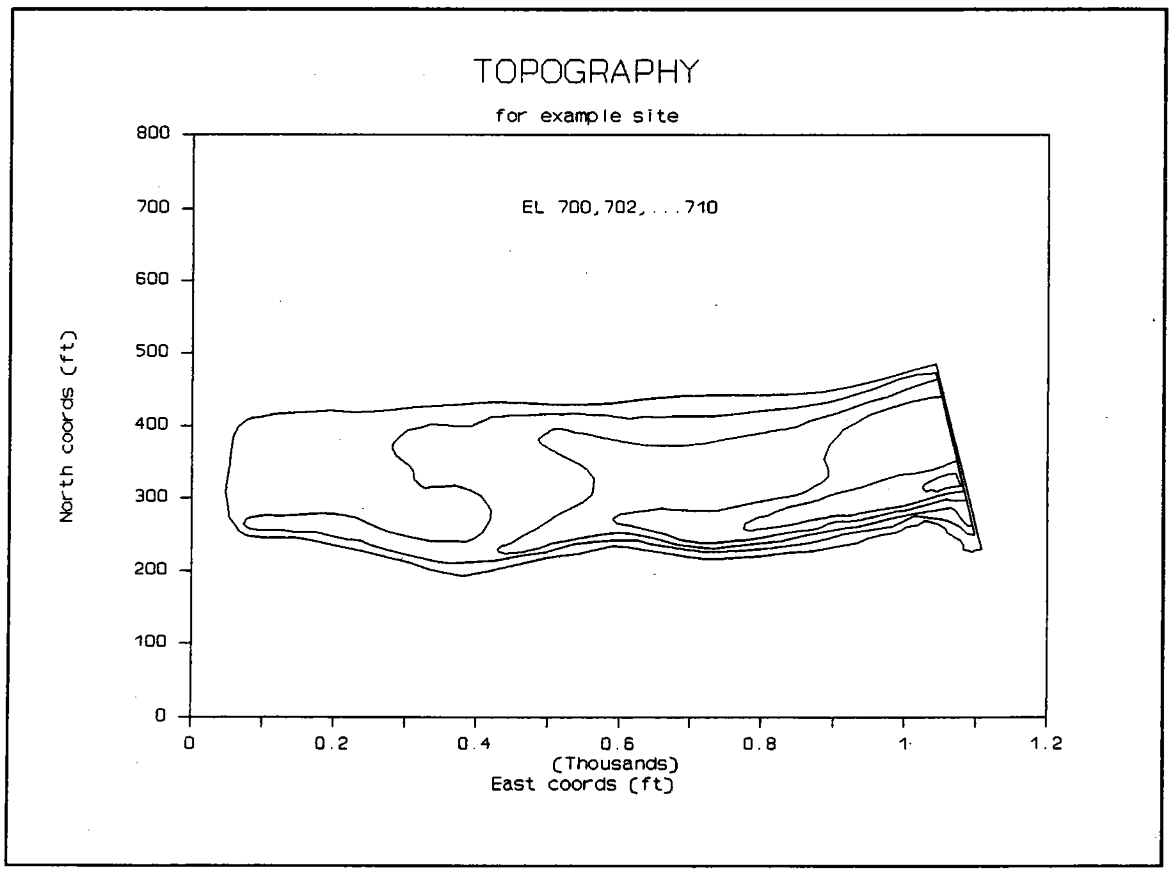

Figure III-1

Table III-1
Computation of a Stage-Storage Function

1	2	3	4	5	6	7	8
CONTOUR	CONTOUR AREA	INCR VOL	S ACCUM VOL	Z STAGE	ln S	ln Z	Z est
	[sq ft]	[cu ft]	[cu ft]	[ft]			[ft]
699	0		0	0			
700	784	392	392	1	5.9713	0.0000	1.10
702	9402	10186	10578	3	9.2665	1.0986	2.99
704	37140	46542	57120	5	10.9529	1.6094	4.99
706	83730	120870	177990	7	12.0895	1.9459	7.05
708	141746	225476	403466	9	12.9078	2.1972	9.03
710	213184	354930	758396	11	13.5390	2.3979	10.93

Regression Output:

Constant	5.647433 ==>	Ks = 284
Std Err of Y Est	0.018680	b = 3.30
R Squared	0.999908	
No. of Observations	5	
Degrees of Freedom	3	↑
X Coefficient(s)	3.299632	------------------------------
Std Err of Coef.	0.018209	

The following notes describe the numbered columnar computations:

1. Enter the contour elevations for which areas are measured. Elev 699 is the estimated invert of the depression.

2. Enter the measured contour areas, most conveniently expressed in square feet.

3. Compute the incremental storage volume (between the contours) by the average-end-area method applied vertically. The incremental volume is the average of the upper and lower contour areas multiplied by the vertical separation of the contours. The result is in cubic feet.

4. Enter the accumulated volume, obtained by adding each incremental volume to the sum of the lower increments. The result is the total storage available at each contour.

5. Enter the stage, or depth, referred to the invert of the pond.

One may usefully plot a stage-storage curve by using the data of columns 4 and 5. However, there is much more information readily obtainable from the stage-storage function of the form of Equation III-7. The columnar computations of Table III-1 continue:

6. Enter the natural logarithm of Storage (the accumulated volume of col. 4).

7. Enter the natural logarithm of Stage (col. 5).

There are two reasonable ways to determine the values of Ks and b of Equation III-7. Both depend upon the assumption that the logarithm of storage is linear with the logarithm of stage. The first few times one does the calculation, it is instructive to plot storage versus stage on log-log axes; or, if a spreadsheet program is being used, to plot ln S versus ln Z, as in Figure III-2.

It is usually true that the lowest point lies somewhat off the line of best fit. The purists among the readers will argue rightly that the fit would be improved by computing the lowest incremental volume by a pyramidal approximation (volume being estimated as one-third of the base area times the height). The author observes that the lowest increment trivially influences the the outcome of the routing, and that it is reasonable to disregard the lowest point in subsequent computations. (We're not making watches; we're merely toting water.)

Algebraic Estimation of Stage-Storage Parameters: One may select two representative points on the curve, preferably in the upper end of the range of stage, and compute values of Ks and b. Writing the logarithmic form of Equation III-7 for two points and solving simultaneously yields

$$b = \frac{\ln\left(\frac{S_2}{S_1}\right)}{\ln\left(\frac{Z_2}{Z_1}\right)} \qquad \text{(III-8)}$$

and

$$Ks = \frac{S_2}{Z_2^{\ b}} \qquad \text{(III-9)}$$

Linear-Regression Estimation of Stage-Storage Parameters: Many spreadsheet programs and programmable calculators have built-in procedures for regression analysis. For the fundamentals of regression analysis, the reader is referred to any basic statistics text. To carry out the operation in a spreadsheet or on a programmable calculator, refer to the operating manual.

NOTES

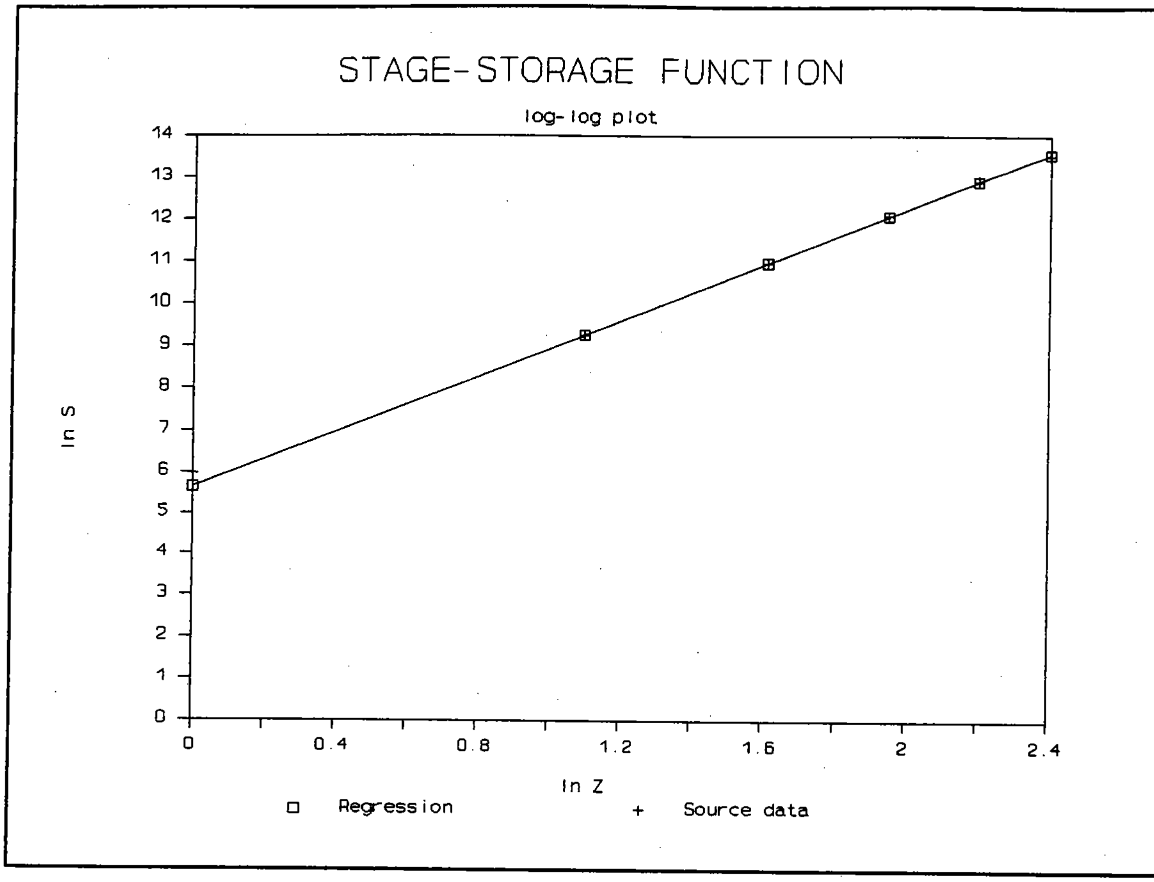

Figure III-2

In the stage-storage function, the independent variable (X- axis) is ln Z, and the dependent variable (Y-axis) is ln S. The value of the exponent, b, is the regression coefficient of the independent variable. The value of the coefficient, Ks, is the antilogarithm of the intercept. These values, as computed in the spreadsheet for the example problem, are shown below the columnar computations.

Validation of the function: Given the assumption of linearity of the logarithms of stage and storage, and given the inherent coarseness of topographic data, it is prudent to test the derived function against the original data. Returning to the columnar computations:

8. Enter estimates of stage as computed for the storage values of col. 4 by Equation III-7 rearranged:

$$\hat{Z} = \left[\frac{S}{Ks}\right]^{1/b} \qquad \text{(III-10)}$$

Compare the values obtained in col. 8 with those of col. 5. If the differences are tolerable, the stage-storage function is valid.

The stage-storage function as derived is plotted in Figure III-3, with the original data values also indicated. Clearly, the function is a valid representation of the topographic data.

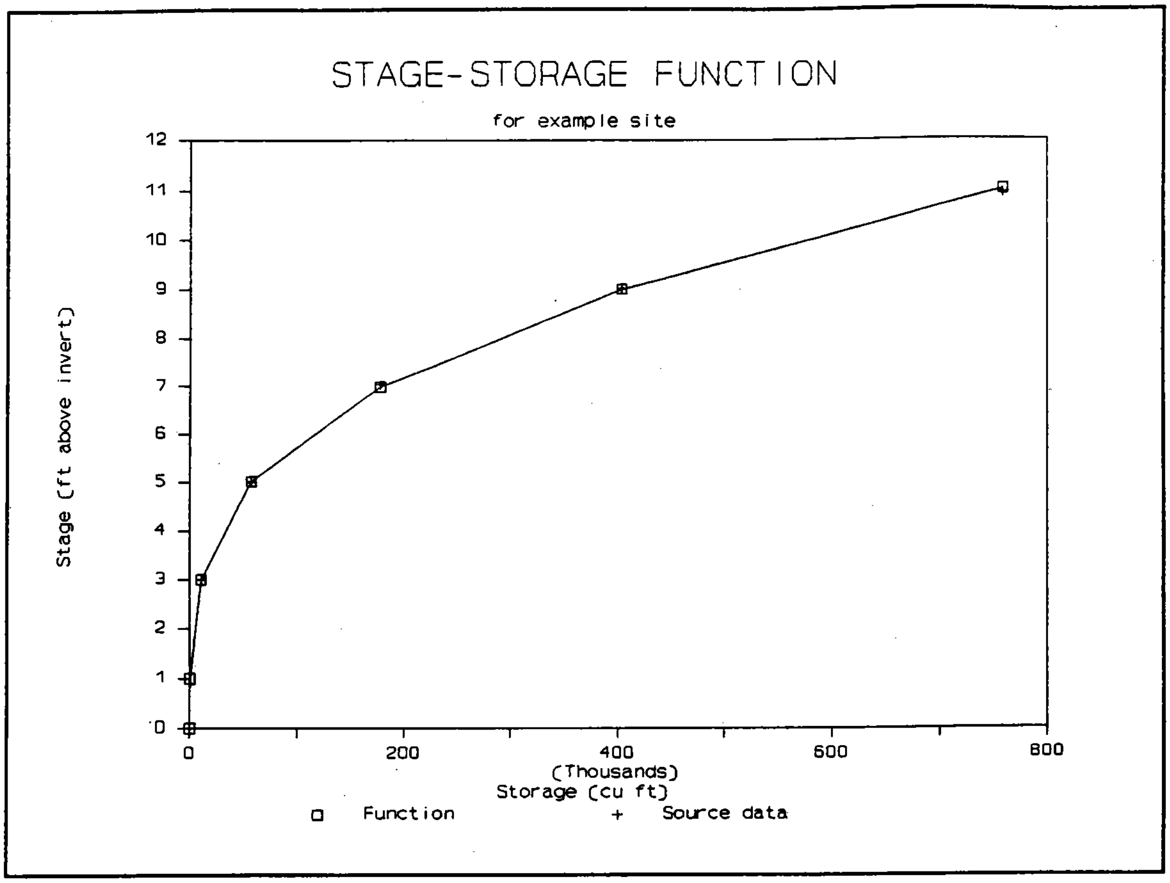

Figure III-3

The stage-storage function may be put to other uses:

1. It is useful early in the design process to consider the impoundment to be a vertical-sided reservoir. To do so, set b equal to one and Ks equal to the surface area in square feet.

2. A stage-area function may be obtained from the stage-storage function by taking the first derivative of storage with respect to stage. The results may be used to determine the area of inundation at a given stage, or to find the stage for a desired surface area. See the example problem.

3. Sometimes the average water depth is of interest. Using the definition of average depth as the volume divided by the surface area, average depth is Z/b.

Stage-Discharge Function

The stage discharge function represents the most important aspects of the hydraulic performance of the outlet device in its influence on the shape of the outflow hydrograph. The stage-discharge function may be presented as a graph of stage (referred to the same datum as the stage-storage function) versus discharge, or reservoir outflow.

The stage-discharge function is derived by hydraulic analysis of the set of outlet devices comprising the spillway system of the reservoir. Usually, these devices can be adequately analyzed by considering the individual outlets as orifices and weirs. For a sample of stages throughout the expected range of water level variation in the reservoir, the total outflow downstream of the dam is computed for each value of stage, and stage versus total discharge is plotted.

Weirs

Most popularly, two kinds of weirs are used -- sharp-crested and broad-crested weirs. For both, the basic equation is:

$$Q = Cw\,L\,H^{3/2} \qquad \text{(III-11)}$$

where:

Q = Discharge (cfs).

Cw = Weir coefficient (dimensionless). See sketches below.

L = Length of weir (ft), measured along the crest.

H = Driving head (ft), measured vertically from the crest of the weir to the water surface at a point far enough upstream to be essentially level.

The weir coefficient depends on the conditions that exist at the crest. For sharp-crested weirs, the value of Cw is theoretically 3.33. For broad-crested weirs Cw is 3.0. For the case of the free overfall, use Cw = 3.0. A useful reference is King, 1963.

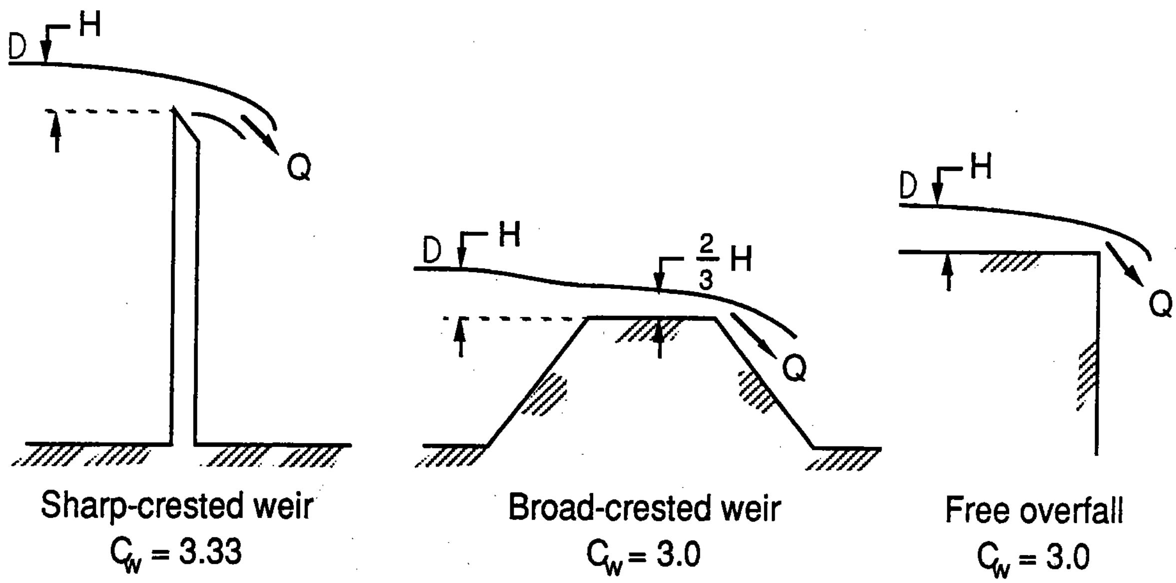

Figure III-4 Schematic sections through weirs.

Orifices

The basic equation for orifices is:

$$Q = C_D A \sqrt{2gh} \qquad \text{(III-12)}$$

where:

Q = Discharge (cfs).

C_D = Coefficient of discharge (dimensionless). See below.

A = Cross-sectional area of flow at the orifice entrance (sq ft).

g = Acceleration of gravity (32.2 ft/sec2).

h = Driving head (ft), measured from the centroid of the orifice area to the water surface.

An idealized sketch of a culvert under inlet control illustrates the orifice application.

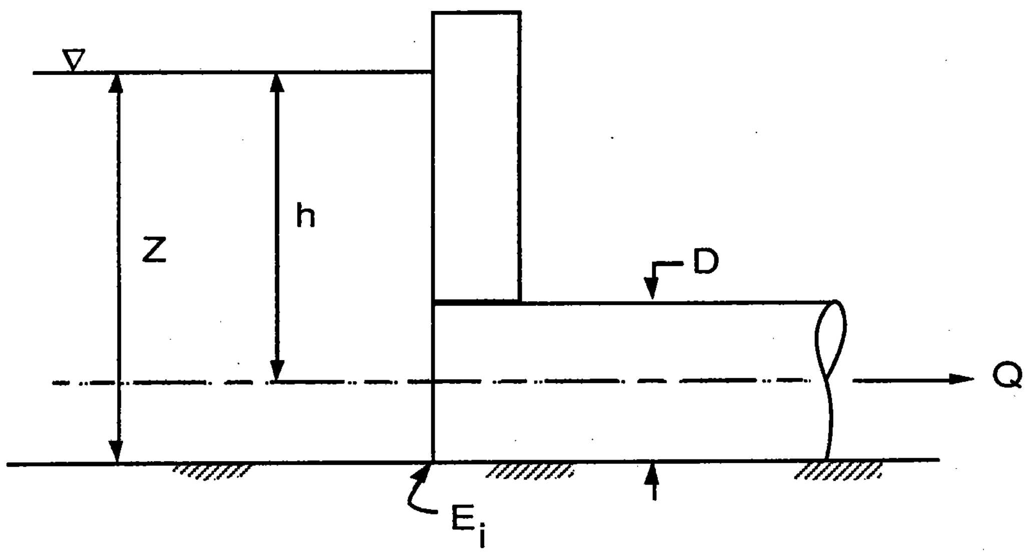

Figure III-5 Schematic section through an orifice.

Table III-2

Values of Coefficient of Discharge, Cd

Entrance Condition	Cd
Typical default value	0.60
Square-edged entrance	0.59
Concrete pipe, grooved end	0.65
Corr mtl pipe, mitred to slope	0.52
Corr mtl pipe, projecting from fill	0.51

Source: These values were back-calculated from the inlet-control culvert-capacity charts of Exhibits 11 and 12 for HW/D = 2.

The orifice equation applies only when the orifice is submerged. When the water surface is below the top of the pipe, a useful approximation of the behavior can be obtained by assuming discharge to be proportional to the three-halves power of depth and fitting the expression to the orifice result at full depth.

The following is a summary step function of stage-discharge for a culvert under inlet control arranged for use in commonly encountered units:

$$\text{For} \quad Z \leq Ei \tag{III-13}$$

$$Q = 0$$

$$\text{For} \quad Ei \leq Z \leq \left(\frac{D}{12} + Ei\right)$$

$$Q = 0.372\, C_D\, D\, (Z - Ei)^{3/2}$$

$$\text{For} \quad Z > \left(\frac{D}{12} + Ei\right)$$

$$Q = 0.0437\, C_D\, D^2 \left(Z - \frac{D}{24} - Ei\right)^{1/2}$$

Q = [cfs] Z = [ft]

D = [in] Ei = [ft] See Figure III-5

The stage-discharge function for pipes under inlet control can be obtained using the Culvert Capacity Charts of the Federal Highway Administration (See Exhibits 11-14 and FHWA, 1985). Figure III-6 illustrates the differences likely to be experienced with the function and chart. While the differences appear to be large, there is usually no significant difference between the two versions in routing results.

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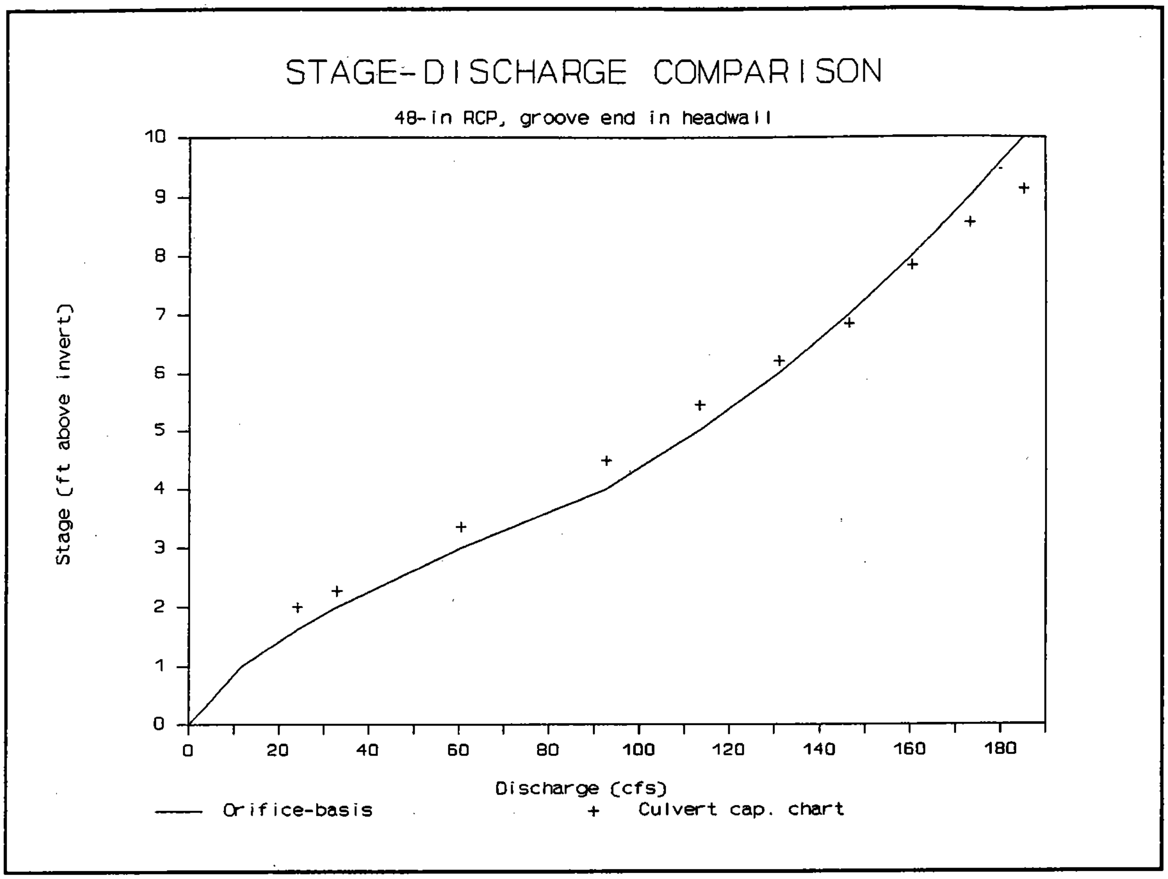


Figure III-6

Composite Stage-Discharge Functions

The typical case to be routed involves combinations of the fundamental orifices and weirs. The composite stage-discharge function can be prepared by applying the fundamental relationships to the outlet components and combining the results as the system behaves. Frequently encountered cases are the overtopped roadway at a culvert, and various combinations of pond spillways, including the riser/barrel spillway.

Culvert and Overtopped Road: The case of a pipe or pipes under a road or dam is illustrated in the schematic section of Figure III-7. For upstream water levels at or below the crest of the weir (top of road), outflow is computed for the pipe acting under inlet control. After the crest of the weir is overtopped, the outflow below the facility is the sum of the flow through the pipe and the flow over the broadcrested weir. Thus for any upstream water level, the outflow can be determined.

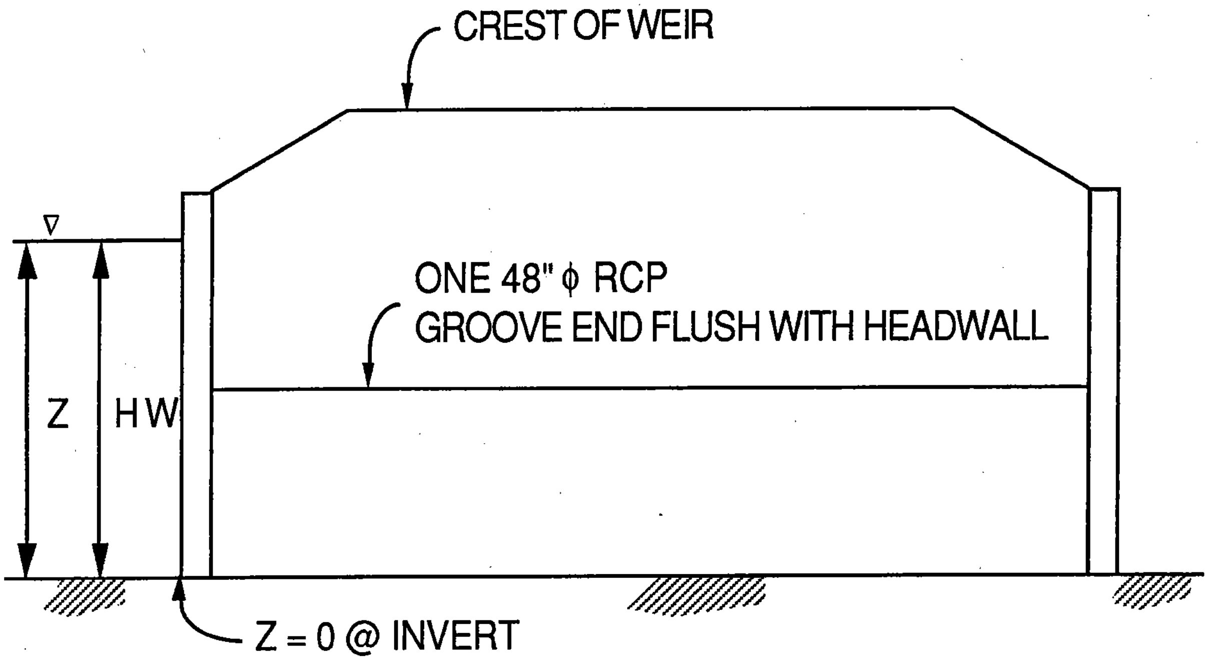


Figure III-7 Schematic Section Culvert Under Road

Riser/Barrel Spillway: A typical configuration for the outlet of a stormwater detention pond is a riser/barrel principal spillway and an emergency weir. There are three modes of behavior in which the riser/barrel can act as the water rises in the pond. One and only one mode of behavior will control at a given stage. Each mode can be analyzed by weir and orifice equations. A schematic of the riser/barrel system is shown in Figure III-8.

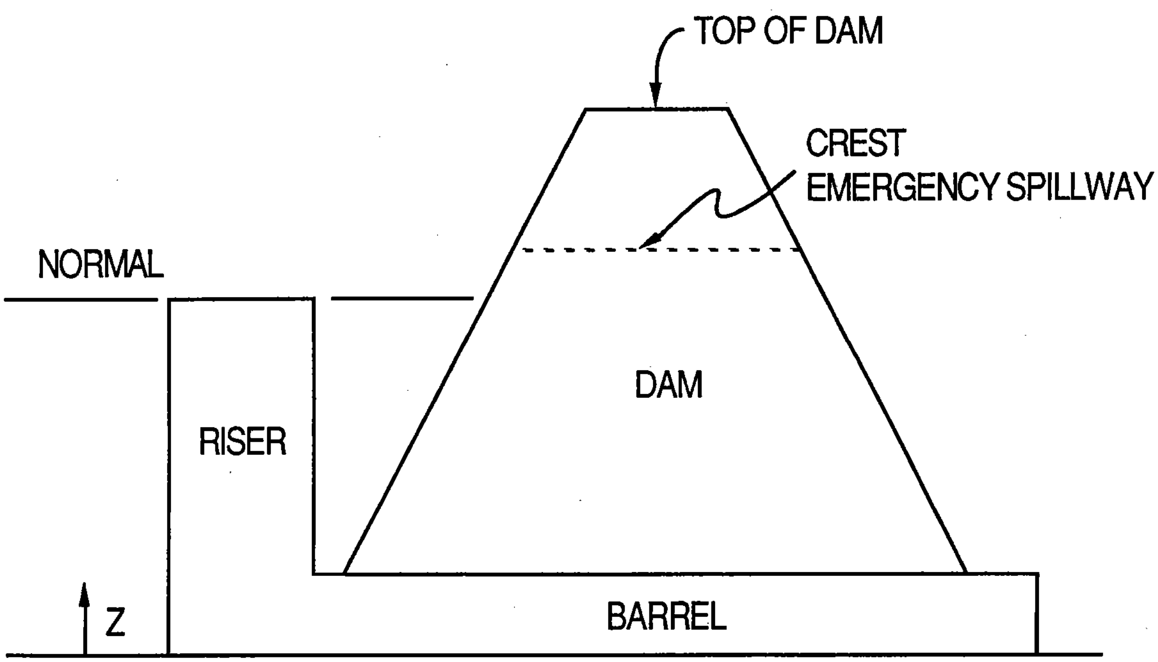


Figure III-8 Schematic Riser/Barrel Spillway

It is instructive to picture the behavior as water rises. To begin, water drops over the rim of the riser. The riser rim acts as a weir with length equal to its circumference and driving head equal to the water-surface elevation minus the elevation of the crest of the riser. As the head increases, one would likely observe a vortex to form as control makes the transition from riser acting as a weir to riser acting as an orifice. The orifice is formed by the top entrance of the riser, the area being the cross-sectional area of the riser. The driving head is measured from the water surface to the horizontal plane of the crest of the riser. These behaviors may be separately computed, as in Table III-3, and plotted, as in Figure III-9. In the figure, the action of the riser as a weir is indicated by the plus signs, and that of the riser as an orifice by the diamonds. Independently of the action at the top of the riser, there is the action of the barrel, which is behaving as a culvert under inlet control. In order to drive the flow through the barrel, water backs up in the inside of the riser. If the barrel is small relative to the riser, the water may rise to submerge the crest of the riser, superseding the action of the riser as a weir. If the barrel is relatively large, the action at the top of the riser may go through the transition from weir to orifice control before barrel control asserts itself. In Figure III-9, the barrel action is plotted with triangles.

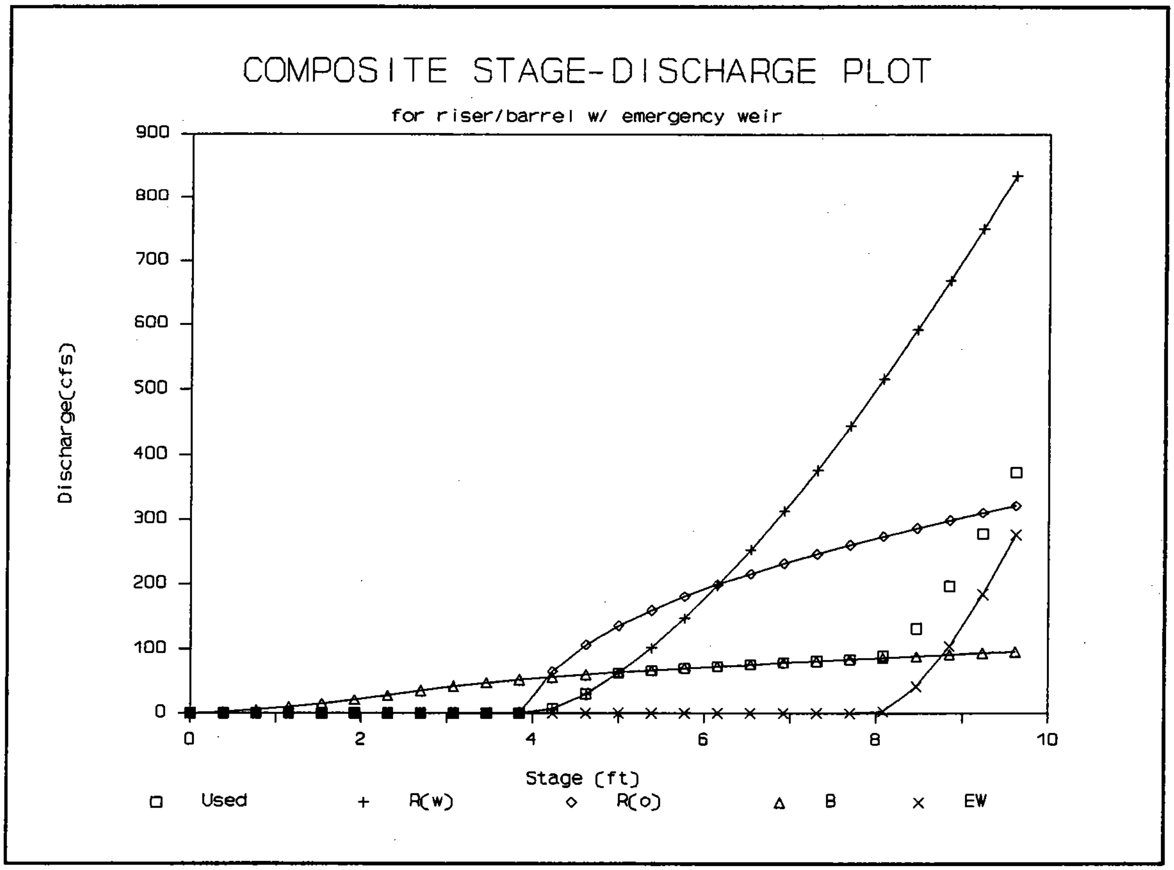


Figure III-9

The three behaviors of the barrel may be computed separately, but at a given stage one will control. At a given stage, choose the least of the discharges. At a given discharge, choose the highest of the three stages.

When the emergency weir is overtopped, the contribution of the weir is added to the contribution of the riser/barrel. In Figure III-9, the values used in the composite function are enclosed in rectangles. The net stage-discharge function is shown in Figure III-10, plotted with stage on the vertical axis and discharge on the horizontal axis as is more conventional.

Table III-3

STAGE-DISCHARGE FOR RISER/BARREL SPILLWAY W/ EMERGENCY WEIR

RISER/BARREL:

Dr	=	72 Riser dia (in)
Etop	=	4.00 Elev top riser (ft)
Cwr	=	3.33 Riser weir coeff
Cdr	=	0.60 Riser orifice coeff
Db	=	36 Barrel dia (in)
Einv	=	0.00 Elev invert barrel (ft)
Cdb	=	0.60 Barrel orifice coeff

EMERGENCY WEIR:

Cw	=	3.0 Weir coeff
Ecr	=	8.00 Elev weir crest (ft)
Lw	=	45 Weir length (ft)

1	2	3	4	5	6	7
Stage	Riser (weir)	Riser (orifice)	Barrel	Principal Spillway	Emergency Weir	Total Outflow
[ft]	[cfs]	[cfs]	[cfs]	[cfs]	[cfs]	[cfs]
0.00	0	0	0	0	0	0
0.38	0	0	2	0	0	0
0.77	0	0	5	0	0	0
1.15	0	0	10	0	0	0
1.54	0	0	15	0	0	0
1.92	0	0	21	0	0	0
2.31	0	0	28	0	0	0
2.69	0	0	35	0	0	0
3.08	0	0	43	0	0	0
3.46	0	0	48	0	0	0
3.85	0	0	52	0	0	0
4.23	7	65	56	7	0	7
4.62	30	107	60	30	0	30
5.00	63	136	64	63	0	63
5.38	102	160	67	67	0	67
5.77	148	181	70	70	0	70
6.15	198	199	73	73	0	73
6.54	254	217	76	76	0	76
6.92	314	232	79	79	0	79
7.31	378	247	82	82	0	82
7.69	445	261	85	85	0	85
8.08	517	274	87	87	3	90
8.46	592	287	90	90	42	132
8.85	670	299	92	92	105	197
9.23	751	311	94	94	184	279
9.62	835	322	97	97	277	374

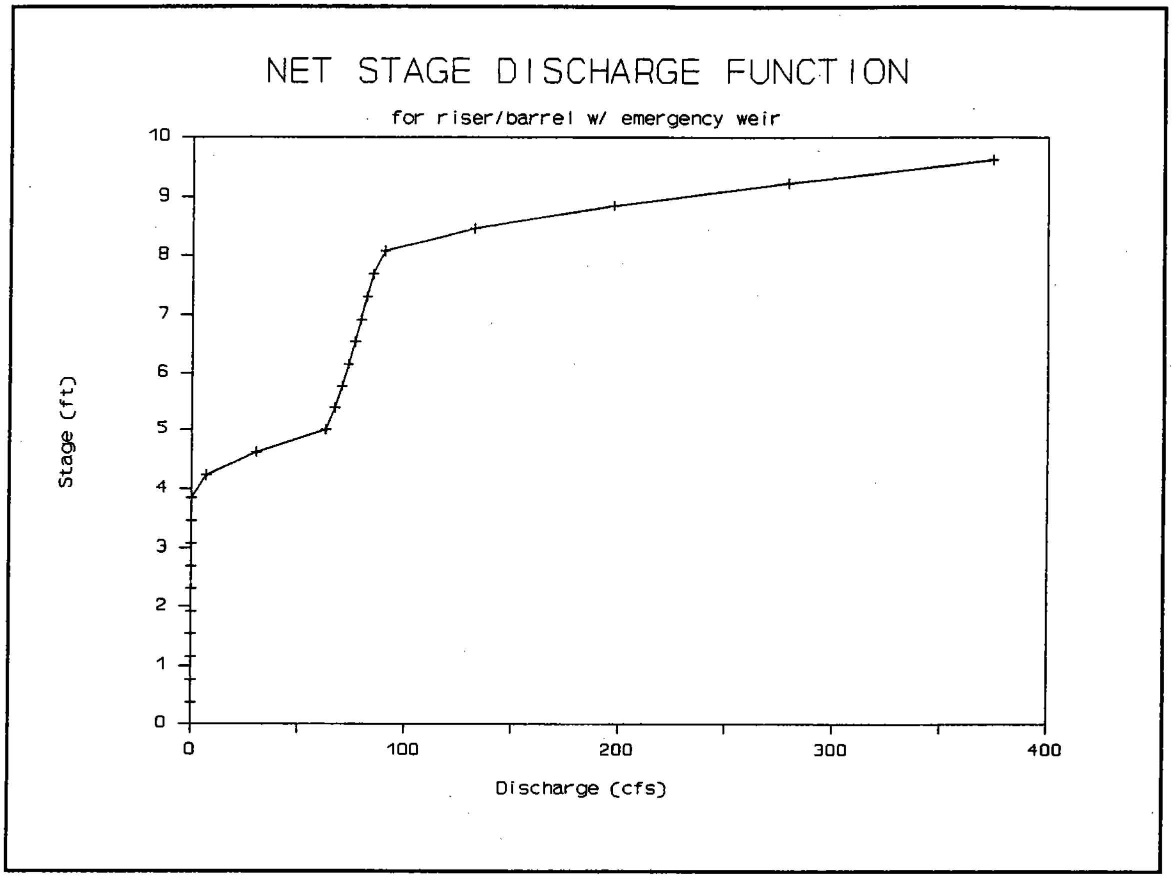


Figure III-10

ROUTING PROCEDURES

Two routing procedures will be discussed. The most widely recognized routing procedure is called the storage-indication method. A second method, devised by the author, is included for its inherent simplicity and efficiency. Several names have been suggested for it: Chainsaw Routing (it's rough and quick), California Routing (it does have its faults). Some of the author's students have given it clever names that are regrettably inappropriate for public documentation. Of them all, the author currently prefers Chainsaw Routing as a reminder of the coarseness of the information on which analysis and design of these systems are based. Our tools are like chainsaws, and we aren't making watches.

To execute either of the methods, one first formulates the three sets of source data described above.

Routing of the flood proceeds by time steps. At each step in time during the passage of the inflow hydrograph through the reservoir, the outflow is computed. The result is a list of values of outflow at stated times -- the outflow hydrograph.

Chainsaw Routing

The continuity principle states that the rate of change of storage with respect to time is the difference between inflow and outflow:

$$\frac{ds}{dt} = I - O \qquad \text{(III-14)}$$

Over a time increment:

$$\frac{\Delta s}{\Delta t} = I - O \qquad \text{(III-15)}$$

The incremental change in storage can be estimated as:

$$\Delta S_{ij} = (I_i - O_i)\, \Delta T_{ij} \qquad \text{(III-16)}$$

in which:

ΔS_{ij} = change in storage in the time increment i to j

I_i = Inflow at time i

O_i = Outflow at time i

ΔT_{ij} = Time increment

The simplification of this method is to consider that the change in storage may be adequately estimated by viewing the time increment as a parallelogram, whereas it is more precisely viewed as a trapezoid (the view taken in the Storage-Indication method). Equation III-16 becomes the basis for taking a step through time in the routing. Note the Equation III-16 is in consistent units. If inflow and outflow are in cfs and if the time increment is in seconds, then storage will be computed in cubic feet. These are the most convenient units.

The routing is conventionally carried out in a table, such as Table III-4, which was executed in a spreadsheet.

The reservoir in this case is a normally dry detention basin designed as a culvert. The inflow hydrograph peaks at 368 cfs and 36 minutes and follows the step function given above. The stage-storage function for the area upstream of the culvert was formulated such that Ks = 284 and b = 3.30. The culvert consists of one 48-inch diameter reinforced concrete pipe with grooved end flush with a headwall for which the coefficient of discharge is estimated at 0.65. There is a roadway that serves as an overflow spillway. The crest elevation is at stage of 10.0 feet, the weir length is 120 feet and the weir coefficient is 3.0 (broadcrested case). The stage reference (Z = 0) is to the invert of the entrance of the culvert which is also the bottom of the dry pond.

A Word about Spreadsheets: In the tabular computations that follow, the values shown in the cells of the tables were rounded back to the precision displayed. Internally in the spreadsheet program, the computations were carried out to several significant figures. As the reader computes values in a given cell based on displayed values in other cells, some differences may be noticed.

Selection of the Time Increment: The time increment should be about one tenth of the time to peak (Tp), where time to peak is considered to be measured from the time of significant rise of the rising limb to the time at which the peak occurs. In this case the time to peak is 36 minutes, so the time increment was conveniently selected at 4 minutes. Note that in calculations the time increment was expressed in seconds.

Table III-4

CHAINSAW ROUTING APPLIED TO A SITE WITH CULVERT AND OVERFLOW WEIR

Input data:

Qp	368
Tp	36
dT	4
Ks	284
b	3.3
N	1
Cd	0.65
D	48
Zi	0
Cw	3
L	120
Zcr	10

RESULTS

OUTFLOW PEAK	173
MAX STAGE	8.94

Routing:

1 TIME [min]	2 INFLOW [cfs]	3 STORAGE [cu ft]	4 STAGE [ft]	5 OUTFLOW [cfs]	6 CULVERT [cfs]	7 WEIR [cfs]
0	0	0	0	0	0.0	0.0
4	11	0	0.00	0	0.0	0.0
8	44	2718	1.98	32	32.4	0.0
12	94	5484	2.45	44	44.1	0.0
16	155	17396	3.48	80	79.7	0.0
20	219	35378	4.31	100	99.7	0.0
24	279	64046	5.17	117	116.6	0.0
28	328	103103	5.97	130	130.5	0.0
32	358	150432	6.69	142	141.9	0.0
36	368	202386	7.32	151	151.1	0.0
40	354	254328	7.84	158	158.4	0.0
44	320	301293	8.26	164	163.9	0.0
48	277	338670	8.56	168	167.8	0.0
52	239	364822	8.75	170	170.2	0.0
56	207	381358	8.87	172	171.7	0.0
60	179	389740	8.93	172	172.5	0.0
64	154	391210	8.94	173	172.6	0.0
68	133	386828	8.91	172	172.2	0.0
72	115	377506	8.84	171	171.4	0.0
76	100	364032	8.75	170	170.2	0.0
80	86	347094	8.62	169	168.6	0.0
84	74	327290	8.47	167	166.6	0.0

Initialization of the Routing Table: In every routing method the routing table must be initialized to represent the state of the system at time zero. In this case, along the row at time zero, the following were set:

Col 2: The initial inflow is zero; the system starts with no inflow. In some cases there may be some trivially low flow to be entered here.

Col 5: The initial outflow is set equal to initial inflow (Col 2) at time zero.

Col 4: Initial stage is set to reflect the water level in the reservoir at the beginning of the storm. In this case the pond is dry, and the stage is zero. In some cases, there is a normally wet pond. Then the stage is set to the initial stage of the water surface.

Col 3: Initial storage is the volume of water (cubic feet) in the reservoir at time zero. In this case, the system is dry and the volume is zero. In the case of a normally wet pond, the initial volume can be computed from the stage-storage function using the stage of the initial water surface.

Taking a Time Step: In every time step, the objective of the computation is to determine the outflow at the end of the time interval. So in this case, one would use the information at time zero and compute the values at time 4 min. Then use the 4 min values to find those at 8 min, and so on. Let time i be the time at the beginning of the interval and time j be the time at the end of the interval. In the chainsaw routine, one begins by using values at time i to estimate the change in storage at time j and to update the storage volume. Then with the known storage at time j, the stage can be computed from the stage-storage function, and from stage the outflow can be computed from the stage-discharge function, all at time j.

As an example, in the interval from time 28 to time 32 min, here is the order of computation:

Col 3: The change in storage, from Equation III-16, is inflow at time i (time 28 min) minus outflow at time i multiplied by the time increment (240 sec). The change is 47,520 cu ft, which is added to the storage at time i (103,103 cu ft) to yield the storage at time j (150,432 cu ft). (The numbers do not agree as printed because the spreadsheet program calculates all values to maximum precision, and the values are rounded for display.)

Col 4: At time j (32 min), calculate stage from updated storage using the rearranged stage-storage function, Equation III-10. At 32 min water has risen to stage 6.69 ft.

Col 6: At time j (32 min), calculate the flow through the pipe by Equation III-13, or by the culvert capacity charts. For stage of 6.69 ft, the equation yields 141.9 cfs.

Col 7: At time j (32 min), calculate the flow over the weir by Equation III-11. Note that for this whole routing, the water level never rises above the crest of the weir, so there is no weir flow.

Col 5: At time j (32 min), the outflow is the sum of the contributions of the pipe (Col 6) and the weir (Col 7).

Col 2: At time j (32 min), the inflow is updated by using the step function (Equations III-1 and -2) to compute the discharge at time 32 min. Alternatively, one could read inflow values from a plotted hydrograph obtained by any other means. (Rounded values are displayed.)

One can run the routing table as far as needed to determine the system responses of interest. Usually, these are the peak outflow, the largest value in Col 5, and the maximum stage, the largest value in Col 4.

Sometimes this method is subject to numerical instability. It will occur if the outlet system is of high discharge capacity and the storage container is of low storage capacity. In the real situation, the outflow hydrograph is tracking closely the inflow hydrograph. The effect of\storage upon outflow is negligible. In the routing table, it will present itself when outflow exceeds inflow on the rising limb of the inflow hydrograph. In the extreme,\change in storage becomes negative and large, perhaps large enough to make total storage go negative, and the computation of stage becomes impossible. Should this happen, it may be corrected by re-initializing the system on the line where the fault occurs as follows:

1. Set outflow (Col 5) equal to inflow (Col 2).
2. Set Stage (Col 4) equivalent to outflow (Col 5), by reference to the stage-discharge function.
3. Set Storage (Col 3) equivalent to stage (Col 4), by using the stage-storage function.
4. Restart the routing, repeating the re-initialization if necessary until the system behaves.

If instability occurs while stage is low in a multiple pipe outlet, it is reasonable to re-initialize where stage is near the top of the pipe. If instability persists to the inflow peak, it indicates that storage is ineffective in the system -- there is no detention effect.

The dry pond routed here would thus reduce the peak of the hydrograph from 368 cfs to 173 cfs by storing water to just under nine feet deep. The hydrographs are shown in Figure III-11. In Figure III-12, the stage-discharge function is plotted showing the points used in the routing.

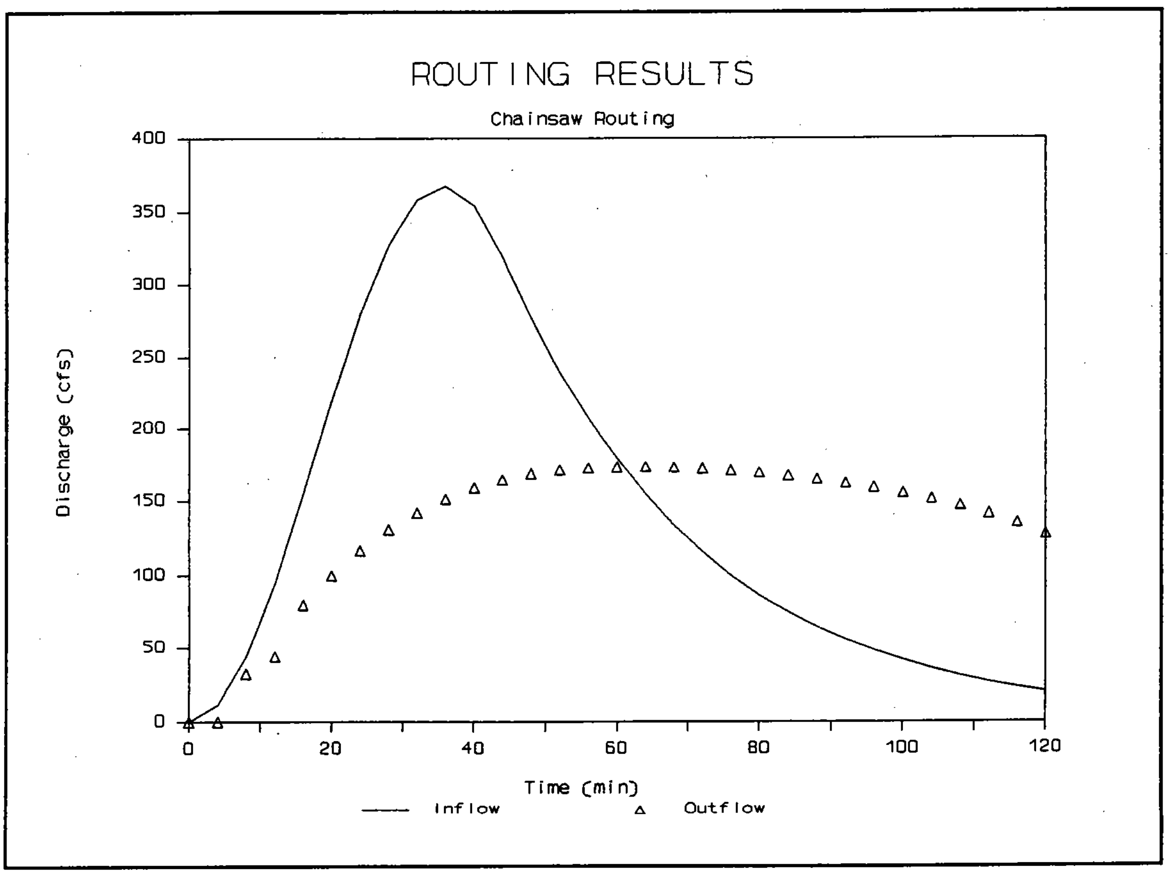


Figure III-11

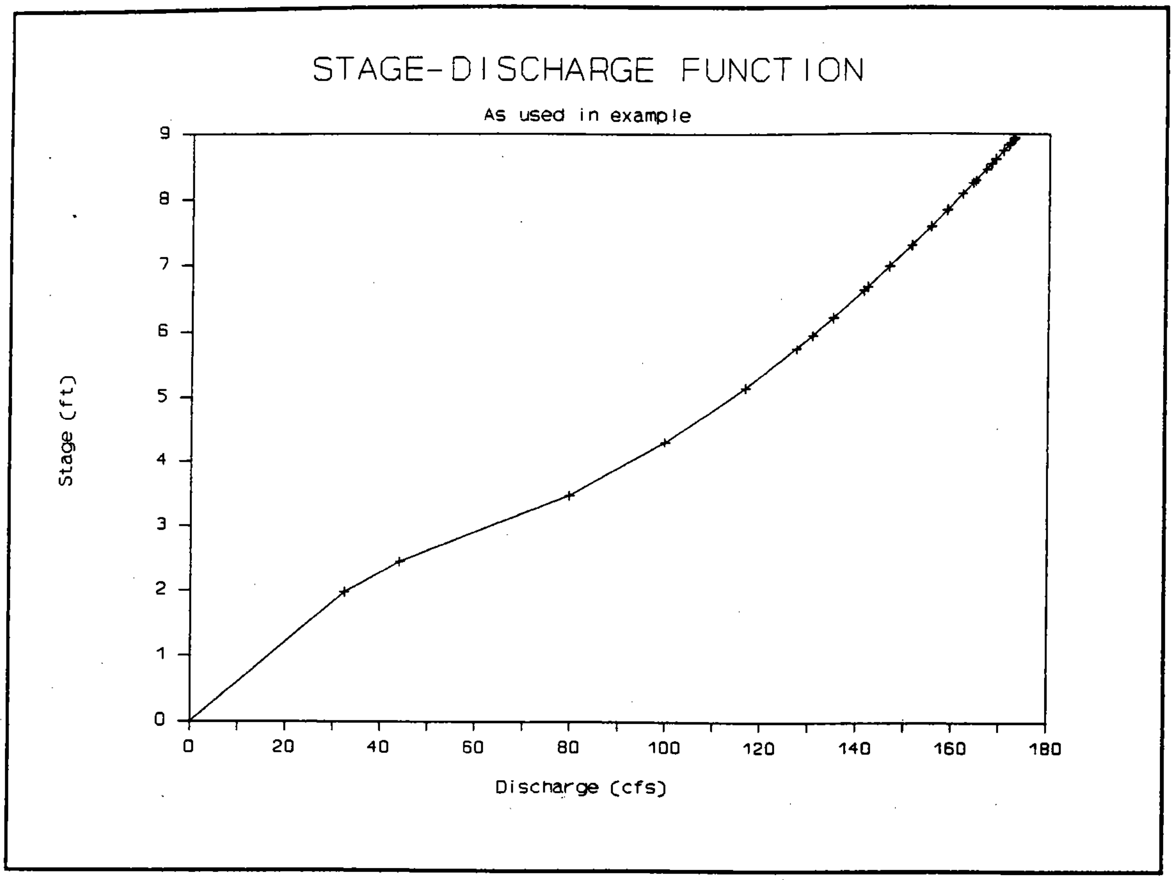


Figure III-12

Storage-Indication Method

The routing procedure most widely accepted is the Storage-Indication method. It is treated in most basic hydrology texts. Like all routing procedures, it proceeds from the continuity principle. To repeat, the continuity principle states that the rate of change of storage with respect to time is the difference between inflow and outflow:

$$\frac{ds}{dt} = I - O \qquad \text{(III-17)}$$

Over a time increment:

$$\frac{\Delta S}{\Delta T} = I - O \qquad \text{(III-18)}$$

The incremental change in storage can be estimated as the area of a trapezoidal element:

$$\Delta S_{ij} = (\bar{I} - \bar{O})\ \Delta T_{ij} \qquad \text{(III-19)}$$

in which:

ΔS_{ij} = change in storage in the time increment i to j

$\bar{I}$ = Average inflow from time i to time j

$\bar{O}$ = Average outflow from time i to time j

ΔT_{ij} = Time increment

Equation III-19 can be manipulated algebraically to obtain the following basis for taking a time step in this procedure:

$$I_i + I_j + \left[\frac{2S_i}{\Delta T} - O_i\right] = \left[\frac{2S_j}{\Delta T} + O_j\right] \qquad \text{(III-20)}$$

in which:

I_i = Inflow at the beginning of the interval

I_j = Inflow at the end of the interval

S_i = Storage at the beginning of the interval

S_j = Storage at the end of the interval

O_i = Outflow at the beginning of the interval

O_j = Outflow at the end of the interval

ΔT = Time increment

The equation is put in this form to collect on the left side the variables known at the beginning of the interval. From these, the sum on the right side may be computed. Note that that the right side has a storage term tied up with the outflow. The outflow may be determined from the value of the right side by means of a chart, or function, prepared from two of the sets of source data, stage-storage and stage-discharge. This chart is\called the storage-indication curve. It is a curious function when first viewed, but its usefulness becomes clear in the tabular computations of the routing. The storage indication curve is a plot of a certain expression, twice the storage divided by the time increment to which is added the outflow, versus the outflow.

The time increment must be selected prior to formulating the storage indication curve.

It is instructive to route the same example problem as was used in illustrating the previous method. The inflow hydrograph, stage-storage and stage-discharge functions remain the same.

Preparation of the Storage-Indication Curve: The curve is plotted by arbitrarily selecting stages and computing the values as shown in Table III-5.

Table III-5

STORAGE-INDICATION CURVE CALCULATION

dT = 240 sec

1	2	3	4	5	6
Stage	Storage	Q Culvert	Q Weir	O Total Q	(2S/dT)+O
[ft]	[cu ft]	[cfs]	[cfs]	[cfs]	[cfs]
0.00	0.00E+00	0	0	0	0
1.00	2.84E+02	12	0	12	14
2.00	2.80E+03	33	0	33	56
3.00	1.07E+04	60	0	60	149
4.00	2.75E+04	93	0	93	322
5.00	5.75E+04	113	0	113	593
6.00	1.05E+05	131	0	131	1006
7.00	1.75E+05	146	0	146	1602
8.00	2.71E+05	160	0	160	2421
9.00	4.00E+05	173	0	173	3508
10.00	5.67E+05	185	0	185	4907
11.00	7.76E+05	196	360	556	7024
12.00	1.03E+06	207	1018	1225	9844

In the table, the columnar computations are:

Col 1: Stages are arbitrarily selected at a convenient interval.

Col 2: Storage (cu ft) is computed at each stage.

Col 3: Discharge through the culvert is computed for each stage by Equation III-13, or equivalent.

Col 4: Discharge is computed at each stage for the weir by Equation III-11.

Col 5: Total outflow is obtained by summing Cols 3 and 4.

Col 6: Enter twice the storage (Col 2) divided by dT (240 sec), plus total outflow (col 5).

For use in the routing computations, these data are usually plotted as a curve. Plot the values of Col 6 versus the values of Col 5 as the storage-indication curve. For the example problem, the storage-indication curve appears as Figure III-13. If values are to be read from the curve for use in routing, the curve should be carefully plotted on good graph paper at large scale. In the routing example here, the actual calculation was done in a spreadsheet with the storage-indication curve computed in a detailed table and interrogated by means of a "lookup" function.

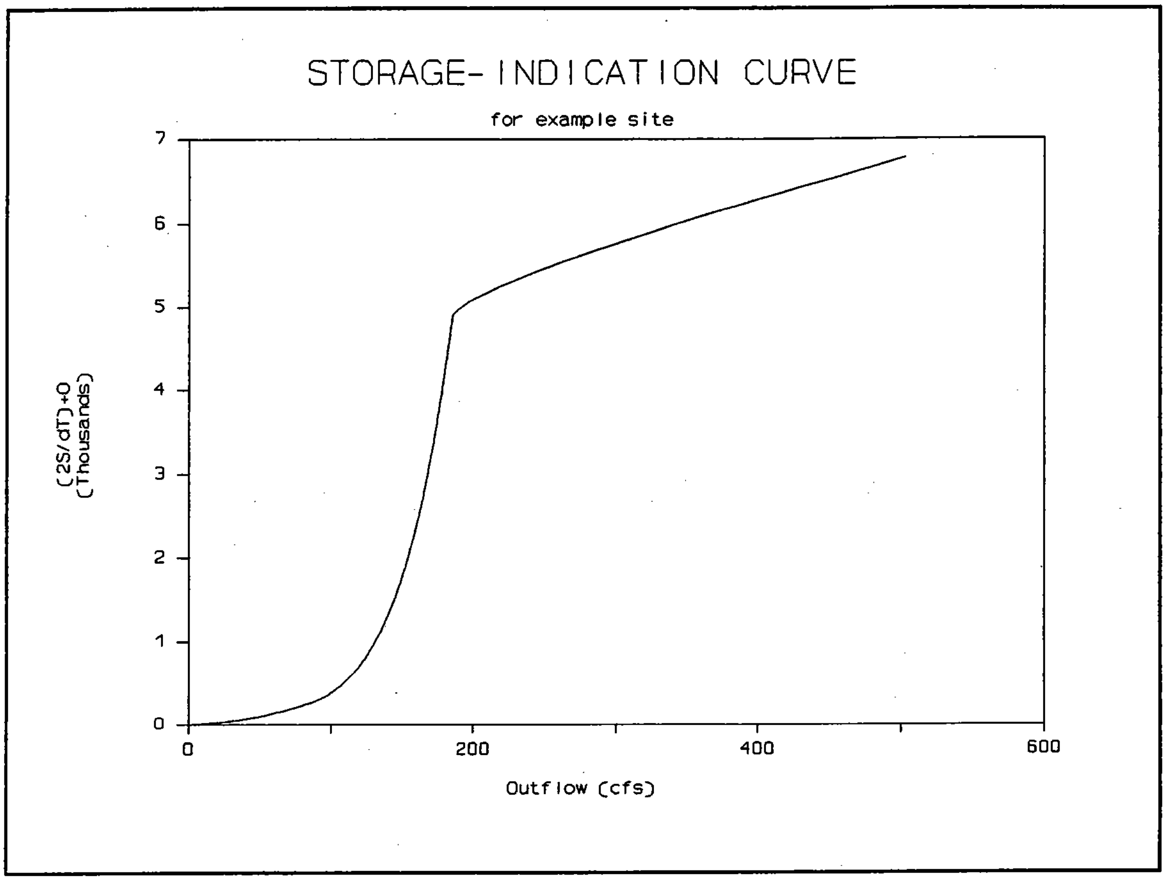


Figure III-13

The routing is carried out in a table such as that in Table III-6. The columns of the table are selected for efficient application of Equation III-20.

Initialization of the Routing Table: As in all such methods, the state of the system must be set at the beginning of the storm. On Line 1, the table is initialized as follows (note the similarity to the previous method):

Col 2: Set initial inflow from hydrograph. Here, at time zero, inflow is zero.

Col 5: Set initial outflow equal to initial inflow.

Col 3: Compute the value of twice the initial storage divided by the time increment (here 240 sec), minus initial outflow. In this case, since at time zero both storage and outflow are zero, the result is zero. If there is a normally wet reservoir, or if initial outflow were non-zero, then a non-zero result would be expected.

Col 4: This cell of the table is not used.

Table III-6

STORAGE-INDICATION ROUTING APPLIED TO A CULVERT AND WEIR

Input data:

Qp = 368
Tp = 36

dT = 4

Ks = 284
b = 3.3

N = 1
Cd = 0.65
D = 48

Cw = 3
L = 120
Zcr = 10

RESULTS:
Max Qo = 171 cfs

Routing:

	1	2	3	4	5
Line	Time	Inflow	(2S/dT)-O	(2S/dT)+O	Outflow
	[min]	[cfs]	[cfs]	[cfs]	[cfs]
1	0	0	0	na	0
2	4	11	-7	11	9
3	8	43	-9	47	28
4	12	92	17	126	54
5	16	152	96	261	83
6	20	216	255	464	105
7	24	276	506	747	121
8	28	325	840	1107	133
9	32	357	1234	1522	144
10	36	368	1653	1959	153
11	40	357	2059	2378	160
12	44	325	2412	2740	164
13	48	282	2684	3019	167
14	52	244	2870	3211	170
15	56	211	2985	3326	171
16	60	183	3037	3379	171
17	64	158	3035	3378	171
18	68	137	2988	3331	171
19	72	119	2904	3244	170
20	76	103	2788	3125	169
21	80	89	2645	2979	167
22	84	77	2481	2811	165
23	88	67	2299	2625	163
24	92	58	2102	2423	160

Taking a Time Step: A time step follows Equation III-20, followed by use of the storage-indication curve. Let us take as an example the entries on Line 3, the time step from time 4 to ime 5 min. Line 2 contains values for time i; Line 3 contains values for time j.

On Line 3:

Col 2: Enter the computed inflow for the time of Col 1. Here, the value was computed by the step function (Equations III-1 and -2). (Differences between inflow values in this table and the chainsaw table are due to the use of rounded values for Qp and Tp in this table, whereas quite precise values were used in the chainsaw table. Again -- a manifestation of the spreadsheet.)

Col 4: Compute the value of the right side of Equation 20. Inflow at time i (4 cfs) plus inflow at time j (8 cfs) plus the value in col 3 at time i (-7 cfs) is entered in col 4 at time j (47 cfs). (Again, the numbers may not add exactly due to rounding of the spreadsheet values.)

Col 5: Enter the storage-indication curve with the value in Col 4, time j (47 cfs), and find the associated outflow (28 cfs).

Col 3: Compute this value from the values at the same time in Cols 4 and 5. Col 4 (47 cfs) minus twice Col 5 (28 cfs) yields the value for Col 3 (-9 cfs).

The routing continues line by line until the response of interest is obtained.

Note that the peak outflow of 173 cfs obtained by chainsaw routing agrees closely with the 171 cfs computed here. The two sets of results of the two methods are plotted in Figure III-14.

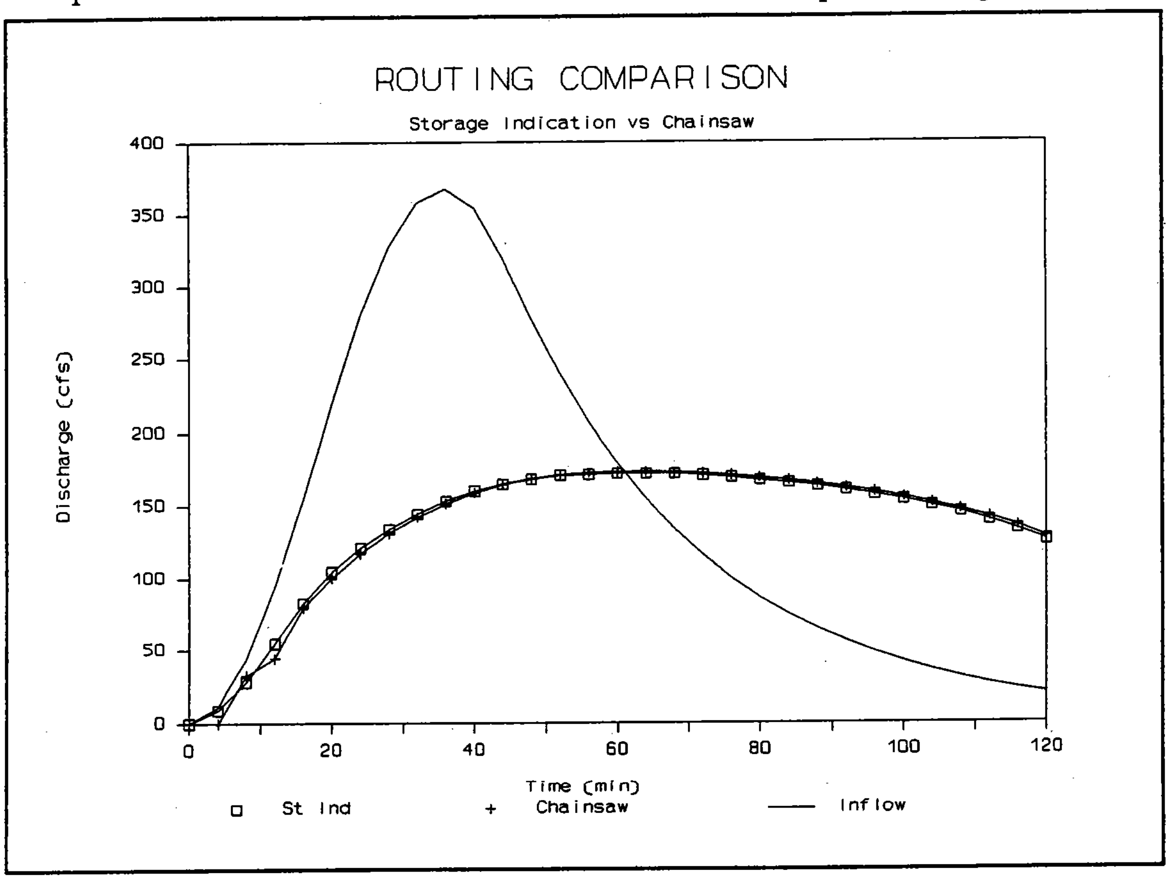


Figure III-14

PRELIMINARY DESIGN

Reservoir design is an indeterminate problem. In order to analyze a reservoir for performance in a design storm, the system must be routed. In order to route, the system must be known. In design, the system is being sought. It is necessary, therefore, to select a tentative system, and then route it. If the site must be selected from a number of alternatives, multiple routings can be laborious. A technique for tentative sizing of a system is useful in reducing the number of alternatives to be routed.

A very good approximation of orifice-based detention reservoirs can be obtained by a gross linearization of the inflow and outflow hydrographs. Let the inflow hydrograph rise linearly from the origin to the peak, Qp, and time to peak, Tp. Let the falling limb descend linearly from the peak to zero discharge at time of twice Tp. Let the outflow hydrograph rise linearly from the origin to its peak, Qo, at its intersection with the falling limb of the inflow hydrograph. In the triangular system thus contrived, the storage is the area between the triangles above the outflow hydrograph. By the triangular relationships, the storage required to reduce Qp to Qo can be estimated as:

$$S = (Qp - Qo)\,Tp \qquad \text{(III-21)}$$

in which:

S = Estimated storage required

Qp = Peak discharge of the inflow hydrograph

Qo = Peak discharge of the outflow hydrograph

Tp = Time to peak of inflow hydrograph, measured from time of significant rise of the rising limb to the time to peak.

The expression is in consistent units. If discharges are in cfs, then time must be in seconds to yield storage in cubic feet.

Equation III-1 may be thought of as PDQ routing. It roughly estimates the storage needed to reduce Qp to the target outflow, Qo, without knowing the system. The storage device may be configured accordingly, and the stage-storage function developed.

The outlet device may also be tentatively sized. In order to pick a pipe, one needs a discharge and driving head. With these, the FHWA culvert capacity charts or the orifice equation may be used to select the pipe or pipes to serve as the outlet. From the stage-storage function, determine the stage necessary to provide the estimated storage required. Using that stage and the target peak outflow, select the pipe or pipes. It is emphasized that the technique is approximate, and that the resulting system should be routed to confirm that it will perform satisfactorily.

SUMMARY

The methods of this section have been assembled to support the design of detention systems. Many jurisdictions have preferred methods for hydrograph formulation and flood routing. The final design must be presented using their methods for acceptance. For preliminary site selection, even design refinement, as in spreadsheets, the methods shown here can improve design efficiency. In some cases, they are acceptable for design submission.

The step-function hydrograph formulation is very quickly executed, and it can be shown to match reasonably well longer methods that use the 24-hr center-weighted design storm. Chainsaw routing is an excellent spreadsheet application. The triangular hydrograph approximation can reduce the number of routings required.

EXAMPLE III-1	DETENTION DESIGN	HRM	1/7

OBJECT: TO EXECUTE A PRELIMINARY DESIGN FOR A DETENTION BASIN IN WHICH STORAGE EXISTS ABOVE THE NORMAL WATER SURFACE OF AN AESTHETIC/RECREATIONAL POND.

GIVEN: (PROBLEM IS HYPOTHETICAL.)

LOCATION: RALEIGH, NC
WATERSHED AREA = 152 AC
HEIGHT OF MOST REMOTE POINT IN WATERSHED ABOVE OUTLET = 61 FT
HYDRAULIC LENGTH OF WATERSHED = 3640 FT
COMPOSITE RUNOFF COEFFICIENT AFTER DEVELOPMENT = 0.62
COMPOSITE RUNOFF COEFFICIENT BEFORE DEVELOPMENT = 0.21
SCS CURVE NUMBER AFTER DEVELOPMENT: 85

STAGE-STORAGE FUNCTION FOR LAKE AREA:

$$S = 332\, Z^{3.15}$$

S = STORAGE (FT^3)
Z = STAGE (FT ABOVE POND INVERT)
POND INVERT = EL 241.17

DESIGN CONSTRAINTS:

1. PEAK OUTFLOW FROM POND MAY NOT EXCEED PEAK FROM WATERSHED PRIOR TO DEVELOPMENT; DESIGN STORM IS 10-YR.

2. NORMAL SURFACE AREA OF POND SHALL BE 4.0 ACRES.

3. OUTLET SHALL BE RISER/BARREL OF CMP. RISER SHALL BE ~~48~~ 72"ϕ CMP.

APPROACH:

1. FORMULATE INFLOW HYDROGRAPH & PERMISSIBLE OUTFLOW PEAK.
2. SET CREST OF RISER TO YIELD 4.0 AC SURFACE AREA.
3. ESTIMATE STORAGE REQ'D; COMPUTE EXPECTED 10-YR STAGE.
4. SELECT OUTLET BARREL FOR 10-YR STAGE & PERMISSIBLE OUTFLOW.
5. CROSS FINGERS & ROUTE.

FORMULATE 10-YR DESIGN HYDROGRAPH:

USE SMALL WATERSHED METHOD (SECTION III):

1. ACCEPT STEP FUNCTION (EQNS III-1,2) AS PATTERN.
2. SET PEAK EQUAL TO AFTER-DEVELOPMENT RATIONAL ESTIMATE.
3. SET VOLUME EQUAL TO RUNOFF FROM 6-HR, 10-YR STORM.

ESTIMATE PEAK:

$$Q_p = C\,I\,A$$

$C = 0.62$ (GIVEN)

$$T_c = \frac{\left[\frac{L^3}{H}\right]^{0.385}}{128} \qquad \text{(EQN I-2)}$$

$$T_c = \frac{\left[\frac{(3640)^3}{61}\right]^{0.385}}{128}$$

$T_c = 20.8$ USE 21 MIN.

$$I_{10} = \frac{195}{22+T} \qquad \text{10-YR, RALEIGH (EXHIBIT 2)}$$

$$I_{10} = \frac{195}{22+21} = 4.53 \text{ IN/HR}$$

$A = 152$ AC (GIVEN)

$Q_p = (0.62)(4.53)(152) = 427$ CFS

USE $Q_p = 430$ CFS ← PEAK, INFLOW HYDROGRAPH

COMPUTE DEPTH OF RUNOFF:

$P = 3.90$ IN — 10-YR, 6-HR PRECIP, RALEIGH (EXHIBIT 2)
$CN = 85$ — (GIVEN)

$$S = \frac{1000}{CN} - 10 \qquad \text{(EQN III-5)}$$

$$S = \frac{1000}{85} - 10 = 1.76$$

FORMULATE 10-YR HYDROGRAPH (CONT.)

COMPUTE DEPTH OF RUNOFF (CONT.)

$$Q^* = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} = \frac{[3.90 - 0.2(1.76)]^2}{3.90 + 0.8(1.76)} \qquad \text{(EQN III-6)}$$

$Q^* = 2.37$ IN ← 10-YR, 6-HR RUNOFF DEPTH

SET VOLUME & COMPUTE TIME TO PEAK:

$$T_p = \frac{VOL}{1.39\ Q_p} \qquad \text{(CONSISTENT UNITS)} \qquad \text{(EQN III-4)}$$

$$T_p = \frac{(2.37\ \text{IN})(152\ \text{AC})}{(1.39)\ (430\ \frac{\text{FT}^3}{\text{SEC}})} \left[\frac{1\ \text{FT}}{12\ \text{IN}}\right]\left[\frac{43560\ \text{FT}^2}{1\ \text{AC}}\right]\left[\frac{1\ \text{MIN}}{60\ \text{SEC}}\right]$$

$T_p = 36.5$ SAY 37 MIN ←

USE AS 10-YR HYDROGRAPH AFTER DEVELOPMENT:

$$\boxed{\begin{array}{l} Q_p = 430\ \text{CFS} \\ T_p = 36.5\ \text{MIN} \end{array}}$$

SHAPE FOLLOWS STEP FUNCTION EQNS III-1,2

SET CREST OF RISER FOR 4.0-AC SURFACE AREA:

INVERT OF POND = EL 241.17 (GIVEN)

STAGE-STORAGE: $S = 332\,Z^{3.15}$ (GIVEN)

Z REFERRED TO EL 241.17

IF STORAGE, $S = K_s Z^b$

THEN SURFACE AREA, $A = \frac{dS}{dZ} = b K_s Z^{(b-1)}$

(THIS FOLLOWS FROM CONSIDERATION OF AVG END AREA METHOD.)

SET CREST OF RISER (CONT.):

SO $A = b K_s Z^{(b-1)}$, OR FOR THIS CASE

$$A = (3.15)(332) Z^{(2.15)}$$

$$Z = \left[\frac{A}{(3.15)(332)}\right]^{1/2.15} \qquad 4 \text{ AC} = 174\,240 \text{ SQFT}$$

$$Z = \left[\frac{174\,240}{(3.15)(332)}\right]^{1/2.15}$$

Z = 10.80 FT ABOVE INVERT OR EL 241.17 + 10.80 = EL 251.97

SET RISER AT EL 252.0 FOR 4.0 AC NORMAL LAKE SURFACE

COMPUTE ALLOWABLE OUTFLOW:

$Q = CIA$ (USE BEFORE-DVLPMT C, LET I & A REMAIN SAME.)

$Q = (0.21)(4.53)(152) = 144.6$ USE 145 CFS ←

ESTIMATE STORAGE NEEDED:

$S = (Q_p - Q_o) T_p$ (CONSISTENT UNITS) (EQN III-21)

Q_p = 430 CFS
T_p = 36.5 MIN
Q_o = 145 CFS

$S = (430 - 145)(36.5 \text{ MIN}) \left[60 \frac{\text{SEC}}{\text{MIN}}\right]$

S = 624,150 CU FT STORAGE REQD ←

(ESTIMATE — MUST ROUTE TO CONFIRM.)

CONFIGURE OUTLET:

AT TOP OF RISER, $Z = 10.83$ FT

$$S = 332\,Z^{3.15} = 332\,(10.83)^{3.15} = 602\,862\ \text{FT}^3$$

AT TOP OF 10-YR STORAGE, TOTAL STORAGE IS

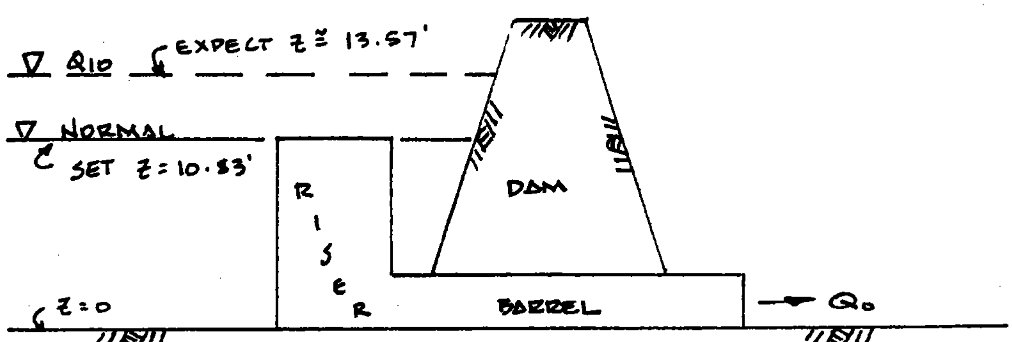
$$S = 602\,862\ \text{FT}^3 + 624,150\ \text{FT}^3$$

TOTAL $S = 1,227,000$ FT^3 WHEN 10-YR STORAGE IS FULL.

THE STAGE FOR TOP OF 10-YR STORAGE IS

$$Z = \left[\frac{S}{K_s}\right]^{\frac{1}{b}} = \left[\frac{1,227,000}{332}\right]^{\frac{1}{3.15}}$$

$Z = 13.57$ FT ← EXPECT POND TO BE THIS DEEP IN 10-YR.



NOT TO SCALE

WHEN $Z = 13.57'$, RISER/BARREL SHOULD DELIVER $Q_0 = 145$ CFS.

SELECT PIPE (BARREL) BY APPLYING ORIFICE EQN (COULD USE EXHIBIT 12).

$$Q = C_D A \sqrt{2gh}$$

$$\sqrt{h} = \frac{Q}{C_D A \sqrt{2g}} = \frac{4Q}{C_D D^2 (\pi \sqrt{2g})} = \frac{0.159\,Q}{C_D\, D^2}$$

$$h = \left[\frac{0.159\,Q}{C_D\, D^2}\right]^2$$

h IS REFERRED TO ℄ BARREL.

EXAMPLE III-1	DETENTION DESIGN (CONT.)	HRM	6/7

CONFIGURE OUTLET (CONT.):

BARREL MAY BE SELECTED BY

$$h = \left[\frac{0.159\,Q}{C_D\, D^2} \right]^2$$

LET $C_D = 0.59$ (TABLE III-2)
$Q = 145$ CFS

SELECT D IN FT ; COMPUTE h
TRY FOR $Z = 13.57'$

D (IN)	D (FT)	h (FT)	Z $h + D/2$	
24	2	95^+	–	WOW.
36	3	18.85	20.3	TOO SMALL
42	3.5	10.18	11.9	TOO BIG

THE SIZE NEEDED IS BETWEEN 36" & 42".
SINCE WE ARE DETAINING, TRYING TO REDUCE OUTFLOW,
SELECT 36" ϕ CMP FOR BARREL.

EXPECT ROUTED $Q_0 < 145$ CFS , $Z_{MAX} > 13.57'$ OR > EL 254.74

SEE NEXT PAGE FOR SPREADSHEET ROUTING OUTPUT.

RESULTS WERE AS EXPECTED $Q = 118$ CFS < 145 CFS OK
$Z_{MAX} = 255.21$

(NOTE: A SECOND ROUTING WAS RUN W/ 42"ϕ BARREL. $Q_0 = 157$ CFS ← NG)
(THE PRELIM DESIGN SEQUENCE POINTED TO A GOOD SOL'N.)

DESIGN DECISION:

1. SET TOP OF RISER @ EL 252.00 (AREA = 4.03 AC)
2. USE BARREL = 36"ϕ CMP. PEAK OUTFLOW = 118 CFS VERSUS ALLOWED 145 CFS.
3. EXPECT MAX 10-YR STAGE = EL 255.21.
4. SET CREST OF EMERGENCY SPILLWAY ABOVE EL 255.21.

ROUTING SPREADSHEET: EXAMPLE III-1

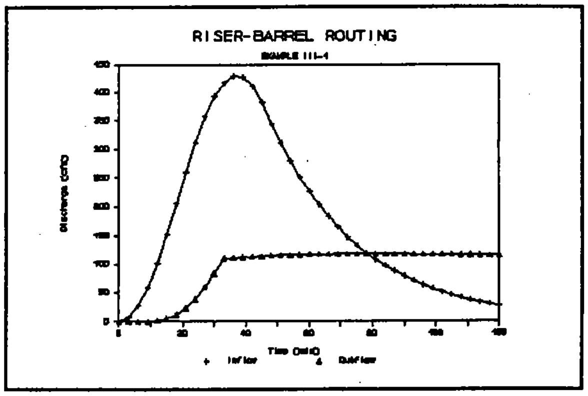
INPUT DATA:

Qp = 430 cfs
Tp = 37 min
dT = 3 min

Ks = 332
b = 3.15
Zo = 241.17 ft (Rf)

Dr = 72 in — Wksht assumes riser acts as weir.

Cw = 3.3
Zcr = 252 ft
Db = 36 in
Zi = 241.17 ft
Cd = 0.59



TIME [min]	INFLOW [cfs]	STORAGE [cu ft]	STAGE [ft]	OUTFLOW [cfs]	RISER [cfs]	BARREL [cfs]
0	0	6.03E+05	252.00	0	na	na
3	7	6.03E+05	252.00	0	0.0	102.2
6	27	6.04E+05	252.01	0	0.0	102.2
9	60	6.09E+05	252.03	0	0.4	102.4
12	102	6.20E+05	252.10	2	1.8	102.7
15	152	6.38E+05	252.20	5	5.4	103.2
18	206	6.64E+05	252.34	12	12.2	104.0
21	260	6.99E+05	252.52	23	23.4	105.0
24	312	7.42E+05	252.74	39	39.3	106.1
27	357	7.91E+05	252.97	60	59.8	107.4
30	393	8.44E+05	253.22	84	84.0	108.7
33	418	9.00E+05	253.47	110	110.7	109.9
36	429	9.55E+05	253.70	111	138.4	111.1
39	427	1.01E+06	253.94	112	167.8	112.3
42	411	1.07E+06	254.16	113	197.5	113.4
45	382	1.12E+06	254.36	114	226.0	114.4
48	346	1.17E+06	254.54	115	251.9	115.3
51	311	1.21E+06	254.69	116	274.3	116.0
54	280	1.25E+06	254.81	117	293.4	116.6
57	252	1.28E+06	254.91	117	309.4	117.1
60	227	1.30E+06	255.00	117	322.6	117.4
63	204	1.32E+06	255.06	118	333.3	117.8
66	184	1.34E+06	255.11	118	341.8	118.0
69	165	1.35E+06	255.15	118	348.2	118.2
72	149	1.36E+06	255.18	118	352.8	118.3
75	134	1.36E+06	255.20	118	355.8	118.4
78	120	1.36E+06	255.21	118	357.3	118.5
81	108	1.37E+06	255.21	118	357.5	118.5
84	98	1.36E+06	255.20	118	356.5	118.4
87	88	1.36E+06	255.19	118	354.5	118.4
90	79	1.35E+06	255.17	118	351.5	118.3
93	71	1.35E+06	255.15	118	347.6	118.2
96	64	1.34E+06	255.12	118	343.0	118.0
99	58	1.33E+06	255.09	118	337.7	117.9
102	52	1.32E+06	255.05	118	331.8	117.7
105	47	1.31E+06	255.01	118	325.3	117.5
108	42	1.29E+06	254.97	117	318.3	117.3

Norm Surf Area = 4.03 ac
Peak outflow = 118 cfs
Peak stage = 255.21 ft,msl