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PRELIMINARY DESIGN PROCEDURES FOR DETENTION BASINS

by

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ABSTRACT

A detention basin reduces flood peaks by providing temporary storage of floodwater. Usually the volume, peak discharge and shape of the inflow flood hydrograph are known, and the design involves calculation of the basin size required to reduce the outflow peak discharge to a specified value. The complete design of a detention basin is a lengthy procedure, involving both selection of a range of design floods and routing the floods through the basin to determine the critical design case. There is a need for a simpler procedure for preliminary design of detention basins. One such procedures involves the use of standard shapes for the inflow and outflow hydrographs. Equations can be derived which directly relate the required basin size to variables representing the standard inflow and outflow flood hydrographs. The present study assesses the accuracy of three preliminary design procedures by comparing predicted results with a complete design using routing of the inflow flood hydrograph through the detention basin. A total of 1029 cases of different combinations of variables was used in the assessment, including watershed sizes ranging from 0.2 - 5 km^2, design recurrence intervals from 1 - 100 years, storm durations from 5 - 720 min, and basin pipe diameters from 300 - 2100 mm. Both Australian and U.S.A. data were used. The recommended preliminary design procedure gave good predictions over wide ranges of contributing watershed size, design recurrence interval, size of pipe draining the detention basin, and basin size.

INTRODUCTION

The use of detention basins to reduce local downstream flooding by temporary storage of part of the flood is well established (Wright-McLaughlin Engineers, 1969; Poertner, 1973). During the early stages of the flood the inflow rate exceeds the outflow rate and water is stored temporarily in the basin. The storage volume S required in the basin is equal to the areas between the inflow and outflow hydrographs.

The complete design of a detention basin involves routing an inflow flood hydrograph through the basin using any of the established reservoir routing procedures. These require details of the basin head-storage volume relation and its head-discharge relation. Several trials are necessary to select the required combination of basin storage volume and outlet pipe diameter. In addition, a range of inflow hydrographs must be

routed through the basin to determine the critical storm duration. Simplified design procedures relating characteristics of the inflow flood hydrograph to the basin storage volume and the outflow flood hydrograph can assist the design process by providing an estimate of the basin volume required. The selected design can then be checked using the complete design procedure. The present study examines the accuracy of several simplified design procedures by comparing predicted results with a complete design using reservoir routing.

Detention storage can be provided in the form of rooftop ponding, parking lots, recreation areas with embankments, and embankment across a watercourse, either as a constructed bank or as a road embankment. In the present study only the latter type of detention storage, embankment across a watercourse, is considered. The method of analysis, however, can be readily extended to the other cases by adopting head-storage volume and head-discharge relations appropriate to the type of basi...

SIMPLIFIED DETENTION BASIN DESIGN PROCEDURES

Preliminary Design Based on Hydrographs

In the complete design procedure, an inflow hydrograph is routed through the basin, and the required basin storage volume is given by the area between the two hydrographs. Simplified hydrograph design procedures are based on the same principle, but avoid the need for basin routing by adopting standard hydrograph shapes. In this way the properties of the inflow and outflow hydrographs can be related to the required basing storage volume as dimensionless ratios which are generally applicable. Three simplified hydrograph procedures are considered here; all adopt triangular inflow hydrographs but differ in the assumed shape of the outflow hydrograph.

Triangular outflow hydrograph -- The simplest standard hydrograph shapes consist of triangular inflow and outflow hydrographs (Fig. 1a). Expression for the volume of the inflow hydrograph, I, and the required basin storage volume, S, can be readily derived in terms of the peak inflow rate i, peak outflow rate q, and inflow duration B, as follows,

$$\frac{q}{i} = 1 - \frac{S}{I} \tag{1}$$

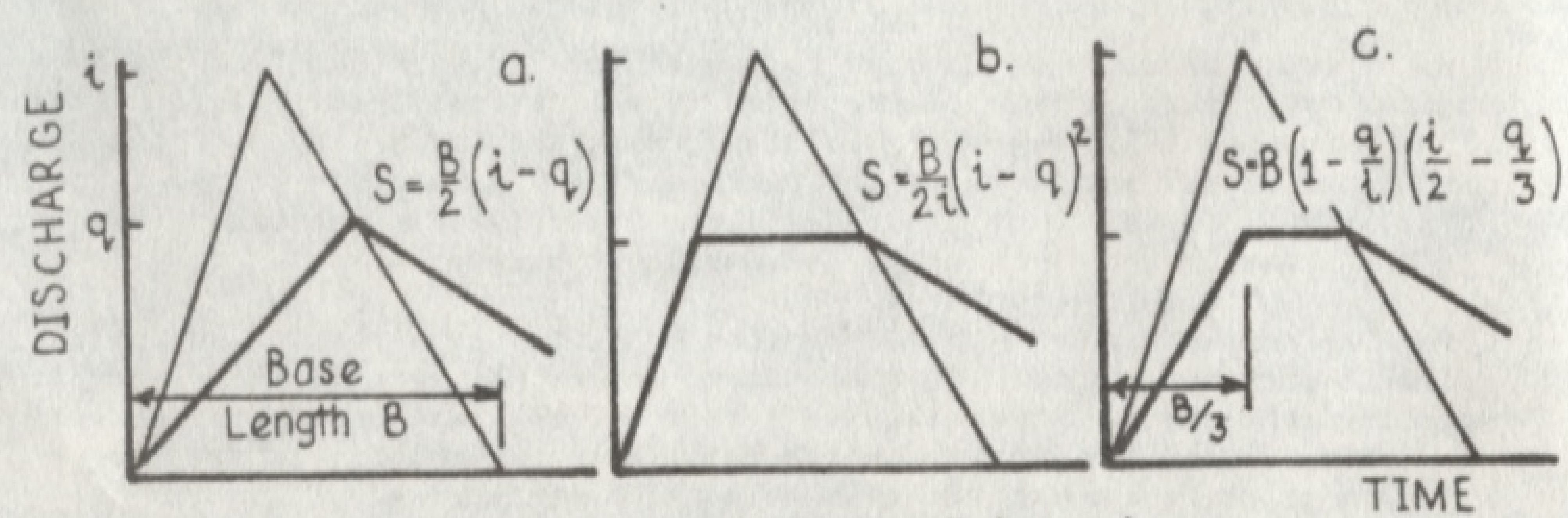

Fig. 1. Inflow and Outflow Hydrographs

Constant outflow rate -- Gould (1967) and Bartlett (1976) have assumed a constant discharge through the outlet pipe (Fig. 1b). For this case

$$\frac{q}{i} = 1 - \left[\frac{S}{I}\right]^{0.5} \qquad (2)$$

Culp formula -- Culp (1948) adopted an outflow hydrograph that increased linearly from zero until a point beneath the inflow hydrograph peak, then remained constant (Fig. 1c). For this case

$$\frac{q}{i} = 1.25 - (1.5\,\frac{S}{I} + 0.0625)^{0.5} \qquad (3)$$

Outflow hydrographs corresponding to the two latter cases are more likely to occur for small outlet pipe diameters, where the pipe flows full with submerged entrance conditions and the outflow does not vary greatly with changes of head in the basin (that is, where $q \propto H^{0.5}$, approximately). Outflow hydrographs corresponding to the first case are more likely to occur for larger pipe diameters, where the entrance is not submerged for an appreciable portion of the flood and the outlet discharge varies accor-ding to the head (that is, $q \propto H^{1.5}$, approximately). It should be noted, however, that assumptions are made with respect to the inflow as well as the outflow hydrograph, and that these may compensate. The most accurate of the three equations can only be determined by comparing the results with complete routing of the inflow flood through the basin. This is done in the following section.

Since Eqs. 1, 2 and 3 relate characteristics of the inflow and outflow flood hydrographs to the basin storage volume, in addition to the design case outlined above, the equations may be used to estimate the peak outflow for any combination of inflow flood and basin volume.

METHOD OF ANALYSIS

General

The accuracy of the three hydrograph methods described previously was tested by comparing the results predicted from Eqs. 1, 2 and 3 with those obtained using a complete analysis involving routing inflow flood hydrographs through detention basins. The accuracy of the equations can be assessed simply by comparing results for any selected inflow hydrographs. In the present study, however, it was considered desirable to make the testing as realistic as possible, and a range of inflow hydrographs was calculated using actual design rainfall data; these were routed through detention basins with a range of outlet pipe diameters. Table 1 gives the range of variables used to calculate inflow hydrographs and route them through the basins.

The procedure used in the accuracy tests was: design rainfall intensities were calculated for all combinations of recurrence interval Y and storm duration T, losses were subtracted to produce rainfall excess hyetographs, and inflow flood hydrographs were calculated for the range of watershed areas. The inflow floods were then routed through a detention basin for the range of outlet pipe diameters d. For each combination of A, Y, and d, only the critical duration storm was used in the accuracy tests, giving a total of 147 events. In some cases, however, the combination

TABLE 1. Range of Variables Used in Accuracy Tests

Watershed Area A (km^2)	0.2	1	5				
Recurrence Interval Y (years)	1.01	2	5	10	20	50	100
Storm Duration T (hours)	0.083	0.25	0.5	1	2	6	12
Outlet Pipe Diameter d (mm)	300	600	900	1200	1500	1800	2100

TABLE 2. Design Rainfall Intensities Used in Study (mm/hr)

Storm Duration T (hours)	Recurrence Interval y (years) 1.01	2	5	10	20	50	100
0.083	119	153	203	228	270	312	349
0.25	78	101	134	150	178	206	231
0.5	55	70	93	104	124	143	160
1	36	47	62	70	82	95	107
3	18	23	31	34	41	47	53
6	11.4	14.6	19.4	22	26	30	33
12	7.6	9.8	13.0	14.6	17.3	20	22

TABLE 3. Range of Properties of Inflow Hydrographs

Watershed Area A (km^2)	Critical Storm duration T_c (hours)	Recurrence Interval Y (years)	Storm Duration T (hours)	Lag Time (hours)	Peak Discharge (m^3/s)
0.2	0.4	1	0.083	0.41	1.3
		1	1	1.33	1.9
		1	12	12.3	0.3
		10	0.083	0.41	2.5
		10	1	1.33	3.7
		10	12	12.3	0.7
		100	0.083	0.41	3.8
		100	1	1.33	5.8
		100	12	12.3	1.1
1.0	0.8	1	0.083	0.80	2.9
		1	1	1.72	7.8
		1	12	12.7	0.8
		10	0.083	0.80	5.6
		10	1	1.72	15.5
		10	12	12.7	3.4
		100	0.083	0.80	8.6
		100	1	1.72	24.1
		100	12	12.7	5.5
5.0	1.5	1	0.083	1.58	3.9
		1	1	2.5	23.5
		1	12	13.5	7.1
		10	0.083	1.58	7.2
		10	1	2.5	46.6
		10	12	13.5	16.8
		100	0.083	1.58	10.8
		100	1	2.5	72.2
		100	12	13.5	27.6

of a large inflow flood with a small outlet pipe diameter produced unrealistically high water depths in the basin. Only those events in which the maximum depth was less than 5.0 m were included, giving a final total of 79 events for accuracy tests.

Design Inflow Flood Hydrographs

Australian Rainfall and Runoff (1977) contains design rainfall intensities for all combinations of recurrence interval Y and storm duration T throughout Australia. For the present study design rainfall intensities for the Wollongong area were selected as being representative. These intensities are given in Table 2. A constant loss rate of 2.5 mm/hr was subtracted from all intensities to give rainfall excess hyetographs. This loss rate is a median value for published data from both Australia and the U.S.A.

The rainfall excess hyetographs were converted into hydrographs of inflow to the basin using the unit hydrograph model of Nash (1960). The two parameters of this model had previously been determined using recorded rainfall and streamflow data for natural catchments in eastern Australia. The parameter values adopted were (K in hours; A in km^2)

$$N = 3$$
$$K = 0.24\ A^{0.46}$$

Both the loss rates and the Nash model parameters refer to natural catchments, and would be different for urban catchments. However this should not affect the accuracy comparisons, since these calculations serve simply to produce realistic inflow hydrographs. Table 3 gives an indication of the wide range of inflow hydrograph shapes used in the study. These ranged from short duration, high intensity storms giving sharply peaked hydrographs to long duration, low intensity storms giving long flat hydrographs. For each combination of catchment area and recurrence ineterval, a critical storm duration which produces the maximum inflow flood peak can be calculated. The critical storm duration T_c increased only slightly with recurrence interval, and depended mainly on the catchment area.

Details of Detention Basin

Figure 2 shows a schematic layout of the detention basin used. A square edge entrance circular outlet pipe was adopted.

Routing of the inflow flood through the basin required details of the basin head-storage volume relation and the outlet pipe head-discharge relation. The head-storage relation adopted was an average for three detention basins constructed across natural watercourses in the Wollongong region, calculated from contour plans. The head-discharge relation used was based on data given in Chow (1959) and extended to cover the case of submerged entrance conditions and full pipe flow. Figure 3 shows the head-storage and head-discharge relations adopted.

Routing of Inflow Flood Through Detention Basin

The basin routing equation was

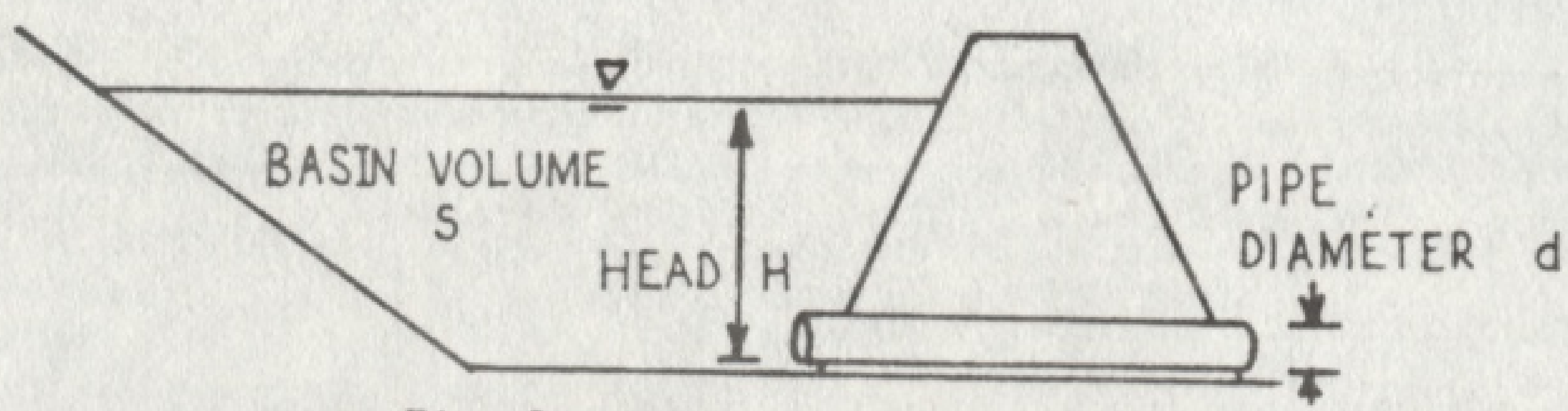

Fig. 2. Schematic Layout of Detention Basin

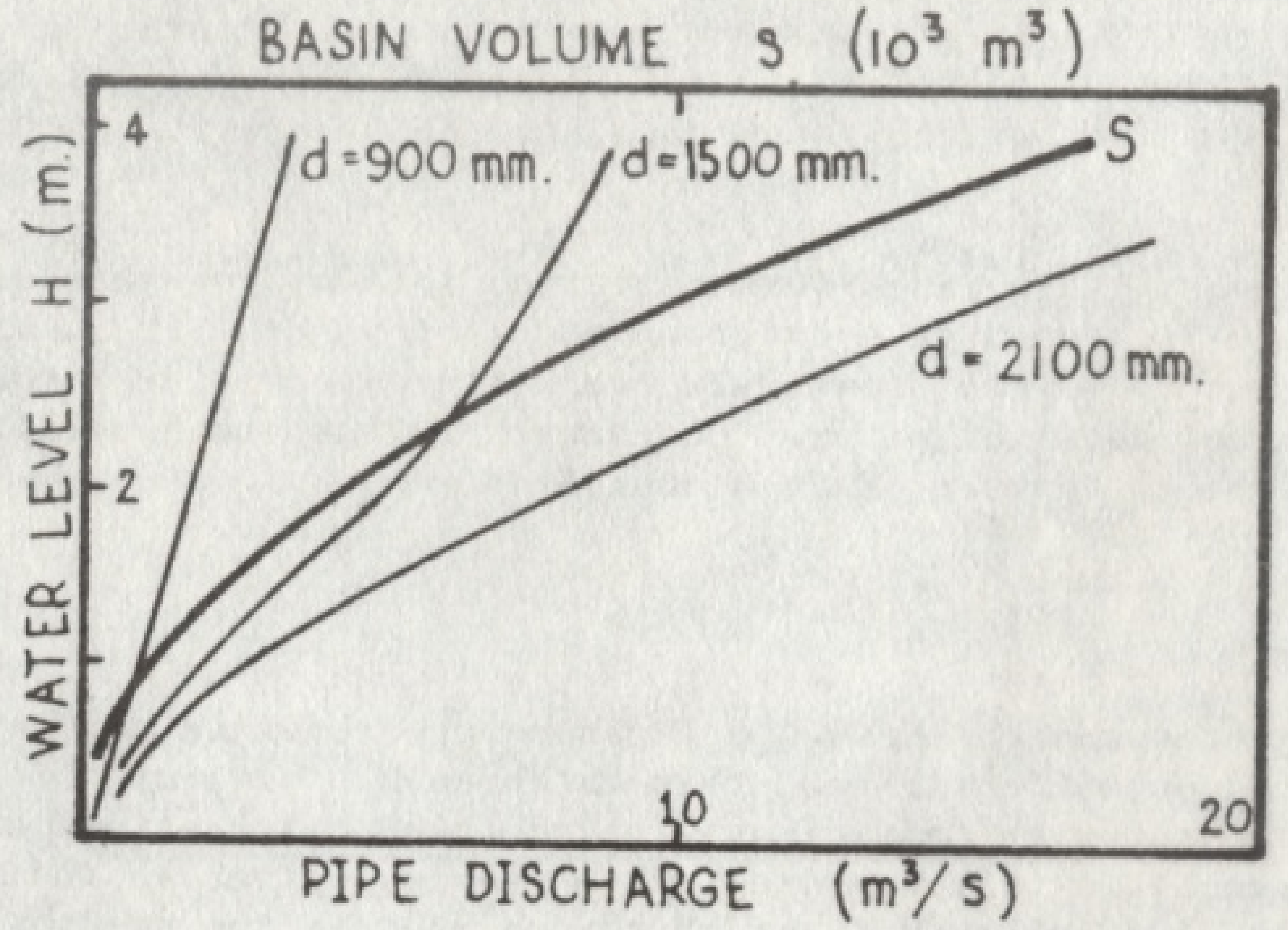

Fig. 3. Detention Basin Storage and Discharge Relations

$$q_2 = \frac{q_1\ (2K_1 - \Delta t) + (i_1 + i_2)\ \Delta t}{2K_2 + \Delta t} \qquad (4)$$

where

i = inflow rate to basin (m^3/s)
q = outflow rate from basin (m^3/s)
Δt = time period used in routing (sec)
K = ratio of basin storage volume to pipe discharge

and subscripts 1, 2 denote respectively the start and end of a routing period. Equation 4 was solved iteratively by a computer.

ACCURACY OF PRELIMINARY DESIGN PROCEDURES

Results of Present Study

Figures 4a to 4f show Eqs. 1, 2 and 3 plotted in dimensionless form, together with plotted points representing the results of reservoir routing for the present and other studies. The plotted points represent results for all combinations of watershed area A, recurrence interval Y and pipe

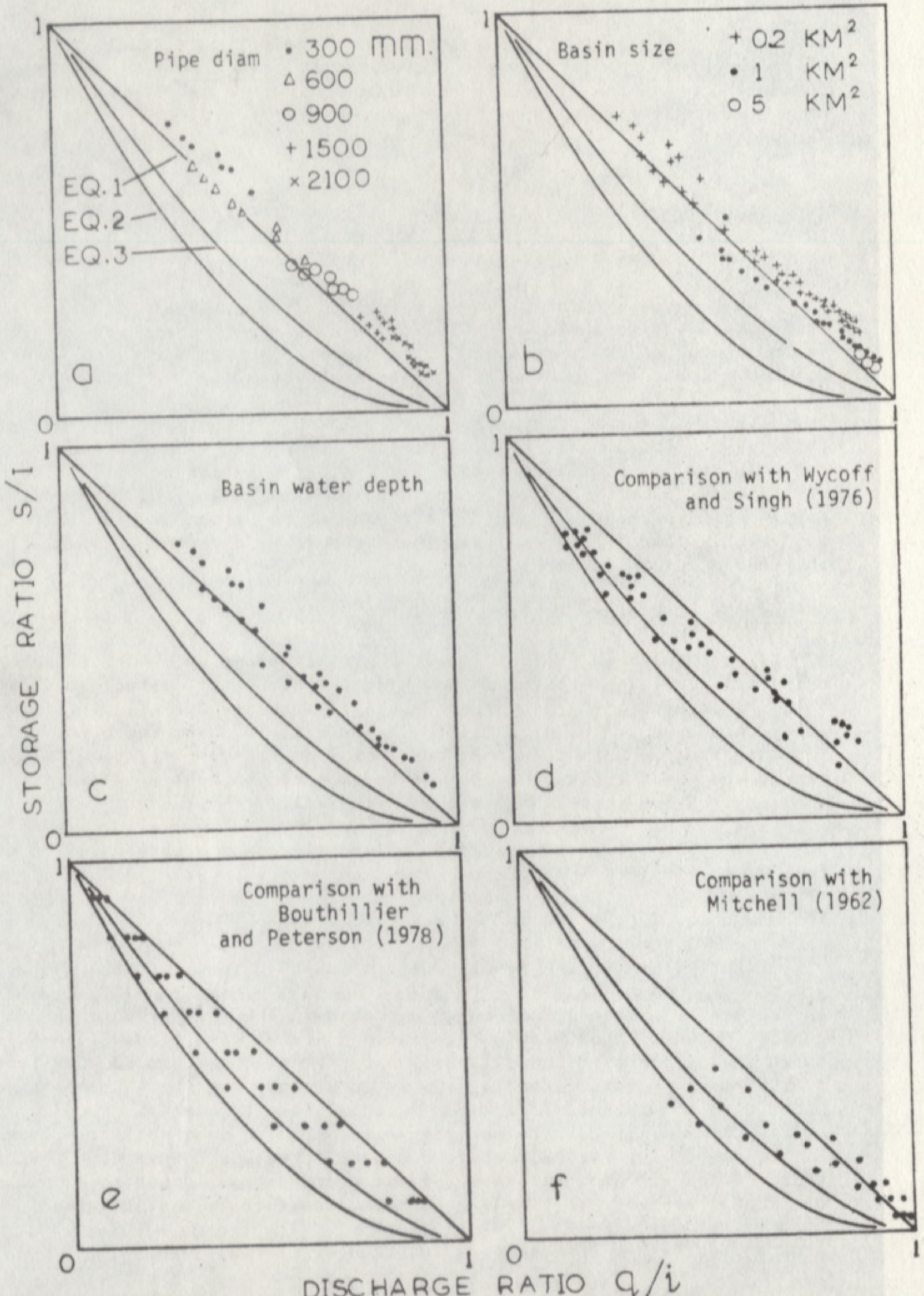

Fig. 4. Results of Detention Basin Routing

diameter d. For each of these combinations only the critical duration storm was used, ensuring that the hydrographs have the realistic shape of design hydrographs. As mentioned previously, those results which had an unrealistically high value of the maximum water depth, $H > 5.0$ m, were excluded.

Figure 4a shows that the pipe diameter has little effect on the lateral location of the points, but shifts them towards the upper or lower range of points. This occurs because small pipe diameters give a greatly reduced outflow peak q, requiring a large volume of basin storage.

Figure 4b shows the effect of watershed area A. Again, the location of the plotted points is not greatly affected. Larger watersheds have larger inflow floods and will require larger diameter outlet pipes (to ensure $H \ngtr 5.0$ m), and thus plot at the upper range of q/i.

Figure 4c shows points for which $2\,d < H < 5.0$ m, and thus excludes those events which have unrealistically low or high values of maximum water depth in the basin. Figures 4a-c all indicate similar results. The effects of the pipe diameter and catchment area are small, and thus the maximum water depth in the basin and the size and shape of inflow hydrograph do not greatly affect the results. These results indicate the Eq. 1. gives the most accurate estimtes.

Comparison with Other Published Results

Figure 4d shows the results of a similar study by Wycoff and Singh (1976) using 50 selected synthetic inflow hydrographs. They fitted a modification of Eq. 1 incorporating a hydrograph shape parameter and allowing for powers other than unity. The data fit Eq. 1 for $q/i > 0.5$ and Eq. 3 for $q/i < 0.5$. Since in the present study only design inflow floods of critical storm duration are used, the hydrograph can be expected to have realistic shapes, and a hydrograph shape parameter is not used.

Figure 4e shows the results of Bouthillier and Peterson (1978) using four adopted inflow hydrograph shapes, three being early, mid and late peaking triangular inflows and the fourth being a cusp shape inflow hydrograph. The points lie in the region between Eqs. 1 and 3.

Figure 4f shows the results of a study by Mitchell (1962) using 16 inflow floods routed through several small reservoirs formed by road embankments and culverts. Mitchell presented his data in the form of various dimensionless ratios; however, these can be rearraged in terms of q/i and S/I. Figure 4f shows the data from Mitchell's Fig. 5 for storm durations ranging from 0.5 to 1.5 times the watershed lag time. In the present study the critical storm duration is seen from Table 3 to be close to the watershed lag time. The points fit Eqs. 1 and 3 reasonably well, although some scatter occures. This may be due to the use of a range of storm durations rather than the critical storm duration. Another source of difference occurs in Mitchell's assumption of linear reservoir routing procedure, whereas it is actually nonlinear.

Accuracy of Design Procedures

The results of this and other three studies quoted indicate that Eq. 1 gives generally the best fit to data from complete design procedures using reservoir routing, followed by Eq. 3; Eq. 2 gives the poorest fit. Equation 1 is more conservative, requiring a slightly larger basin storage volume S to reduce the outflow flood peak to a given q. Of the four studies quoted, only the present study used realistic inflow flood hydrographs resulting from critical duration storms, hence these results are more likely to be applicable to the design case. From the results presented, Eq. 1 tends to underestimate slightly the value of basin storage volume S calculated using reservoir routing by approximately 5%, while Eqs. 2 and 3 underestimate by approximately 50% and 30%, respectively.

CONCLUSIONS

The accuracy of three preliminary design procedures for detention basins has been assessed by comparing predicted results with a complete design using reservoir routing of an inflow flood through the basin. A wide range of watershed sizes, storm recurrence intervals and durations, and basin outlet pipe diameters was used. The results indicate that a simple dimensionless relation (Eq. 1) based on triangular inflow and outflow hydrographs gives the most accurate results. This conclusion is generally supported by the results of three independent studies using various inflow hydrographs.

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