Site," *Engineering Research Bulletin B-108*, The Pennsylvania State University, University Park, Penn., Jan., 1973.
13. Weber, W. J., Jr., *Physiochemical Processes for Water Quality Control*, Wiley-Interscience, New York, N.Y., 1972.
14. Williams, J. R., and Berndt, H. D., "Sediment Yield Computed with Universal Equation," *Journal of the Hydraulics Division*, ASCE, Vol. 98, No. HY12, Proc. Paper 9426, Dec., 1972, pp. 2087-2098.
15. Wischmeier, W. H., and Smith, D. D., "Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains," *Agriculture Handbook No. 282*, United States Department of Agriculture, Agricultural Research Service, May, 1965.

**12938 DESIGN OF STORMWATER DETENTION BASINS**

KEY WORDS: **Detention reservoirs;** Erosion; **Flood routing;** Hydrology; **Runoff; Sediment control; Storm water;** Urban development; Water management (applied)

ABSTRACT: In addition to increased flood runoff, urban development has caused a significant increase in sediment loads in streams. While many means of sediment and runoff control have been proposed, stormwater detention has been shown to be one of the more cost efficient means. Because detention facilities have not been used extensively in the past, a data base is not available for determining the effect of design factors on sediment trap efficiency and runoff control characteristics. A mathematical model, which includes erosion, sedimentation, and detention facility components, was developed from principles of hydraulics and classical settling mechanics. The model was used to examine the effect of: (1)Detention basin location; (2)soil particle size distribution; (3)basin depth; (4)initial storage, and (5)orifice diameter. An understanding of the relative importance of these factors may lead to better design of stormwater detention facilities.

REFERENCE: Curtis, David C., and McCuen, Richard H., "Design Efficiencies of Stormwater Detention Basins," *Journal of the Water Resources Planning and Management Division*, ASCE, Vol. 103, No. WR1, **Proc. Paper 12938**, May, 1977, pp. 125-140



JOURNAL OF THE WATER RESOURCES PLANNING AND MANAGEMENT DIVISION

# Design Efficiency of Stormwater Detention Basins

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## Introduction

Millions of dollars are expended annually in the design, construction, and maintenance of facilities to control urban runoff. Consequently, it is important that the effects of urbanization on watershed response be researched and analyzed. As research information becomes available, appropriate policies can be defined to help assure that adequate drainage facilities are provided to the public at lowest possible cost. Many state and local governments have already acknowledged urban runoff problems by introducing various forms of legislation designed to achieve stormwater management on a watershed basis. For example, the State of Maryland established justification for the control of increased runoff from developing areas through an interpretation of the existing Maryland Sediment Control Law. The interpretation put forth by the Attorney General of Maryland in 1971 stated that the Maryland Sediment Control Law included the control of storm water to prevent off-site erosion and subsequent downstream sedimentation resulting from increased runoff generated by development (3). During the years since implementation of Maryland sediment and stormwater ordinances, engineers have utilized a wide variety of techniques to control sediment and stormwater discharges from developing areas. A sample of these control methods includes the construction of terraces, checkdams, dikes, diversion ditches, and detention basins. Soil stabilization methods such as the use of natural or artificial mulches and reestablishment of vegetation are also common practices.

Note.—Discussion open until October 1, 1977. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Water Resources Planning and Management Division, Proceedings of the American Society of Civil Engineers, Vol. 103, No. WR1, May, 1977. Manuscript was submitted for review for possible publication on August 3, 1976.

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The most popular on-site control technique is the detention basin. Detention of storm water generally refers to storage of runoff prior to its release into the drainage system. "Detained runoff" is released at a controlled rate, such that the peak flow rates entering downstream conveyance systems are less than those which would have occurred without detention. In many cases, storage is accomplished using flood storage available in existing ponds and lakes or by excavating new detention sites designed to capture the excess runoff. The basins may be designed to control storm water only or to control sediment as well, or both of these. Detention basins may either be temporary structures designed to survive only the duration of construction activity, or permanent structures designed for use both during construction and upon project completion, even serving various recreational purposes.

As the number of detention basins in operation increases, several important questions arise, e.g.: (1) What is the effect of detention basins on the hydrologic response of a watershed? (2) How effective are detention basins in the removal of sediment? (3) What are the important design parameters that affect detention basin performance? and, most importantly, (4) How sensitive is watershed response to detention facility design?

In an attempt to answer the preceding questions, a mathematical model of an on-site detention basin was developed and is presented herein. Used in conjunction with a hydrologic model to simulate watershed response, the detention model will enable a researcher to study the effectiveness of detention basins under a wide variety of conditions. Similarly, the model will enable study of watershed response sensitivity to various design parameters of detention facilities.

## Model Formulation

The detention basin model is assumed to be rectangular with sloping sides and horizontal bottom. Outlet structures chosen for the simulated basin were based on observations of detention facilities currently in operation. Often, a vertical corrugated metal pipe riser and a grass-lined trapezoidal emergency spillway are used to control flow from the detention basin (Fig. 1). In practice, the riser pipes generally have a number of perforations along their heights to enable controlled drawdown to a desired pool level without manual assistance. To simulate the operation of detention basins for control of storm water and sediment discharge, three major components were required. First, a procedure was required to simulate soil erosion resulting from a rainfall stimulus to provide sediment inflow to the detention basin. Then a method was needed for estimating the amount of sediment detained by the basin. Finally, a routine was necessary to route storm water through a detention structure. In addition to these components, a linked-system hydrograph simulation (LSHS) model was utilized to develop the appropriate inflow hydrographs.

**Erosion.**—The most common technique for estimation of erosion or soil loss is the Universal Soil Loss (USL) equation (15). The USL equation, the result of a statistical analysis of many years of test plot data, is

$$A = RKLSCP \quad (1)$$

in which $A$ = soil loss, in tons per acre; $R$ = rainfall erosivity index; $K$ = soil erodibility factor; $L$ = slope length factor; $S$ = slope gradient factor; $C$



area available for flow, and a change in vertical spacing varies the effect of hydraulic head at each orifice.

## Summary

The erosion, sedimentation, and detention components presented herein have been shown to be flexible and capable of reflecting a wide variety of design conditions. The components in conjunction with the LSHS model will enable engineers, planners, and conservationists to evaluate the performance of detention basins in controlling sediment and storm water discharges. Also, the components can be used to identify the relative importance of various detention basin design parameters. In this manner, policies controlling the use of detention basins may be devised such that the impact of urbanization on the environment may be minimized.

## Acknowledgments

The research was supported in part by funds provided by the U.S. Department of the Interior, Office of Water Research and Technology, as authorized under the Water Resources Act of 1964. The project was administered by the Water Resources Research Institute, University of Maryland. Computer funds were provided by the Computer Science Center, University of Maryland, College Park, Md.

## Appendix.—References

1. Brune, M., "Trap Efficiency of Reservoirs," *Transactions of the American Geophysical Union*, Vol. 34, No. 3, June, 1953.
2. Camp, T. R., "Sedimentation and the Design of Settling Tanks," *Proceedings*, ASCE, Vol. 71, Paper No. 4, Apr., 1945, pp. 445-486.
3. Cordero, C. T., "Management of Runoff from Minor Storms in Montgomery County, Maryland," *Paper No. 72-714*, American Society of Agricultural Engineers, St. Joseph, Mo., 1972.
4. Fair, G. M., Geyer, J. C., and Okun, D. A., *Water and Waste Water Engineering*, John Wiley and Sons, Inc., New York, N.Y., 1968.
5. McCuen, R. H., "A Regional Approach to Urban Storm Water Detention," *Geophysical Research Letters*, Vol. 1, No. 7, Nov., 1974.
6. Meyer, L. D., and Wischmeier, W. H., "Mathematical Simulation of the Process of Soil Erosion by Water," *Transactions of the American Society of Agricultural Engineers*, Vol. 12, No. 6, Dec., 1969.
7. Poertner, H. G., Practices in Detention of Urban Storm Water Runoff, *Special Report No. 43*, American Public Works Association, Chicago, Ill., June, 1974.
8. *Rainfall Frequency Atlas of the United States, Weather Bureau Technical Paper No. 40*, United States Department of Commerce, Washington, D.C., 1961.
9. Rendon-Herrero, O., "Estimation of Washload Produced on Certain Small Watersheds," *Journal of the Hydraulics Division*, ASCE, Vol. 100, No. HY7, Proc. Paper 10638, July, 1974, pp. 835-848.
10. Roth, C. B., et al., "Prediction of Subsoil Erodibility Using Chemical, Mineralogical, and Physical Parameters," *Project No. 15030HIX*, Office of Research and Development, United States Environmental Protection Agency, Washington, D.C., 1974.
11. *Standards and Specifications for Erosion and Sediment Control in Urbanizing Areas*, United States Department of Agriculture, Soil Conservation Service, College Park, Md., Nov., 1969.
12. Swerdon, P. M., and Kountz, R. R., "Sediment Runoff Control at Highway Construction

for storm water entering a basin with no initial storage are greatly increased and sedimentation is improved. Also, if an initial storage exists in the basin, flood storage is sacrificed. This could be a significant problem if a sufficient amount of time does not pass to allow discharge or evaporation of detained water from one storm before a second storm occurs.

**Effect of Orifice Diameter.**—The effect of various orifice diameters on peak discharge and on trap efficiency is shown in Table 6. When the orifice diameter was increased from 0.5 in. (13 mm) to 2.0 in. (51 mm), peak discharge increased

**TABLE 5.—Effect of Initial Storage on Trap Efficiencies**

| Basin depth, in feet (1) | Orifice diameter, in inches (2) | Initial depth, in feet (3) | Peak flow, in cubic feet per second (4) | Trap Efficiencies,[a] as a percentage | | | |
|---|---|---|---|---|---|---|---|
| | | | | Over-all (5) | Basin 1 (6) | Basin 2 (7) | Basin 3 (8) |
| 10 | 1.5 | 0.00 | 0.54 | 83 | 85 | 93 | 73 |
| 10 | 1.5 | 2.16 | 1.62 | 52 | 50 | 66 | 42 |
| 10 | 1.0 | 0.00 | 0.46 | 86 | 88 | 97 | 78 |
| 10 | 1.0 | 2.16 | 1.5 | 61 | 60 | 75 | 52 |

[a]Particle size distribution $A$.
Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 cfs = 0.028 $m^3/s$.

**TABLE 6.—Effect of Orifice Diameter on Trap Efficiencies with Initial Storage = 0.0**

| Basin depth, in feet (1) | Riser height, in feet (2) | Orifice diameter, in inches (3) | Peak flow, in cubic feet per second (4) | Trap Efficiencies, as a percentage | | | |
|---|---|---|---|---|---|---|---|
| | | | | Over-all (5) | Basin 1 (6) | Basin 2 (7) | Basin 3 (8) |
| 8 | 5 | 2.0 | 0.82 | 83 | 87 | 94 | 73 |
| 8 | 5 | 1.5 | 0.74 | 85 | 88 | 96 | 76 |
| 8 | 5 | 1.0 | 0.72 | 88 | 92 | 97 | 81 |
| 8 | 5 | 0.5 | 0.69 | 93 | 96 | 99 | 88 |

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 cfs = 0.028 $m^3/s$.

18%, and overall trap efficiency decreased 11%. The reduction in trap efficiency was expected because, as the orifice diameters increase, flow rates increase and the length of time that the sediment-laden storm water remains in the basin decreases. Reduced detention times result in decreased trap efficiencies, as individual particles have less opportunity to settle out. Changing the orifice diameter, in effect, changes the total cross-sectional area available for orifice flow. The same effect can be obtained by altering the number and spacing of orifices on the riser. An increase in the number of orifices increases total



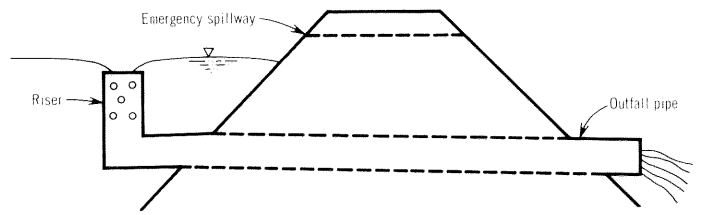


**FIG. 1.—Detention Basin Configuration**

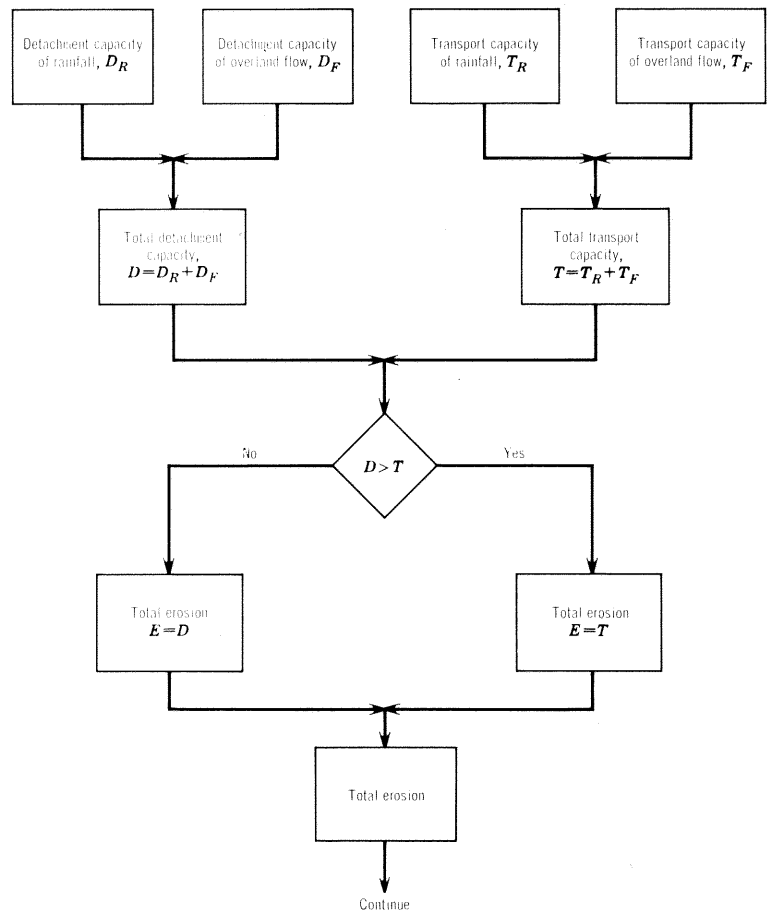


**FIG. 2.—Erosion Component Solution Procedure**

= crop management factor; and $P$ = conservation practice factor. The USL equation has been applied not only on individual fields (15) but on whole watersheds (9) and construction areas as well (10,12). However, use of the USL equation yields only one singular time-invariant estimate of soil loss. A more dynamic approach was required if the erosion component was to be used with a LSHS model. The amount of eroded soil should reflect watershed conditions throughout the duration of a storm event.

A method for estimating soil erosion, readily adaptable to the LSHS model, was proposed by Meyer and Wischmeier in 1969 (6). Essentially, Meyer and Wischmeier developed a conceptual model based on Ellison's description of the soil erosion process (6). Ellison suggested that erosion is composed of the processes of soil detachment and soil transport by erosive agents. Meyer and Wischmeier reported mathematical relationships that reflect: (1) Soil detachment by rainfall; (2) soil transport by rainfall; (3) soil detachment by runoff; and (4) soil transport by runoff. In summary, these mathematical relationships are given by Eqs. 2-5. Rainfall detachment capacity is estimated by

$$D_R = S_{DR} A_S I^2 \quad (2)$$

in which $S_{DR}$ = rainfall detachment factor (a constant parameter reflecting soil type and land surface conditions); $A_S$ = land surface area; and $I$ = rainfall intensity. The overland flow detachment capacity is estimated by

$$D_F = S_{DF} S^{2/3} Q^{2/3} A_S \quad (3)$$

in which $S$ = slope in the direction of flow; $Q$ = volume rate of overland flow; and $S_{DF}$ = flow detachment factor (a constant introduced to reflect soil types and surface conditions). The rainfall transport capacity is estimated by

$$T_R = S_{TR} SI \quad (4)$$

in which $S_{TR}$ = rainfall transport factor (a constant introduced to reflect soil and cover conditions). The overland flow transport capacity is estimated by

$$T_F = S_{TF} S^{5/3} Q^{5/3} \quad (5)$$

in which $S_{TF}$ = overland flow transport factor (a constant introduced to reflect soil and cover conditions).

The solution procedure for the determination of total soil erosion is represented by the flow diagram in Fig. 2. Total detachment capacity is determined by summing detachment capacities due to rainfall and overland flow. Total transport capacity is found by summing transport capacities due to rainfall and overland flow. It is assumed that if total transport capacity is greater than total detachment capacity (i.e., total amount of soil available for transport), then total amount of soil eroded in a particular time period is equal to total detachment capacity. Conversely, if total detachment capacity exceeds ability of rainfall and overland flow to transport sediment, then total erosion is just equal to the transport capacity.

**Erosion Component Formation.**—In their present form, Eqs. 2, 3, 4, and 5 are not compatible with the LSHS model used in this study. The flow rate, $Q$, in Eqs. 3 and 5 is the volume rate of overland flow. The LSHS model determines overland flow as a uniform depth over an entire subcatchment during one time increment, $\Delta t$. Thus, the flow rate, $Q$, must be expressed as depth



settle at a faster rate (particle density remaining constant), and therefore, more particles in distribution $B$ were able to settle out.

It is apparent that the effectiveness of a particular detention basin in the removal of suspended sediments depends on the shape of particle size and weight distribution. If a detention basin is located in an area where eroding soil is composed of a high portion of large heavy particles, trap efficiencies will be correspondingly high. Conversely, if the incoming particle distribution includes smaller and lighter particles, trap efficiencies will generally be smaller.

**Effect of Basin Depth.**—Table 4 shows the effect of basin depth on trap efficiencies. The design volume of 0.5 in./acre $3.14 \times 10^{-3}$ mm/m$^2$ of drainage area for each basin remained constant as the depth varied. To maintain a constant volume at various basin depths, length and width dimensions were adjusted accordingly (length was always assumed to be twice the basin width). However, initial ponded depth varied because riser height also changed with depth. The elevation of the lowest orifice was approx one-third of riser height.

Results indicate that trap efficiency varies significantly with basin depth. As total basin depth was decreased from 15 ft (4.6 m) to 7 ft (2.1 m), the overall trap efficiency increased from 54%-75%. Increased trap efficiencies observed for shallower basins result in part from lower discharge rates and from increased surface area of the basins. At shallower depths, lower discharges, resulting both from fewer orifices available on a shorter riser to convey flow and from a lower hydraulic head at each orifice, effectively increase basin detention times. Longer detention times enable entrapment of a greater number of sediment particles. Increased surface area results in greater horizontal travel distance and decreased vertical settling distance for sediment particles reaching the basin.

Results in Table 4 also indicate that, even though each basin located in subcatchments 1, 2, and 3 was designed using identical design criteria, vastly different sedimentation performances can occur. For example, when design depth was 11 ft (3.4 m), basins located in subcatchments 1, 2, and 3 obtained trap efficiencies of 62%, 77%, and 54%, respectively. Due to variation in the rainfall-runoff properties of each subcatchment, the ratio of runoff volume to available detention storage volume varies. Thus, if hydraulic performance of each subcatchment varies, then sedimentation performance must also vary between individual basins.

**Effect of Initial Storage.**—Standards suggested for detention basin design (11) state that the top one-half to two-thirds of risers in basins used to trap sediment should be perforated. If no other means of drainage is provided, a permanent pool will will be maintained in the basin at a depth equal to the height of the lowest riser orifice. To show the effect of an initial ponded depth on trap efficiencies and on peak watershed discharge, basin performance was simulated for two cases. First, no initial ponded depth was considered and second, an initial ponded depth of 2.16 ft (0.7 m) was specified. (Table 5.) Initial ponded depth reduced available stormwater storage capacity and is reflected by higher peak discharges. Increasing initial depth from 0.0 ft-2.16 ft (0.7 m) reduced available storage by about 30% and increased peak discharge by 200%, with an orifice diameter of 1.5 in. (38 mm).

When no initial pool is provided, all inflow was detained until the elevation of the lowest orifice was reached [i.e., 2.16 ft (0.7 m)]. If an initial depth of 2.16 ft exists, outflow begins almost immediately. Thus, detention times

**TABLE 2.—Particle Size Distribution**

| Particle size, in $10^{-6}$ m (1) | Weight fraction distribution $A$ (2) | Weight fraction distribution $B$ (3) |
|---|---|---|
| 1,500 | 0.15 | 0.20 |
| 750 | 0.15 | 0.30 |
| 375 | 0.20 | 0.15 |
| 137 | 0.25 | 0.10 |
| 94 | 0.10 | 0.10 |
| 46 | 0.05 | 0.05 |
| 24 | 0.05 | 0.05 |
| 12 | 0.03 | 0.03 |
| 6 | 0.01 | 0.01 |
| 1 | 0.01 | 0.01 |
| Total | 1.00 | 1.00 |

**TABLE 3.—Effect of Particle Size on Trap Efficiencies[a]**

| Riser height, in feet (1) | Orifice diameter, in inches (2) | Weight distribution (3) | Trap Efficiencies, as a percentage | | | |
|---|---|---|---|---|---|---|
| | | | Overall (4) | Basin 1 (5) | Basin 2 (6) | Basin 3 (7) |
| 8 | 1.0 | $A$ | 63 | 62 | 77 | 54 |
| 8 | 1.0 | $B$ | 71 | 71 | 81 | 65 |

[a]Initial depth = 2.67 ft (0.814 m).
Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.

**TABLE 4.—Effect of Basin Depth on Trap Efficiencies**

| Total depth, in feet (1) | Riser height, in feet (2) | Initial depth, in feet (3) | Peak flow,[a] in cubic feet per second (4) | Trap Efficiencies,[b] as a percentage | | | | Change in overall trap efficiencies, as a percentage (9) |
|---|---|---|---|---|---|---|---|---|
| | | | | Overall (5) | Basin 1 (6) | Basin 2 (7) | Basin 3 (8) | |
| 15 | 12 | 4.00 | 1.02 | 54 | 52 | 70 | 45 | — |
| 13 | 10 | 3.33 | 1.01 | 58 | 56 | 74 | 49 | +7 |
| 11 | 8 | 2.67 | 1.00 | 63 | 62 | 77 | 54 | +17 |
| 9 | 6 | 2.00 | 0.97 | 67 | 66 | 79 | 58 | +24 |
| 7 | 4 | 1.33 | 0.92 | 75 | 75 | 87 | 67 | +39 |

[a]Orifice diameter = 1.00 in. (25 mm).
[b]Particle size distribution $A$.
Note: 1 ft = 0.305 m; 1 cfs = 0.028 $m^3/s$.



of flow. Combining Manning's equation and the continuity equation yields

$$Q = CA_x S^{1/2} R^{2/3} \quad (6)$$

in which $A_x$ = cross-sectional area of flow; $S$ = channel slope; and $R$ = hydraulic radius. The coefficient, $C$, is a constant equal to $1.49/n$, in which $n$ = Manning's roughness coefficient. If the flow width is assumed constant, $A_x$ may be expressed as the product of the flow width, $w$, and the flow depth, $d$. By the same assumption, the hydraulic radius can be expressed as the depth of flow, $d$. Thus, Eq. 6 becomes

$$Q = CS^{1/2} wd^{5/3} \quad (7)$$

Substitution of Eq. 7 into Eq. 3 yields

$$D_F = S^*_{DF} w^{2/3} d^{10/9} SA_S \quad (8)$$

in which $S^*_{DF} = S_{DF} C$. If the flow plane is assumed rectangular (i.e., subcatchment is considered as an equivalent rectangle), the product of the flow width, $w$, and the flow length, $L$, yields the flow surface area, $A_S$. Substituting $wL$ into Eq. 8 for $A_S$ produces:

$$D_F = S^*_{DF} Lw^{5/3} d^{10/9} \quad (9)$$

By similar reasoning and development, Eq. 5 was modified to yield

$$T_F = S^*_{TF} w^{5/3} s^{15/6} d^{25/9} \quad (10)$$

An algorithm was written to solve Eqs. 2, 4, 9, and 10 according to the erosion solution procedure outlined in Fig. 2. The solution estimates total soil loss from a subcatchment during one time increment, $\Delta t$.

**Sedimentation.**—Previously, computational techniques for estimation of sediment removal have been developed by two separate fields. Sanitary engineers have utilized classical settling mechanics in the estimation of the removal of discrete particles in settling tanks for water treatment facilities (4,13). Reservoir engineers have utilized empirical relations to estimate sedimentation in large reservoirs (1). The use of detention basins as sediment traps, however, does not fall distinctly within the realm of either reservoir or sanitary engineering, and in fact has characteristics reflecting both disciplines. Detention basins are generally not well defined hydraulic structures, as might be found in a water treatment plant. Rather, these basins are often irregular in shape, as are the much larger reservoirs. However, due to the relatively small size of on-site detention basins, operation of detention basins may be similar to that of treatment plant sedimentation tanks. Thus, classical theories of settling particles developed for treatment plant design have been applied to the analysis of sedimentation in on-site detention basins (7).

To develop a settling component for use in an LSHS model, Camp's (2) approach to clarification theory for an ideal basin was used. Camp defined an ideal basin as a hypothetical settling tank in which settling takes place in exactly the same manner as in a quiescent settling container of the same depth. The advantages to this approach are that mathematical expressions are straightforward and solutions can be quickly obtained. The use of techniques accounting for turbulence, flocculation, short-circuiting, etc., would render solutions more difficult and time-consuming to obtain.

**Sedimentation Mechanics.**—Discrete nonflocculating particles that settle without hindrance from other particles can be described by the equations of classical mechanics. Settling of particles under these conditions is essentially a function of properties of the surrounding fluid and particle characteristics. The principal equation resulting from the classical approach, commonly known as Stoke's Law, describes the settling velocity of a discrete particle. If settling velocity of a particle is assumed to be sufficiently low, such that fluid flowing past the particle is within the laminar regime, and if the particle is assumed spherical, Stoke's Law has the form

$$v_t = \frac{g}{18} \frac{\rho_s - \rho_f}{\rho_f} \frac{d_p^2}{\nu} \quad (11)$$

in which $v_t$ = settling velocity; $d_p$ = particle diameter, $\nu$ = kinematic viscosity; $\rho_s$ = particle density; $\rho_f$ = fluid density, and $g$ = gravitational constant.

In order to utilize relationships for settling of discrete particles in continuous-flow sediment basins as developed by Camp (2), the following assumptions must be observed: (1) Particles and velocity vectors are distributed equally over the basin cross section; (2) the liquid moves as an ideal slug; and (3) any particle striking the bottom is effectively removed. A discrete particle settling in a rectangular horizontal flow basin follows a path that is the vector sum of its own velocity, $v_t$, and the velocity of the surrounding fluid, $v_d$ (2). From Fig. 3, it is clear that all particles settling with a velocity, $v_t$, greater than the velocity, $v_o$, will be removed. The parameter, $v_o$, can be defined as

$$v_o = \frac{h_o}{t_d} \quad (12)$$

in which $v_o$ = velocity of a particle settling height, $h_o$, during basin detention time, $t_d$. Basin detention time is defined as

$$t_d = \frac{C_d}{Q_d} \quad (13)$$

in which $C_d$ = volumetric capacity of basin; and $Q_d$ = volumetric flow rate through the basin.

It was assumed that the particles are distributed evenly over the basin cross section. Thus, a portion of the particles with settling velocities, $v_t$, less than $v_o$ is within a vertical distance $(h_o - t_d v_t)$ from the bottom and would be removed. If $X_o$ particles possessing a settling velocity $v_t$ that is less than or equal to $v_o$ compose a particle size fraction, the proportion $X/X_o$ of particles to be removed is

$$\frac{X}{X_o} = \frac{h}{h_o} = \frac{v_t t_d}{v_o t_d} = \frac{v_t}{v_o} \quad (14)$$

The size-weight composition of a particular suspension can be expressed by a frequency distribution as shown in Fig. 4 (2,4,13). From Fig. 4 it is evident that a proportion $(1 - f_o)$ of the particles with $v_t \geq v_o$ are totally removed. The proportion of the remaining particles with velocities $v_t < v_o$ that are removed can be determined from the cumulative frequency curve. Eq. 14 gives the portion



size distribution on detention basin performance, two different size distributions were tested. The respective distributions are shown in Table 2. Weight of sediment

**TABLE 1.—Effect of Detention Basin Location**

| Detention location (1) | Peak discharge, in cubic feet per second (2) | Time-to-peak, in minutes (3) | Reduction of peak flow, as a percentage (4) |
|---|---|---|---|
| No detention | 13.7 | 215 | — |
| Subwatershed | | | |
| 1 | 8.9 | 225 | 34 |
| 2 | 10.8 | 208 | 21 |
| 3 | 9.8 | 210 | 28 |
| 1,2 | 4.5 | 221 | 67 |
| 1,3 | 4.9 | 230 | 64 |
| 2,3 | 7.5 | 204 | 45 |
| 1,2,3 | 1.0 | 272 | 93 |

Note: Orifice diameter = 1.00 in. (25 mm); riser height = 10.0 ft (3 m); initial depth = 3.33 ft (1 m); 1 cfs = 0.028 $m^3/s$.

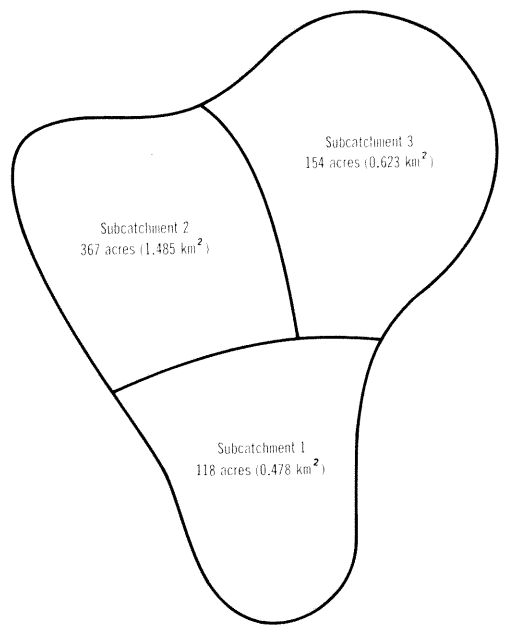


**FIG. 5.—Configuration of Manor Run Watershed**

estimated for each particle size is expressed as a decimal fraction of total sediment weight. Results, shown in Table 3, indicate that trap efficiency for distribution *B* was significantly higher than that for distribution *A*. The larger particles

To estimate the effect of detention basins on the stormwater and sediment discharge from Manor Run, design storm hydrographs were simulated using no detention storage, detention in all subcatchments, and detention storage in combinations of one or two subcatchments. The volume of available detention storage and the minimum discharge rates were computed based on local standards and specifications (11). Specifications for detention basins require a minimum design storage of 0.5 in./acre of drainage area. In this study, all detention basin configurations used were designed to store the required volume below the crest of the emergency spillway.

## Results and Analysis

The LSHS model, which included erosion, sedimentation, and detention components, was used to show: (1) Effect of detention basin location on peak watershed discharge and time-to-peak; (2) effect of particle size distribution on trap efficiencies; (3) effect of initial basin storage on trap efficiencies; (4) effect of orifice diameters on peak discharge and trap efficiencies; and (5) effect of basin depth on trap efficiencies.

**Detention Basin Location.**—The effect of detention basin location on watershed discharge was shown by specifying a volume of available storage in each subcatchment. Relative orientation of individual subcatchments is shown in Fig. 5. An initial ponded depth of 3.3 ft (1 m) [i.e., height of lowest orifice location on the 10-ft (3-m) riser] was specified. Table 1 shows the effect on watershed response of various storage combinations. The reduction of peak discharge ranged from 21%-93%, as compared to the peak flows that resulted when no detention was specified. In all cases, sufficient storage was available to significantly delay release of storm water from individual subcatchments and reduce total watershed peak discharge.

Detention storage location also has an important impact on time-to-peak. Times-to-peak resulting from various combinations of storage ranged from 204 min-272 min. With no detention, watershed discharge reached a peak at 215 min. When detention storage is specified in a particular subcatchment, storm water is stored and released at a controlled rate. The storage effectively delays timing and magnitude of peak flows contributing at the watershed outfall from an individual subcatchment.

The model was also used to examine changes in timing of subcatchment contribution to watershed discharge. The variability of peak discharges and times-to-peak with various storage combinations suggests that it may be possible to create higher peak watershed discharges than experienced with no detention. If runoff from lower portions of the watershed are delayed to coincide with arrival of runoff from upper portions, peak flows higher than those for the nodetention condition may result. This tends to support the contention by the second writer (5) that partial detention may, in certain cases, actually increase peak discharges.

**Effect of Particle Size Distribution.**—The trap efficiency of a basin equals the ratio of sediment retained in the basin to the weight of sediment reaching the basin, expressed as a percentage. Similarly, overall trap efficiency of a watershed is total sediment retained by all the detention basins divided by the weight of sediment reaching the basins. To demonstrate the effect of particle



of each particle size fraction with $v_t < v_o$ that is removed; therefore, if $df$ is a particle size fraction with settling velocity, $v_t$, then

$$dr = \frac{v_t}{v_o}\, df \qquad (15)$$

in which $dr$ = portion of particles removed from $df$. Integrating the left-hand

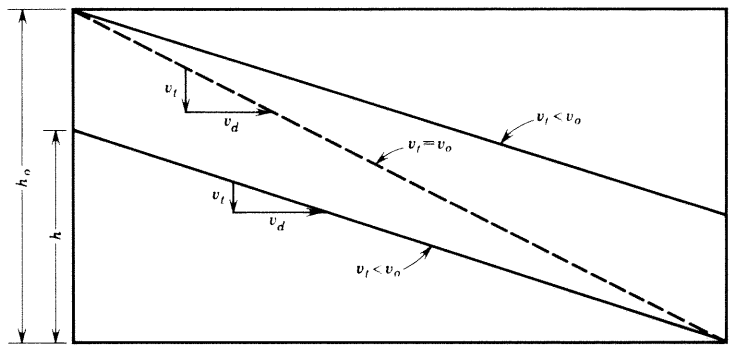


**FIG. 3.—Paths Taken by Discrete Particles**

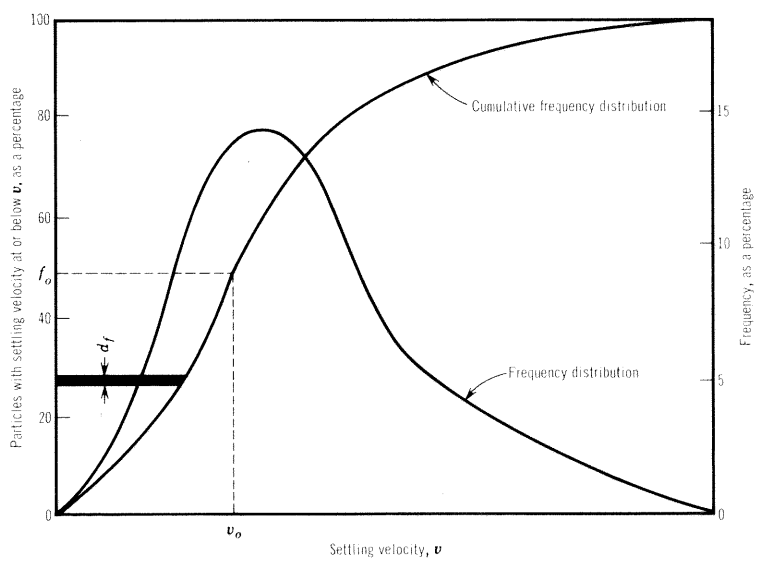


**FIG. 4.—Frequency Distribution of Settling Velocity**

side of Eq. 15 from 0 to $r_o$ and the right-hand side from 0 to $f_o$ yields

$$r_o = \int_o^{f_o} \frac{v_t}{v_o}\, df \qquad (16)$$

Total removal, $R$, is then

$$R = (1 - f_o) + \int_0^{f_o} \frac{v_t}{v_o} df \quad (17)$$

To evaluate Eq. 17, the frequency distribution, the settling velocity of each particle size, and the relationship, $h_o/t_d$, must be known.

**Sediment Component Formation.**—The mechanics of sedimentation, as developed by Camp (2), consider a basin operating under steady-state conditions. Most often, however, a detention basin encounters stormwater and sediment flow rates that vary with time. Thus, values $v_o$ and $t_d$ described by Eqs. 12 and 13 become time-dependent [i.e., $v_o(t)$ and $t_d(t)$]. The efficiency with which sediment is trapped in a basin that operates under unsteady conditions also varies with time. Thus, sediment removal as presented by Eq. 17 is

$$R(t) = [1 - f_o(t)] + \int_o^{f_o(t)} \frac{v_t}{v_o(t)} df \quad (18)$$

To determine total sediment removal in each time increment, $\Delta t$, an algorithm was written utilizing a finite difference form of Eq. 18:

$$R(t) = \sum_{i=m}^{n} \Delta f_i + \frac{1}{v_o} \sum_{j=1}^{m-1} v_{tj} \Delta f_j \quad (19)$$

in which $\Delta f_j$ is the $j$th of $(m - 1)$ fractions of particles with settling velocities $v_t < v_o$; $\Delta f_i$ = $i$th of the $(n - m)$ fractions of particles with settling velocities $v_t \geq v_o$; and $n$ = total number of particle size fractions.

To evaluate Eq. 19, the frequency distribution of settling velocities can be assumed or estimated based on knowledge of soils and ground cover conditions in the watershed. A more appropriate determination of frequency distribution would be an actual analysis of surface runoff and determination of the particle size distribution. From the particle sizes, settling velocities can be calculated by Eq. 11 and frequency distribution of settling velocities can be formulated. It should be obvious that the shape of particle size distribution does not remain constant during a storm event. However, equations predicting such a phenomenon have yet to be adequately defined, and the shape of particle size distribution was assumed constant.

**Detention Component Formulation.**—The procedure for routing a storm hydrograph through a detention basin was based on the Law of Continuity, which states that the rate of change in detention basin storage equals the difference between basin inflow and outflow. If one time period is assumed and a finite difference form is used, the continuity equation can be expressed as

$$\Delta S = I - O \quad (20)$$

in which $S$ = basin storage; $I$ = inflow during one time unit; and $O$ = outflow during one time unit.

In the physical system, inflow and outflow occur continuously over the entire time interval, $\Delta t$. However, to apply a digital computer program, inflow and outflow are assumed to occur in incremental steps. All inflow for the time interval, $\Delta t$, is assumed to occur at the beginning of $\Delta t$, while the outflow is considered to occur at the end of $\Delta t$. In Eq. 20, basin inflow, $I$, was determined by the LSHS model for each time increment. At the beginning of each time period, basin water depth and water surface area were known. Therefore, if $\Delta t$ is small (in this study, $\Delta t$ = 1 min), an incremental depth, $\Delta h$, can be calculated by dividing inflow volume by water surface area. A new water depth was then found by adding $\Delta h$ to depth $h$, at the beginning of $\Delta t$. The total outflow, $O$, was then determined by using appropriate hydraulic relationships for each outlet structure and the basin water depth, $h$.



Storm water could be released from the detention basin configuration used in this study through four outflow devices: (1) Flow through riser perforations; (2) flow over the top edge of riser; (3) flow through an emergency spillway; and (4) basin overflow. Flow through an orifice is described by

$$Q_o = C_o A_o (2gH_o)^{1/2} \quad (21)$$

in which $Q_o$ = flow through one orifice; $A_o$ = orifice area; $H_o$ = effective head; and $C_o$ = discharge coefficient. Flow over the top edge of riser was assumed as flow over a sharp-crested weir and calculated as

$$Q_w = C_w L_w H_w^{3/2} \quad (22)$$

in which $Q_w$ = flow rate; $L_w$ = weir length equal to riser circumference; $H_w$ = effective head; and $C_w$ = discharge coefficient. To circumvent complexities introduced into the hydraulic computations for a trapezoidal emergency spillway, it was determined that no loss in concept would be incurred if emergency flow was assumed to pass through a broad-crested weir with vertical sidewalls. Thus, the function governing flow through the emergency spillway has the form

$$Q_e = C_e B_e H_e^{3/2} \quad (23)$$

in which $Q_e$ = flow rate; $B_e$ = corrected width of emergency spillway; $H_e$ = effective head; and $C_e$ = discharge coefficient. If the maximum basin storage was exceeded, basin excess or overflow, $Q_{ex}$, was calculated as

$$Q_{ex} = \text{(current storage)} + \text{(input volume)} - \text{(maximum storage)} \quad (24)$$

An algorithm was written to calculate incremental changes in basin depth and stormwater discharges through each of the outflow structures during time period $\Delta t$. The algorithm is available from the writers.

## Study Procedure

Data to adequately calibrate a linked-system hydrograph simulation model including a detention component are presently unavailable. Therefore, the model was initially calibrated without the detention component in operation, to observed hydrograph and sediment discharge records for a storm occurring over the Manor Run watershed in Montgomery County, Md., on November 3, 1970. The Manor Run watershed is a developing watershed with a land surface area of approx 1 sq. mile (2.6 km$^2$). Three subcatchments of 118 acres (478,000 m$^2$), 367 acres (1,490,000 m$^2$), and 154 acres (623,000 m$^2$) were delineated. A design storm hyetograph was formulated having a duration of 6 hr and a return period of 2 yr. A 2-yr period was chosen because the 2-yr storm has been commonly used in design of detention basins in Maryland. Total volume of the storm was estimated from Ref. 8.