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Contents

A Storm-Sewer Flow Measurement and Recording System

FREDERICK A. KILPATRICK and WILLIAM R. KAEHRLE

ABSTRACT

A comprehensive study and development of instruments and techniques for measuring all components of flow in a storm-sewer drainage system were undertaken by the U.S. Geological Survey under the sponsorship of FHWA. The study involved laboratory and field calibration and testing of measuring flumes, pipe insert meters, weirs, and electromagnetic velocity meters as well as the development and calibration of pneumatic bubbler and pressure transducer head-measuring systems. Tracer dilution and acoustic-flowmeter measurements were used in field verification tests. A single micrologger was used to record data from all the foregoing instruments as well as from a tipping-bucket rain gauge and also to activate on command the electromagnetic velocity meter and tracer dilution systems. A system was developed and suggested for use in measuring all components of flow in a typical storm-sewer system.

In recent years, with advances in modeling techniques for watershed rainfall runoff, there has been an emphasis on the modeling approach to storm-sewer design. The literature on this subject is full of statements about the need for more and better data bases to aid model development (1-3). For example, it was concluded by the American Society of Civil Engineers in 1965 that a technological hiatus exists largely because of an absence of suitable measuring devices. Progress has been made since then, and it was the objective of this study to evaluate existing devices and techniques as well as to consider new approaches and instruments. These would be tested and evaluated both in the laboratory and the field and an instrumentation package would be proposed that would accurately measure all of the flow components that make up a storm-sewer drainage system.

Figure 1 shows conceptually the total system chosen for the measurement of stormwater drainage. A detailed description of the development and testing of the system is described elsewhere (4); all figures are taken from that report. Head measurements are made by using a pneumatic bubbler with pressure transducers to convert sensed pressure to feet of water. There can be as many orifices and their respective pressure transducers serving as many measuring devices as required. Output from the transducers is sensed by the micrologger and stored if flow or change in head is indicated. Above a given threshold of stage in any of the measuring devices, such as the trunkline, an electromagnetic velocity meter is activated and then deactivated with falling stage. Data from other measuring devices, such as rain gauges, are stored by the micrologger.

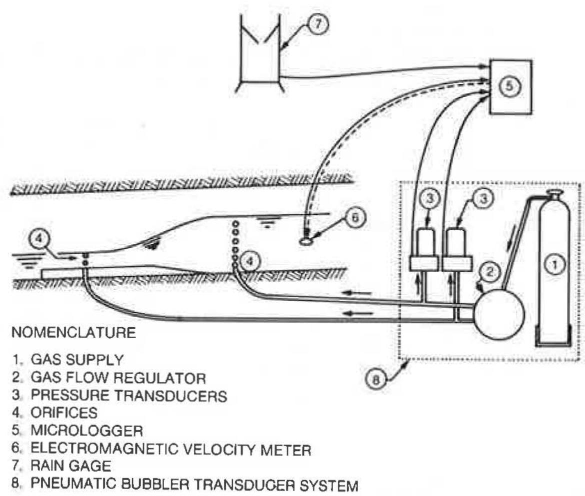

FIGURE 1 Conceptual diagram of storm-sewer flow measurement system.

APPROACH

This study consisted of laboratory testing and calibration followed by field verification of the various instruments and equipment selected for consideration in measuring storm-sewer flows. Laboratory tests were carried out at the U.S. Geological Survey (USGS) Hydrologic Instrumentation Facility located at the National Space Technology Laboratories, Mississippi. This was followed by field testing and verification of the instruments selected and, in some instances, in situ calibration of instruments and structures, which was accomplished at the field site by independently measuring selected flows from a nearby fire hydrant with an acoustic flowmeter. The hydrant flows were then diverted to the desired measuring device. Concurrent with the acoustic-flowmeter measurements, tracer dilution-discharge measurements (5,6) were made of each hydrant flow. This method involved the measurement of the degree of dilution of a known amount of water-soluble tracer following its injection into and mixture with the flow to be measured. The dilution-measuring system can be either manual or automatic (7); the dilution-discharge measurements were performed manually when steady hydrant flows could be utilized. In this study an automatic system also was installed to acquire discharge measurements with the occurrence of storm runoff (8). This system was activated by the micrologger on the basis of water stages in the storm-sewer system.

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Stormwater flows might be expected to be very unsteady, especially if the surface drainage area is small and largely paved. This approach to measurement, if successful, not only might be used to check various measuring devices at flows greater than those that can be obtained from the hydrant, but its possibilities for the measurement of stormwater flows in general might also be investigated. Such an installation offers the possibility of calibrating measuring devices or structures in situ and might also provide directly all the storm-runoff data desired, as has been suggested by Wenzel (9).

The various elements of a storm-drainage system and the approach or instrument selected in this study to perform and record the measurement were as follows:

1. Precipitation: Select a rain gauge for accuracy and compatibility with the data-recording system.
2. Catchment outflows: Design and calibrate a meter to be inserted in catchment outflow pipes.
3. Street-gutter bypass flows
   a. Rate in situ gutter and catchment as a unit,
   b. Design and rate, in situ, a weir for measuring gutter flows.
4. Trunkline flows
   a. Design and calibrate a modified Palmer-Bowlus type flume for both open and pipe-full flow,
   b. Test and determine velocity coefficients for an electromagnetic point-velocity meter installed in a trunkline to measure velocities near and during pipe-full flow conditions.
5. Head measurement system
   a. Combine pressure transducer and pneumatic-bubbler orifice into a working system,
   b. Test and calibrate pressure transducers.
6. Control and recording system: Test and evaluate microloggers to store and interpret data and to activate instruments on programmed command.

RESULTS AND CONCLUSIONS

From the inception of rainfall to flow in roadways and gutters, collection via curb inlets and drop structures or catchments, and final conveyance from the area via lateral and trunkline pipes, the whole gamut of flow hydraulics can occur in a storm-sewer drainage system. In this section, each of the measuring devices or techniques considered part of an overall storm-sewer drainage collection and recording system will be briefly described and the results of laboratory and field tests presented.

Precipitation Measurements

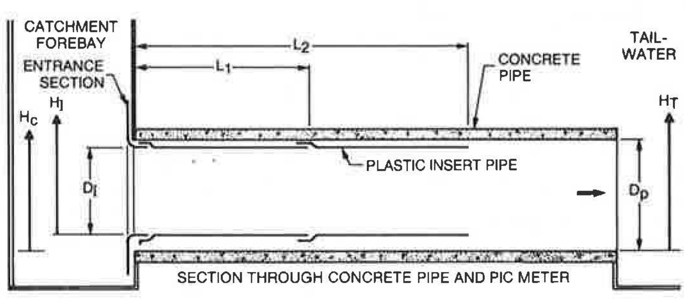
The rapidity of runoff in response to rainfall in predominantly paved areas such as highways makes it vital that the rain gauges being used have rapid responses (10). For this reason various investigators have recommended the use of tipping-bucket rain gauges (11). Furthermore, the tipping feature is also the most suitable for digital recording of rainfall volumes and intensities.

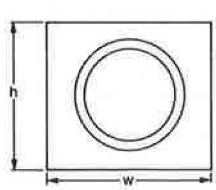
Several manufacturers currently produce tipping-bucket rain gauges, but the market is continuously changing, and final selection should depend on the quality of construction and on laboratory tests of accuracy and responsiveness and compatibility with the data-recording system.

Catchment Outflows

It was found that most pipes draining catchments were seldom larger or smaller than 12 to 24 in. in diameter. Flow in catchments typically is very turbulent, making it difficult to measure head in this location. If head could be measured, it might be possible to rate the existing outlet or calibrate a structure to be placed in the outflow pipe. Their size precluded the ready construction of measuring flumes or other devices in the outflow pipes. For this reason a pipe insert contraction (PIC) meter was designed that could be placed in the outflow pipe away from the catchment (see Figures 2 and 3). Two meters, 10 and 15 in. in diameter, were designed to nest inside 12- and 18-in. concrete pipes, respectively.

Both the 10- and 15-in. PIC meters were successfully calibrated in the laboratory; only a 15-in. one was field tested. Figure 4 shows the calibration curves for the 15-in. PIC meter for both free-surface and pipe-full flow (see Figure 2 for definition of





DIMENSIONS OF PIC METER

TEST METER	DIAMETERS: PLASTIC PIPE, D_p (1)	DIAMETERS: INSERT PIPE, D_I (2)	LENGTHS: ONE INSERT TUBE, L_1 (7)	LENGTHS: TWO INSERT TUBES, L_2 (8)
12 INCH	1.00	0.833	1.67	3.00
18 INCH	1.50	1.250	3.00	5.67

ALL DIMENSIONS ARE IN FEET EXCEPT AS NOTED

FIGURE 2 Sketch and dimensions of PIC meter.

FIGURE 3 Placement of 15-in. insert pipe through 18-in. manhole into catchment; 20-in.-long pipe subsequently turned 90 degrees and inserted into 18-in. sewer line.

terms). Although there is a slight scatter in the data for the different barrel lengths, it is not significant. The flow conditions in the long barrel configuration appeared more stable. The long barrel PIC meter could not be made to flow full except by artificially raising the tailwater in the laboratory. Despite the agreement of the data at low values of $\Delta H_{IT}/D_I$ (ΔH_{IT} is the difference between the head on the insert in the catchment and the tailwater head), the calibration should probably not be used at relative heads less than 0.05 because sizable errors are likely with such small differences (0.062 ft).

To verify the calibration of the 15-in. PIC meter installed at the field test site, eight hydrant discharges were measured through the meter. Figure 4B compares the field rating with the laboratory calibration for the condition of free surface flow below the crown of the PIC meter. As can be seen, there is close agreement between the tracer dilution and acoustic-flowmeter measurements; a curve fitted to both is to the right of the laboratory calibration at lower heads and both converge at higher heads. The discrepancy is considerable at the lower heads but not surprising considering the violent and skewed flow conditions in the catchment. As heads increase, the flow conditions in the catchment smooth out.

The tests point to the need to place any measuring device away from the immediate catchment and to also measure head away from the catchment.

Gutter Bypass Flows

The difficulties of measuring bypass flow in the gutters are primarily due to the adverse conditions created by vehicular traffic and debris. The placement of sophisticated measuring instruments in the gutter or curb adjacent to inlets was judged to have a limited chance of success. Two approaches were tested: (a) an in situ rating of the inlet and catchment as a unit to distinguish the flow components and (b) an in situ rating of a weir structure placed in the street gutter.

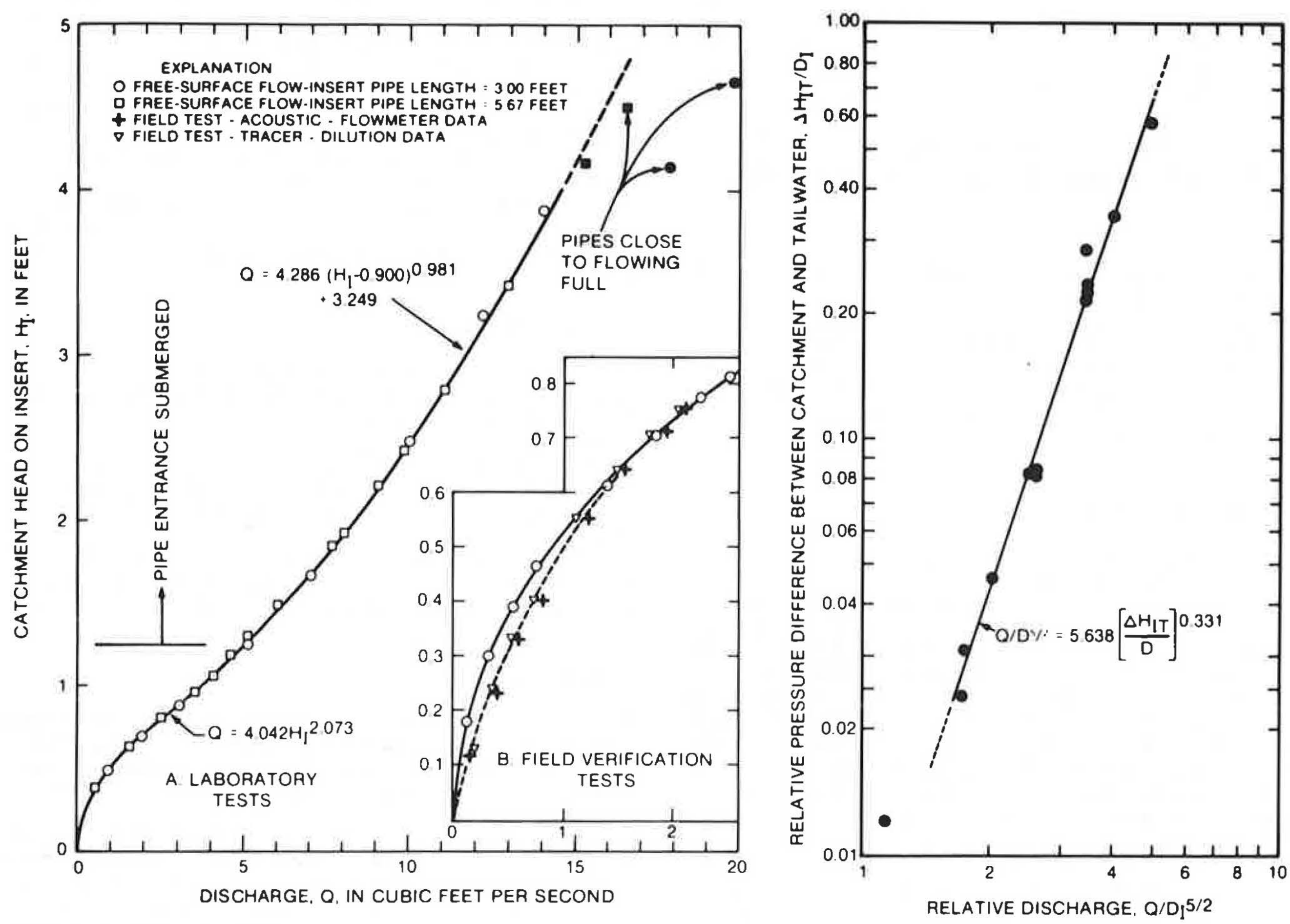


FIGURE 4 Fifteen-inch PIC meter calibration: left, free surface flow; right, pipe-full flow (laboratory tests only).

Inlet In Situ Rating

Conceptually for each catchment and inlet design and configuration a unique relationship existed between the flow into and out of the catchment and that bypassing the inlet. This unique relationship was determined in the field for a single inlet by measuring, for selected discharges, the total flows from the fire hydrant up to and past the point at which a portion of the flow bypassed the inlet. Flows out of the catchment were measured with the PIC meter. Once the rating was established, the outlet flow was a measure of the flow bypassing the inlet. The unique discharge relationship for this inlet is shown in Figure 5 in which the bypass discharge is plotted versus the catchment outlet discharge. The results are good, although it was not possible to extend the rating as high as might have been desired because of the limited hydrant discharges available.

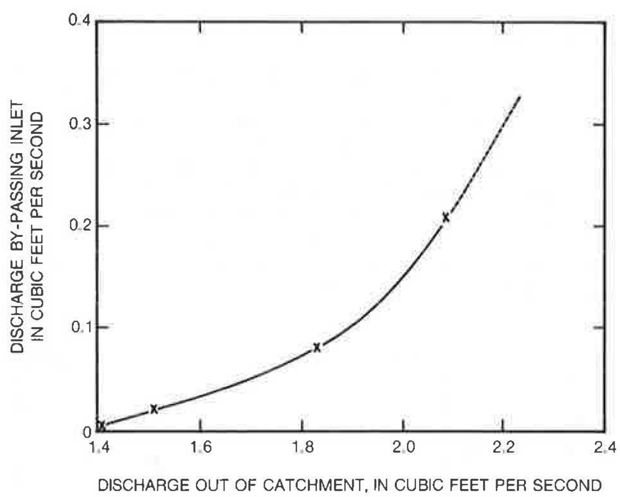


FIGURE 5 In situ bypass discharge rating for inlet at field test site.

Although this method is accomplished with considerable effort, it is the ultimate in assessing the hydraulics of a given storm-drainage system. It can be expected that in practice every inlet will have different physical and hydraulic characteristics regardless of similarity in design. The in situ rating of each would resolve this problem, though with considerable effort.

Curb Weir In Situ Rating

An alternative to the inlet rating technique was desirable. It was decided to try a broad-crested weir structure located in the street gutter to intercept and measure bypass flows. The structure was to have a low profile and be sturdy, capable of withstanding vehicular traffic, and preferably self-cleaning. Figure 6 shows the design of the curb dual weir that was field rated. It consists of a section of aluminum channel 8 in. wide, 1 in. thick, and 2 ft long secured and sealed to the street gutter.

The initial tests using hydrant flows diverted down the street gutter and through the weir structure indicated that supercritical flow was occurring in the gutter with a hydraulic jump forming just upstream of the weir. To produce subcritical flow in the approach to the measuring weir, a second weir was placed 2.0 ft upstream, hence the name "dual weir." The hydraulic jump then formed upstream of

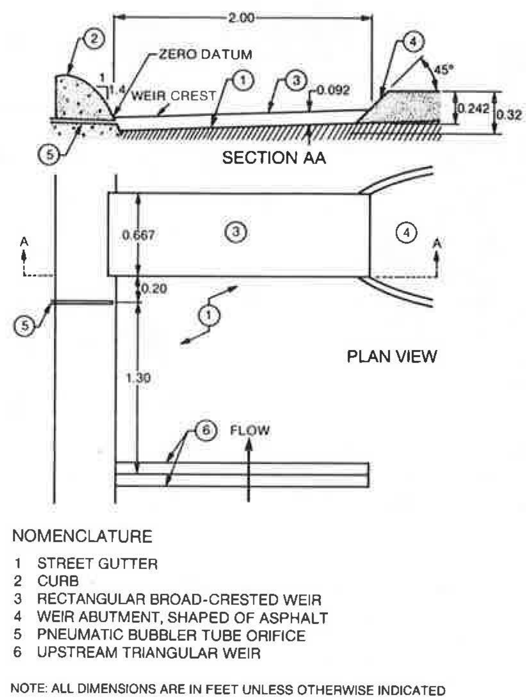


FIGURE 6 Curb dual weir installation.

this second weir with subcritical flow between the two weirs, providing reasonably good head-measuring conditions in the approach to the broad-crested measuring weir. The field rating for this curb dual weir is shown by the solid curve in Figure 7.

As noted in Figures 6 and 7, the curb dual weir is not horizontal and the abutment and curb are not vertical; as a result, the top width (B_t) increases with head. For the same reason, the effective head (H_e) is not the same as the head measured from zero datum at the low point next to the curb (H_{cw}). If the effective width (B_e) is determined for each flow as the mean width for that head and H_e, computed as a mean head, the discharge equation for this broad-crested weir becomes

$$Q = C_b \times 2/3\ B_e (2g)^{1/2} H_e \tag{1}$$

The theoretical rating curve for $C_b = 0.86$ is also shown in Figure 7 and agrees closely with the field rating. The coefficient C_b for the normally higher broad-crested weir is about 0.50 to 0.57. As pointed out by Vennard (12), C_b will be larger for a low-profile weir with a high velocity of approach; obviously that is the case for this broad-crested weir.

These tests, although limited, lend credibility to using this kind of curb dual weir installation and the theoretical rating to measure bypass flows.

Trunkline Flows

Trunklines are typically 3 ft in diameter and larger and hence it is feasible to install flumes, weirs, and other measuring devices in them. Despite their size, trunklines are accessible only through small manholes, which is usually a limiting factor.

Flows in trunklines may be subcritical or supercritical. The presence of constrictive flow measurement devices such as flumes will almost invariably cause subcritical flow to occur upstream, if it is not subcritical already, and rapid transition to pipe-full flow when discharge and heads become large

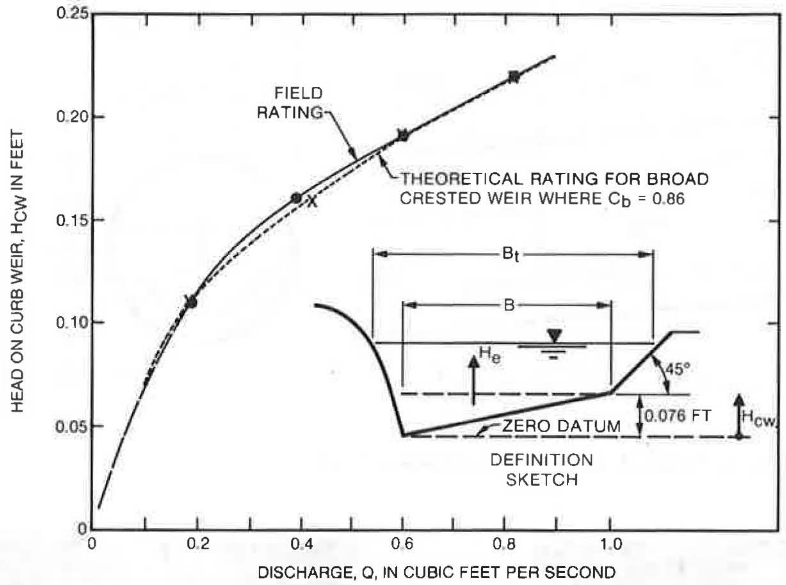


FIGURE 7 Discharge ratings for curb dual weir.

enough. Establishing a discharge rating in the transition zone between open-channel and pipe-full flow is a particular problem.

Surcharge conditions occur when piezometric heads are above the pipe crown elevations and the system is experiencing pipe-full pressure flow.

Backwater conditions, such as when trunklines discharge into nearby streams, may further complicate the hydraulics. Typically, trunkline pipes become submerged as the receiving stream reaches flood stage, thus causing the trunklines to fill as free flow in the pipes ceases to exist. In extreme cases, negative flow may exist in which flow enters from the stream. Flow-measuring devices in these trunklines that depend on only head measurements and free flow are no longer valid under such conditions.

The decision in this study was to rate a Palmer-Bowlus flume modified to function as a venturi meter for pipe-full flow and as both a subcritical and a supercritical flume during open-channel flow. An electromagnetic velocity meter was to be installed upstream of the flume to aid in measuring transition flows as well as flows during backwater conditions. Head would be measured upstream of the flume and in the throat, allowing the applicable calibration to be chosen.

Design and Calibration of a Modified Palmer-Bowlus Flume

The most common type of device used for measurement of flow in larger trunklines is the Palmer-Bowlus flume (13-15). The decision in this study was to design and rate a modified type of Palmer-Bowlus flume (MPB) for both open-channel flow and as a venturi meter with the occurrence of pipe-full flow. It was also intended that the flume perform as a supercritical-flow flume when head was measured in the throat. To accomplish this, the typical Palmer-Bowlus flume design was modified as shown in Figure 8 by making it longer with flatter side slopes and a greater floor thickness. This MPB flume was also designed so that its components could be passed through small manhole openings as shown in Figure 9 for assembly in a trunkline (Figure 10). The 1:1 side slopes were to make construction of concrete possible without having to employ forms.

An 18-in. MPB flume was calibrated in the laboratory for four pipe slopes--0, 1, 2, and 3 percent--for both the approach and the throat. A 48-in. MPB flume was installed at the field test site and rated for low discharges by using hydrant flows measured with tracer dilution and the acoustic velocity meter. Dilution-type discharge measurements were also obtained in the 48-in. MPB flume during several runoff events.

Generalized calibrations for the MPB flume were successfully obtained for both open-channel and pipe-full flow conditions and are shown in Figure 11 (terms are defined in Figure 8; ΔH_{at} is the head difference between the approach and the throat when the system is flowing full). In addition, satisfactory calibrations for the MPB flume functioning as a supercritical flowmeter were obtained (Figure 11B), which extended the measurement range capability because the throat does not fill as quickly as the approach., Nevertheless a transition zone exists between open-channel and pipe-full flow during which flow may pulsate because of alternate filling and opening.

As soon as pipe-full flow exists throughout the MPB flume, pressurized flow exists, and the flume can be treated as a venturi meter. At the steeper slopes considerable difficulty was experienced in getting the 18-in. MPB flume to flow completely full without raising the tailwater and creating backwater. Discharges on the order of 20 ft^3/sec would just barely cause the pipe and the flume to fill. Most of the data in Figure 11C are based on tests in which tailwater was increased to force the system to flow full. Data are also shown for those tests where pipe-full flow occurred or almost occurred without tailwater. As can be seen, most of the scatter is for those measurements where pipe-full flow was not definitely established. These data are purposely shown to emphasize that fully pressurized flow must exist through the MPB flume for this calibration to apply.

From a practical standpoint, it is suggested that the throat calibrations using the MPB as a supercritical-flow flume in conjunction with the venturi calibration when the flume is flowing full will yield good results with a limited transition zone. Care must be taken to determine which flow condition exists. The advantages of using the MPB flume as a

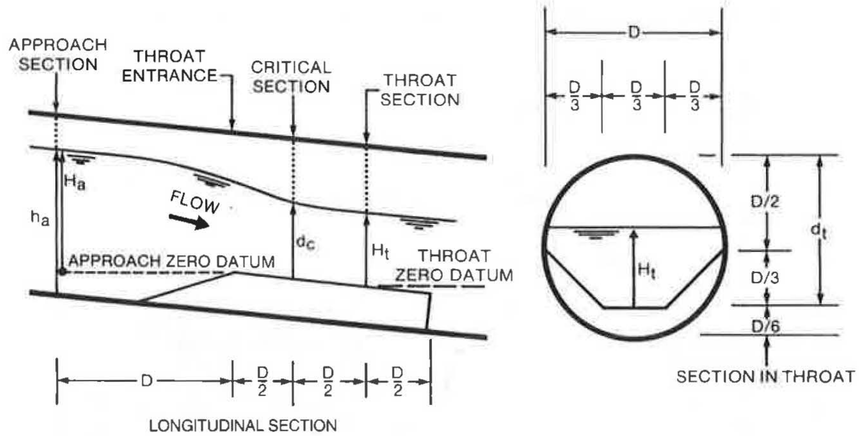


FIGURE 8 Design of modified Palmer-Bowlus flume.

FIGURE 9 Passage of part of modified Palmer-Bowlus flume form through 18-in. manhole.

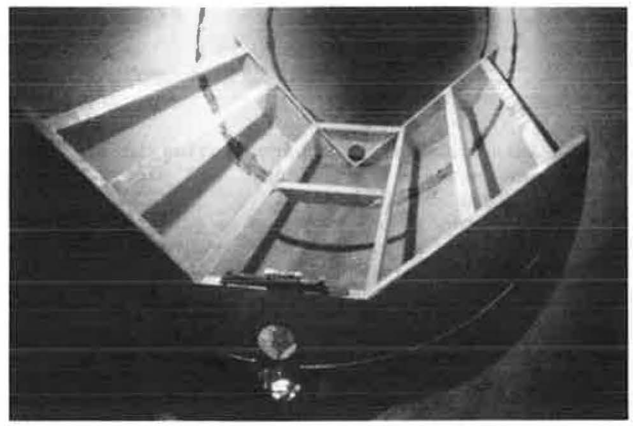


FIGURE 10 Framework for forming modified Palmer-Bowlus flume of concrete.

supercritical-flow flume are the improved self-cleaning characteristics and the greater range in discharge available before the pipe nears flow-full conditions (about 10.5 ft^3/sec compared with 7.5 ft^3/sec for the 18-in. MPB flume). The chief disadvantage is the less sensitive throat calibration.

Electromagnetic Velocity Meter

It was recognized that measuring the flows in pipes with meters such as the MPB flume was apt to be poor in the transition range. In order to measure discharges in the transition range, an electromagnetic point velocity meter (EVM) was installed in the approach to the MPB flume (16). The EVM was to be activated when stages approached the crown of the trunkline to measure velocities as flow went from free surface to pipe full and back. With known velocities, it would be possible to compute discharges through the transition.

In conjunction with the laboratory calibration of the 18-in. MPB flume, an EVM was calibrated to determine what velocity coefficients should apply to different flows and flow conditions in the approach to the flume. These tests were less than satisfactory because it was found that approach conditions to the flume (a hydraulic jump formed upstream when flow in the upstream pipe was initially supercritical) probably affected the velocity distributions in an unpredictable manner. Furthermore, in the transition zone, pulsating flow resulted, which added uncertainty to the applicable velocity coefficients. Only with pipe-full flow did the EVM velocity coefficients appear to be reasonably stable at about 1.00, regardless of pipe slope. Uncertainty exists as to what, if any, coefficient should be used at other than pipe-full flow. Therefore it would appear that the scheme of using an EVM located in the approach to the MPB flume to measure transition flows is of questionable value.

Field Tests of the MPB and EVM

Fire hydrant discharges were directed into the 48-in. trunkline leading to a 48-in. MPB flume installed at the field site. Once each flow had stabilized, it was measured by both tracer dilution and acoustic meter; these discharges were in close agreement and also agreed closely with the calibration curves shown in Figure 11A and 11B. No attempt has been made to show these data in Figure 11A or 11B.

Several periods of storm runoff were experienced

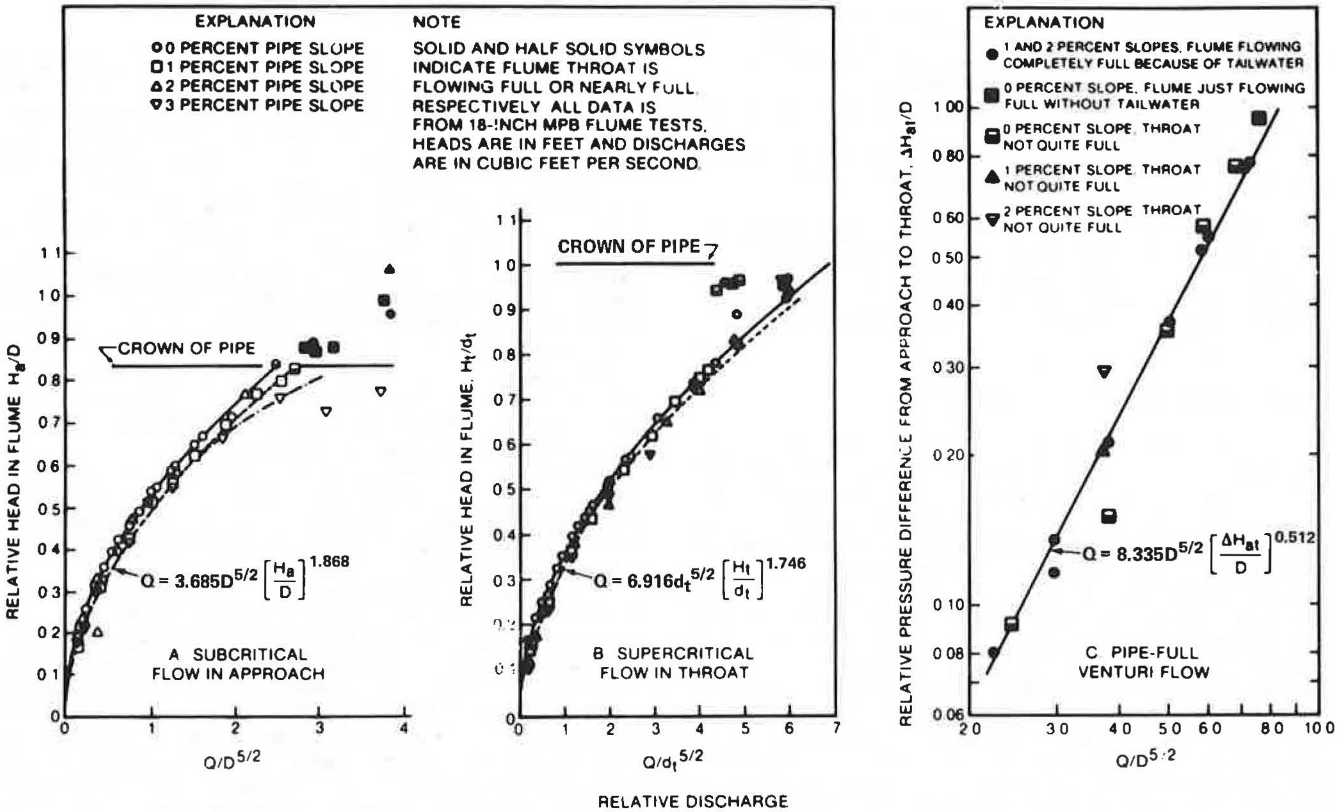


FIGURE 11 Calibrations for modified Palmer-Bowlus flume.

during the field test period. A total of 12 dilution measurements was obtained automatically, some on the rapidly rising limbs of the hydrographs and others on the slower recessions. As might be expected, these measurements scatter more than the dilution measurements made during the steady-flow hydrant tests. Nevertheless, agreement was good and verified the laboratory calibrations. Normally the dilution measurements made on the slower-changing recessions would be expected to give the most accurate results. In this case no distinction could be made, probably because mixing was so nearly instantaneous. Factors causing such good mixing were the turbulence in a junction box at the point of injection and the hydraulic jumps above and possibly below the MPB flume; sampling was downstream of the MPB flume.

At the field site the EVM was installed on a vertical support rod in the approach to the 48-in. MPB flume 0.4D above the invert and 1D upstream of the entrance. It was activated only when the micrologger sensed that sufficient stage existed during a given flow event. The voltage output was then recorded on the micrologger.

The system worked without flaw during all field tests. The EVM was adequately submerged only at head values at and above 1.2 ft. Unfortunately, the field site was found to experience backwater at about this stage, so a direct comparison of discharges computed from the EVM velocity data and the MPB flume calibration could not be made. Unfortunately too, the highest dilution-discharge measurements thus far obtained are at approximately the same head as that when the EVM is activated; thus the validity of the EVM-based discharges cannot be confirmed at this time. Field tests are continuing in hopes that higher dilution-discharge measurements will confirm this backwater effect. If so, the validity of using the EVM to measure flows under backwater conditions would be confirmed.

Stage and Head Measurement System

In the normal storm-sewer drainage system, it is impractical, if not impossible, in most instances to employ fluid intake systems with stilling wells and floats or other such direct means of measuring water stage or pressure heads in connection with flow measurement devices. It was elected in this study to use the gas purge or pneumatic-bubbler and orifice system because of the flexibility and reliability it offered. With this system, a gas is bubbled out through an orifice and the pressure exerted by the overlying fluid column is measured and converted to head in feet of water (17). Various types of manometers have been developed to convert gas pressure to feet of water. Although not new, pressure transducers were tested to measure gas pressures and relay the corresponding voltage output to a micrologger recorder.

Five pressure transducers were initially calibrated in the laboratory before installation at the field test site. These ranged from 10-in. to 100-in. water-stage sensing units. They were tested from -25°C to +50°C. The calibrations of the five transducers were then periodically checked at the field test site by using a water-filled standpipe. Three of the five transducers were then retested in the laboratory following 6 months of use.

The following conclusions were drawn from laboratory and field tests of the five transducers:

1. Each transducer should be tested and calibrated in the laboratory over a range of depths and temperatures.
2. Calibrations should exclude the use of data extremes and transducers should be installed if possible to avoid their use at very low heads (0.1 to 0.2 ft of water).

3. Temperature effects are not normally significant but high temperatures significantly over 100°F should be avoided or the transducer should be insulated from extremes in temperature.
4. Hysteresis is not significant and is lowest at lower temperatures.
5. Transducers may experience an aging process and should be periodically calibrated in the field.

In general it was concluded that the combination of the pressure transducer and pneumatic bubbler and orifice into a head-measuring system can be very successful if used with the foregoing precautions, in particular the avoidance of shallow depths.

Control and Data Recording System

In recent years (18) numerous solid-state digital recorders have become available. Certain of the data loggers or microloggers have been tailored for the collection, storage, and processing of hydrologic data. The Campbell Scientific CR-21 micrologger is such a unit. The CR-21 is an extremely low-power, battery-operated, temperature-stable data logging and system control device. The unit uses a 12-V power supply and has nine input channels and four output control ports. This micrologger can be programmed to sense and process data and emit commands to activate various instruments. This latter feature was important because the power consumption of an electromagnetic velocity meter would be excessive for prolonged battery operation alone unless it could be turned on only when needed. Furthermore, the CR-21 can be programmed to store data only if runoff occurs. This unit stores data internally and unloads it automatically to cassette or solid-state storage.

The single CR-21, shown in Figure 12, was used at the field test site to record pressure-transducer data from five measuring devices: a curb weir, four voltages from a 15-in. PIC meter and a 48-in. MPB flume, the voltage output from an EVM, and the counts from a tipping-bucket rain gauge. Furthermore, the same micrologger was programmed to activate the EVM and two automatic tracer dilution-discharge measurement systems above selected heads.

The CR-21 unloads its data to a solid-state recorder that can interface with most computers. Suitable programs may be used to query the storage module and output the data as desired.

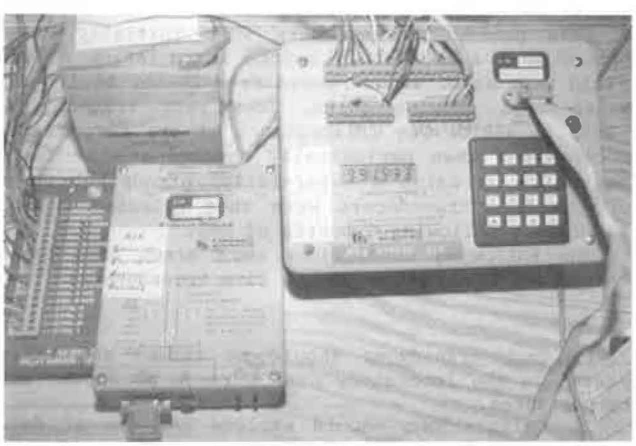


FIGURE 12 Micrologger unit in center with relay board on left and data storage module at lower center.

SUMMARY AND SUGGESTIONS

This study has been successful in identifying certain measuring devices and techniques that can be suggested for inclusion in any storm drainage measurement system. The MPB flume can be relied on for measuring flows in trunklines whether open channel or pipe full; the electromagnetic velocity meter can be used for measuring pipe-full flow caused by backwater conditions; the in situ rating of catchment inlets, although difficult, can be used to evaluate gutter bypass flows; and quality tipping-bucket rain gauges are available commercially that will yield quick response and accurate measurement of rainfall. The pneumatic bubbler and transducer system is a reliable means of measuring head as long as the transducers are kept calibrated. The micrologger can serve all of the devices, both recording data and activating instruments as needed.

Additional work needs to be performed on developing a device to measure flows out of catchments. The PIC meter developed as part of this study was not wholly satisfactory because it depends on the measurement of head in the catchment.

Furthermore, the dilution-gauging approach should be considered in any study if mixing conditions appear favorable. It is suggested that this approach to rating existing catchments in situ be used without any modifications except the addition of pneumatic bubblers and stilling wells. Consideration also should be given to using dilution gauging to measure entire runoff events in catchments; these data are used alone without resorting to adding measurement devices, rating existing catchments, or making any changes in the existing storm-sewer system except to sample the tracer.

The acquisition of comprehensive storm precipitation-runoff data sets for selected segments of highway systems could involve instrumenting several miles of highway. The first attempt might be to try to make such a monitoring system a single, interconnected, centrally controlled network. Instead, it is recommended that a highway system under consideration for study be divided into small units in which one or more inlets, trunkline gauges, and a rain gauge are served by one transducer-pneumatic network, which in turn interfaces with its own micrologger recorder. This is suggested because experience with microloggers indicates that they have good timing accuracy; hence the primary argument for centralized control and the expense and complexity of a central system are eliminated.

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New Studies of Urban Flood Frequency in the Southeastern United States

VERNON B. SAUER

ABSTRACT

Five reports dealing with flood magnitude and frequency in urban areas in the southeastern United States have been published during the past 2 years by the U.S. Geological Survey (USGS). These reports are based on data collected in Tampa and Tallahassee, Florida; Atlanta, Georgia; and several cities in Alabama and Tennessee. Each report contains regression equations useful for estimating flood peaks for selected recurrence intervals at ungauged urban sites in their respective study area. A nationwide study of urban flood characteristics by the USGS published in 1983 contains equations for estimating urban peak discharges for ungauged sites throughout the United States. At the time that the nationwide study was conducted, data from only 35 sites in the southeastern United States were available. The five new reports contain data for 88 additional sites in the southeastern United States. These new data show that the seven-parameter estimating equations developed in the nationwide study are unbiased and have prediction errors less than those described in the nationwide report. On the other hand, the new data indicate that the three-parameter equations are biased and significantly underestimate flood discharge in four of the new study areas. The five new reports on the southeastern United States and the nationwide report provide reliable methods for estimating design discharges.

Rapid expansion and development of urban areas in the United States bring many informational needs. An important one is flood data, which are necessary in the design of stream channels, canals, storm sewers, detention ponds, roadways, bridges, and culverts. Likewise, the magnitude, frequency, and boundaries of floods are required for zoning and insurance purposes. Gauging, or measuring, all streams and locations where data are needed is not feasible. Instead, flood data are collected at a few selected sites, and the information is transferred to ungauged sites by various regionalization procedures. To this end, the U.S. Geological Survey (USGS) has collected urban flood data in many cities throughout the United States during the past 20 to 30 years. These data have been published for public use and analyzed for flood-frequency regionalization studies. Reports have been published describing flood characteristics for individual cities, metropolitan areas, or selected groups of cities. References to and brief abstracts of many of these reports as well as other urban flood-frequency procedures may be found in a literature review by Rawls et al. (1). A nationwide regionalization of urban flood characteristics by Sauer et al. (2) describes techniques for estimating flood magnitude and frequency for cities throughout the United States, including Alaska. That report contains flood data and basin characteristics for 269 urban sites in 56 cities and 31 states. It also has an extensive list of references for both urban and rural flood-frequency procedures.

After analysis of the nationwide regionalization (2), five new urban studies were prepared for cities, metropolitan areas, and states in the southeastern region of the United States. These include a statewide study for Alabama (3); the Atlanta, Georgia, metropolitan area (4); the Leon County, Florida, area, which includes Tallahassee (5); the Tampa Bay, Florida, area, which includes the cities of Tampa, Clearwater, and St. Petersburg (6); and a statewide study for Tennessee (7). These five studies contain useful equations and techniques for estimating urban flood frequency in each study area. In addition, they contain specific flood and basin data for 88 gauged sites. Only three of these sites had been available for inclusion in the nationwide study (2), which used only 35 sites from the southeastern region, so the new data represent a significant increase in available data.

The purpose of this paper is to summarize and briefly describe the new urban studies in the southeastern region of the United States and to compare the new data with estimates based on the nationwide equations (2).

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DESCRIPTION OF VARIABLES

Several equations for the five new urban studies and the nationwide study are presented in this paper. Flood characteristics and basin and climatic parameters are symbolized in these equations by abbreviations. In this paper some symbols have been changed from those shown in the original publications in order to present a consistent set of symbols. These are as follows:

A = contributing drainage area (mi^2). In urban areas drainage systems sometimes cross topographic divides. Such drainage changes should be accounted for when computing A.

BDF = basin development factor, an index of the prevalence of the drainage aspects of (a) storm sewers, (b) channel improvements, (c) impervious channel linings, and (d) curb-and-gutter streets. The range of BDF is 0

to 12. A value of zero for BDF indicates that the foregoing drainage aspects are not prevalent but does not necessarily mean that the basin is nonurban. A value of 12 indicates full development of the drainage aspects throughout the basin. See the paper by Sauer et al. for details of computing BDF (2).

IA = percentage of the drainage basin occupied by impervious surfaces, such as houses, buildings, streets, and parking lots.

n = number of gauging sites used in error analysis.

RI2 = rainfall intensity (in.) for the 2-hr, 2-year occurrence [determined from Weather Bureau (8)].

RI24 = rainfall intensity (in.) for the 24-hr, 2-year occurrence [determined from Weather Bureau (8)].

RMS = Root-mean-square error (logarithmic units and percent) is considered for this study as an approximation of the standard error of prediction. It is used for comparing new data with existing estimating equations. See text for method of computation.

RQx = peak discharge (ft^3/sec) for an equivalent rural drainage basin in the same hydrologic area as the urban basin and for recurrence interval x. For this study equivalent rural discharges were computed from applicable USGS regional flood-frequency reports.

s = average standard deviation (logarithmic units and percent) of the errors between observed and estimated discharges. See text.

SEP = average standard error of prediction (logarithmic units and percent) of the regional regression equations for all urban basins, both gauged and ungauged. SEP can be computed either theoretically or from split-sample methods. It is an estimate of how well an equation can predict flood magnitude for a given recurrence interval at any urban site within a designated study area.

SER = average standard error of regression (logarithmic units and percent) of the regional regression equations for the gauged urban basins. It is based only on the data used to derive the regression equations, and it is usually less than the standard error of prediction (SEP).

SL = main channel slope (ft/mi), measured between points that are 10 and 85 percent of the main channel length upstream from the study site. For sites where SL is greater than 70 ft/mi, 70 ft/mi is used in the nationwide equations.

ST = basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetlands. In-channel storage of a temporary nature resulting from detention ponds or roadway embankments is not included in the computation of ST.

STDT = basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, detention basins, and retention basins. This variable is used only in the Tampa Bay area study (6).

UQx = peak discharge (ft^3/sec) for the urban watershed for recurrence interval x. That is, UQ10 = 10-year urban peak discharge, UQ100 = 100-year urban peak discharge.

$\bar{X}$ = mean (logarithmic units and percent) of the errors between observed and estimated discharges. See text.

THE FIVE NEW URBAN STUDIES

The collection of basic data is similar for all five recent southern region urban studies and follows USGS standards. At each gauge site, streamflow and storm rainfall data were measured for 15 or more floods over a period of 2 or more years. These data were used as input for calibration of a rainfall-runoff model, and where sufficient data permitted, split-sample techniques were used to verify the calibration. The models used were either the USGS model (9) or the DR3M model (10). Following a successful calibration at each site, long-term rainfall and evaporation records from a nearby National Weather Service (NWS) station were used to synthesize a flood record of maximum annual peak discharges. In lieu of long-term synthesis, some investigators chose the map-model method (11) to compute flood-frequency data for the site. Flood-frequency analysis of the long-term synthesized annual peaks was done according to log Pearson III procedures (12). For purposes of comparison of the five studies in this paper, only the 10- and 100-year recurrence intervals are presented.

Alabama Urban Study

Olin and Bingham assembled urban flood data for 23 sites from the Alabama cities of Dothan, Alexander City, Montgomery, Homewood, Greenwood, Ketona, Birmingham, Ensley, Adamsville, Tuscaloosa, Mobile, Huntsville, Lily Flagg, and Florence (3). The USGS model was calibrated for each site and a flood-frequency curve was synthesized through application of the map-model method. The synthesized flood-frequency data for the 23 sites were regionalized by relating basin and climatic parameters to urban flood discharge by multiple regression methods. The final equations for the 10- and 100-year urban floods are

$$UQ10 = 266A^{0.69}IA^{0.39} \quad \text{(SER = 24 percent)} \quad (1)$$

$$UQ100 = 444A^{0.69}IA^{0.39} \quad \text{(SER = 25 percent)} \quad (2)$$

These equations can be used throughout the state of Alabama.

Atlanta, Georgia, Study

Inman used flood data from 19 sites in the Atlanta, Georgia, metropolitan area to study the effects of urbanization (4). He calibrated both the USGS and the DR3M rainfall-runoff models for each site. A 76-year period of rainfall data from the Atlanta NWS station was used to synthesize two sets of annual flood peaks, one set based on the USGS model and another based on the DR3M model. The primary difference between the results from the two models is that the USGS model calculates flood discharges for an as-is condition of the drainage basin, whereas one feature of the DR3M model allows the user to add or remove storage elements in the basin. For the Atlanta study sites, all significant storage elements, such as detention storage upstream from road embankments, were removed from the model. The flood-frequency results from the DR3M model are therefore essentially storage-free and can be considered a maximum flood potential for the respective basins under their current state of development. The results from the USGS model represent flood potential for existing conditions in the study basins and include the effects of detention storage, which tends

to reduce flood peak discharges. The 10- and 100-year flood-estimating equations are as follows:

Equation based on USGS model (storage included):

$$UQ10 = 15.5A^{0.92}\ (SL - 12)^{0.50}IA^{0.58}$$
$$(SER = 15\text{ percent; } SEP = 25\text{ percent}) \quad (3)$$

$$UQ100 = 30.7A^{0.95}\ (SL - 12)^{0.53}IA^{0.51}$$
$$(SER = 13\text{ percent; } SEP = 25\text{ percent}) \quad (4)$$

Equation based on DR3M model (storage free):

$$UQ10 = 46.2A^{0.80}\ (SL - 12)^{0.37}IA^{0.50}$$
$$(SER = 20\text{ percent; } SEP = 31\text{ percent}) \quad (5)$$

$$UQ100 = 72.4A^{0.79}\ (SL - 12)^{0.39}IA^{0.54}$$
$$(SER = 20\text{ percent, } SEP = 32\text{ percent}) \quad (6)$$

The difference between the results of the USGS and DR3M model equations can be attributed to the average effects of detention storage in the Atlanta area. Because detention storage will almost always be present, it is likely that the USGS model equations are more suitable for design purposes.

Leon County, Florida, Study

Franklin and Losey (5) used flood data from 15 sites in the Leon County, Florida, area (mainly Talahassee) to study urban flood frequency. They calibrated the USGS model for each site and computed two sets of long-term synthesized annual peak discharges for each site by using two long-term NWS rain gauges--the Thomasville-Coolidge gauge and the Pensacola gauge. A flood-frequency curve was computed from each of the separate sets of annual peak discharges and averaged by a weighting procedure. Two stations could not be used because model calibrations were poor; therefore, the data set was reduced to 13 stations. In addition, all streams in the Lake Lafayette basin were found to have large amounts of channel and detention storage. The five sites in the Lake Lafayette basin were in the regression analysis but were assigned a qualitative code to distinguish them from sites where storage was not excessive. The resulting regression analysis gives the following equations:

Lake Lafayette Basin:

$$UQ10 = 7.98A^{0.776}IA^{0.867}$$
$$(SER = 20\text{ percent; } SEP = 33\text{ percent}) \quad (7)$$

$$UQ100 = 32.4A^{0.808}IA^{0.687}$$
$$(SER = 25\text{ percent; } SEP = 40\text{ percent}) \quad (8)$$

Other Leon County Sites:

$$UQ10 = 39.1A^{0.776}IA^{0.867}$$
$$(SER = 20\text{ percent; } SEP = 33\text{ percent}) \quad (9)$$

$$UQ100 = 118A^{0.808}IA^{0.687}$$
$$(SER = 25\text{ percent; } SEP = 40\text{ percent}) \quad (10)$$

Tampa Bay Area, Florida, Study

Lopez and Woodham (6) used nine urban sites to study flood frequency in the Tampa Bay area, which included the cities of Tampa, Clearwater, and St. Petersburg. They calibrated the USGS rainfall-runoff model for each site and simulated a 47-year record of annual peak discharges by using long-term climatic records from the NWS station at Tampa. A flood-frequency analysis was made for each site by using the simulated annual peaks. Transferability of the site data to ungauged sites was accomplished through a regional regression analysis and resulted in the following equations:

$$UQ10 = 12.9A^{1.04}BDF^{0.75}SL^{0.83}\ (STDT + 0.01)^{-0.10}$$
$$(SER = 35\text{ percent}) \quad (11)$$

$$UQ100 = 282A^{1.16}\ (13 - BDF)^{-0.51}SL^{0.76}$$
$$(SER = 42\text{ percent}) \quad (12)$$

Note that the storage factor (STDT) is not included in the 100-year flood equation, indicating that it is not significant at high flood levels. This logically implies that at high flood levels, storage becomes satisfied and no longer has a reducing effect.

Tennessee Urban Study

Robbins (7) used flood data from 22 urban runoff sites in the Tennessee cities of Gallatin, Donelson, Nashville, Avondale, Franklin, Dickson, Greeneville, Morristown, Alcoa, Cleveland, Fayetteville, Manchester, Paris, Jackson, Humboldt, Covington, Germantown, Memphis, and Chattanooga to study urban flood frequency in Tennessee. He calibrated the USGS rainfall-runoff model for each site and used the calibration results in the map-model method to estimate synthetic flood-frequency data. These data were used in a statewide regression analysis to regionalize the data for use at ungauged sites. The following equations resulted from the regression analysis:

$$UQ10 = 11.8A^{0.75}IA^{0.43}RI24^{2.12}$$
$$(SER = 27\text{ percent; } SEP = 37\text{ percent}) \quad (13)$$

$$UQ100 = 77.0A^{0.75}IA^{0.40}RI24^{1.10}$$
$$(SER = 25\text{ percent; } SEP = 39\text{ percent}) \quad (14)$$

These equations can be used throughout the state of Tennessee.

NATIONWIDE URBAN FLOOD-FREQUENCY STUDY

Sauer et al. (2) used urban flood data from 199 of the 269 urban sites listed in their report to develop regression equations for use at ungauged sites throughout the United States. The seven-parameter equations are

$$UQ10 = 2.99A^{0.32}SL^{0.15}\ (RI2 + 3)^{1.75}\ (ST + 8)^{-0.57}\ (13 - BDF)^{-0.30}IA^{0.09}RQ10^{0.58}$$
$$(SER = 38\text{ percent; } SEP = 45\text{ percent}) \quad (15)$$

$$UQ100 = 2.50A^{0.29}SL^{0.15}\ (RI2 + 3)^{1.76}\ (ST + 8)^{-0.32}\ (13 - BDF)^{-0.28}IA^{0.06}RQ100^{0.63}$$
$$(SER = 44\text{ percent; } SEP = 53\text{ percent}) \quad (16)$$

The basin development factor (BDF) proved to be highly effective in these equations for explaining the effects of urbanization on flood peaks. The equations include an estimate of the equivalent rural discharge (RQx) for a similar basin in the same hydrologic area. This serves as a regional or geographic factor, thus allowing use of the equations throughout the United States. The equations essentially adjust the equivalent rural peak to the urban condition. The equations do not include an adjustment for detention or temporary in-channel storage. Therefore, they should not be used if detention storage is highly significant in the basin--such as in the Lake Lafayette basin in Leon County,

Florida. Slope (SL) is limited to a maximum value of 70 ft/mi and 70 ft/mi should be used for any basin in which SL exceeds this amount.

The seven-parameter equations were simplified by eliminating the less significant variables and recalibrating the regression constants and coefficients. The following three-parameter equations resulted:

$$UQ10 = 9.51A^{0.16} (13 - BDF)^{-0.36} RQ10^{0.79} \quad (SER = 41 \text{ percent; } SEP = 43 \text{ percent}) \tag{17}$$

$$UQ100 = 7.70A^{0.15} (13 - BDF)^{-0.32} RQ100^{0.82} \quad (SER = 46 \text{ percent; } SEP = 49 \text{ percent}) \tag{18}$$

COMPARISONS OF URBAN ESTIMATING EQUATIONS

Each of the five new urban studies was carried out by the USGS using similar techniques. One would logically expect that the results would be similar. Rainfall-runoff modeling was similar for each study, synthesization was similar, and regionalization was made by regression analysis in all cases. Each investigator explored numerous basin and climatic variables for explaining the variation in urban flood characteristics, and only those that were statistically significant were used. These are compared in Table 1, which also includes parameters from the nationwide study. For the five new studies, drainage area size and an index of urbanization, either impervious area or the basin development factor, are significant in all. Slope is significant in only two, Atlanta and Tampa. The rainfall index used in the Tennessee study accounts for statewide variations in rainfall, which probably reflect variations in flood potential across the state.

The three- and seven-parameter nationwide equations were used to estimate the 10- and 100-year discharges for 78 of the 88 sites for which data are available from the five new urban studies. Three sites were not used because they were in the original nationwide analysis; two sites from the Leon County, Florida, study could not be used because of poor calibrations; and the five Lake Lafayette basin sites in the Leon County study were not used because of the large amounts of detention storage. Average error ($\bar{X}$) and standard deviation of the errors (s) were computed for each study for both the three- and seven-parameter equations. The root-mean-square (RMS) error was computed as follows:

$$RMS = (\bar{X}^2 + s^2)^{1/2} \tag{19}$$

The RMS error is considered an approximation of the standard error of prediction and was used for comparison with the standard errors reported for each study. All error analysis was performed by transforming the discharge data to logarithmic units and computing the logarithmic residual between the estimated discharge and the observed discharge. The final logarithmic values of $\bar{X}$, s, and RMS of the residuals were then converted to percentages.

The mean error ($\bar{X}$) is an indication of the bias present in the equations. Student's t-test was used to determine whether any $\bar{X}$-values were significantly different from zero. The results of the error analyses are shown in Tables 2 and 3. These comparisons show that the three-parameter nationwide equations have a negative average error for the 10- and 100-year floods for each of the new study areas. Student's t-test, at the 0.01 level of significance, indicates that these negative errors are statistically significant for both return levels for the Alabama, Atlanta, and Leon County studies and for the 10-year floods in the Tampa study. The 10- and 100-year floods in the Tennessee study and 100-year floods in the Tampa study show no significant bias, but overall the new data appear to indicate that the three-parameter nationwide equations are biased, at least in parts of the southeastern United States. Data from the original 35 sites were reexamined and found not to show the same bias. Tests are now being made in an attempt to explain the reason for the bias in the new data.

Conversely, the seven-parameter nationwide equations show both negative and positive mean errors, and in no case are these significantly different from zero. These equations are not biased and can be used for estimating urban floods in all of the new study areas.

The RMS errors for the 100-year three-parameter nationwide equations are less than the nationwide standard error of prediction when those equations are applied to the new data in Alabama, Atlanta, and Tennessee, in spite of the fact that the equations are biased in Alabama and Atlanta. In both Florida studies, the equations have large RMS errors. The RMS error for all the data combined is ±47 percent as compared with the nationwide SEP of ±49 percent. Except for Tennessee, the three-parameter equation RMS error is larger than the SEP for the individual regionalized equations of the five new studies.

The RMS errors for the 10- and 100-year seven-parameter nationwide equations are less than the nationwide standard error of prediction for those equations when applied to the data from each of the five new studies. This, plus the fact that the equations are not biased, shows them to be good estimating equations throughout the southeastern United States. The RMS error for both the 10- and 100-year equations is ±35 percent for the combined data set of 78 sites. This is significantly better than the

TABLE 1 Comparison of Variables Used in Urban Flood-Frequency Estimating Equations

	Equation Variable								
Study Area	A	SL	RI2	RI24	ST	STDT	BDF	IA	RQx
Alabama	X							X	
Atlanta, Georgia	X	X						X	
Leon County, Florida	X							X	
Tampa Bay, Florida	X	X				X[a]	X		
Tennessee	X			X				X	
Nationwide									
Three parameters	X						X		X
Seven parameters	X	X	X		X		X	X	X

Note: X = variable is significant.

[a]Storage of 10 yr or less.

TABLE 2 Comparison of Errors for Nationwide Equations and Five Southeastern Studies

		Nationwide Three Parameters (log units/%)			Nationwide Seven Parameters (log units/%)			Reported Study Error (%)	
Study Area	n	X	s	RMS	X	s	RMS	SER	SEP
10-Year Floods[a]									
Alabama	22	−0.1462/−29	0.1450/±34	0.2059/±49	−0.0897/−19	0.1547/±36	0.1788/±42	±24	(±37)
Atlanta	18	−0.1325/−26	0.1262/±29	0.1830/±43	−0.0091/−2	0.1085/±25	0.1089/±25	±15	±25
Leon County	8	−0.3015/−50	0.1331/±31	0.3296/±83	−0.1157/−23	0.1394/±33	0.1811/±43	±20	±33
Tampa	9	−0.2160/−39	0.1779/±42	0.2798/±69	−0.0417/−9	0.1624/±38	0.1677/±40	±35	(±48)
Tennessee	21	−0.0381/−8	0.1530/±36	0.1577/±37	−0.0093/1	0.1418/±33	0.1419/±33	±27	±37
Total	78	−0.1379/−27	0.1627/±38	0.2133/±51	−0.0430/−9	0.1446/±34	0.1508/±35	—	—
100-Year Floods[b]									
Alabama	22	−0.1284/−26	0.1263/±29	0.1801/±43	−0.0589/−13	0.1392/±33	0.1512/±35	±25	(±38)
Atlanta	18	−0.1100/−22	0.1205/±28	0.1632/±38	−0.0417/3	0.1039/±24	0.1049/±24	±13	±25
Leon County	8	−0.2478/−43	0.1576/±37	0.2937/±73	−0.0703/−15	0.1753/±41	0.1888/±45	±25	±40
Tampa	9	−0.1903/−35	0.2215/±53	0.2921/±72	−0.0412/−9	0.2101/±50	0.2141/±51	±42	(±54)
Tennessee	21	−0.0428/−9	0.1512/±36	0.1571/±37	−0.0182/4	0.1405/±33	0.1417/±33	±25	±39
Total	78	−0.1205/−24	0.1575/±37	0.1983/47	−0.0203/−5	0.1470/±34	0.1484/±35	—	—

Note: Values in parentheses were estimated for this paper only.
[a]Nationwide values for SER and SEP (n = 199) are as follows. Three parameters: ±41 percent and ±43 percent; seven parameters: ±38 percent and ±45 percent.
[b]Nationwide values for SER and SEP (n = 199) are as follows. Three parameters: ±46 percent and ±49 percent; seven parameters: ±44 percent and ±53 percent.

TABLE 3 Nationwide Urban Equation Bias Based on Student's t-Test

	Three-Parameter Equation		Seven-Parameter Equation	
Study Area	10 Yr	100 Yr	10 Yr	100 Yr
Alabama	Yes	Yes	No	No
Atlanta	Yes	Yes	No	No
Leon County	Yes	Yes	No	No
Tampa	Yes	No	No	No
Tennessee	No	No	No	No
Total	Yes	Yes	No	No

Note: Level of significance = 0.01.

±44 percent (10-year) and ±53 percent (100-year) reported SEP for the nationwide equations. For each new study area, the seven-parameter RMS error is approximately the same as the prediction errors reported for the individual study regional equations.

SUMMARY AND CONCLUSIONS

During the past 2 years, significant new urban flood data have become available in the southeastern United States. Five new studies have been published by the USGS in Alabama, Georgia, Florida, and Tennessee. In each study, regional equations were developed for estimating urban flood magnitude and frequency. A comparison shows the regional equations to be similar, using drainage area size and an index of urbanization as primary estimating parameters. Standard errors of prediction are generally less than 40 percent.

A comparison with the nationwide urban equations published by Sauer et al. shows their three-parameter equations to significantly underestimate urban floods in four of the new study areas (2). Only for Tennessee and the 100-year flood levels in the Tampa Bay area are the equations unbiased. On the other hand, the nationwide seven-parameter equations are unbiased in all new study areas and show standard errors of prediction less than those published for the nationwide equations.

In conclusion, the individual study regional equations can be used within their specific region and parameter limits. The seven-parameter nationwide equations can be considered good estimating equations throughout the southeastern United States.

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Simulation of Flood Hydrographs for Georgia Streams

E. J. INMAN and J. T. ARMBRUSTER

ABSTRACT

Flood hydrographs are needed for the design of many highway drainage structures and embankments. A method for simulating these flood hydrographs at urban and rural ungauged sites in Georgia is presented. The O'Donnell method was used to compute unit hydrographs from 355 flood events from 80 stations. An average unit hydrograph and an average lag time were computed for each station. These average unit hydrographs were transformed to unit hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lag time and then reduced to dimensionless terms by dividing the time by lag time and the discharge by peak discharge. Hydrographs were simulated for these 355 flood events and their widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. The dimensionless hydrograph based on one-half lag-time duration provided the best fit of the observed data. Multiple-regression analysis was used to define relations between lag time and certain physical basin characteristics, of which drainage area and slope were significant for the rural equations, with impervious area being added for the Atlanta urban equation. A hydrograph can be simulated from the dimensionless hydrograph, peak discharge of a specific recurrence interval, and lag time obtained from regression equations for any site of less than 500 mi² in Georgia. For simulating hydrographs at sites larger than 500 mi², the U.S. Geological Survey computer model CONROUT can be used. CONROUT produces a simulated outflow discharge hydrograph with a peak discharge of a specific recurrence interval. The diffusion analogy routing method with single linearization was used in this study.

The design of many highway drainage structures and embankments requires an evaluation of the flood-related risk to the structures and to the surrounding property. Risk analyses of alternative designs are necessary to determine the design with the least total expected cost. In order to fully evaluate these risks, a runoff hydrograph with a peak discharge of specific recurrence interval may be necessary to estimate the length of time of inundation of specific features, for example, roads and bridges. For ungauged streams, this information is difficult to obtain; therefore, there is a need for a method based on Georgia hydrologic data to estimate the flood hydrograph associated with a design discharge. The objective of this study was to define techniques for simulating flood hydrographs for specific design discharges at ungauged sites in Georgia. The scope of this study was statewide for rural basins and the Atlanta metropolitan area for urban basins up to 25 mi².

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HYDROGRAPH SIMULATION PROCEDURE

Several traditional methods for simulating a hydrograph for a flood of selected recurrence interval at an ungauged watershed were considered for this study. However, a new procedure based on observed streamflow data was developed for this study and is presented in this section.

Basins Less Than 500 mi²

A dimensionless hydrograph was developed for use in basins up to 500 mi². Peak discharge of a selected

recurrence interval and lag time are necessary parameters to convert the dimensionless hydrograph to a simulated hydrograph for a given basin. Price (1) presents a technique for estimating the peak discharge of a selected recurrence interval for rural streams in Georgia. Inman (2) presents a technique for estimating the peak discharge of a selected recurrence interval for basins less than 25 mi^2 in the Atlanta urban area. Lag-time estimating equations were developed for Georgia streams as part of the current study and will be presented in a later section.

The dimensionless hydrograph was developed from observed flood hydrographs. Using data from 80 basins having drainage areas less than 20 mi^2, the method is as follows:

1. Compute a unit hydrograph and lag time for three to five storms for each of the 80 gauging stations. All unit hydrographs should be for the same time interval (duration) at a station. Lag time is computed as the time at the centroid of the unit hydrograph minus one-half the time of the computation interval (duration). The unit hydrograph computation method is by O'Donnell (3,pp.546-557).
2. Eliminate the unit hydrographs with inconsistent shapes and compute additional unit hydrographs if needed.
3. Compute an average unit hydrograph for each station by aligning the peaks and averaging each ordinate of discharge for the final selection of unit hydrographs. The correct timing of the average unit hydrograph is obtained by averaging the time of the center of mass of the individual unit hydrographs and plotting the average center of mass at this average time. The time of the center of mass of the discharge hydrograph is obtained by adding one-half the unit hydrograph computation interval (duration) to that hydrograph's lag time.
4. Transform the average unit hydrographs computed in step 3 to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lag time. These durations must be to the nearest multiple of the original duration (computation interval). These transformed unit hydrographs will have durations of two times, three times, four times, and six times the duration of the original unit hydrograph. The transformation of a short-duration unit hydrograph to a long-duration unit hydrograph (for instance, a 5-min to a 20-min duration) can be accomplished through the use of the following equations:

$$TUHD(t) = 1/2[TUH(t) + TUH(t-1)] \quad (D/\Delta t = 2) \tag{1}$$

$$TUHD(t) = 1/3[TUH(t) + TUH(t-1) + TUH(t-2)] \quad (D/\Delta t = 3) \tag{2}$$

$$TUHD(t) = 1/4[TUH(t) + TUH(t-1) + TUH(t-2) + TUH(t-3)] \quad (D/\Delta t = 4) \tag{3}$$

$$TUHD(t) = 1/n[TUH(t) + TUH(t-1) \ldots TUH(t-n+1)] \quad (D/\Delta t = n) \tag{4}$$

where

Δt = computation interval (the original unit hydrograph has an actual duration equal to Δt),
D = design duration of the unit hydrograph (this must be a multiple of Δt),
TUHD(t) = ordinates of the desired unit hydrograph at time t, and
TUH(t), TUH(t - 1), etc. = ordinates of the original unit hydrograph at times t, t - 1, t - 2, etc.

Duration may be thought of as actual duration or design duration, so a distinction must be made between the two. Actual duration, which is highly variable, may be defined as the time during which precipitation falls at a rate greater than that of the existing infiltration capacity. It is the actual time during which rainfall excess is occurring. Design duration is that duration which is most convenient for use on any particular basin. The design duration is that for which the unit hydrograph is computed. For this paper, design duration is expressed as a fractional part of lag time, such as one-fourth, one-third, one-half, and three-fourths. It is later shown that the design duration of one-half lag time provides the best fit of observed data.

5. Reduce the one-fourth, one-third, one-half, and three-fourths lag-time hydrographs to dimensionless terms by dividing the time by lag time and the discharge by peak discharge.
6. For Hydrologic Regions 1, 2, and 3 as defined by Price (1) and the Atlanta urban area as reported by Inman (2), compute an average dimensionless hydrograph by using the dimensionless hydrographs at the stations within that area or region. The hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio (Q/Q_p).

Steps 1 through 5 were done for all stations having data in the U.S. Geological Survey (USGS) WATSTORE unit-values file, which had hydrographs plotted from earlier studies. A total of 355 unit hydrographs from 80 stations, including 19 Atlanta urban sites, was used to develop the one-fourth, one-third, one-half, and three-fourths lag-time dimensionless hydrographs. A statistical analysis to select the best-fitting design duration was done by comparing the widths of hydrographs estimated (or computed) from the one-fourth, one-third, one-half, and three-fourths lag-time dimensionless hydrographs from each region or area with the observed hydrograph widths from their respective region or area. The one-half lag time was the best fit of width at 50 percent and 75 percent of peak flow. In Figure 1 plots of the one-half lag-time dimensionless hydrograph for Regions 1, 2, and 3 and for the Atlanta urban area are shown. On the basis of these plots, one dimensionless hydrograph was selected for both rural and urban conditions for the entire state as shown in Figure 2 and Table 1.

Another statistical analysis to test the accuracy of the dimensionless hydrograph application technique was done by comparing the simulated hydrograph widths at 50 and 75 percent of peak flow from simulated hydrographs using the statewide one-half lag-time dimensionless hydrograph with the 355 observed hydrographs. One example of this comparison is shown in Figure 3. The resulting standard error of estimate for the 50 percent of peak flow width comparison was ±31.8 percent and that for the 75 percent comparison was ±35.9 percent. The standard error of estimate of the width comparisons is based on mean-square difference between observed and simulated widths. Based on verification and bias testing, which are presented in a later section, this dimensionless hydrograph can be used for flood-hydrograph simulation for ungauged basins up to 500 mi^2. Steps 3 through 6 of the dimensionless hydrograph development and the statistical analyses were programmed for computer use by S.E. Ryan of the USGS.

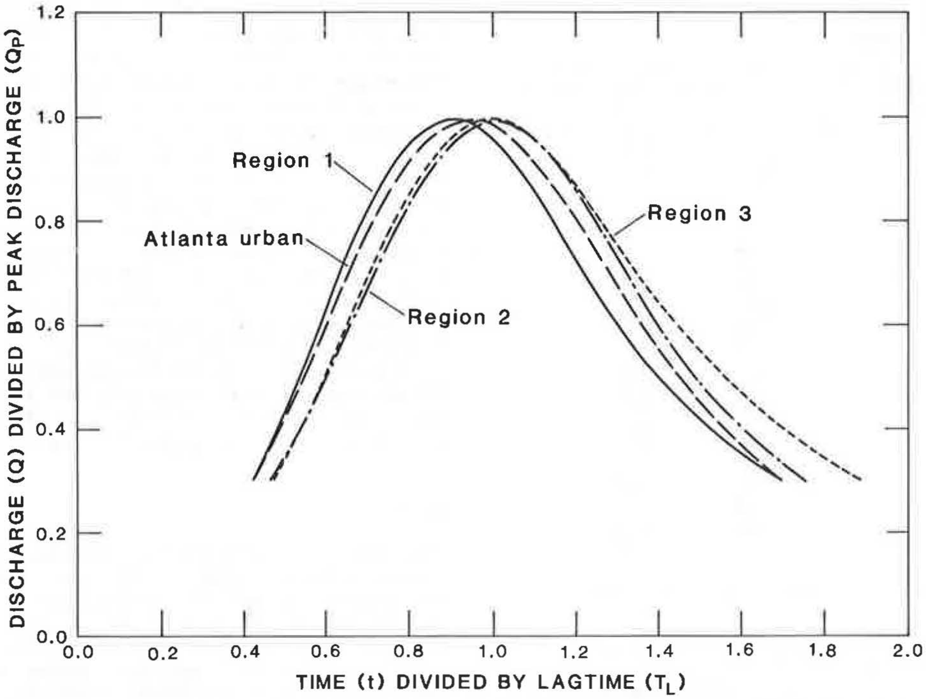

FIGURE 1 Average one-half lag-time dimensionless hydrographs for Regions 1, 2, and 3 and the Atlanta urban area.

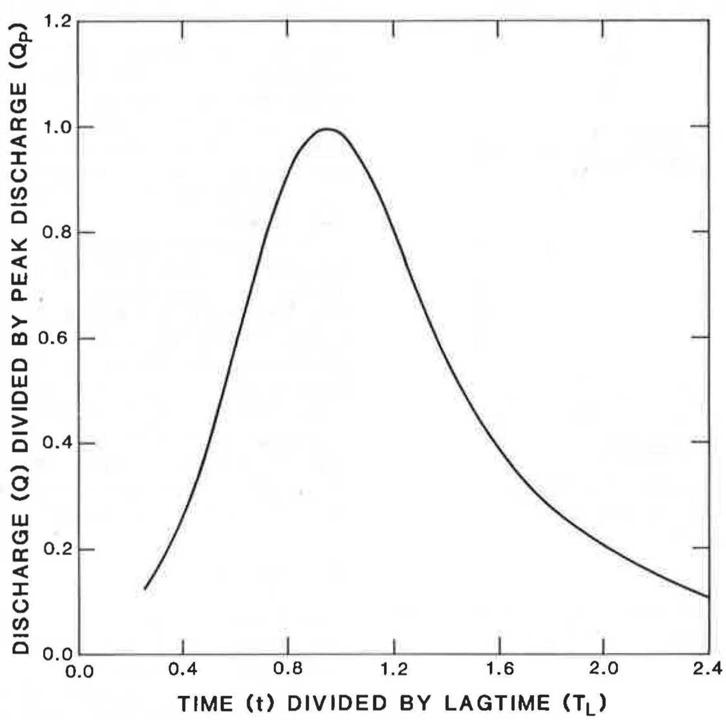

FIGURE 2 Statewide dimensionless hydrograph.

Basins Greater Than 500 mi²

The method for simulating a hydrograph at basins greater than 500 mi² uses the USGS computer model CONROUT. The model routes streamflow from an upstream channel location to a user-defined location downstream. CONROUT is described in detail by Doyle et al. (4).

CONROUT provides the user with two methods of routing: diffusion analogy and storage continuity. The diffusion-analogy method with single linearization as recommended by Keefer (5) was used in this study.

TESTING OF DIMENSIONLESS HYDROGRAPHS

Four tests are generally required to establish the soundness of models. The first test is the standard error of estimate, which has been explained and pre-

TABLE 1 Time and Discharge Ratios of Statewide Dimensionless Hydrograph

Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)	Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)
0.25	0.12	1.35	0.62
0.30	0.16	1.40	0.56
0.35	0.21	1.45	0.51
0.40	0.26	1.50	0.47
0.45	0.33	1.55	0.43
0.50	0.40	1.60	0.39
0.55	0.49	1.65	0.36
0.60	0.58	1.70	0.33
0.65	0.67	1.75	0.30
0.70	0.76	1.80	0.28
0.75	0.84	1.85	0.26
0.80	0.90	1.90	0.24
0.85	0.95	1.95	0.22
0.90	0.98	2.00	0.20
0.95	1.00	2.05	0.19
1.00	0.99	2.10	0.17
1.05	0.96	2.15	0.16
	0.92	2.20	0.15
1.15	0.86	2.25	0.14
1.20	0.80	2.30	0.13
1.25	0.74	2.35	0.12
1.30	0.68	2.40	0.11

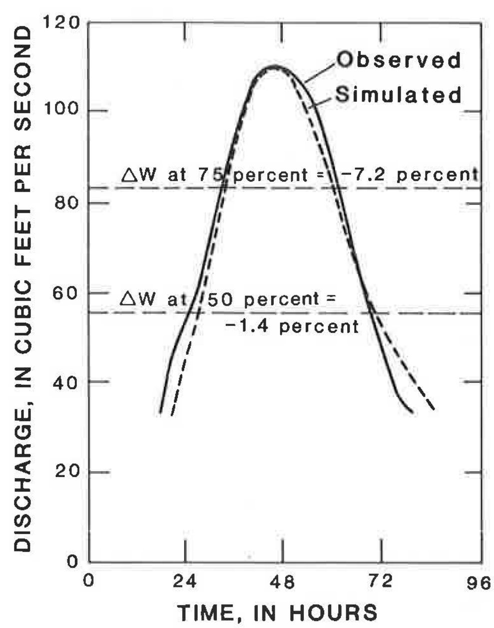

FIGURE 3 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Atlanta urban station.

sented in prior sections of this paper. The other tests are for verification, bias, and sensitivity.

Verification

For verification the dimensionless hydrograph was applied to other hydrographs not used in its development. This test included the use of 138 flood events from 37 stations having drainage areas of 20 to 500 mi^2 and located throughout the state. The average station lag time and peak discharge for each flood event were used to simulate a theoretical flood hydrograph, which was compared with the observed hydrograph. At 50 and 75 percent of peak flow widths the standard errors of estimate were ±39.5 and ±43.6 percent, respectively. Figure 4 gives an example of this comparison.

An additional verification, or test, of the entire simulation procedure was conducted on the highest peaks (simple or compound) having unit values available in the Georgia District and a station flood-frequency curve. Thirty-one stations having drainage areas of 20 to 500 mi^2 were tested as follows. The recurrence interval of this observed peak discharge (Q) was determined from the station frequency curve. The appropriate regional frequency equation from Price (1) was used to compute the corresponding peak discharge for this recurrence interval. The lag time (T_L) for this station was computed from the appropriate regional lag-time equation. The regression Q and regression T_L were then used to simulate a flood hydrograph. A comparison of the simulated and observed hydrograph widths at 50 and 75 percent of peak flow yielded standard errors of estimate of ±51.7 and ±57.1 percent, respectively. Figure 5 gives an example of this comparison.

Bias

Two tests for bias were conducted, one for simulated versus observed hydrograph width and the other for geographical bias. The width-bias test was performed on the widths at 50 and 75 percent of peak flow at the 31 stations used in the additional verification step. As explained earlier, these were the highest available floods at these stations. The average recurrence interval was about 30 years. The mean error ($\bar{x}$) indicated that there was a positive error (simulated greater than observed) in the hydrograph widths at 50 percent of peak flow and a negative error (observed greater than simulated) in the hydrograph widths at 75 percent of peak flow. Also, there was a negative error (estimated less than observed) in the comparison of peak Q from regional regression equations and peak Q from station frequency curves. However, Student's t-test indicated that these errors are not statistically significant at the 0.01 level, and therefore the simulated hydrograph widths are not biased.

The test for geographical bias was done by comparing the widths at 50 and 75 percent of the ratio (Q/Q_p) of the dimensionless hydrographs simulated for Regions 1, 2, and 3 as defined by Price (1), and shown in Figure 6, and for the Atlanta metropolitan area with the widths of the statewide dimensionless hydrograph. Figure 1 shows these four dimensionless hydrographs. There was no significant bias. In fact, the mean error ($\bar{x}$) was very small in both the 50 and the 75 percent tests, which further confirmed the decision to use one dimensionless hydrograph statewide for basins up to 500 mi^2.

Sensitivity

The fourth test was to analyze the sensitivity of the simulated hydrograph widths to errors in the two independent variables (Q and T_L) that are used to simulate the hydrograph. This test was done by holding one variable constant and varying the other by ±10 and ±20 percent at the hydrograph widths corresponding to 50 and 75 percent of peak flow, respectively. When peak Q was varied, the test results indicated that the hydrograph width did not change at 50 or 75 percent of that varied peak Q. When lag time was varied, the test results indicated that the hydrograph width varied by the same percentage.

REGRESSION ANALYSIS OF LAG TIME

So that lag time could be estimated for ungauged sites, the average station lag times obtained from

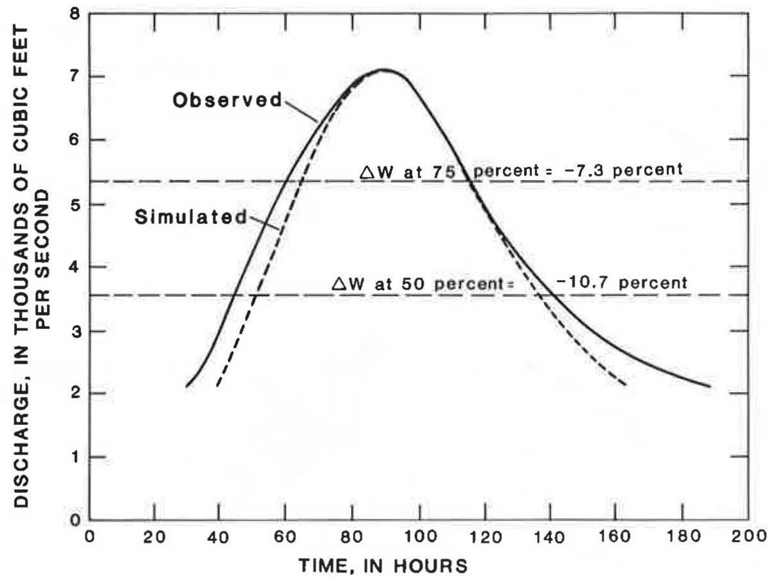

FIGURE 4 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Spring Creek near Iron City.

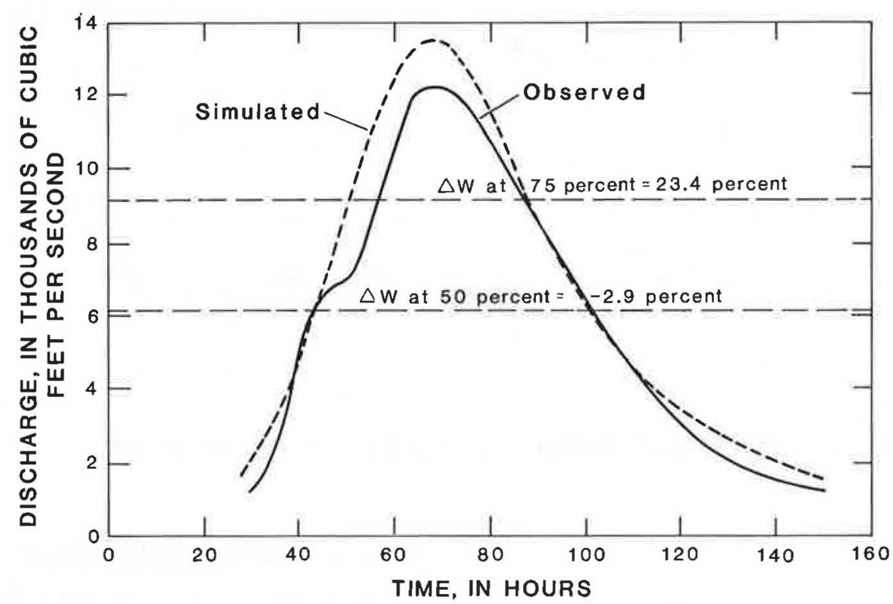

FIGURE 5 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Flint River near Griffin.

the stations used in the dimensionless hydrograph development were related to their basin characteristics. This was done by the linear multiple-regression method described by Riggs (6). Lag times were computed for each flood event with the same program that computed the t-hour unit hydrographs. These storm-event lag times were then averaged to compute an average station lag time, which was in turn used in the regression analyses. Lag time is generally considered to be constant for a basin and is defined by Stricker and Sauer (7) as the time from the centroid of rainfall excess to the centroid of the runoff hydrograph. Lag time for the 19 Atlanta urban stations was analyzed separately because of the effect of urbanization.

The regression equations provide a mathematical relation between the dependent variable (lag time) and the independent variables (the basin characteristics found to be statistically significant). All variables were transformed into logarithms before analysis to (a) obtain a linear regression model and (b) achieve equal variance about the regression line throughout the range. In the analyses performed, a 95 percent confidence limit was specified to select the significant independent variables.

The independent variables, or physical basin characteristics, are defined in the following paragraphs.

Lag Time (T_L)

T_L is the elapsed time in hours from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph. Lag time is computed from the unit hydrograph.

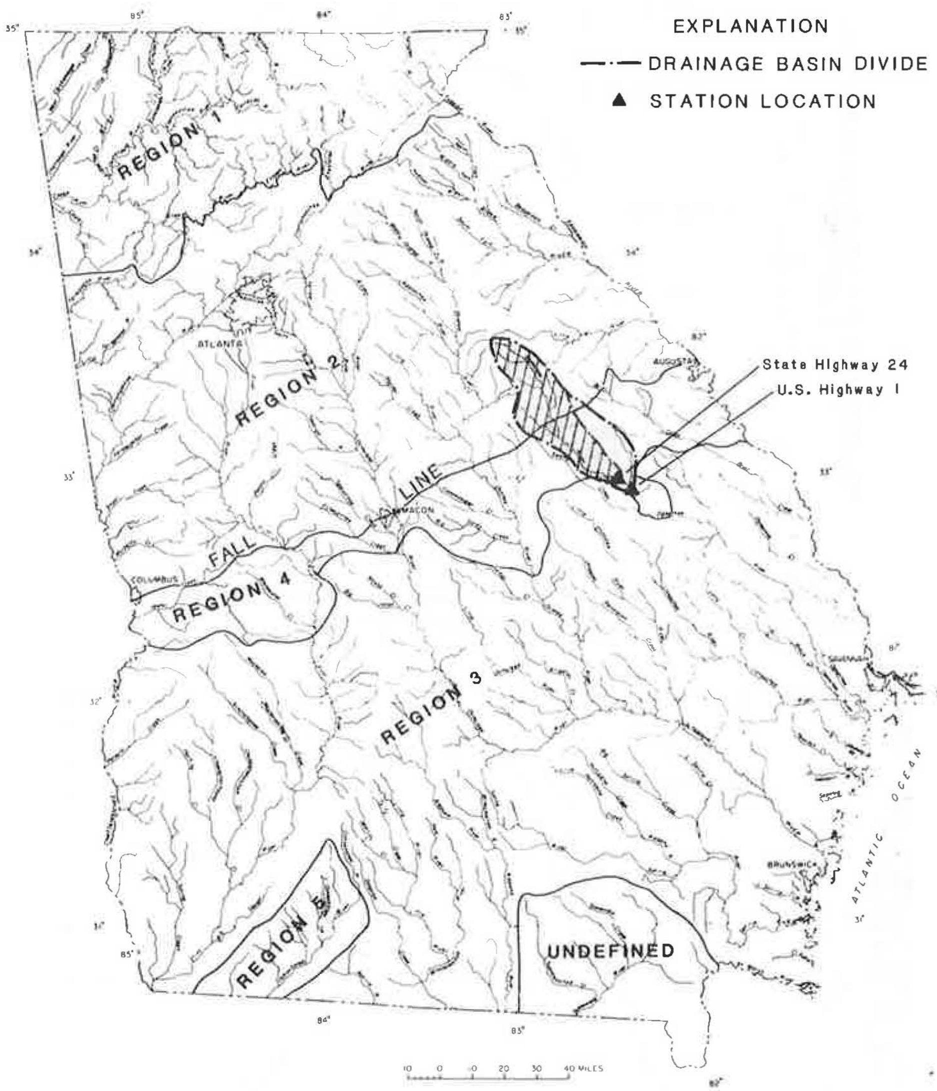

FIGURE 6 Regional boundaries for flood-frequency and lag-time estimating equations.

Drainage Area (A)

Area of the basin is in square miles and is planimetered from USGS 7.5-min topographic maps. Basin boundaries were all field checked.

Channel Slope (S)

The main channel slope is in feet per mile, as determined from topographic maps. The main channel slope was computed as the difference in elevation in feet at the 10 and 85 percent points divided by the length in miles between the two points.

Channel Length (L)

The length of the main channel is in miles, as measured from the gauging station upstream along the channel to the basin divide.

$L/S^{0.5}$

L and S have been previously defined.

Measured Total Impervious Area (IA)

The percentage of drainage area that is impervious to infiltration of rainfall is determined by a grid-overlay method using aerial photography. According to Cochran (8,pp.71-86), a minimum of 200 points, or grid intersections, per area or subbasin will provide a confidence level of 0.10. Three counts of at least 200 points per subbasin were obtained and the results averaged for the final value of measured total impervious area. On several of the larger basins where some development occurred during the period of data collection, this parameter was determined from aerial photographs made in 1972 (near the beginning of data collection) and then averaged with the values obtained from aerial photographs made in 1978 (near the end of data collection).

Measured Effective Impervious Area (MEIA)

The percentage of impervious area, which is directly connected to the channel drainage system, was obtained in conjunction with measured total impervious area. Noneffective impervious area, such as house rooftops that drain onto a lawn, is subtracted from

this total. When the minimum of 200 points was counted, three totals per subbasin were obtained. The first total was pervious points; the second, definite impervious points such as streets and parking lots; and the third, rooftops. One building out of three was field checked to determine the percentage of effective impervious area of its roof and gutter system. An average percent effective impervious area was determined for the buildings field checked in the subbasin, and this factor was multiplied by the total number of building points. The resulting product was added to the definite impervious points, and this total of effective impervious area points was divided by the total number of points counted in the subbasins to determine the MEIA percentage.

Regionalization

The initial regression run utilized data from 91 rural stations of less than 500 mi^2 located throughout the state. A geographical bias was detected. The area north of the fall line, consisting of Regions 1 and 2 as defined by Price (1) and shown in Figure 6, tended to overpredict lag time, whereas the area south of the fall line, consisting of Regions 3, 4, and 5 as defined by Price (1) and shown in Figure 6, tended to underpredict lag time.

The next step was to make separate regression runs for each of the five regions. Region 1 had no equations with two or more variables significant at the 95 percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 43 to 51 percent. Such large standard errors are not desirable. Region 2 also had no equations with two or more variables significant at the 95 percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 34 to 37 percent, with a tendency to overpredict at the lower end of the curve and underpredict at the upper end.

Regions 1 and 2 were combined and analyzed as one region. Two equations each have two variables significant at the 95 percent confidence limit. The equation selected was

$$T_L = 4.64A^{0.49}S^{-0.21} \quad (5)$$

Region 4 had only five stations and Region 5 only three. Therefore, neither region could be analyzed separately. Regions 3, 4, and 5 were combined and analyzed as one region. Only one equation had two variables significant at the 95 percent confidence limit. The equation was

$$T_L = 13.6A^{0.43}S^{-0.31} \quad (6)$$

The Atlanta urban area was analyzed separately because of the effects of urbanization on lag time. IA and MEIA were added as independent variables in the analysis. The following equation was selected:

$$T_L = 161A^{0.22}S^{-0.66}IA^{-0.67} \quad (7)$$

It is similar to the rural equations in that both rural and urban equations have area and slope as independent variables. Impervious area accounts for the urbanization effect. Drainage area (A) had a significance level of 6.8 percent but was retained in order to provide continuity with the rural equations. The Atlanta urban equation (7) should be considered preliminary and subject to revision after more urban data are analyzed in the Rome, Athens, Augusta, and Columbus metropolitan areas. If these additional data show the same regionalization pattern as do the rural data north of the fall line, then these data will be analyzed with the Atlanta data, which could possibly change the Atlanta urban equation.

The accuracy of regression equations can be expressed by two standard statistical measures: the coefficient of determination, R^2 (the correlation coefficient squared), and the standard error of regression. R^2 measures how much variation in the dependent variable can be accounted for by the independent variables. For example, an R^2 of 0.94 would indicate that 94 percent of the variation is accounted for by the independent variables and that 6 percent is due to other factors. The standard error of regression (or estimate) is, by definition, one standard deviation on each side of the regression line and contains about two-thirds of the data within this range. A summary of the lag-time equations and their related statistics is given in Table 2.

Limits of Independent Variables

The effective usable range of basin characteristics for the rural equations is as follows:

Variable	Minimum	Maximum	Unit
North of fall line			
A	0.3	500	Square miles
S	5.0	200	Feet per mile
South of fall line			
A	0.2	500	Square miles
S	1.3	60	Feet per mile

The effective usable range of basin characteristics for the Atlanta urban equation is as follows:

Variable	Minimum	Maximum	Unit
A	0.2	25	Square miles
S	13	175	Feet per mile
IA	14	50	Percent

TESTING OF LAG-TIME REGRESSION EQUATIONS

The lag-time regression equations were tested with the same four tests as those used for the dimensionless hydrograph. The standard error of estimate has been explained and presented in a prior section. Verification, bias, and sensitivity are the other tests.

TABLE 2 Summary of Lag-Time Estimating Equations

Area	Equation	Standard Error of Regression (%)	Coefficient of Determination (R^2)
North of the fall line (rural)	$T_L = 4.64A^{0.49}S^{-0.21}$	±31	0.94
South of the fall line (rural)	$T_L = 13.6A^{0.43}S^{-0.31}$	±25	0.96
Metropolitan Atlanta (urban)	$T_L = 161A^{0.22}S^{-0.66}IA^{-0.68}$	±19	0.94

Verification

Split-sample testing is the process by which part of a data set is used for calibration and the remaining part for verification or prediction. The standard error of estimate, obtained from the calibration phase, is a measure of how well the regression equations will estimate the dependent variable at the sites used to calibrate them. The standard error of prediction, on the other hand, is a measure of how well the regression equations will estimate the dependent variable at other than calibration sites according to Sauer et al. (9). Split-sample testing was used for verification of the regression equations both north and south of the fall line. It was also used to estimate the magnitude of the average prediction error and to determine whether the same variables were significant. The stations from each region were divided into two groups of about equal size. The sites were arrayed in ascending order according to drainage-area magnitude. The odd-numbered events made up the first sample and the even-numbered events the second sample. Multiple-regression analyses were performed on both regions using only the sites in one of the samples; then the equations were recalibrated using the sites in the other sample. The results were all acceptable, as shown in Table 3. The regression analyses yielded new regression equations similar to the equations originally developed by using all the sites in each region.

The first set of equations tentatively selected had area (A) and $L/S^{0.5}$ as the two independent variables. The standard errors of regression were about the same as for the equations with A and slope (S) as independent variables for both regions. However, when split-sample testing was performed, $L/S^{0.5}$ was not significant at the 95 percent confidence limit for either the odd or the even sample above the fall line. The equations with A and $L/S^{0.5}$ was split-sample tested for the area south of the fall line, and A was not significant at the 95 percent confidence limit for either the odd or the even sample. No attempt was made to analyze the Atlanta urban equation with split-sample testing because of the limited number of stations available.

Bias

Two tests for bias were performed, one for variable bias and the other for geographical bias. The variable-bias tests were made by plotting the residuals (difference between observed and predicted lag time) versus each of the independent variables for all stations. These plots were visually inspected to determine whether there was a consistent overprediction or underprediction within the range of any of the independent variables. These plots also verified the linearity assumptions of the equations. The equations were found to be free of variable bias throughout the range of all independent variables.

Geographical bias was tested by plotting the residuals of observed lag times minus predicted lag times on a state map. The plot was visually inspected to determine whether any area of the state was consistently overestimated or underestimated. Because this test indicated no consistent overestimation or underestimation in any part of the state, it can be concluded that no geographical bias exists.

The same bias analyses were performed on the Atlanta urban equation. There was no geographical or variable bias.

Sensitivity

The fourth test was to analyze the sensitivity of lag time to errors in the two independent variables in the regression equations. The computation of these independent variables is subject to errors in measurement and judgment. To illustrate the effect of such errors, the equations were tested to determine how much error was introduced into the computed lag time from specified percentage errors in the independent variables. The test results are shown in Table 4, which was computed by assuming that all independent variables were constant except the one being tested for sensitivity.

The Atlanta urban equation was tested for sensitivity of lag time to errors in the three independent variables in the same manner as that for the two rural equations. The test results are shown in Table 4.

SUMMARY

A dimensionless hydrograph was developed for Georgia streams having drainage areas of less than 500 mi². This dimensionless hydrograph can be used to simulate flood hydrographs at ungauged sites for both rural and urban streams statewide. More than 350 observed flood hydrographs were used for its development. For verification, the dimensionless hydrograph was applied to 169 flood hydrographs not used in its development.

Multiple-regression analysis was used to define relations between lag time and selected basin characteristics, of which drainage area and slope were significant for the rural basins and drainage area, slope, and impervious area were significant for the Atlanta urban basins. The rural equation was regionalized into one equation for the area north of the fall line and one equation for the area south of the fall line. Both rural equations were verified by split-sample testing. There was neither variable nor

TABLE 3 Split-Sample Test Results for Lag-Time Equations

Area	No. of Stations	Equation	Standard Error of Regression (%)	Standard Error of Prediction (%)	Coefficient of Determination (R^2)
North of fall line					
Odd	25	$T_L = 4.88A^{0.48}S^{-0.22}$	±32	–	0.94
Even	24	–	–	±32	0.93
Even	24	$T_L = 4.51A^{0.50}S^{-0.21}$	±31	–	0.94
Odd	25	–	–	±32	0.94
South of fall line					
Odd	21	$T_L = 36.8A^{0.35}S^{-0.57}$	±18	–	0.98
Even	21	–	–	±41	0.92
Even	21	$T_L = 8.63A^{0.48}S^{-0.21}$	±26	–	0.96
Odd	21	–	–	±29	0.96

Note: Dashes indicate data not applicable.

TABLE 4 Sensitivity of Computed Lag Time to Errors in Independent Variables

Percentage Error in Independent Variable	Percentage Error in Computed Lag Time by Independent Variable		
	Area	Slope	Impervious Area
Equation for North of Fall Line			
+50	+21.9	-8.2	
+25	+11.5	-4.6	
+10	+4.8	-2.0	
-10	-5.0	+2.2	
-25	-13.1	+6.2	
-50	-28.5	+15.7	
Equation for South of Fall Line			
+50	+19.2	-11.8	
+25	+10.1	-6.7	
+10	+4.2	-2.9	
-10	-4.5	+3.3	
-25	-11.7	+9.4	
-50	-25.9	+24.1	
Atlanta Urban Equation			
+50	+9.9	-23.4	-23.9
+25	+5.4	-13.5	-14.0
+10	+2.7	-5.9	-6.3
-10	-2.2	+7.2	+7.2
-25	-5.9	+21.2	+21.2
-50	-14.0	+58.1	+59.0

geographical bias in either the rural equation or the Atlanta urban equation. Sensitivity tests indicated drainage area as the most sensitive basin characteristic in the rural equation and impervious area as the most sensitive in the Atlanta urban equation.

A simulated flood hydrograph may be computed by applying lag time, obtained from the proper regression equation, and peak discharge of a specific recurrence interval to the dimensionless hydrograph. The coordinates of the runoff hydrograph can be computed by multiplying lag time by the time ratios and peak discharge by the discharge ratios in Table 1.

For basins larger than 500 mi^2 the USGS computer model CONROUT is used for simulating flood hydrographs. CONROUT routes streamflow from an upstream channel location to a user-defined location downstream. The product of CONROUT is a simulated outflow discharge hydrograph with a peak of a specific recurrence interval at the end of a reach.

ACKNOWLEDGMENTS

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Effect of Main-Channel Orientation on Flood Peaks for Streams in Ohio

JOHN OWEN HURD

ABSTRACT

Flood data from 205 unregulated rural drainage basins ranging in size from 0.012 to 5,993 mi^2 were analyzed to determine whether main-channel orientation had any effect on flood peaks. No significant effect on flood peaks was found for unregulated rural streams in Ohio except for steep basins (main-channel slope > approximately 1 percent) that flow in nearly the same direction as the prevailing storm movement (<20-degree deviation). These basins in general produce higher flood peaks than those predicted by the U.S. Geological Survey regional regression equations for Ohio.

Numerous studies (1-12) have addressed the fact that storm patterns affect peak runoff rates for small drainage basins. Abraham et al. (13) and Masch (14) showed that storms traveling in the same direction as the main-channel flow of the basin produce flood peaks of shorter duration and greater magnitude than storms traveling directly opposite the flow of the main channel. This effect is normally increased with an increase in the ratio of length to drainage area of the basins. For most areas of the United States there is a prevailing direction of rainstorm movement (15,16). Considering the foregoing, it would be reasonable to question whether streams flowing in the same direction as the prevailing direction of storm movement would have significantly different flood peaks than streams flowing in the opposite direction.

Previous studies (17-20) that developed peak-runoff prediction equations for ungauged and unregulated rural basins in Ohio have not addressed the effect of drainage-basin orientation on floods. The latest report by Webber and Bartlett (20), based on data from 215 gauging stations, presents predictive equations for flood peaks with recurrence intervals of 2, 5, 10, 25, 50, and 100 years for five separate geographic areas of the state (see Figure 1). Basin characteristics that had a significant effect on discharge for all five geographic areas were drainage area and main-channel slope. Basin characteristics that had a significant effect on discharge for one or more but not all five areas were surface storage, average basin elevation, and average annual precipitation.

Because there is a prevailing direction of rainstorm movement in Ohio generally from west to east except along Lake Erie, this study was undertaken to determine whether drainage-basin orientation had any effect on peak-runoff rates for unregulated rural streams in Ohio. If possible with available data, a quantitative relationship was to be developed and an adjustment factor provided for the existing predictive equations. Thus an improved estimate of flood flows would be provided to the designers of hydraulic structures.

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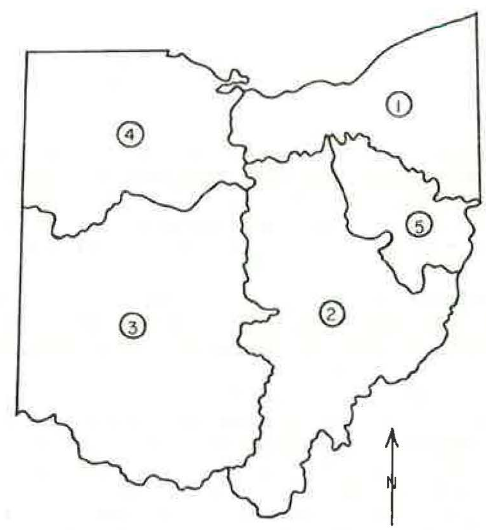

FIGURE 1 Geographic areas of Ohio for U.S. Geological Survey flood-prediction equations.

DATA DEVELOPMENT

The 215 unregulated gauged sites used by Webber and Bartlett (20) to develop their predictive equations were used as the data base for this study. Ten stations were dropped from the data base because a large percentage of their basin areas was in forest, which tends to attenuate flood peaks. Drainage areas for these streams ranged from 0.012 to 5,993 mi^2. Peak-flow data developed from station records by Webber and Bartlett with methods described in Water Resources Council (WRC) Bulletin 17B (21) were used as the observed station flood peaks for this study. Although an additional 7 years of records were available on some stations used by Webber and Bartlett, most of these stations with additional record length were already long-term ones. Therefore, it was believed that the 10-fold or greater increase in study cost to reestablish estimated station flood peaks was not warranted for this study. This increase in cost would have been even more infeasible if several methods of flood peak estimation were used at each station as recommended by Wallis and Wood (22).

Gauging-station record lengths used by Webber and Bartlett are as follows:

Record Length (years)	No. of Stations
15 or less	45
16-25	35
26-35	66
36-45	26
>45	33

Most of the stations with short-term records that had not been discontinued before the report were discontinued after its publication (20). The Ohio Department of Transportation (ODOT) currently has a research contract with the Water Resources Division, U.S. Geological Survey (USGS), to perform an update of the report by Webber and Bartlett that will include additional data on strip-mined, forested, and northwest Ohio basins. The station flood estimates will also be updated. Predicted flood peaks were generated by using the equations developed by Webber and Bartlett from the basin characteristics listed in that report.

In order to quantify drainage-basin orientation for this study, the main-channel orientation angle of each drainage basin was initially developed as shown in Figure 2. A main-channel vector was constructed from a point on the main channel located at 85 percent of the total main-channel distance from the gauging station to a point at 10 percent of the distance from the gauging station. The angle (from 0 to 180 degrees measured north or south) between this vector and a vector running due west to east was defined as the main-channel orientation angle for the basin. The 85 and 10 percent points were selected because they are the stream distances commonly used in determining main-channel slope and mean basin elevation in the USGS regression equations used by numerous states for flood estimates. If initial data analyses provided quantifiable relationships between channel orientation angle and flood peaks, other means of defining the angle were to be studied. Channel orientation angles were determined for the data sites by using topographic maps with scales ranging from 1:2,400 to 1:625,000. The scale of map selected was that most convenient for the size of drainage area for each basin.

DATA ANALYSES

The ratios of previously defined observed flood peaks to predicted flood peaks for Q_{10}, Q_{25}, Q_{50}, and Q_{100} were computed for each station. The flood peaks Q_2 and Q_5 were not considered because ODOT does not use floods of these frequencies in the design of hydraulic structures. These ratios were compared initially with both main-channel orientation angles and the cosines of these angles by using several linear and log-linear regression relations. The cosine of the angle was initially selected because it was thought to be the most representative of the theoretical relationship between orientation angle and flood ratios. If initial analyses provided quantifiable results, other measures of angle were to be studied. Comparisons were done for the total data set and several subsets based on drainage area:

Drainage Area Size (mi²)	No. of Sites
All areas	205
1,000 or less	191
>1,000	14
300 or less	151
>300	54
100 or less	105
>100	100
30 or less	76
>30	129

The ratios of observed 25-year flood peaks to predicted 25-year flood peaks for basins of 30 mi² or less are plotted against main-channel orientation angles in Figure 3.

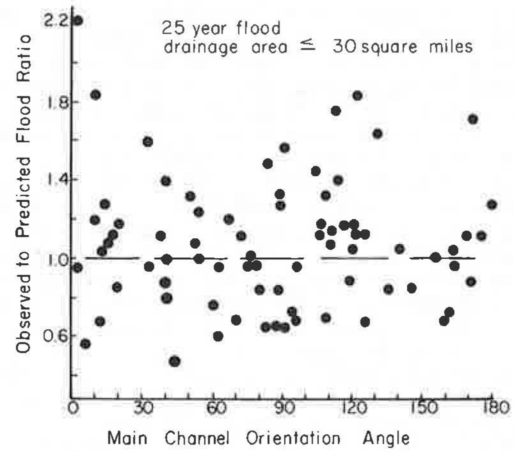

FIGURE 3 Observed to predicted flood ratios for basins with drainage areas 30 mi² or less.

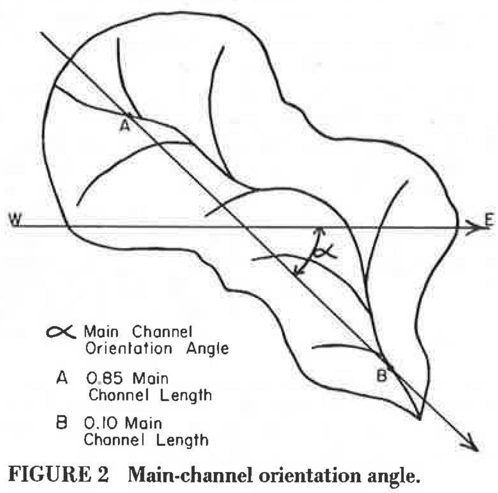

FIGURE 2 Main-channel orientation angle.

Results of the regression analyses showed that there was no significant relation between the ratio of observed flood peaks to predicted flood peaks and main-channel orientation for any combination of drainage area data set and recurrence interval.

Because channel orientation was not considered by Webber and Bartlett, there was some question whether the geographic breakdown of the state or the variables used in the predictive equations in that report could have accounted for any effect of channel orientation. In order to determine whether there was any bias in stream-channel orientation among the geographic areas considered by Webber and Bartlett, chi-square analyses were performed on the channel orientation data.

The main-channel orientation angles were divided into three groups for each geographic area: west to east-flowing (angle ≤ 45 degrees), east- to west-flowing (angle ≥ 135 degrees), and north- to south- and south- to north-flowing (angle between 45 and 135

degrees). The number of stations in each group for each geographic area was as follows:

Geographic Area	No. of Stations by Flow Direction		
	E to W	N to S and S to N	W to E
1	4	26	6
2	2	28	17
3	23	37	15
4	11	16	6
5	4	7	3

There were significant differences at the 99 percent confidence level in the number of streams in each orientation-angle category among the geographic areas. Geographic Area 2 had significantly more west- to east-flowing streams than the other areas and Geographic Area 1 had significantly more south- to north-flowing streams than the other areas.

Because of this bias it was decided to rerun the stepwise regression analysis for Q_{10} through Q_{100} previously performed for the statewide data set by Webber and Bartlett. The independent variables for channel orientation angle and cosine of that angle were included with those variables found to have a significant effect on floods in that study. Results showed that neither channel orientation angle nor the cosine of that angle had any significant effect on floods for any recurrence interval. Therefore, any bias in main-channel orientation angle among different geographic areas shown by Webber and Bartlett did not affect the results of that study.

A possible explanation for the lack of effect of channel orientation on flood magnitudes for the data sets studied is the large difference in the response time of the rural basin to a rainfall event and the time it takes the rainstorm to cross the basin. Rainstorms that produce flood peaks on most drainage areas in Ohio travel at approximately 35 to 45 mph (or approximately 50 to 65 ft/sec). This rate of storm movement produces a rather short storm travel time across the basin. The average velocity of flood flows in most natural channels in Ohio is in the range of 5 to 10 ft/sec. The rate of flood wave movement is even less than this. A difference in the two travel times of approximately one order of magnitude would create an actual condition very close to the idealized case of "uniform rainfall" across the basin. In such a case main-channel orientation would be of little consequence.

In order to test whether main-channel orientation might affect peak flows in basins with faster response times, a data subset of basins with main-channel slopes of 50 ft/mi (approximately 1 percent) or greater was analyzed. This included 31 basins that ranged in size from 0.012 to 14.0 mi². The ratios of observed 25-year flood peaks to predicted 25-year flood peaks for these basins are plotted against main-channel orientation angles in Figure 4.

The same regression analyses performed on the drainage-area data set and subsets shown at the beginning of this section were applied to the data for the foregoing 31 basins. Results of the regression analyses showed that there was no significant relation between flood-peak ratios and main-channel orientation for these basins with steeper main-channel slopes.

Although no regression relation could be developed for the entire range of main-channel orientation angle, observation of Figure 4 and similar plots for Q_{10}, Q_{50}, and Q_{100} showed that 5 of the 6 basins with main-channel orientation angles <20 degrees had ratios of observed to predicted floods >1.0, whereas only 13 of the remaining 25 basins had ratios >1.0. Chi-square analyses comparing these 6 basins with the other 25 basins in this data set showed with 80 percent confidence that this difference could not have occurred randomly.

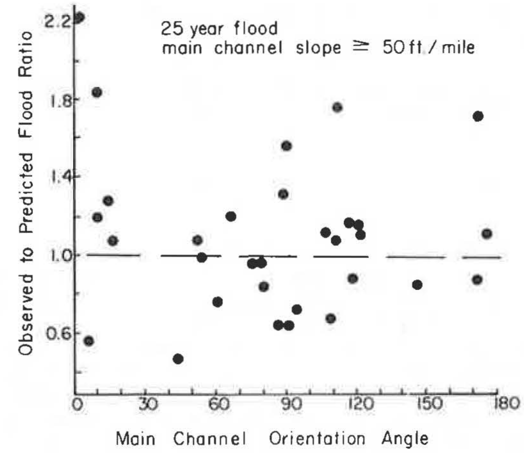

FIGURE 4 Observed to predicted flow ratios for basins with main-channel slopes 50 ft/mi or greater.

Considering the foregoing finding, the plots of flood ratio versus angle for the data sets used by Webber and Bartlett (shown in the section on data development) were tested to compare ratios of basins with main-channel orientation angles ≤20 degrees with ratios from basins with angles >20 degrees. This comparison was also made for a data set composed of 33 basins with drainage areas <15.0 mi² and channel slopes <50 ft/mi. There was no significant difference between ratios for basins with angles >20 degrees and those with angles ≤20 degrees for any of these data sets.

These findings indicate that main-channel orientation angle has no effect on flood peaks except for steep (main-channel slope approximately 1 percent or greater) basins with orientation angles ≤ approximately 20 degrees. These basins appear to have significantly larger observed flood peaks than those predicted by Webber and Bartlett. The mean value of observed to predicted 25-year flood ratios for these six basins was 1.36. However, insufficient data exist to develop any reliable quantifiable relationship or to determine whether the ratio of basin length to drainage area or other factors affect the relationship.

Although it was beyond the scope of this study, an examination of existing stream-flow data on gauged sites in neighboring states (Pennsylvania, West Virginia, Kentucky, etc.) should be made. Steep basins with orientation angles ≤ approximately 20 degrees with sufficient years of record should be added to the six Ohio basins to form a larger data set. At this point an update of estimated gauging-station flood peaks could be made at stations where there are significant increases in record lengths from those used in existing estimates. This data set, it is hoped, would produce a quantifiable adjustment factor for existing flood-peak prediction models. If such is not the case, new gauging stations would be required to expand the set of stations.

Floods for urbanized areas that have much faster basin response times may also be affected by channel orientation. However, at this time data were not available to investigate floods from urban basins in Ohio.

CONCLUSION

Main-channel orientation had no significant effect on flood peaks for unregulated rural streams in Ohio except for steep (main-channel slope > approximately 1 percent) basins that flow in nearly the same direction as the prevailing storm movement (<20-degree deviation). These basins produce higher flood peaks than those predicted by USGS regional regression equations as given by Webber and Bartlett (20) by an average of 36 percent for 25-year floods.

RECOMMENDATIONS

The effect of basin orientation on urban flood peaks in Ohio should be investigated when data become available.

Additional rural basins in and around Ohio with steep main-channel slopes and flow directions with small deviations from prevailing storm movement should be studied to more precisely quantify the effect of direct channel alignment with prevailing storm movement. Adjustment of prevailing storm movement from due west to east should be made based on geographic location, and the effect of basin length-to-width ratio should be considered.

At those sites where failure of the hydraulic structure to adequately convey flood flow is critical, an increase in the flood peaks predicted by the USGS regression equations should be made on those basins with main-channel slopes >50 ft/mi and main-channel orientation angles <20 degrees.

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Evaluation of Alternative Hydrograph Methods for Hydraulic Design

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ABSTRACT

A protocol is established to evaluate nine methods for defining hydrographs. Data from one site are used. The context is a need for dynamic hydraulic analysis to downsize drainage structures resulting from designs developed by using conservative steady-state flow assumptions. In addition, the analysis addresses a situation in which measured rainfall or runoff data are missing at a site. Data are generated for a hypothetical site and data requirements and computational methods are evaluated. The test results indicate that Snyder's method and its modification by Constant are superior. The protocol appears to be general enough to use in a comprehensive evaluation of hydrograph generation methods for use in ungauged watersheds. The comprehensive evaluation would draw on a national database and would determine criteria for selecting appropriate hydrograph generation methods to support dynamic hydraulic analysis.

The objective of this paper is to propose a protocol for reviewing methods used to generate hydrographs of small to medium-sized ungauged watersheds. The hydrographs are used to design culverts, bridges, and storm sewers and for inputs to dynamic hydrologic simulations. Dynamic simulation is used to help safely economize drainage structures. Design selections that result from static or steady-state analysis tend to be larger because the influence of system storage is not exploited. Smaller system elements can be sized under dynamic hydraulic conditions because of the effects of temporary storage as well as the effects of dynamic interactions of drainage system elements.

For very small drainage areas, the rational method has become the standard for sizing drainage structures on the basis of peak flows. For larger drainage areas, in which basin storage characteristics need to be accounted for, several competing hydrograph methods have been proposed, each with its own set of adherents. No hydrograph method has yet emerged as a de facto standard. Thus, the objective of this paper is to examine various methods that can be used with microcomputers and to suggest a protocol by which further investigations might be conducted to encourage the adoption of one or more methods as de facto standards.

This paper is divided into four sections: background, a discussion of several hydrograph methods, application of the methods to hypothetical and actual watersheds, and conclusions and recommendations.

BACKGROUND

Procedures developed for hydraulic and hydrologic analysis span a rather broad range. Each procedure has its own strengths and weaknesses and its own set of devotees. Requirements for analytical consistency cause specific analytic procedures to emerge as standards. For example, in the area of floodplain backwater calculations, the U.S. Army Corps of Engineers HEC-2 model has become the de facto standard, accepted by both legal and technical professionals. More recently, the Federal Emergency Management Agency designated the FHWA-U.S. Geological Survey (USGS) WSPRO model as an acceptable method in this area. As a result of federal interagency committee decisions, the Log Pearson Type III extreme-value methods became the standard for flood-peak estimation of gauged stream flows. It is clear in specific cases that standard methods do not necessarily work as well as other alternatives; however, standard methods provide a uniform basis of comparison between studies and also ensure that errors of judgment in selecting alternatives are avoided.

Synthetic hydrographs have been developed because of a lack of stream-flow and rainfall data for many watersheds. The synthetic methods require significantly less data and effort to develop compared with construction of a hydrograph solely from actual gauge data. Unlike the examples cited earlier, no single synthetic unit hydrograph method is universally accepted throughout the United States because of the wide range of geographical and climatic regimes and different institutional approaches. It may be that it is now time to consider standardization of hydrograph generation methods.

The rainfall-runoff response of watersheds can influence the design of highway stream crossings and highway surface drainage systems. Sophisticated data analysis and modeling can be used to predict the response of a watershed to a precipitation event in the presence of adequately measured data. However, the use of synthetic unit hydrographs may be appropriate for watersheds for which there are no rainfall data. Watershed attributes are used instead.

A hydrograph is a continuous graph depicting the discharge from a watershed with respect to time. It characterizes the response of a watershed to a specific precipitation event and integrates geometric and climatologic factors.

A unit hydrograph is "the hydrograph that results from one inch of precipitation excess generated uniformly over the watershed at a uniform rate during a specified period of time" (1). Unit hydrographs are commonly used to predict peak discharge rates and the pattern of that discharge over time using the runoff produced during the precipitation event. For a

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unit hydrograph it is assumed that the runoff occurs from precipitation excess (i.e., the difference between precipitation and losses) and that the excess is created at a uniform rate and with a uniform spatial distribution (1).

For rainfall events producing an excess of other than 1 in. of runoff, the amount of excess rainfall is simply multiplied by the unit graph ordinates. It is assumed that the time base of the hydrograph remains unchanged and that the ordinates are directly proportional to the amount of rainfall. The shape of a unit hydrograph derived from measured data depends on the temporal and spatial distribution of rainfall excess.

The design approach for using unit hydrographs to secure design information for highway drainage is

- Select a method,
- Estimate the unit hydrograph from watershed characteristics,
- Select a return period,
- Estimate the rainfall excess or the peak flow associated with the return period, and
- Develop the hydrograph from the unit hydrograph and the rainfall excess or peak flow.

METHODS

Nine methods for developing synthetic unit hydrographs are examined. These methods and the agencies that use them are given in Table 1. There are, of course, other methods that are not investigated in this paper. Six common hydrologic-hydraulic models that incorporate eight of the nine methods are as follows:

TABLE 1 Synthetic Unit Hydrograph Methods Examined

Method	Reference	User Agency
Snyder's	*2, 3, 4*	Corps of Engineers
SCS		
Dimensionless	*1, 3*	Soil Conservation Service
Triangular	*1, 3*	Soil Conservation Service
Clark's	*5*	Corps of Engineers
Grey's	*2*	–
Constant's	*6, 7*	Corps of Engineers
SBUH	*8, 9*	–
HYMO	*2*	Agricultural Research Service
USGS dimensionless	*10, 11*	U.S. Geological Survey

Model and Developer	Method Used
HEC-1 [Corps (5)]	Snyder's, Clark's
TR-20 [SCS (1)]	SCS dimensionless
CDS [FHWA (10)]	USGS
HYMO [ARS (2)]	Grey's, HYMO
HYDRO [FHWA (7)]	Constant's
SSAD [Golding (8)]	SBUH

A discussion of each of the nine methods follows.

Snyder's Method

Snyder's method was developed for Appalachian watersheds ranging from 10 to 10,000 mi². It has been applied to watersheds in most of the continental United States.

The method uses seven input parameters: watershed area (A), overall length (L), length to watershed centroid (L_{CA}), volume of excess rainfall (Q), rainfall duration (D), and two empirical coefficients (C_p and C_t) to calculate peak flow (Q_p) and time to peak (t_p). The unit hydrograph is constructed by calculating 50 and 75 percent of the peak flow, the corresponding times, and the time at which the flow returns to zero (or base) flow. Spline techniques are used to fit the points into a smooth unit curve.

C_t, the first empirical coefficient, represents the variation of the unit hydrograph lag time with respect to watershed slopes and storage. A typical value used for C_t is 2.0. Values of C_t have been found to vary from 1.8 to 2.2 in the Appalachian highlands and from 0.4 in Southern California to 8.0 in the eastern Gulf of Mexico (3). Schultz (4) provides the values of C_t for several differing watershed types.

The second empirical coefficient is C_p, which represents the variation of unit hydrograph peak discharge with watershed slope, storage, lag time, and effective area. Values of C_p usually range from 0.4 to 0.94 with a typical value of 0.6 (3).

Soil Conservation Service (SCS) Methods

The SCS unit hydrograph methods were designed for watersheds from 0 to 2,000 acres. The dimensionless method was created by averaging a large number of hydrographs collected from watersheds across the continental United States and calculating a scaled table of dimensionless ordinates. The triangular method uses a simple application of the unit hydrograph theory for derivation of the hydrograph shape.

An important input parameter for determining the peak flow is the time of concentration (t_c) for the watershed. The SCS developed two methods to calculate t_c: the graphical and the curve number. The graphical method relates overland and channel slopes and topographical features to the velocities of the runoff. With these velocities and the overland and channel lengths, t_c is readily calculated. The curve-number method allows the input of more precise geographical and hydrological factors. This method reflects actual watershed characteristics and is one of the more detailed empirical means of evaluating watershed t_c.

A second direct input parameter in the SCS unit hydrograph methods is an empirical constant (K), which represents the fraction of the area under the rising limb of the hydrograph. A typical value of K is 484. This value represents a hydrograph with 3/8 of its area under the rising limb. The fraction is less for flat, swampy areas ($K \simeq 300$) and greater for mountainous watersheds ($K \simeq 600$).

Time of concentration and the constant K, along with A, D, and Q, allow the calculation of Q_p. Q_p and t_c are used to create the unit hydrograph shape.

Clark's Method

Clark's method routes the incremental runoff from a watershed through a linear reservoir and translates the data into hydrograph ordinates. Clark's method can be applied to a wide range of watershed areas.

The input parameters are A and t_c. These parameters are used to section the watershed along its primary watercourse into subareas containing equal travel times. The translated flows are then routed through storage that is assumed to be at the outflow location. An attenuation constant (R), evaluated at the point of inflection of the recession limb of the hydrograph, is defined by successive iterations of the method.

As a result of the iterative process, Clark's method lends itself to use on a computer. In addi-

tion, it offers the ability to make adjustments for changes in drainage characteristics without requiring a complete reworking of the problem.

Grey's Method

Grey's method was designed for Midwestern watersheds of 0 to 94 mi^2. This method consists of a two-parameter derivation and is sensitive to the accuracy of the input data. It does not lend itself to simple applications. It requires calculation of a gamma distribution parameter (Γ), construction of an S-curve, and routing of the curve for the desired time interval. The final calculated values do not require the use of empirical constants. A practical drawback is the amount of analysis time required to determine the ordinates of the hydrograph.

Constant's (Modified Snyder's) Method

Constant's method was developed by the Albuquerque District of the U.S. Army Corps of Engineers. It is a variation of the techniques used in Snyder's unit hydrograph method and, as such, is similar in application and data requirements. In Constant's method it is recognized that the rising limb of a unit hydrograph can be expressed as a parabolic function and that the falling limb can be expressed as a function of exponential decay. A general equation was developed by using curve-fitting techniques to include the points calculated by Snyder's method.

The inputs for the method are Q_p, A, t_p, and the desired time interval. These features allow Constant's method to be applied to any hydrologic technique that calculates Q_p and t_p (or, indirectly, t_c). The method can be used for any technique that closely simulates the available watershed data. It is a convenient method because it uses peak flow rather than rainfall excess as an input variable. The peak-flow return can be estimated by USGS ungauged techniques that use watershed attributes and have been developed by using regression analysis for most of the country.

Santa Barbara Method

The Santa Barbara unit hydrograph (SBUH) method is a linear reservoir routing method similar to Clark's. An important difference is that the SBUH method uses Horton's equation to calculate infiltration rates (and thus rainfall excess). This approach allows the creation of a hydrograph that is adjusted for an estimated volume of rainfall excess. The SBUH method also uses such watershed characteristics as fraction of impervious area, antecedent moisture conditions, and soil classification as input parameters. The requirement of a hyetograph necessitates more complete rainfall data than are required by other unit hydrograph methods.

HYMO Method

The HYMO method is a dimensionless unit hydrograph developed for the Agricultural Research Service (ARS). It is based on data from watersheds of 0.5 to 25 mi^2 located in Arkansas, Louisiana, Mississippi, Oklahoma, Tennessee, and Texas. The dimensionless unit hydrograph is synthesized by applying computed parameters to three segment equations. Input variables are A, Q, L, watershed width (W), elevation difference (SLP), and two empirical parameters (n,B). There are three calculated parameters; two are regional regression equations--one for t_p and a regional recession constant K_R--and the third parameter is for Q_p.

The empirical parameters are dimensionless and are related to the shape of the watershed. The parameter B is a function of n. It has a purpose and range similar to those of the SCS empirical constant (K) and is used to determine the peak flow. The parameter n is a function of K_R and t_p.

USGS Urban Method

The USGS developed a dimensionless unit hydrograph by using a method similar to that used to develop the SCS dimensionless unit hydrograph. The difference is that the USGS data used to develop the ordinates consisted of urban and western watershed characteristics (as opposed to the rural nationwide watershed characteristics of the SCS dimensionless hydrograph). The USGS method can be applied to urban watersheds of most sizes.

Input data necessary for the method include Q_p and time lag (t_l). Q_p can be determined by using USGS empirical equations. Time lag can be calculated by using USGS techniques (or indirectly from t_c or t_p). Input data are applied to a scaled table of ordinates to create the unit hydrograph shape.

Because t_l and Q_p are the only required input data, the method is similar to Constant's in its scant data requirements. Constant's method uses three inputs--Q_p, A, and t_p--and mathematical equations instead of a table of ordinates.

Standardized Test Case

Synthetic unit hydrograph methods discussed previously are applied to a hypothetical watershed (Figure 1). The hypothetical watershed was invented

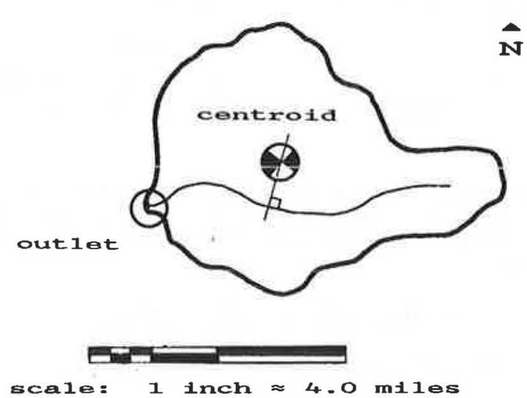

FIGURE 1 Hypothetical watershed.

to test the data requirements and complexity of the nine methods. It is ungauged and has the following characteristics:

- Drainage area (A) = 87.5 mi^2 (56,000 acres)
- Overland statistics
  - Forested, rural watershed with slope (S_o) of 1.5 percent
  - Length of watershed (L) = 8.3 mi from outlet to watershed divide
  - Width of watershed (W) = 12.0 mi at widest point
  - Length to centroid (L_{CA}) = 4.1 mi from outlet to centroid of watershed area

- Channel statistics
  - Grassy waterway ($n \approx 0.030$) with a slope (S_c) of 0.7 percent
  - Length of channel (L_c) = 8.0 mi
- Volume of rainfall excess (Q) = 1.0 in.
- Duration of rainfall event (D) = 1.0 hr
- Time of concentration (t_c) = 10.8 hr (overland plus channel)

COMPARISON OF HYPOTHETICAL UNIT HYDROGRAPHS

Table 2 provides data and peak flows resulting from the application of the nine unit hydrograph methods to the hypothetical test case. The table is significant because it demonstrates the information that is required for each of the nine methods. Included in Table 2 are input parameters and watershed characteristics. For Snyder's method, a peak flow of 5,843 ft^3/sec was calculated assuming a C_t of 2.0 and a C_p of 0.6. This peak-flow value was also applied to Constant's and the USGS methods. Comparison of peak flows provides an indication of the degree of consistency of the various methods. Several analyses of data from actual sites are necessary for a sound comparison. The SBUH method could not be applied to the test case because of a lack of a suitable hyetograph; the need for data on rainfall versus time imposes a level of complexity that may be unwarranted for the intended use of a synthetic unit hydrograph method.

From a computational standpoint, all unit hydrograph methods except HYMO, Grey's, Constant's, and SBUH are relatively quick and easy. The methodology for HYMO is lengthy and complicated, but the results are consistent with those from other unit hydrograph methods. Grey's, Constant's, and SBUH methods require extensive calculations or a computer program, or both, to be practical.

All unit hydrograph methods except USGS use measurements of drainage area. The most common driving variable is rainfall excess. From a practical standpoint, this means that an analysis of rainfall, infiltration, and runoff must precede the development of a hydrograph. In contrast, Constant's and USGS methods are driven by peak flow. Peak flow is a convenient driving variable because its use eliminates determination of rainfall excess. Peak flow can be estimated by using

- The rational method for very small drainage areas (i.e., <300 acres). In this case the rainfall return period is assumed to be the hydrograph return period.
- USGS empirical regression equations that are developed for selected return periods. These equations are in widespread use across the country and are available in USGS publications.
- A frequency analysis of a nearby flow gauge, which can identify a flow with a given return period. This flow can be transferred to an ungauged site by using drainage-area ratios.

Overall, it can be seen from Table 2 that the SCS methods produce the highest peak flow, whereas Clark's method produces the lowest. All methods consistently develop a hydrograph volume of 1 in. over the watershed; this is a constraint and is necessary in order for a unit hydrograph to be considered valid.

The shapes of the unit hydrographs are shown in Figure 2. All the hydrographs appear qualitatively similar except those from the SCS triangular, Grey's, and USGS methods, which appear to depart significantly from the others. A comparison of the methods shows that

- Peak flows vary within a range of ±5 percent. It should be noted that peak flow from Snyder's method is used as input for peak flows for Constant's and USGS methods.
- All methods produce a volume consistent with the definition of a unit hydrograph.
- For all but SCS triangular, Grey's, and USGS methods, the hydrographs are similar in shape.
- The following differences exist with respect to computation: Snyder's, SCS dimensionless and triangular, Clark's, Constant's, HYMO, and USGS methods are relatively quick and easy to compute. Grey's and SBUH methods require significant computations to determine gamma-distribution parameters (Grey's) or to calculate infiltrations (SBUH).
- The following differences exist with respect to input data: There is some overlap, but different watershed attributes are used in the different methods. The peak-flow methods (Constant's and USGS) are convenient in that rainfall excess is not used and the methods directly input the peak flow.

COMPARISON OF UNIT HYDROGRAPHS WITH ACTUAL DATA

The synthetic unit hydrograph methods are compared with the actual data for a 910.9-acre (1.432-mi^2) subcatchment of the Bloody Run Catchment, located in the northwest section of Cincinnati, Ohio. The Bloody Run Catchment has been the subject of an

TABLE 2 Data Comparison for Standardized Test Case

	Method								
Parameter	Snyder's	SCS Dimensionless	SCS Triangular	Clark's	Grey's	Constant's	SBUH	HYMO	USGS
Peak flow Q_p (ft^3/sec)	5,843	6,062	6,062	5,741	5,852	5,843[a]		5,769	5,843[a]
Time of concentration t_c (hr)		10.8	10.8	10.8			X		
Time to peak t_p (hr)	6.25					6.25		6.4	
Time lag t_l (hr)									6.2
Lag-time factor C_t	2.0								
Peak-flow factor Cp	0.6								
Shape factor K		484	484						
Gamma distribution Γ					7.9				
Watershed factor B								420	
Shape factor n								5.2	
Computational interval δt (hr)						1.0	X		
Hyetograph i(t) (in./hr)							X		
Infiltration rate f(t) (in./hr)							X		
Fraction impervious Imp							X		

Note: X = data required but not run, as noted in text.

[a]Peak flow calculated indirectly (Snyder's method).

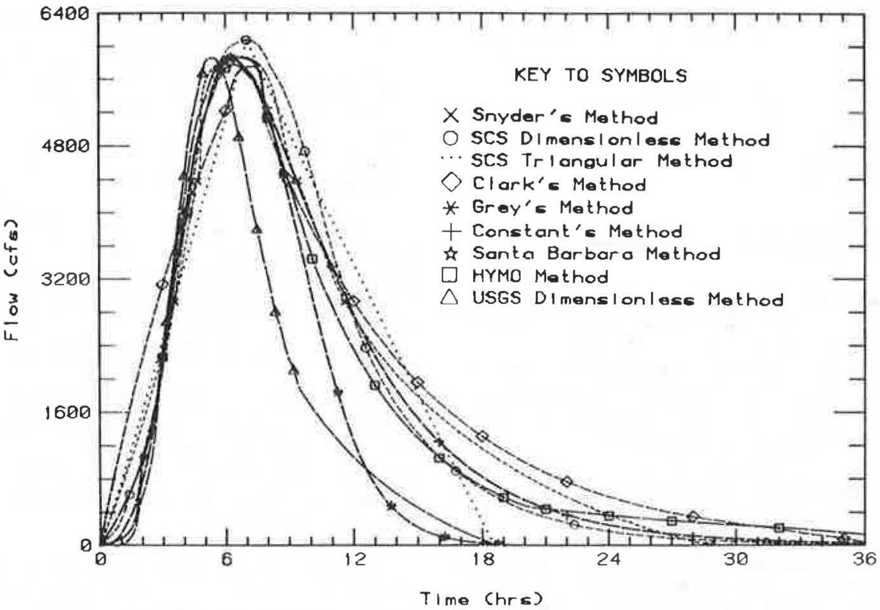

FIGURE 2 Unit hydrographs for standardized test case.

Environmental Protection Agency (EPA) report (12). These data were available to the authors and were used to demonstrate the methods described here.

The subcatchment is a urban drainage area consisting of primarily pasture and open parkland (24 percent) and single-family and multifamily residential (55 percent) and commercial (21 percent) property. The area is shown in Figure 3. The topography consists of rolling terrain (average slope is 4.8 percent) with ridges running east and west. The parameters for the Bloody Run subcatchment are as follows:

- Drainage area (A) = 1.423 mi^2 (910.9 acres)
- Overland statistics
  - Developed, urban watershed with slope (S_O) of 4.8 percent
  - Length of watershed (L) = 1.8363 mi from outlet to watershed divide
  - Width of watershed (W) = 0.8 mi
  - Length to centroid (L_{CA}) = 0.8755 mi from outlet to centroid of watershed area
  - Impervious area, 55 percent
- Channel statistics
  - Grassy waterway ($n \simeq 0.030$) with a slope (S_C) of 4.8 percent
  - Length of channel (L_C) = 1.5322 mi
- Volume of rainfall excess (Q) = 0.0174 in.
- Duration of rainfall event (D) = 1.0 hr
- Time of concentration (t_C) = 0.918 hr (overland plus channel)

The subcatchment contains a flow gauge and rainfall stations located near the subcatchment outlet from which data are measured. The storm-runoff event that is compared by using different synthetic unit hydrograph methods occurred on November 9, 1970, between 6:00 and 7:00 p.m. The average intensity of the rainfall event is 0.109 in./hr. The storm duration approximately equals the time of concentration; this enabled the authors to avoid the process of hydrograph separation that would be necessary to evaluate most natural storms and to estimate unit hydrographs. The resulting output hydrograph contained a volume of 0.0174 in. This volume is assumed to be due to the rainfall excess. The peak flow of the hydrograph was 9.72 ft^3/sec.

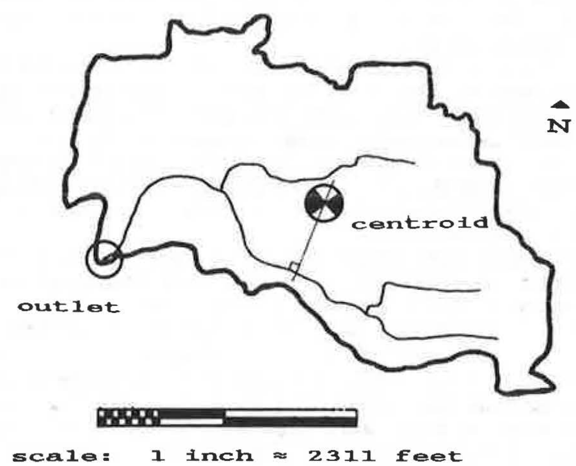

FIGURE 3 Bloody Run subcatchment watershed.

The verification rainfall is relatively light; that is, it is not a flood event. This implies that this particular verification measures the ability of the various methods to estimate an event with a low return period.

Figure 4 shows the hydrographs computed for the November 9, 1970, 6:00 p.m. rainfall event for the Bloody Run subcatchment. Qualitatively, the best-fitting hydrographs are the result of Snyder's and Constant's methods, both of which match the peak well and match the shape for the first 2 hr.

Clark's and the SBUH methods, both similar linear reservoir-routing methods, underestimate the peak but match the recession. Grey's method underestimates the peak and overestimates the recession. HYMO, SCS dimensionless and triangular, and USGS methods match the peak but underestimate the recession. HYMO matches the recession more closely than the others. The SCS triangular method visually appears to produce a poor approximation of a smooth hydrograph curve.

Table 3 presents a quantitative comparison of the methods. The data used to generate Figure 4 are analyzed by using a standard error (SE) test. The difference between the calculated and the observed

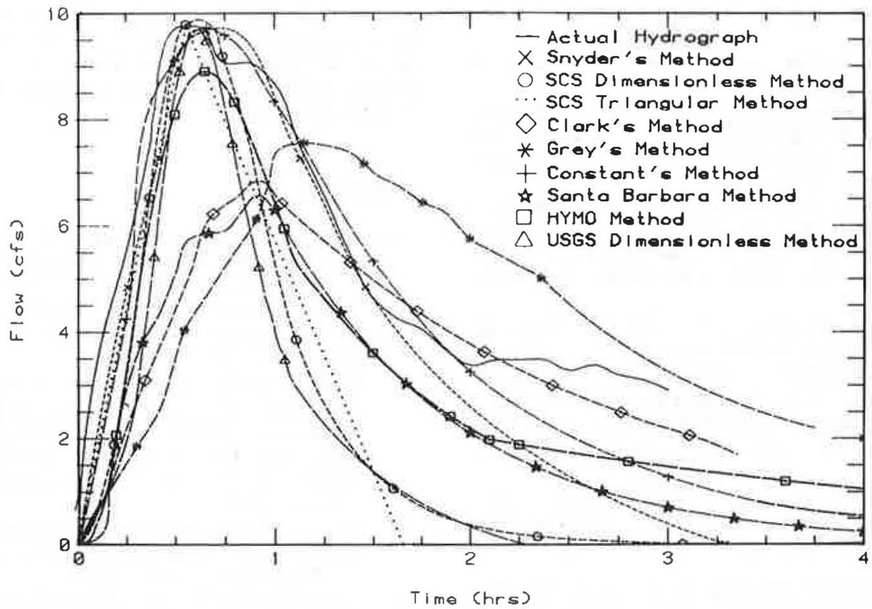

FIGURE 4 Unit hydrographs for actual test case.

TABLE 3 Quantitative Comparison of Unit Hydrograph Methods

Method	Peak Flow (ft^3/sec)	Standard Error
Actual data	9.72	–
Snyder's	9.67	0.53
Constant's	9.67	0.56
HYMO	8.89	1.51
Clark's	6.83	2.34
SBUH	6.56	2.45
SCS		
Triangular	9.87	2.56
Dimensionless	9.87	2.65
USGS	9.61	2.87
Grey's	7.54	3.12

values at 15-min intervals is squared and cumulated over the interval 0 to 2 hr. The square root of the cumulated value divided by the number of intervals (9) is reported as the SE in Table 3. This SE is similar to a chi-squared goodness-of-fit statistic. Also reported are the peak flows estimated by the various methods.

The standard errors indicate a relative ranking as follows:

• Snyder's and Constant's methods have SE's less than 1 ft^3/sec;

• SE for the HYMO method is between 1 and 2 ft^3/sec;

• SCS dimensionless and triangular, Clark's, SBUH, and USGS methods have SE's between 2 and 3 ft^3/sec; and

• Grey's method has an SE greater than 3 ft^3/sec.

It is notable that the SCS triangular method is slightly better in terms of SE than the SCS dimensionless and USGS methods. Thus the "un-eye-appealing" SCS triangular method provides a slightly better fit than either dimensionless method. However, all three methods have similar SE performance.

CONCLUSIONS AND RECOMMENDATIONS

A protocol for testing hydrograph generation methods is recommended:

- Assemble the alternative procedures and their documentation (nine methods are tested to check the protocol in this paper).
- Identify and assemble data on specific watersheds:
  - Input hyetograph,
  - Output hydrograph,
  - Land use and geometric data sufficient to estimate watershed attributes (similar to the information in Table 2).
- Provide quality control on the watershed.
- Specify a goodness-of-fit criterion or criteria (such as standard error of estimate or chi-square).
- Apply the watershed information to each watershed (independently of the input hyetograph and output hydrographs) to estimate the unit hydrograph for each alternative procedure.
- Employ the input hyetograph or rainfall excess to calculate a hydrograph from each unit hydrograph.
- Compare the calculated hydrograph with the measured hydrograph and determine goodness of fit.
- Analyze the goodness of fit and develop criteria for hydrograph method selection:
  - Focus on methods that are suitable for ungauged watersheds, and
  - Determine whether suitable methods can be applied universally or if they must vary according to geography or climate.

As an example of the protocol, an examination of nine hydrologic methods by using a hypothetical watershed to examine data requirements and methods and by using one actual runoff event to test performance indicates the following:

• Snyder's and Constant's methods performed best in terms of standard error of prediction. In practice, Snyder's method would be used with rain-

fall excess as the driving variable and Constant's method would be used with peak flow as the driving variable.

• HYMO method performed nearly as well as Snyder's and Constant's methods.

• USGS, SCS dimensionless, and SCS triangular methods all performed similarly. USGS method used the least data, and SCS dimensionless and triangular methods used the same data.

• Clark's and SBUH methods require considerable analysis and probably need computerized implementation. The other methods are simpler and, with the exception of Grey's, provided the same or better performance.

• Grey's method provided the least accurate results and was difficult to implement.

• The methods requiring a computer or significant calculation are Clark's, Grey's, Constant's, SBUH, and HYMO. Of these, Constant's gave the most accurate results and the most straightforward computations.

• The desk-top methods are Snyder's, SCS dimensionless and triangular, and USGS. Of these, Snyder's was the most accurate and easiest to implement.

The example presented in this paper demonstrates the feasibility of implementing the protocol on a large-scale test of many sites.

It is recommended that watershed data be assembled for evaluation with this protocol and that they be distributed across the country so as to represent various geophysical and climatologic regions. Assuming that this can be accomplished with approximately two sites per state, on the average, a database of 100 sites is implied.

Sources of data can be USGS studies, EPA Areawide Wastewater Management Studies, and state, local, and university studies.

Data assembly is expected to be a challenge; however, no new measurement program is recommended. It is the authors' experience that sufficient data exist and that it will take diligence and effort to assemble, check, tabulate, and refine these data into a uniform and consistent database for testing alternatives.

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Gabions Used in Stream Grade-Stabilization Structures: A Case History

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ABSTRACT

Streams in western Iowa have been degrading since the turn of the century and this entrenchment has endangered many highway and railroad bridges. Although grade-stabilization structures have been effective in controlling this erosion, the cost of reinforced-concrete structures has risen to the point that less expensive materials need to be considered. In an effort to evaluate alternative materials for this purpose, a gabion drop structure was designed and built and its performance monitored for 2 years after completion. The demonstration structure has performed satisfactorily with minimal differential settling and minor erosional problems downstream of the structure. Sedimentation occurred upstream of the structure during construction but little additional sediment has accumulated since. A cost analysis that normalizes several variables is used to compare the gabion structure with concrete structures and indicates that the cost of building the gabion structure was about 20 percent of that of a comparable-size concrete structure. It is concluded that this type of structure is an effective and economic alternative.

Since the turn of the century, tributaries to the Missouri River in western Iowa have entrenched their channels to as much as six times their original depth. This channel degradation is accompanied by widening as the channel side slopes become unstable and landslides occur. The deepening and widening of these streams have endangered about 25 percent of the highway bridges in 13 counties (1).

Grade-stabilization structures have been recommended as the most effective remedial measure for stream degradation (2). In western Iowa within the last 7 years, reinforced-concrete grade-stabilization structures have cost between $300,000 and $1,200,000. Recognizing that the high cost of these structures may be prohibitive in many situations, the Iowa Department of Transportation (Iowa DOT) sponsored a study at Iowa State University (ISU) to find low-cost alternative structures. Analytical and laboratory work led to the conclusion that alternative construction materials such as gabions and soil-cement might result in more economical structures (1). The ISU study also recommended that experimental structures be built and their performance evaluated.

The supervisors of Shelby and Pottawattamie counties agreed to participate in the construction of these demonstration structures; the counties were to provide 25 percent of the construction costs and the Iowa DOT Highway Research Board was to provide 50 percent. The Iowa State Water Resources Research Institute (ISWRRI) provided sufficient funds for 25 percent of two structures, one in Shelby County and the other in Pottawattamie County.

CONSTRUCTION COSTS AND PROBLEMS

Plans were developed for a soil-cement structure in Shelby County and a gabion structure in Pottawattamie County. The original cost estimate for the Shelby County structure was about $60,000; however, the final cost estimate was twice that because of problems anticipated with the excavation of the stilling basin. No bids were received at the scheduled March 1982 letting, and the construction money allocated to this project reverted to ISWRRI and was reallocated to other projects within the Institute.

Although the laboratory studies at ISU suggested that soil-cement was a feasible construction material for grade-stabilization structures (3), the lack of any contractor willing to bid on the project indicated that a major practical problem existed with the use of soil-cement in this type of structure. The problem may have been associated with conditions at this specific site or with the lack of contractor experience in constructing soil-cement water-control structures. A third possibility was that the contractors did not accept the results of the laboratory studies and needed evidence of the field performance of such structures. If lack of construction experience was the reason for the lack of bids, specifications outlining construction procedures need to be developed. If the third reason was the primary cause for the lack of bids on the Shelby County structure, field scale research needs to be conducted to support or reject the validity of the laboratory work. It is the Shelby County engineer's opinion that lack of contractor experience in mixing and placing soil-cement was the major problem; in addition, the practical construction problems may drive the cost of the structure so high that they will offset any savings in material cost (Eldo Schornhorst, personal communication, Nov. 7, 1984).

The Pottawattamie County gabion structure was originally estimated at a cost of $60,000, but after detailed design and modifications suggested by the county engineer, the cost estimate increased to $85,000. Bid letting was September 16, 1982, when three bids were received; the lowest was $97,000. The required additional funds were provided by the county supervisors, and construction began November 29.

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Except for 4 weeks during January, construction continued through the winter. Although several problems were encountered during construction, the contractor was able to get water through the structure by May 16. The structure was completed by June 30, 1983, at a final cost of $108,000. The cost overrun was due largely to construction problems. A comparative cost analysis of this gabion structure and reinforced-concrete structures is presented later in this paper.

DESCRIPTION OF GABION GRADE-STABILIZATION STRUCTURE

The demonstration gabion grade-stabilization structure is located on Keg Creek 3 mi east of McClelland at Section 1-75-42. The drop structure is situated 100 ft downstream from a bridge where, since 1958, 14 ft of channel degradation has exposed bridge piers and caused landslides that removed soil from the east abutment. The drainage area of Keg Creek at this location is approximately 90 mi². Before construction, the stream gradient was from 6 to 8 ft/mi with a channel width of about 50 ft at the top.

The structure consists of a gabion wier and ramp with a net drop of 12.6 ft, which is intended to reduce the effects of the degradation at the bridge site. Figure 1a and b show the plan and profile of the structure. The bottom width of the weir and ramp is 21 ft with 2:1 side slopes extending 27 ft upward. Figure 1c is the plan of the gabion arrangement. Gabions 1 ft by 3 ft and 3 ft by 3 ft were used. The ramp is 51 ft long with a 4:1 downstream slope. The stilling basin is 63 ft long. The fill rock in the gabions has a maximum rock size of 8 in. with 75 percent greater than 4 in. and not more than 5 percent passing the 1/2-in. sieve.

The structure was designed for the 50-year-frequency flood of 9,930 ft³/sec to contain the hydraulic jump and avoid overbank flooding. As a point of comparison, the 2-year-frequency flood is estimated at 2,190 ft³/sec. Before construction of the structure, the channel had the capacity to contain the 100-year-frequency flood of about 12,000 ft³/sec. It is expected that the structure will cause a 4-ft rise in the water surface elevation upstream of the structure during the 100-year flood but will not cause overbank flow.

Monitoring Performance

The monitoring of the structure included differential settlement measurements, measurements of upstream aggradation and downstream degradation subsequent to placement of the structure, measurements of stream flow through the structure, and qualitative observations of structural deterioration.

Settlement Measurements

In order to monitor the differential movement of the structure, concrete monuments were placed on the

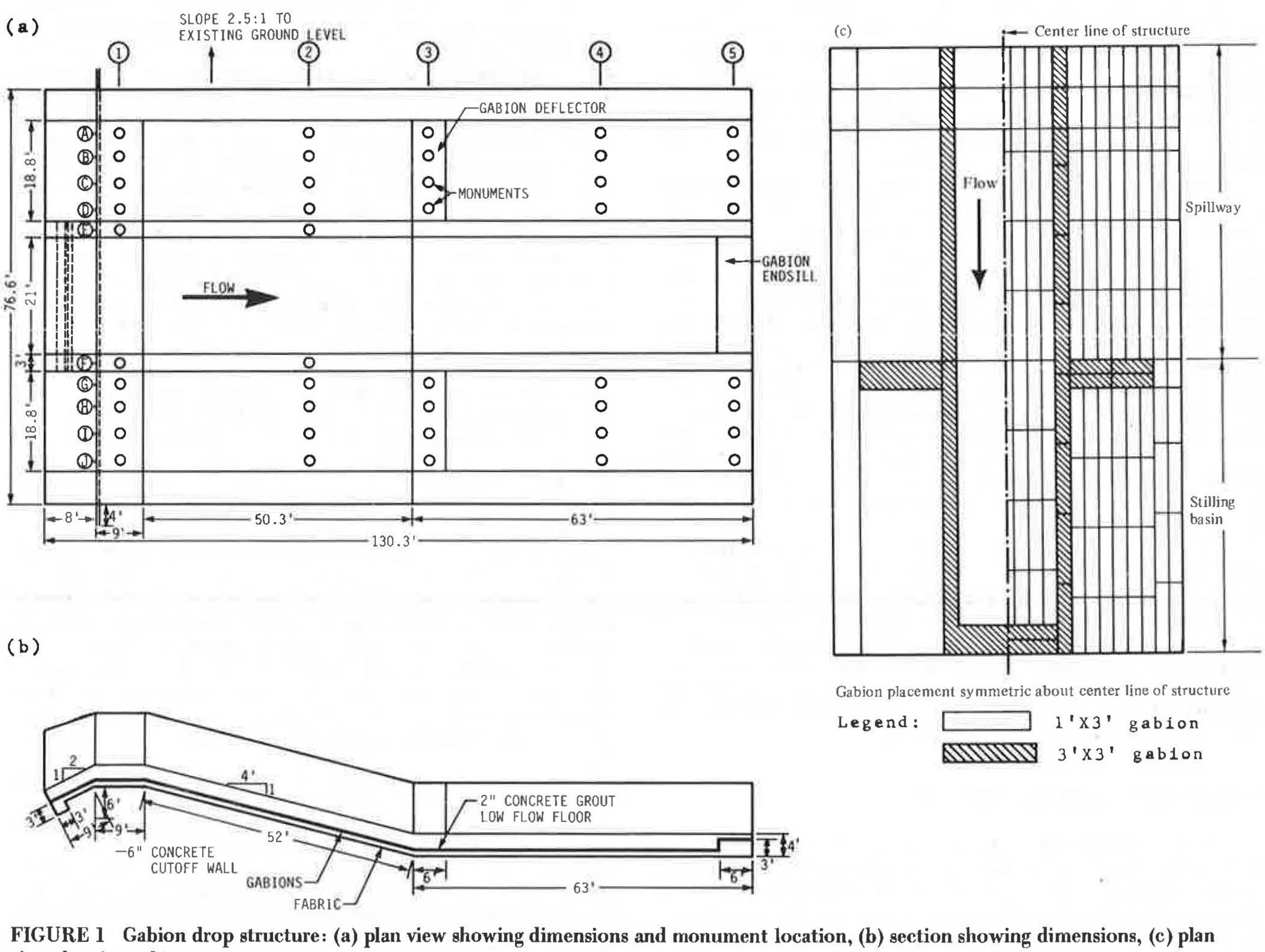

FIGURE 1 Gabion drop structure: (a) plan view showing dimensions and monument location, (b) section showing dimensions, (c) plan view showing gabion arrangement.

surface as shown in Figure 1a. Elevations of the monuments were measured at five different times: June 29 and November 17, 1983; June 8 and August 22, 1984; and June 5, 1985. The elevation data were used to plot all five of the transverse cross sections at the various dates. These plots revealed that virtually no differential settlement occurred within the structure throughout the course of the investigation. Figure 2 is a typical cross section, and Figure 3 is the cross section at about the middle of the structure. In Figure 3, Monument 1A (at the top of the side slope immediately downstream from the crest) settled about 4 in. between the first two observation dates. No differential movement has been observed since November 17, 1983; thus it has been concluded that differential settlement is not a problem. Because soil consolidation occurs most rapidly soon after loading, it appears that settlement will not be a problem.

Observations of Deterioration

Minor deterioration of the structure is being observed visually and has been recorded in photographs. Some deformation of the side slope is apparent in the vicinity of Monuments 2D and 2E, but this movement occurred during construction after a high-runoff event. Because runoff had filled the channel, the contractor, in an effort to dewater the site and resume work quickly, pumped the water out of the channel in a short time. It is interpreted that this rapid drawdown condition created instability and caused slippage. A rapid drawdown condition is not likely to occur during normal operation of the structure; consequently, the side slopes are expected to be stable in the future. The stability of the side slopes is verified by the constant elevations of the monuments. Although the observed slope deformation is not of great concern, anchors were placed on the slope as a precaution.

A scour hole has developed immediately downstream of the stilling basin on the west bank. The hole is roughly 10 ft long parallel to the channel and extends about 3 ft into the bank. This is where the construction diversion channel was located, and the backfill in this area may have been improperly compacted. The scour hole should be continuously observed for any signs of expansion. If erosion proceeds in the upstream direction, it may undermine

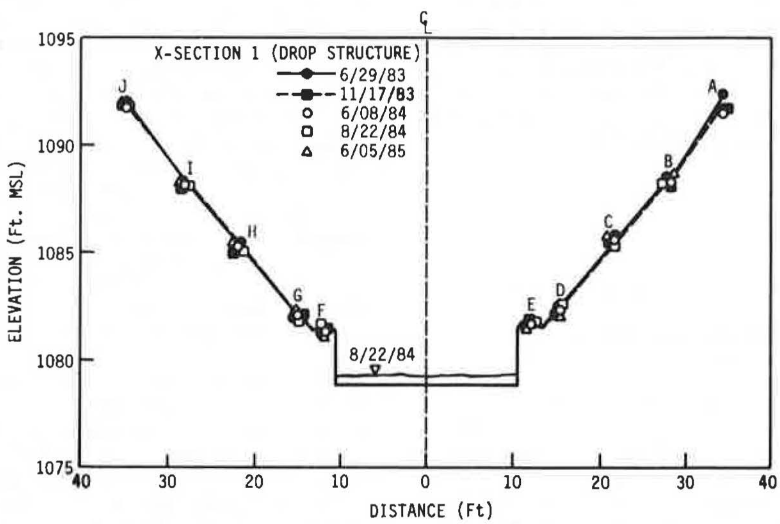

FIGURE 2 Cross section at top of structure (note differential movement at monument 1A, the only measurable movement observed in the structure).

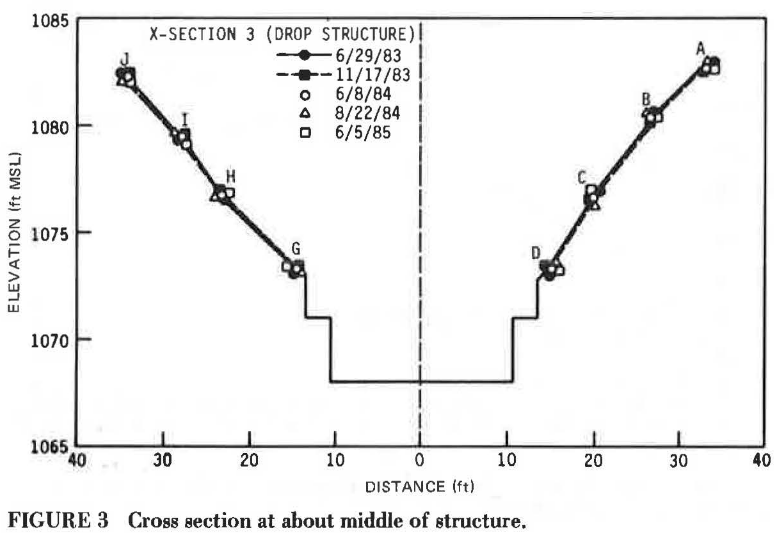

FIGURE 3 Cross section at about middle of structure.

the side slope and stilling basin. If the hole expands, it should be protected with riprap or additional gabions.

Flow Estimates

Seven gauges to measure stream flow were placed on the bridge piers and on posts upstream and downstream of the bridge within a 3-mi reach. Each gauge consists of a vertical tygon tube attached to a staff gauge. The tube has a one-way valve at the bottom that allows water to flow into but not out of the tube. The gauges were positioned on the posts to measure high-flow events only. Water enters the bottom of the tygon tubing through the one-way valve and rises in the tube as the stream stage rises. After the maximum stage has been reached, the water is trapped in the tube by the one-way valve. This allows measurement of the maximum stage from the staff gauge attached to the tube. The gauges have not functioned as well as anticipated. Debris has plugged the one-way valves and prevented the collection of data. For future applications of these gauges, an attempt should be made to design some type of debris trap on the intake end of the system.

Because the gauges failed to perform adequately, an alternative method of estimating flows was devised. The spillway structure produces critical flow at its crest and therefore acts as a control in the stream channel. Controls are defined as certain features in a channel that tend to produce critical flow (4). At any feature that acts as a control, if the flow depth is known, the discharge can be calculated by using the following relationship:

$$Q = A(gA/B)^{1/2}$$

where

- Q = discharge flowing through the crest of the spillway (ft^3/sec),
- A = area of the wetted section (ft^2),
- B = corresponding width of the water surface (ft), and
- g = acceleration due to gravity (32.2 ft/sec^2).

The geometry of the spillway crest of the gabion grade-control structure was used to calculate the discharge for various depths of flow from the foregoing equation; that relationship is shown in Figure 4. Details of the calculation may be found in a report by Hanson et al. (5). Note that for the 50-year flood frequency with a discharge of 9,930 ft^3/sec, the depth of flow through the structure is 14 ft. For the design of this structure the HEC-2 backwater calculation program was used, and it estimated the flow depth at 13.5 ft. The data shown in Figure 4 are in good agreement with the design estimates.

Debris deposited on the sidewalls of the structure during a flood event are physical evidence of the maximum stage for that event. The elevations of the debris lines were measured during summer 1983 and spring 1984 and on June 8, 1984, and June 5, 1985. These depths of flow are plotted in Figure 4 and indicate that, to date, the flows have been well below the design flood with discharges less than 1,200 ft^3/sec.

The water-surface profile for the design flow of 9,900 ft^3/sec was estimated with the HEC-2 program. The design water-surface profile and the water-surface profiles for 120- and 1,200-ft^3/sec discharges, which were estimated from the debris lines, are shown in Figure 5. These curves suggest that the downstream effects of the structure and stream force the hydraulic jump upstream onto the spillway to create a submerged jump.

Sedimentation Observations

Changes in the upstream channel geometry caused by sedimentation have been monitored by surveying transverse profiles at the bridge and at 500-ft intervals upstream to a distance of 5,000 ft. Transverse profiles at the bridge were measured on June 28 and November 17, 1983; June 8 and August 22, 1984; and June 5, 1985. Before construction the slope of the stream was 13.2 ft/mi. A set of transverse profiles is shown in Figure 6. These sections show that sedimentation to a depth of 6 ft had occurred before June 28, 1983, but little change has been noted since then. This indicates that the major amount of sedimentation above the crest occurred during construction and that the 1,200-ft^3/sec event since construction has had little effect on deposition.

A longitudinal profile surveyed on August 23, 1984, is shown in Figure 7. The water surface extends 5,500 ft upstream from the crest of the structure and the sediment surface extends approximately 4,000 ft upstream from the structure. It is expected that this sediment will extend further upstream in the future. A conservative estimate is that it will continue to the point where the water-surface profile intersects the streambed profile, that is,

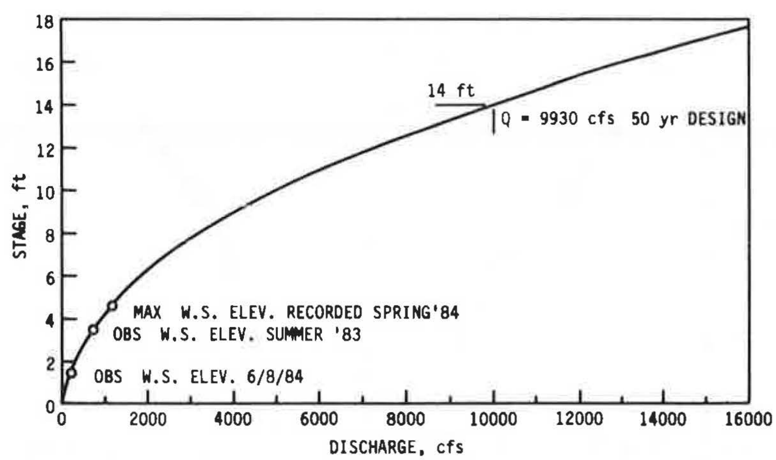

FIGURE 4 Stage-discharge relationship for drop structure at its crest (elevation, 1,079 ft above mean sea level).

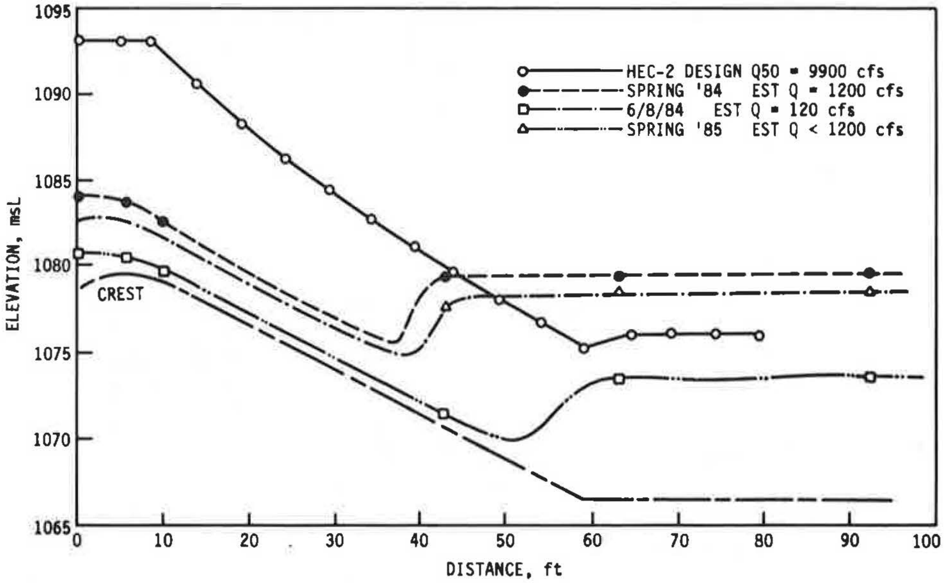

FIGURE 5 Water-surface profiles from HEC-2 for design and as estimated from debris lines at various dates.

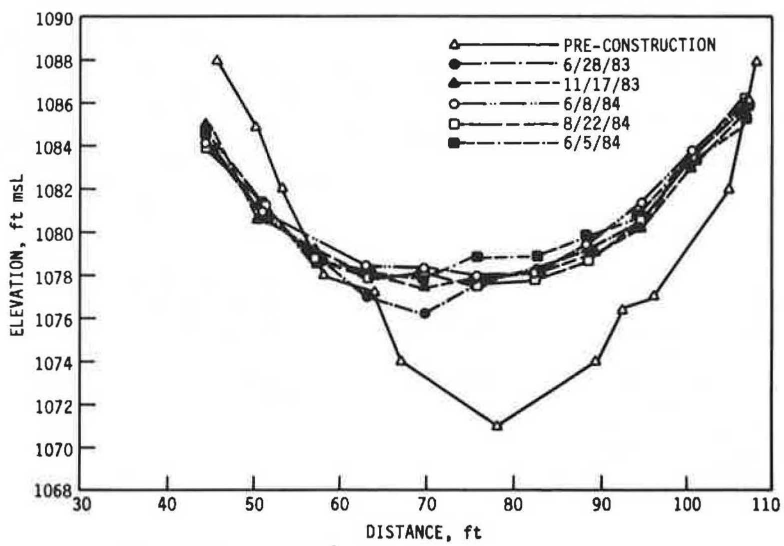

FIGURE 6 Transverse cross sections of channel bottom for various dates.

about 5,500 ft. This would produce a slope of about 5.7 ft/mi.

A less conservative estimate of the ultimate upstream extent of the sediment is calculated from the method suggested by Maccaferri (6). The stable slope of a channel can be estimated from the following equation:

$$i = (vu)_{\ell}^{10/3}B^{4/3}n^{2}/Q^{4/3}$$

where

- i = stable slope,
- u_{ℓ} = maximum permissible velocity (which depends on the size of bed material at which bed erosion starts),
- v = ratio between mean water velocity and the corresponding velocity at the channel bottom,
- B = wetted perimeter,
- n = roughness coefficient, and
- Q = design flow.

This relationship is an extension of Manning's equation, and the detailed analysis with application to the gabion structure has been given by Hanson et al. (5). That analysis estimated a stable slope of 4 ft/mi, which would cause the sediment to extend upstream for about 6,500 ft. That slope is also plotted in Figure 7, where it can be seen that the wedge of sediment would intersect the knickpoint at about midheight. Depending on the estimate used, the grade-control structure has reduced the slope of the stream to 30 or 40 percent of the original and thereby caused aggradation.

Even though the sedimentation effects of the structure may extend 6,500 ft upstream, there is field evidence that the channel banks are barely stable and that sloughing of the side slopes may cause further loss of land and damage to roads. Also, the upstream knickpoint was not submerged by the grade-control structure, so it is likely that the knickpoint will continue to progress upstream.

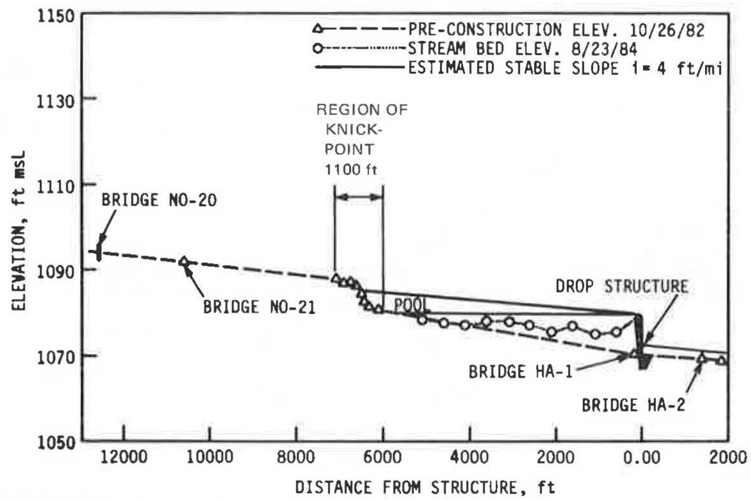

FIGURE 7 Longitudinal profile of channel bottom and water surface in pool upstream of structure.

Downstream Erosion

Bank erosion is occurring downstream beyond the stilling basin to a distance of approximately 80 ft. This may be partly because of the submerged jump, which was discussed earlier. The submerged jump provides relatively inefficient energy dissipation, which may be cause for future concern. An extension of the stilling basin may be required to provide better energy dissipation.

ECONOMIC COMPARISON OF GABION STRUCTURE WITH CONCRETE STRUCTURE

The major objective of this research was to find low-cost alternatives for the stabilization of degrading streams. In western Iowa, the conventional approach to grade stabilization has been the use of reinforced-concrete drop structures. In Pottawattamie County gabions have served well in various applications for over 10 years, and it is thought that the service life of a gabion structure is essentially equivalent to that of a reinforced-concrete structure.

It is difficult to compare the cost of the gabion drop structure that was built and evaluated as part of this study with the cost of reinforced-concrete structures that have been used in the past because the cost of the structure increases with increasing size, slope, drainage area, and design discharge. The cost of four reinforced-concrete drop structures that were constructed in western Iowa within the last 7 years and that of the gabion structure are shown in Table 1. These data are the 1982 costs based on Iowa DOT's construction index; the calculations of these costs may be found elsewhere (5). Although the gabion structure is less than one-third the cost of the least expensive concrete structure, it also has the smallest drop. On the other hand, the gabion structure has the second-largest drainage area and the largest design flow of the structures listed. The foregoing comparisons illustrate the problem. In order to develop a normalized size and discharge factor that would account for all the size and hydrologic variables, a dimensional analysis was performed to incorporate all the relevant properties into one term that could be used for the cost comparison of the structures.

Design discharge and structure width can be combined to provide a flow area at the crest based on the assumption that critical flow occurs at the crest of the structure (4):

$$A = (Q^2B/g)^3$$

where

- A = area of the wetted section at the critical depth,
- Q = design flow,
- B = corresponding width of the water surface, and
- g = acceleration due to gravity.

The wetted section (A) has been calculated for the five drop structures and the data are shown in Table 2.

The wetted area can be combined with drainage area (D.A.) to form a semidimensionless term:

$$a = D.A./A$$

TABLE 1 Geometry, Design Discharge, and Cost of Grade-Control Structures in Western Iowa

Structure Type and County	Creek	Drainage Area (mi^2)	Slope (ft/mi)	Drop (ft)	Length (ft)	Width (ft)	Design Q (ft^3/sec)	Cost[a] ($)
Reinforced-concrete structures								
Harrison	Willow	67.2	9.3	38.4	142	67.5	5,800	376,022
Monona	Willow	32	19.5	36.6	142	67.5	7,500	372,447
Harrison	Willow	100.2	8.3	24.0	115	80.0	7,250	434,562
Harrison	Pigeon	56.5	8.0	18.6	110	80.0	8,100	345,147
Gabion structure								
Pottawattamie	Keg	90	8.0	12.6	131	–[b]	9,930	101,000

[a] 1982 costs based on Iowa DOT construction index.
[b] Stilling basin is trapezoidal with average width of 51 ft.

TABLE 2 Dimensional Analysis to Develop Size Factor for Cost Comparison of Structures

Structure Type and County	Creek	A (ft)	a (mi^2/ft^2)	Sc (ft/mi)	Sc (ft/ft)	aScSs (mi/ft)
Reinforced-concrete structures						
Harrison	Willow	413.1	0.163	9.38	3.70	5.63
Monona	Williow	490.7	0.065	19.5	3.88	4.92
Harrison	Willow	507.3	0.198	8.33	4.79	7.90
Harrison	Pigeon	546.3	0.103	8.00	5.89	4.85
Gabion structure						
Pottawattamie	Keg	602.0	0.100	8.00	10.35	12.43

The term is semidimensionless because the drainage area is in square miles and the wetted area is in square feet. The values of a for all five structures are also shown in Table 2. The channel slope (Sc) is a semidimensionless term in feet per mile, and a dimensionless term (Ss) can be generated by dividing the overall length of the structure by its drop. These terms, along with their combined values, are shown in Table 2. This combined term describes the structure according to size, design flow, and drainage area and is defined here as the size factor. The cost of each structure is plotted versus the size factor in Figure 8; it can be seen that the cost of the concrete structures increases linearly with increasing size factor. Note that the cost of the plotted gabion structure versus its size factor falls considerably below the line projected for the concrete structures. This analysis suggests that the cost of the gabion structure may be about 20 percent of the cost of an equivalent reinforced-concrete structure.

CONCLUSIONS

The gabion grade-stabilization structure has shown satisfactory structural performance throughout the 2-year observation period, with minimal differential settling and no evidence of side-slope instability since construction was finished. It should be recognized that the maximum flow to date has been less than 15 percent of the design flow.

The major amount of sedimentation occurred during construction and is likely to extend at least 5,500 ft upstream of the structure. A more optimistic estimate is that the depositional wedge will extend 6,500 ft upstream. In any event the sedimentation effects of the structure will not submerge the knickpoint that exists upstream, so continued erosion problems are likely upstream of the sedimentation area.

The sedimentation beneath the bridge has been sufficient to cover the piles to their original depth of soil cover and to stabilize the slope beneath the abutment.

Erosion downstream of the structure could be a problem, especially if it undermines the stilling basin. However, the gabions are deformable and may collapse into any scour hole that forms, thereby becoming somewhat self-protecting. This downstream erosion is the result of inefficient energy dissipation by the stilling basin.

An analysis of the cost of the gabion structure as compared with costs of four concrete structures included the size, drainage area, and design flow of each of the structures. This analysis suggests that the cost of the gabion structure is about 20 percent of that of an equivalent concrete structure.

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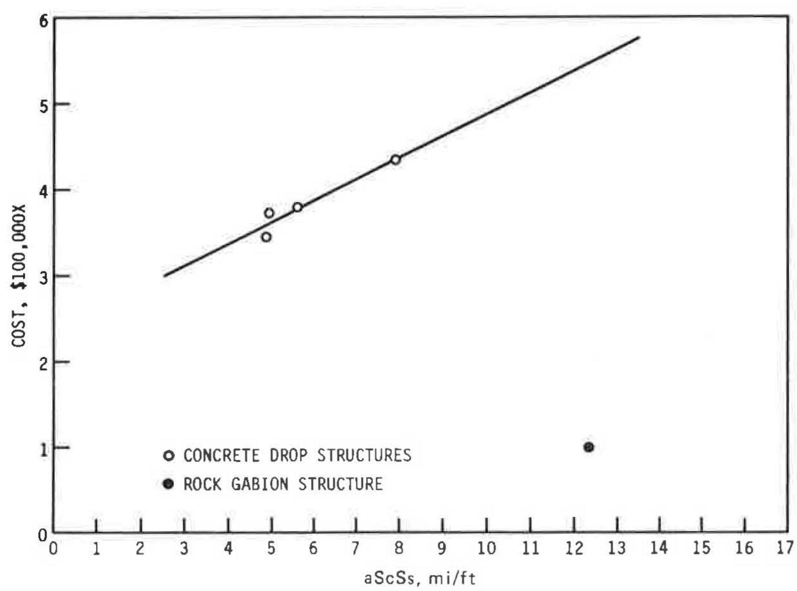

FIGURE 8 Dimensionless size factor versus construction costs for four concrete drop structures and one gabion drop structure.

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