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Prediction of Channel Bed Grade Changes at Highway Stream Crossings

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Changes in channel bed-level elevation associated with channel aggradation and degradation have been found to be a significant cause of hydraulic problems at bridges. Numerous techniques for evaluating the impact of these grade changes, from simple, qualitative geomorphic principles to complex, fully developed, computer models, are available in the literature. These techniques represent a wide range of levels of analysis data needs, difficulty of application, time requirements, and costs. The number of techniques available and the variety in their levels of accuracy emphasize the need to base grade-change predictions on more than one technique or model. An appropriate procedure starts with general observations and the evaluation of geomorphic principles and relations to establish the cause and direction of the change and is then built on by applying some of the simpler quantitative techniques. From this base, one or more of the levels of mathematical modeling can be applied. The level of sophistication used in the modeling process should be based on the physical processes causing the grade change, the economic importance of the particular crossing, and available time, manpower, and financial resources.

A recent study by Brice and others (1) of countermeasures for hydraulic problems at bridges revealed that a primary cause of hydraulic problems at highway crossing sites is a change in the base level of the river. Changes in channel bed level can be described by three interrelated phenomena: local scour, general scour, and aggradation-degradation (changes in channel grade). This paper deals with the prediction of changes in channel bed level due to aggradation and degradation.

The terms aggradation and degradation have been defined in a variety of ways. The differences in the definitions come from defining the limits of the perspective used to view the river in time and space. In this paper, aggradation and degradation are considered to be changes in stream-bed elevation resulting from a change in channel gradient over an extended reach of a river. Aggradation results in a general rising of the channel bed, and degradation results in a general lowering of the channel bed. The time period involved ranges from days and months to years. The change in channel gradient can be caused by natural factors and events or by human activities. Natural causes include channel cutoffs, alluvial fan development, tectonic activity, landslides, and climatic changes. Human activities include land use changes, construction activities, channelization, floodplain clearing, streambed mining, and damming and reservoir regulation. The length of reach affected by the change in gradient can range from several hundred feet to hundreds of miles and is defined by the location of channel bed-level controls both upstream and downstream of the activity causing the change.

TECHNIQUES FOR EVALUATING MAGNITUDE OF GRADE CHANGES

Grade changes in river systems can be predicted through the use of various levels of analysis, from simple to extremely complex. The simplest techniques involve the application of mathematical statements of geomorphic principles, and the most complex techniques require analysis of entire drainage systems by using detailed computer modeling of water and sediment transport processes. However, the majority of highway crossing design situations do not initially justify the application of extremely complex computer models. Therefore, a less-complex, intermediate range of prediction techniques is presented here, including geomorphic and basic engineering relations as well as some simpler modeling techniques.

Before the various techniques available are described, it is important to point out that the prediction of grade changes should never be based solely on one prediction technique or model. The art of predicting grade changes is not an exact science. The most reliable results can be obtained by applying several techniques and tempering the quantitative results with engineering judgment and experience.

Geomorphic and Engineering Relations

Useful geomorphic and engineering relations include Lane's relation (2), hydraulic geometry relations, incipient motion considerations, analysis of changes in bed material volume, and engineering relations developed for specific applications.

Lane's Relation

Analysis of actual grade changes is based on the concept of equilibrium. The most widely known geomorphic relation embodying the equilibrium concept is Lane's principle:

$$QS \sim Q_s D_{50} \quad (1)$$

where

Q = water discharges,
S = channel slope,
Q_s = sediment discharge, and
D_{50} = median sediment size.

Equation 1 can be used to qualitatively predict channel response to modifications within the water-sediment regime of a channel. Application of Equation 1 has been documented by Richardson and others (3).

Although Lane's relation produces only qualitative results, its use is an important initial step in analyzing channel response. The technique indicates the direction of a grade change and the potential severity of a given problem.

Hydraulic Geometry Relations

Relations describing the geometric shape of regime channels have been developed by numerous investigators (4-8). These relations denote proportionalities between bankfull or dominant discharge, sediment transport, and channel geometry.

Leopold and Maddock (4) have shown that in a drainage basin the types of hydraulic geometry relations that can be defined are those that relate channel width (W), depth (v), velocity (V), slope (S), and sediment load (Q_s) to the variation of discharge (Q) at a station and those that relate these parameters to the change in mean annual or bankfull flow in the downstream direction. The regime formulas describing these relations are given in the following equations:

$$W \sim Q^b \quad (2)$$

$$y_o \sim Q^f \quad (3)$$

$$V \sim Q^m \quad (4)$$

$$Q_s \sim Q^j \quad (5)$$

$$S \sim Q^z \quad (6)$$

Table 1 gives mean values for the exponents b, f, m, j, and z as reported by Leopold and others (9). Also included are theoretical values developed at Colorado State University (CSU). The variation among the values reported can be explained by differences in the physiographic characteristics of the regions in which the exponents were evaluated. Factors important to the morphologic development of stream networks include local geology, hydrology, type and density of vegetation, type and depth of valley alluvium, and controlling valley slope.

It is important that the appropriate exponent be used in applying the proportionalities in Equations 2-6. This can be done by using the exponents in Table 1 developed for the physiographic region that most closely resembles the region of interest. An alternative technique would be to develop new exponents based on information from watersheds in the same physiographic region as the stream system of interest. The second technique would provide the most reliable information.

An example of the application of hydraulic geometry relations can be found in Chapter 8 of the report by Richardson and others (3).

Incipient Motion Considerations

Before soils can be transported through a system, some critical condition must be reached above which sediment motion will occur. This incipient motion condition is reached when the hydrodynamic forces acting on the grain of sediment have attained a value that, if increased even slightly, will move the particle. Under these critical conditions, or at incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle.

For noncohesive channel bed material, the beginning of motion is known to be a function of the Shields parameter, which can be represented as

$$\tau_c/(\gamma_s - \gamma)D_s \quad (7)$$

where

τ_c = critical boundary shear stress;
γ_s and γ = specific weights of sediment and water, respectively; and
D_s = characteristic diameter of the sediment particle.

The Shields parameter has been found to be approximately equal to 0.047 in situations where the flow boundary layer is fully turbulent. Theoretically, fully turbulent boundary flow occurs at shear Reynolds numbers larger than 400. This condition is satisfied by most river-flow situations.

Incipient motion considerations can be used effectively to calculate the limiting slope of a degrading channel bed. An example of such a calculation is provided in Chapter 8 of the report by Richardson and others (3).

Potential Change in Bed Material Volume

Another technique available for estimating the magnitude of potential grade changes is an analysis of the potential change in bed material volume in the vicinity of the crossing. This technique involves analysis of the transport capabilities of upstream reaches as well as those within the local reach of interest.

The analysis consists of applying sediment continuity to the reach of interest. To maintain continuity, the following equation must be satisfied:

$$\Delta V_o = [\Delta t/(1 - \lambda)]\,(Q_{so} - Q_{si}) \quad (8)$$

where

ΔV_o = sediment volume change within the reach,
Δt = time increment under consideration,
λ = bed material porosity,
Q_{so} = sediment transport rate out of the reach, and
Q_{si} = sediment transport rate into the reach.

The sediment continuity equation is applied with the aid of an appropriate sediment transport equation and an annual flow duration curve or other flow hydrograph. The choice of an appropriate sediment transport equation should be based on characteristics of the bed material in the reach of interest. The annual flow duration curve or other flow hydrograph can be constructed from stream-flow records.

The technique can be based on an annual flow hydrograph, a single storm hydrograph, or both. If average annual flows are anticipated to have the most significant impact on grade changes, an annual flow duration curve should be used for the analysis. If the major problem is anticipated to come from a single storm event or a series of storm events, the design storm flow hydrograph should be used for the analysis. In many cases, it is difficult to anticipate the type of flow event that will have the most impact on channel grade. In these cases, both flow events should be analyzed.

Applying the sediment continuity equation presented above requires the following steps:

1. Divide the flow hydrograph (annual or single event) into incremental time steps;

Table 1. Values of exponents in equations for hydraulic geometry of rivers.

	At Station					Downstream (bankfull or near annual flow)				
Location	b	f	m	j	z	b	f	m	j	z
Avg values for midwest United States	0.26	0.40	0.34	2.5		0.5	0.4	0.1	0.8	-0.49
Brandywine Creek, Pennsylvania	0.04	0.41	0.55	2.2	0.05	0.42	0.45	0.05		-1.07
Ephemeral streams in semiarid United States	0.29	0.36	0.34			0.5	0.3	0.2	1.3	-0.95
Appalachian streams						0.55	0.36	0.09		
Avg of 158 U.S. gauging stations	0.12	0.45	0.43							
Ten gauging stations on Rhine River	0.13	0.41	0.43							
Theoretical values (Colorado State University)	0.26	0.46	0.30		0.00	0.46	0.46	0.08		-0.46

2. With the channel geometry and bed material size gradation as input, compute the transported volume within each reach for each time interval (by using an appropriate sediment transport equation);
3. Apply the sediment continuity equation at each time step to compute the volume of material eroded during the time step;
4. Sum all ΔV_o's to find the total volume of material deposited or eroded during the time span of interest; and
5. Estimate a depth of deposition or erosion based on channel geometry and typical bed material porosity.

This technique provides an estimate of the magnitude of an aggradation or degradation problem at a local site on an annual or single-storm-event basis. It is not intended to provide a final estimate of the base level of the channel at the site. Such an estimate requires repeated application of the procedure over longer time spans and adjustments in channel slope and other hydraulic and geometric parameters at each step. This type of analysis is better suited for use with a digital computer and will be discussed later.

The technique described provides a relatively quick method of estimating the magnitude of an aggradation-degradation problem. It can be used to estimate changes in channel grade at a crossing and provide useful information for establishing the magnitude of a required maintenance program. It can also be used to evaluate quickly the relative importance of single storm events and average annual flow hydrographs to the grade change problem. An example application of this technique is presented by Brown, McQuivey, and Keefer (10, Appendix C).

Engineering Relations Developed for Specific Situations

In addition to the techniques already presented, there are a number of prediction techniques available that are based on assumptions that make them applicable only to specific cases. Time will not be taken here to explain these techniques in detail. They are described in the references cited below:

Problem	Reference
Degradation downstream of dams	Komura and Simons (11) Mostafa (12)
Aggradation upstream of dams	Bruk and Milorodov (13) Garde and Swamee (14)
Aggradation in streams from overloading	Soni (15) and others

Mathematical Modeling

Modeling techniques encompass sediment and flow routing computations over an extended reach of river. Sediment transport models range from methods that use hand calculation procedures aided by the use of small computers to fully developed dynamic models capable of handling unsteady, nonuniform flow routing and sediment routing by size fraction. Although this last model type could theoretically give excellent results, the data input requirements make it impractical to apply in many cases.

An analysis of typical highway-related aggradation-degradation problems and a review of available modeling techniques have revealed that most highway design situations do not require extremely complex models (10). In fact, a significant number of hydraulic problems at highway crossings occur on small- to medium-sized channels (less than 500 ft wide) (1), and a large number of these are in the 50- to 300-ft range. These channels can often be analyzed by using hand and/or computer-aided computation techniques or some of the simpler, fully computerized models.

An approach to modeling aggradation-degradation problems is presented below. Several levels of analyses found to be applicable to typical highway design situations are included in the discussion.

Background Data and Analysis

The development of an appropriate transport model requires a significant amount of background data and analysis before the actual profile analysis can be conducted. Background information includes locating channel controls, defining subreaches, collecting the hydraulic and geometric input, defining an appropriate flow event, establishing sediment boundary conditions, and selecting an appropriate time step for the analysis.

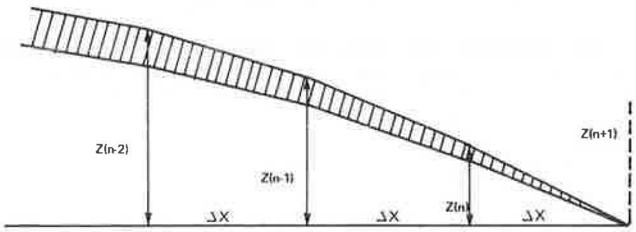
The first step in developing a system model is to locate the channel controls with respect to the particular aggradation or degradation problem. This defines the affected reach of the river and determines the extent of the river that should be included in the model. Channel controls can be either natural or man-made. Natural controls include outcrops of bedrock, heavily armored channel conditions, clay plugs, buried vegetation, fixed or controlled water surface levels, and, in some cases, tributary junctions. Most hydraulic structures built on rivers represent man-made controls. Typical examples include dams, weirs, spillways, free overfalls, underflow gates, sluice gates, culvert crossings, and some bridge crossings. Submerged pipelines can also act as controls.

With the controls established, the reach of interest is divided into subsections. The criterion for selecting the subsections is uniformity among hydraulic and geometric properties. These properties include channel slope, channel geometry, bed profiles, channel roughness, and bed material characteristics. The number of subsections or subreaches used depends on the degree of variance in the above parameters as well as the level of accuracy required in the results. Theoretically, the number used could range from one into the hundreds; however, increasing the number of subsections increases the number of computations required. Subreaches should be selected to provide the accuracy required by using the least number of sections. It is recommended that, if more than five subreaches are required, a digital computer be used for at least some of the computational steps.

The next step is to obtain the average hydraulic and geometric properties for the subreaches. These data include average channel geometry, reach lengths and slopes (defining the bed profile), estimates of channel roughness, bed material size distributions, and an approximate voids ratio for the bed material. The required precision of the input data will depend on the level of sophistication of the model being developed as well as the desired accuracy of the results. For example, channel geometry could be obtained in several ways. It could be estimated from site inspections, measured by using modern surveying techniques at one location per reach, or determined by surveying several locations within a reach and then averaging the resulting sections to get a section characteristic of the reach. Similarly, various levels of analysis could be used to obtain other required information.

The channel profile is then given in terms of (n + 1) bed elevations as shown in Figure 1. The ΔX's shown in the figure do not have to be equal, but for best results they should not vary by more than 35 percent.

Figure 1. Division of reach into finite elements.



Another input requirement for aggradation-degradation models is the selection of a flow event on which to base the grade-change prediction. The flow event chosen could be a single design storm, an annual flow duration curve, a series of synthetically generated annual flow hydrographs, or some other "average" discharge condition. The selection of an appropriate flow event is based on the hydraulic conditions that most influence the grade change. Often, several flow conditions must be evaluated to determine which will have the greatest impact.

Model boundary conditions must also be established. Sediment transport loads entering the model at the upstream control and other influent locations (at tributaries or other point load sources) must be estimated. Sediment loads entering the model at the upstream end can be computed by using an appropriate transport equation and assuming that the volume of material transported into the reach is equal to the capacity of the upstream reaches to transport sediment. If some other condition is known to exist, the sediment transport loads must be estimated by other methods.

The last step in setting up the model is to select an appropriate time interval (Δt) over which to compute incremental changes in the channel bed profile. The time interval selected does not have to remain the same for all steps; in fact, it is best to increase the Δt with each computational step because degradation and aggradation processes proceed most rapidly in the period immediately after the initiation of the grade change. If digital computer techniques are being used, however, the time steps should be equal in length. The value of Δt used depends on the type and cause of the grade-change problem as well as the hydraulic and geometric characteristics of the river system. The initial time interval is based on trial estimates of the initial grade-change rate. Subsequent changes in Δt at each step of the process are based on the actual rate of change documented in the preceding step. Consideration must also be given to the time steps in the discharge hydrograph to be used in the analysis.

Once the model is set up, the repetitive steps in computing the new profile can be started. This involves evaluating backwater conditions within the reach, computing sediment transport rates, and applying sediment continuity within each reach of each time step. As with other parts of the model setup, various levels of sophistication can be applied to each of the components involved.

Backwater Computation

Backwater profiles can be estimated by using normal depth consideration or calculated by using step backwater techniques. For the simplest applications, these computations could be made by hand. However, they can be easily programmed for repetitive use.

Normal depth can be calculated by using the Manning equation (given here in metric units):

$$Q = (1.0/n)\, AR^{2/3} S^{1/2} \tag{9}$$

where

n = Manning resistance parameter,
A = flow area, and
R = hydraulic radius.

The equation for trapezoidal channels can be expressed as

$$Q = (1.0/n)\left([(zy^2 + by)^{5/3} S^{1/2}] / \left\{ [b + 27(1 + z)^{1/2}]^{2/3} \right\} \right) \tag{10}$$

where z equals side slope (horizontal to vertical) and b equals bottom width. Either of these equations can be solved for y in terms of the other known parameters by a trial-and-error method.

Step backwater techniques have been covered in many texts (3,16,17). The calculations can be performed by hand or programmed for computer solution.

For complex situations that involve many bridges, culverts, and long reaches of river, it is often necessary to use a developed computer program such as the U.S. Army Corps of Engineers HEC-2 or others (18) to compute a water surface profile. The use of such a model is the only way to analyze large systems efficiently.

Each of the backwater techniques discussed is based on rigid boundary hydraulics, and care must be used in applying them to analyze alluvial systems. To account for the change in geometry produced in alluvial systems, the channel boundary must be adjusted during each time step to account for the computed aggradation and/or degradation. A technique developed by Chang and Hill (19) can be used to adjust the channel geometry, or aggradation and degradation volumes can be distributed uniformly across the channel.

Sediment Transport

The second step involved in computing the new profile is to evaluate the sediment transport along the channel within each subreach. Many transport equations are available, ranging in complexity from those that require simple graphical procedures (20) to those that require many calculations (21). Numerous texts cover the development of sediment transport relations; included among these are reports by Simons and Senturk (22) and the American Society of Civil Engineers (ASCE) Task Committee (23). In most cases, the development of sediment transport relations is based on specific bed material types (sand, gravel, cobble, or cohesive clays) and transport mechanisms (wash load, bed load, suspended load, or total bed material load). Therefore, it is important that bed material type and transport mechanism be considered in choosing an appropriate transport equation.

Brown, McQuivey, and Keefer (10), Simons and Senturk (22), and the ASCE Task Committee (24) provide comparisons of available transport relations. Table 2 illustrates the applicability of some of these relations to various conditions. The relations presented do not constitute a complete list of available equations.

Sediment Continuity

Once the sediment transport rates are established for each subreach, the following sediment continuity equation can be applied over segments of the channel

Table 2. Applicability of sediment transport relations.

Material	Method	Wash Load	Bed Material Load		
			Suspended	Bed Load	Total Load
Sand	Einstein		X	X	
	Modified Einstein	X	X	X	X
	Colby	Measured or estimated	X	X	X
	Schoklitsch			X	
Gravel and cobble	Schoklitsch			X	
	Meyer-Peter, Muller			X	
Cohesive	Ariathurai, Krone/ Parthenaides	Measured or estimated	X		

for the given time interval to determine the amount of aggradation or degradation:

$$[1/(1-\lambda)](\partial q_s/\partial x) = (\partial Z/\partial t) \quad (11)$$

where

q_s = sediment transport rate per unit width,
x = channel distance, and
Z = elevation.

Equation 11 is applied for each section, starting with the section between cross section (n + 1) and n in Figure 1. In the figure, the hatched area represents the volume to be eroded, which is (1/2) • $\Delta Z(n)$ • Δx. According to Equation 11, this equals the difference in sediment load at (n + 1) and n times the time interval Δt and times a coefficient converting weight into corresponding volume. The resulting equation is solved for Z(n), which produces

$$\Delta Z(n) = -2 \cdot \left\{ [g_s(n+1) - g_s(n)]/\Delta x \right\} \cdot \Delta t \quad (12)$$

The result is negative in the case of degradation and positive in the case of aggradation.

A similar procedure is then applied to the next section, which yields

$$\Delta Z(n-1) = \Delta Z(n) - 2\left\{[q_s(n) - q_s(n-1)]/\Delta x\right\} \quad (13)$$

and so on until the ΔZ's of all cross sections are determined. The geometry of all cross sections is then adjusted to reflect the degradation or aggradation. A new profile is determined by adding ΔZ to the previous Z.

The procedure described is repeated until the limits of the final equilibrium profile are approached. Because of the asymptotic nature of grade changes, it becomes evident that the final profile is being reached when successive ΔZ's become small. This final equilibrium profile can be evaluated separately by using the incipient motion criteria discussed earlier.

Several parts of the above procedures can easily be programmed on calculators or small computers. They include the sediment transport equation, the hydraulic properties of cross sections, normal depth or backwater computations, and the sediment continuity equation. Small programs for each of these procedures would greatly reduce the number of individual computations required and would therefore reduce the time required to apply the technique.

Fully Computerized Models

A number of fully computerized mathematical models dealing with sediment transport have been published in recent years. These models are based on the modeling concepts discussed above. Three models are representative of the types currently available.

Thomas and Prashuhn (25) present a sediment model coupled with a step backwater calculation. This results in a fairly inexpensive computational procedure and can be used over long periods by assuming a series of steady-state discharges (say, mean monthly). Chang and Hill (19,26) use a similar steady-state analysis. However, their model incorporates procedures to evaluate sediment transport through channel bends and to distribute scour and deposition across the cross section based on relative tractive force. The Chen-Simons model was developed at CSU and is one of a variety of models available there. The Chen-Simons model consists of a linear, implicit, finite-difference flow model coupled with a sediment transport model. This is a sophisticated technique but one that provides a high degree of accuracy.

LEVEL OF ANALYSIS

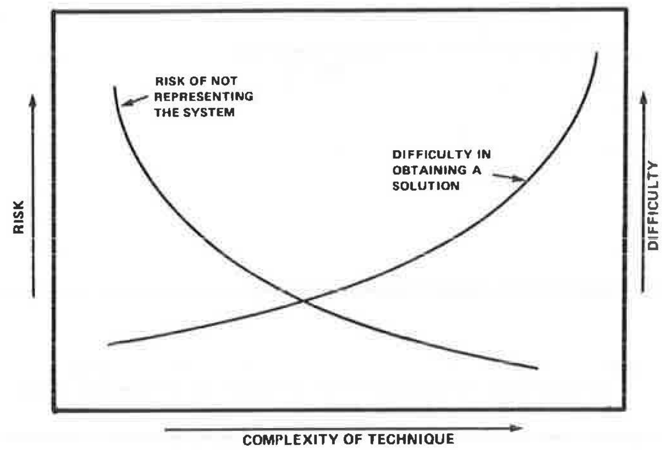
The cost of any engineering analysis is a function of the level of effort, which in turn depends on the accuracy required, the time available, and the analysis techniques used in reaching the solution. Techniques for estimating the magnitude of grade changes, ranging in complexity from simple geomorphic principles to complex computer models, have been discussed. Generally, the more complicated the analysis technique, the more accurate the solution becomes; however, the time, application difficulty, and associated costs also increase.

Because time, manpower, and money are always limited, decisions must be made regarding the degree of complexity needed to evaluate a given situation. According to Overton and Meadows (28), if a highly complex mathematical representation of the system under study is made, the risk of not representing the system will be minimized but the difficulty of obtaining a meaningful solution will be maximized. Many data will be required, and programming effort and computer time will be significant. Furthermore, the resource constraints of time, money, and manpower may be exceeded.

On the other hand, if a greatly simplified solution technique is used, the risk of not representing the physical system will be maximized but the difficulty in obtaining a solution will be minimized. Figure 2 shows the general concept of "trade-offs" involved in considering the complexity of the solution technique.

The application of all analytic procedures requires trained personnel to evaluate and interpret the results. Geomorphic concepts do not require extensive mathematical or computer analysis; however, they do require a very well-founded knowledge of the significant physical processes. Engineering relations and mathematical models can be used as "black box" calculations by support personnel. How-

Figure 2. Model complexity trade-offs.



ever, the design engineer must provide the proper input and be capable of interpreting the output.

The choice of an appropriate level of analysis must be based on available time, manpower, and financial resources. These are influenced heavily by the economic importance of a particular highway crossing as determined through risk analysis (29-31). It is also important to consider the governing physical processes and the sensitivity of system response at each site in determining an appropriate level of analysis.

CONCLUSIONS

Changes in channel bed-level elevation caused by aggradation and degradation processes have been found to be a significant cause of hydraulic problems at highway stream crossings. The number, variety, and level of analysis of the techniques available for predicting grade changes emphasize the uncertainty involved in calculations of sediment transport. It is important that grade-change predictions be based on more than one prediction technique or model and that the quantitative results be tempered by engineering judgment and experience. An appropriate solution procedure starts with the evaluation of geomorphic principles and relations (such as Lane's relation) to establish the cause and direction of the grade change. This can be built on by applying quantitative geomorphic and engineering relations (incipient motion considerations, change in bed material volume, etc.) as well as relations developed for specific grade-change problems. At this point a decision must be made to determine the level of analysis desired for additional work based on available time, manpower, and financial resources as well as risk considerations. Based on this decision, one or more levels of mathematical modeling can be applied.

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Abridgment

Stream Channel Grade Changes and Their Effects on Highway Crossings

STEPHEN ARNE GILJE

Stream channel degradation and aggradation are significant hydraulic problems at river crossings. Degradation is lowering of the streambed, independent of scour caused by obstructions or constrictions. Rapid or long-term degradation is usually due to a significant change in normal sediment-transport relations. Aggradation is stream infilling and occurs when more sediment is supplied to a stream than the stream is capable of transporting. Problems caused by stream degradation are far more common than those caused by aggradation. If actions are to be taken to protect a highway crossing against grade changes, early recognition of these hazards is imperative. Techniques for determining whether a crossing is experiencing degradation or aggradation are observation of stream characteristics (geomorphology), anticipation of gradation changes based on watershed activities, and measurement of pertinent stream dimensions. Streams in areas of high sediment yield are most prone to grade changes. Severe grade changes are often due to human intervention in natural stream processes. The problems associated with degradation and aggradation warrant special attention because protective measures effective against local hydraulic hazards are ineffective for protection against grade changes.

Degradation is lowering of a stream channel caused by a significant change in normal sediment-transport relations. It is independent of scour created by isolated obstructions or constrictions. Aggradation occurs when more sediment is supplied to a stream than the stream is capable of transporting. To deal with grade changes, it is necessary to understand what causes them and how to recognize their features.

The processes of streambed grade changes have been defined differently by various authors (1-3). The differences in definition stem from differences in the limitations on temporal and spatial perspective; as a result, grade changes are sometimes confused with scour and fill.

The pervasive nature of grade changes is often described in terms of a long time and great distance. This is an accurate description of many case studies but fosters the misconception that all grade changes progress similarly. When basic stream-forming factors are altered, the stream response is to change channel geometry. The direction of change, whether vertical or lateral; the rate, whether in seconds or in decades; and the distance affected, whether in meters or kilometers, is dictated by the physics of the situation involved. Degradation or aggradation that is significant enough to be of engineering concern is caused by major changes in the river environment.

HIGHWAY PROBLEMS DUE TO GRADE CHANGES

The extent of stream degradation and aggradation in the United States is demonstrated by a 1978 Federal Highway Administration (FHWA) research study (4). Data from 224 sites that were experiencing various hydraulic problems were assembled and carefully analyzed. Thirty-nine of these sites (17.4 percent) had undergone changes in streambed elevation.

Degradation is the lowering of a stream channel; therefore, a problem at crossings is the exposure of footings, pilings, and foundations (see Figure 1). Undermining of channel banks or highway fill results in failure of bridge approaches, revetment, and other countermeasures. Undermining of channel banks causes general stream instability and exacerbates debris problems (5). Channel instability is often a clue to future degradation problems; on the other hand, some instability problems (channel cutoffs) result in degradation. Degradation alters crossing conditions so that hydraulic hazards that under design conditions pose no significant threat to the integrity of crossings become critical. For example, local scour considered acceptable under design conditions could cause bridge failure when superimposed on a degraded channel.

Aggradation--general infilling of a stream channel--causes a reduction in the flow area available at crossings (see Figure 2). In extreme cases, the flow area is less than that necessary for design discharges, which results in overtopping of the roadway or bridge deck. During flooding enough horizontal or turning movement can occur to cause bridge damage. Aggradation increases stream instability because excessive sediment carried by an aggrading stream is prone to deposit in point and lateral bars. As these bars grow, stream sinuosity increases and flow is redirected into stream banks (located on the outside of meander bends across and slightly downstream from point bars). The most dramatic--and, fortunately, uncommon--aggradation hazard occurs when a stream is filled sufficiently to cause floodwater to overflow streambanks and seek a new channel. In evaluating grade problems nationwide, it was found that for each aggradation hazard identified there were three degradation hazards identified (3).

In the evaluation of highway problems, more than 80 percent of serious grade changes were caused by human intervention in natural stream processes (3). In fact, it was difficult to isolate severe grade changes that were caused naturally and were unaffected by human activities.

Recurrent gravel mining or dredging, reservoir-regulated flows, and land use changes can have continuing effects (see Figure 3). This is in contrast to natural and other human-induced causes, the effects of which are most severe following the impact but diminish thereafter (7). The magnitude of the impact is easily underestimated, as is the stream distance affected. Many streams have degraded more than 5 m (8). Grade changes have progressed many kilometers on small streams and hundreds of kilometers on major rivers.

Figure 1. Degradation and resulting channel erosion at Middle Creek crossing of I-80 near Lincoln, Nebraska.

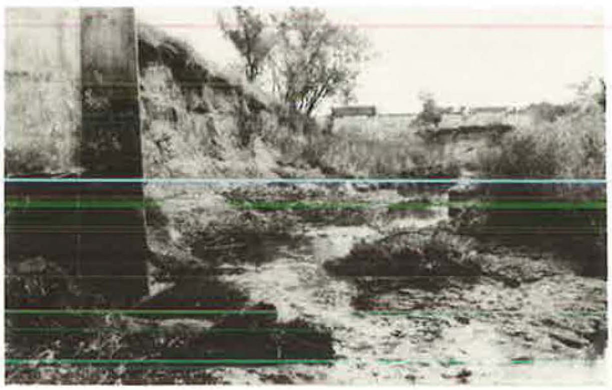

Figure 2. Aggradation on Badwater Creek at US-20 near Shoshoni, Wyoming.

Figure 3. Degradation due to gravel mining upstream and downstream on Amite River at LA-37 near Grangeville, Louisiana.

EARLY RECOGNITION OF DEGRADATION AND AGGRADATION

Techniques that can readily be applied to determine whether a crossing is experiencing a grade change are observation of stream characteristics (geomorphology), prediction of a grade change based on watershed activities, and measurement of pertinent stream dimensions. Evaluation should be based on experience with the stream in question, analysis of aerial photographs taken at different times over as long a period as possible, and field investigations. Hydraulic hazards can be recognized by drawing on experience with crossings elsewhere on the stream or on streams of a similar nature. A significant change in the character of a stream--a change of any type--is often the first signal that a pervasive hydraulic hazard is imminent (see Figure 4). Aerial photographs can be used to analyze long reaches at different periods in the past. Klingman (9) discusses the use of aerial photography in highway design. All stream sites should be evaluated in the field after review of maps and aerial photographs.

Observation of Stream Characteristics

Degradation and aggradation are opposites. Because degradation is a more common problem, the emphasis here is on the characteristics of degrading streams.

Direct observation of degradation in the early stages is difficult under perennial flow conditions. In addition, the dynamic nature of streams normally creates channel fluctuation, which can be mistaken for a progressive change. It is commonly easier to recognize the geomorphic effects of a degrading channel in the early stages than the bed change itself. An exception to this guideline is when degradation occurs as a headcut or sharp. A headcut is recognized as a local high slope area on a stream and in many cases is exhibited as rapids or falls. This type of degradation is more common on small, ephemeral, or intermittent streams that have channels composed of relatively cohesive bed material or armor. Headcuts are uncommon on larger perennial streams or streams with erodible beds.

Degradation can occur solely as downcutting (minor degradation where banks are resistant, well-vegetated, reveted, or in very small streams), as downcutting resulting in bank slumping (moderate degradation and cohesive bank materials), or as downcutting associated with severe bank erosion. All of these stream aspects are observable in aerial photographs or, as shown in Figure 5, in the field (note in the figure the slumping of the banks, the erosion on the convex section of the meander, and the toppled tree in the background).

Bank erosion on both sides of a meander, disappearance of lateral or midchannel bars, and exposure of unvegetated banks that are normally covered indicate degradation. Evidences of degradation that can be seen in aerial photographs are slumping and scalloping of channel banks, an increase in vegetative debris, trees overhanging the channel, and excessive bank erosion without resultant downstream growth of point bars. Streambed armoring is indicative of potential degradation. If the armor layer is breached during a flood, degradation may be rapid. Streams with high sediment yields have been shown to be more prone to degradation.

Figure 4. Degradation on Homochitto River at US-61 crossing near Doloroso, Mississippi, resulting in severe impact on lateral stability (indicated by change in stream form at right of photograph).

Figure 5. Severe bank erosion caused by degradation on Perry Creek at I-55 near Grenada, Mississippi.

Figure 6. Aggradation on Yellowstone River in Yellowstone National Park, illustrated by deposition at confluences and culverts.

Degradation can often be subtle on a mainstream. It is normal for some streams to be incised and exhibit bank erosion at bends and some armoring. In these cases, evaluation of small tributaries will indicate how deep the stream may have degraded. Because tributaries are higher-slope streams that carry less perennial flow, they maintain irregularities longer. Their banks and beds may also be composed of more resistant material than the mainstream, and the influence of vegetative controls is greater for small streams. If a tributary has an abrupt change in slope at its junction with a mainstream or if it contains headcuts, this indicates recent degradation. While the floodplain is being evaluated, the elevation of abandoned meanders and overflow channels should be documented. The age of these features can be estimated by the growth of vegetation and sedimentation characteristics. If these features are considerably higher than the present streambed (taking into consideration infilling due to silting and flood stages), the difference in elevation is indicative of degradation.

Aggradation is easier to evaluate than degradation because depositional features are usually obvious and can develop longer and more fully before they pose a danger to crossings. Aggradation can occur on any type of stream and is common in bedrock or mountainous streams. The first sign of aggradation is often accelerated growth of point bars as well as deposition elsewhere. A shift from a meandering to a braided pattern is indicative of aggradation (7). Aggradation is most active wherever there is a change of slope or in areas of backwater. Therefore, the first signs of aggradation are often at confluences, behind culverts, and at bridges (see Figure 6). The growth of natural levees and the occurrence of more frequent bank overtopping during moderate floods indicate stream aggradation.

PREDICTION OF GRADE CHANGES BASED ON WATERSHED ACTIVITIES

Dramatic examples of degradation and aggradation have been documented by Lane (8). These cases were caused by mining in the river channel, landslides, dams, and water diversion. Dams, channelization projects, gravel mines, and flow diversion projects are easy to spot in aerial photographs. However, establishing the connection between impacts and observed geomorphic characteristics is more difficult. The occurrence of an activity within a watershed does not necessarily mean that grade changes will occur. The extent of the activity and the stability and flow history of the stream are significant factors that determine final effects (10). Case histories illustrating the types of grade problems that result from different watershed activities have been documented, as have methods for calculating the extent of grade change (3,6).

MEASUREMENT OF STREAM PROPERTIES

The most direct method for evaluating a grade change is measurement from a fixed object (i.e., a bridge deck) to the streambed. Progressive and continual trends in bed elevations over an extended time period indicate a grade change. Original bridge design plans often take note of the distance from the bridge deck to the streambed at the time of construction. Comparison of the results of a quick field measurement with these existing data is useful to establish whether a change has occurred.

Engineers should be careful to avoid the measurement of local effects. Streams, as part of their natural character, have streambeds that fluctuate up and down. Engineers should also avoid comparing bed elevations that result from high flows with those of a dry period. Repeated measurements from different positions across the stream will yield the best results for comparative purposes.

Analysis of stream cross sections and longitudinal profiles can be used to determine the effects of grade changes. Comparisons of cross sections measured in the past with those taken later can be used to show whether the river has incised or risen (11).

If it is feasible to measure the profile of the stream along its length, these data are very useful (7). Measurements will identify hard points or base-level controls within the channel. Stream profiles drawn for stable streams are often concave upward. A stream profile concave downward is indicative of a degrading channel (8,12). In addition, stage-discharge relations reflect changes in bed elevation and can be used to indicate progressive degradation or aggradation.

CONCLUSIONS

Degradation or aggradation severe enough to be of significance to river engineering is not occurring on every stream nationwide. However, approximately one-fifth of the hydraulic hazards that are severe in nature are due to grade changes. Most of these incidences can be traced directly to an obvious cause: channelization, streambed mining, or a dam. Hydraulic engineers should be aware of the potential consequences of a significant change in the use or control of a stream. When such an impact is made on a stream with crossings in place, highway engineers should inspect the stream and crossings annually.

Degradation problems are far more common than aggradation problems. Streams carrying large sediment loads are more prone to problems. Methods of evaluating and calculating degradation or aggradation, as well as an evaluation of countermeasures for hydraulic problems caused by grade changes, are presented in the literature (3,6).

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Assessment of Commonly Used Methods of Estimating Flood Frequency

DONALD W. NEWTON AND JANET C. HERRIN

A study made to determine what are likely to be the most accurate and consistent procedures for determining peak flood flow frequencies for ungaged watersheds in the Tennessee Valley region is described. The study was based on information developed during a pilot test, which is currently the most comprehensive and objective method available of examining the performance of commonly used procedures for estimating peak flow frequencies at ungaged locations. Test results showed significant differences in performance when procedures were evaluated by using the criteria of accuracy, reproducibility, and practicality. Although the pilot test evaluation was limited to the Midwest and Northwest, it is concluded that the observed differences in performance result from fundamental differences in procedure formulation that can be expected to occur in the Tennessee Valley and in other regions as well. The most accurate and reproducible procedures evaluated were found to be regression-based procedures in which prediction equations are calibrated to flood-frequency determinations at gaged locations by using statistical estimating procedures. The most obvious reasons for this superior performance are the definition of the parameters and the formulation of the prediction equation. The procedures that performed best (a) used parameters that were well-defined and could be consistently determined, (b) were formulated so that the flood-frequency estimates were not sensitive to parameter variations, and (c) were calibrated to a large number of gage records in a relatively small, well-defined hydrologic region.

A common problem for hydrologists is estimation of peak flow frequencies at locations for which there are few or no flood records--ungaged locations. The different procedures in common use often provide different estimates at the same site. Furthermore, estimates made by different hydrologists using the same procedure will often vary.

An interagency Work Group of the Hydrology Committee of the Water Resources Council was assigned the task of developing national guidelines for defining peak flow frequencies at ungaged, unregulated stream locations. The Work Group found that the lack of agreement about the most commonly used procedures precluded developing a national guide without first obtaining objective information about procedure performance. A nationwide test was proposed that would provide an objective, authoritative means of discriminating between procedures by using the criteria of accuracy, reproducibility, and practicality. The scale of such a test presented problems of experimental design, quality control, and management. Consequently, a pilot test was conducted to evaluate the feasibility of a nationwide test and to provide information to design and implement it.

The pilot test currently provides the most comprehensive, objective examination available of the performance of commonly used procedures for estimating peak flow frequencies at ungaged, unregulated locations under field conditions. Test results show that it is possible to detect differences in procedure performance. The pilot test provides the information necessary to design and conduct a nationwide test from which procedures can be objectively selected for inclusion in a national guide.

The scope and quality of the pilot test were believed inadequate to permit the definitive conclusions necessary to write a national guide (1, p. 7). However, the pilot test data were found to provide valuable information for the practicing hydrologist who must select or develop procedures to estimate peak flow frequencies at ungaged locations.

The focus of the pilot test and Work Group analyses was examination of the feasibility of a nationwide test and, if the test were feasible, development of its design and implementation. This paper evaluates two questions:

1. What produced the differences in procedure performance observed in the pilot test?
2. Do these differences reflect inherent differences in procedure performance?

This evaluation was undertaken as a first step in selecting and designing improved flood-frequency estimating procedures for the Tennessee Valley region.

In this paper, the pilot test is described in sufficient detail to provide an understanding of the data base and to summarize the Work Group analyses relating to differences in procedure performance. A complete description of the pilot test, including the data and its analyses, is contained in the Work Group report (1).

PROCEDURE CLASSIFICATION

The Work Group adopted a classification scheme for categorizing procedures based on the assumptions made and methods used for estimating peak flow frequencies (1, Chapter II). The scheme included eight categories:

1. Statistical estimation of peak flows for a given exceedance probability,
2. Statistical estimation of moments,
3. Index flood,
4. Transfer methods,
5. Empirical equations,
6. Single storm event,
7. Multiple discrete events, and
8. Continuous record.

To evaluate procedure performance, we divided the eight categories into two groups: (a) those that involve transferring an estimate from a gaged to an ungaged site (categories 1-4) and (b) those that are based on the rain-runoff process (categories 5-8). The only transfer methods tested were regression-based procedures in categories 1 and 3. Thus, the terms "regression-based" and "rain-runoff" procedures are used for convenience in this paper. It is of interest that the Work Group found hydrologists about equally divided in their preference for regression-based or rain-runoff procedures for estimating flow frequency (1, Chapter III).

PILOT TEST

The pilot test consisted of five independent estimates of peak flow frequency for the 1-, 10-, and 50-percent-chance floods at 70 sites (42 in the Midwest and 28 in the Northwest) made by using as many as 10 different procedures. Test sites were restricted to watersheds with negligible man-made effects and with a gaged streamflow record of 20 or more years. Drainage areas varied from 0.08 to 943 $miles^2$.

The procedures selected for testing are identified in Table 1 (1, Table V-1, pp. 28-29). These were selected based on availability and currency of procedure, frequency of use, representativeness of procedure category, and applicability to the test sites. Procedures in categories 4, 7, and 8 were excluded from the pilot test because of the limited resources for testing. In tests of two complex watershed modeling procedures in category 6, Soil Conservation Service (SCS) Technical Release (TR) 20 and U.S. Army Corps of Engineers HEC-1, it was assumed that no site data were available for calibrating the procedures. However, some regional data were supplied for calibration in a number of the HEC-1 applications.

About 200 persons participated in the test. Estimates were made based on the assumption that no site data were available. Each tester was given a procedure package and a resource package. The procedure package provided instructions for conducting the test, a description of the procedure, and a record sheet for documenting answers, input parameters used in the solution, and information about the tester's years of experience, prior use of the procedure, and whether a field visit was made. The resource package provided topographic maps, aerial photographs, soils data, and rain data needed to apply the procedures.

Of 1784 procedure applications made, 1505 were included in the Work Group's statistical analyses. Table 2 (1, Table V-6, p. 47) gives the watershed-procedure test matrix. It excludes 140 applications of the Snowmelt procedure, 75 applications of the Reich procedure (19) in the Northwest, 10 applications of the Index Flood procedure, and 54 procedure applications made without resource packages. Four procedures (260 applications) were applied outside the intended range of application to evaluate their performance under such conditions. These are also identified in Table 2. Our graphical comparisons and those of the Work Group and our analyses of data effects on conclusions are based on the 1245 remaining applications.

CRITERIA FOR COMPARISON

The criteria for evaluating procedures were accuracy, reproducibility, and practicality. The standard used to evaluate accuracy was the log-

Table 1. Procedures selected for testing.

Category	Type	Procedure No.	Procedure
1	Statistical estimation of Q_P	2	Fletcher (2)
		4	U.S. Army Corps of Engineers Snowmelt (3)
		1	U.S. Geological Survey State Equations (4-9)
3	Index flood estimation	5	U.S. Geological Survey Index Flood (10-17)
5	Empirical equations	6	Rational method (18)
		3	Reich (19)
6	Single storm event, rain frequency proportional to runoff frequency	9	Soil Conservation Service Technical Release 20 (20)
		7	Soil Conservation Service Technical Release 55 (21, Appendix D, Charts)
		8	Soil Conservation Service Technical Release 55 (21, Chapter 5, Graph)
		10	U.S. Army Corps of Engineers HEC-1 (22)

Table 2. Watershed-procedure test matrix.

Location	Drainage Area (miles^2)	No. of Sites	No. of Procedure Replicates: 1	2	3	5	6	7	8	9	10	Total
Midwest	0-3	8	40	40	40	-	40	40	40	-	-	240
	3-10	8	40	40	40	-	40[a]	-	40	15	15	230
	1050	8	40	40	40[a]	40	-	-	40[a]	20	20	240
	50-100	10	50	50[a]	-	50	-	-	-	15	15	180
	>100	8	40	-	-	40	-	-	-	10	10	100
Northwest	0-3	5	20[b]	25		-	25	25	25	-	-	120
	3-10	5	15[b]	25		-	25[a]	-	25	15	15	120
	10-50	5	10[b]	25		25	-	-	25[a]	10	10	105
	50-100	8	25[b]	40[a]		40	-	-	-	10	10	125
	>100	5	10[b]	-		15	-	-	-	10	10	45
Total		70	290	285	120	210	130	65	195	105	105	1505

[a]Outside range of intended application. [b]No state equations in Oregon and Idaho.

Pearson Type III flood-frequency estimate (1, Appendix 2) at the site based on the available stream gage record (gage estimate).

Accuracy is a measure of the closeness of the flood-frequency estimate to the standard. The criterion variable selected to numerically represent accuracy in the statistical analyses was bias. It was defined as the difference between the mean of the flood-frequency estimates for a watershed and procedure and the gage estimate, standardized by dividing by the gage estimate. Reproducibility is the ability of different people to get the same result at a site by using the same procedure. The criterion variable used in the analysis was the standard deviation of the flood-frequency estimates for a watershed and procedure, standardized by dividing by the square root of the gage estimate. Accuracy is measured against the gage estimate and reproducibility against the performance of other testers. Practicality is a user decision that involves balancing the effort involved and the study requirements. The time in hours required to apply the procedure was used to represent practicality because it is the factor of major importance.

PERFORMANCE COMPARISONS

The box plots shown in Figure 1 (1, Figure VIII-4, p. 84) provide a graphical comparison of procedure performance in estimating the 1-percent-chance flood. Each box plot shows the distribution of individual estimates for a given procedure expressed as a percentage deviation from the gage estimate. The height of the box defines the 25th and 75th percentiles. The median and mean are shown by solid and dashed lines, respectively. The 10th and 90th percentiles and the minimum and maximum values are shown by lines. The width of the box is a function of the sample size, and the sample size is given inside the box. Procedures are identified by number.

The plots show that for the pilot test there were differences in procedure performance. These differences were found to be similar for each probability of exceedance. These same differences are evident when procedure performance is evaluated by the criterion variables of bias, shown in Figure 2 (1, Figure VIII-7, p. 88), and reproducibility, shown in Figure 3 (1, Figure VIII-8, p. 89). Note that bias is an average watershed value in contrast to Figure 1, which compares individual estimates.

Figure 4 (1, Figure VIII-9, p. 90) shows the time required to apply the procedures. As expected, there is a distinct difference in the times required to obtain results by using the simple estimating procedures 1-8 and the more complex watershed modeling procedures 9 and 10.

Statistical analyses of the data made by using both analysis of variance (ANOVA) and multivariate analysis of variance (MANOVA) techniques showed that there were statistically significant differences in procedure performance (1, pp. 87-99).

EFFECTS OF TEST DESIGN FACTORS

The pilot test was designed to evaluate the variation in procedure performance with respect to watershed size, exceedance probability, region, and use of a procedure beyond its intended range. Also evaluated were the effects of variations in the gage estimate, the unbalanced nature of the experimental design, the use of average watershed values rather than individual values of bias, and the method of standardization.

The ANOVA and MANOVA results changed slightly with these factors, but conclusions about the relative performance of the procedures were not altered. As expected, the time required to apply a procedure increased with increasing watershed area (1, pp. 99-112).

Figure 1. Percentage deviation from gage estimate: total analysis.

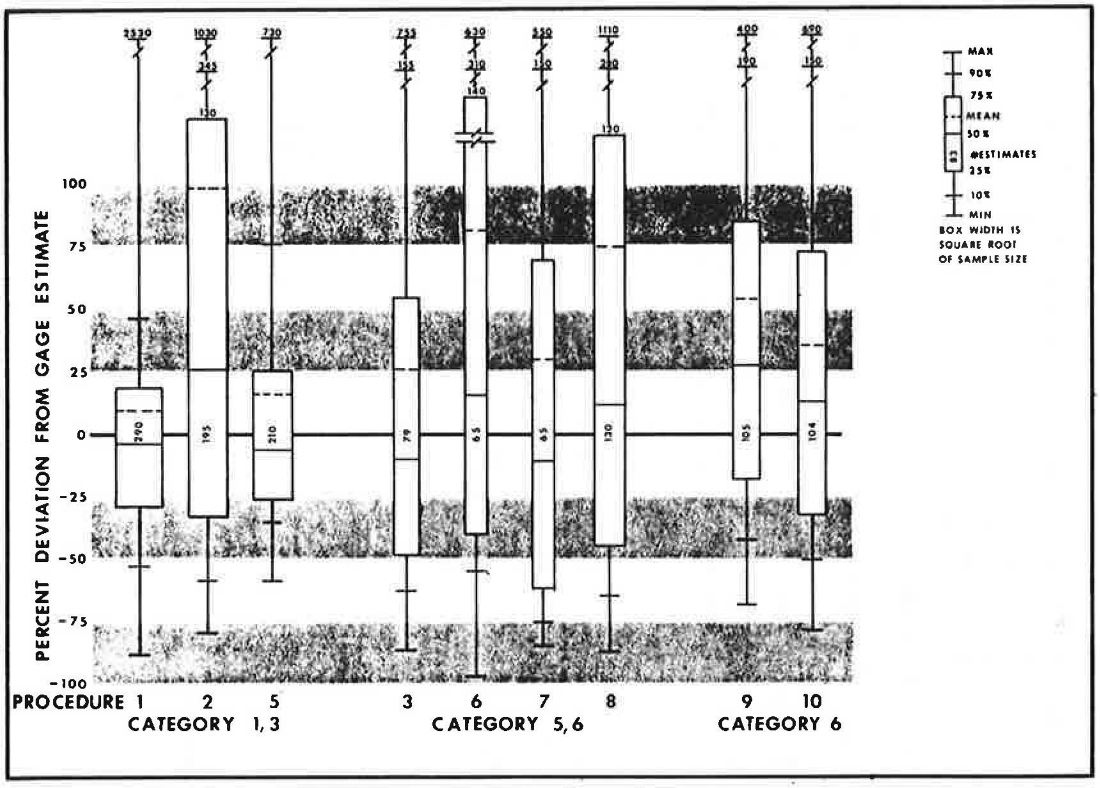


Figure 2. Total analysis: bias.

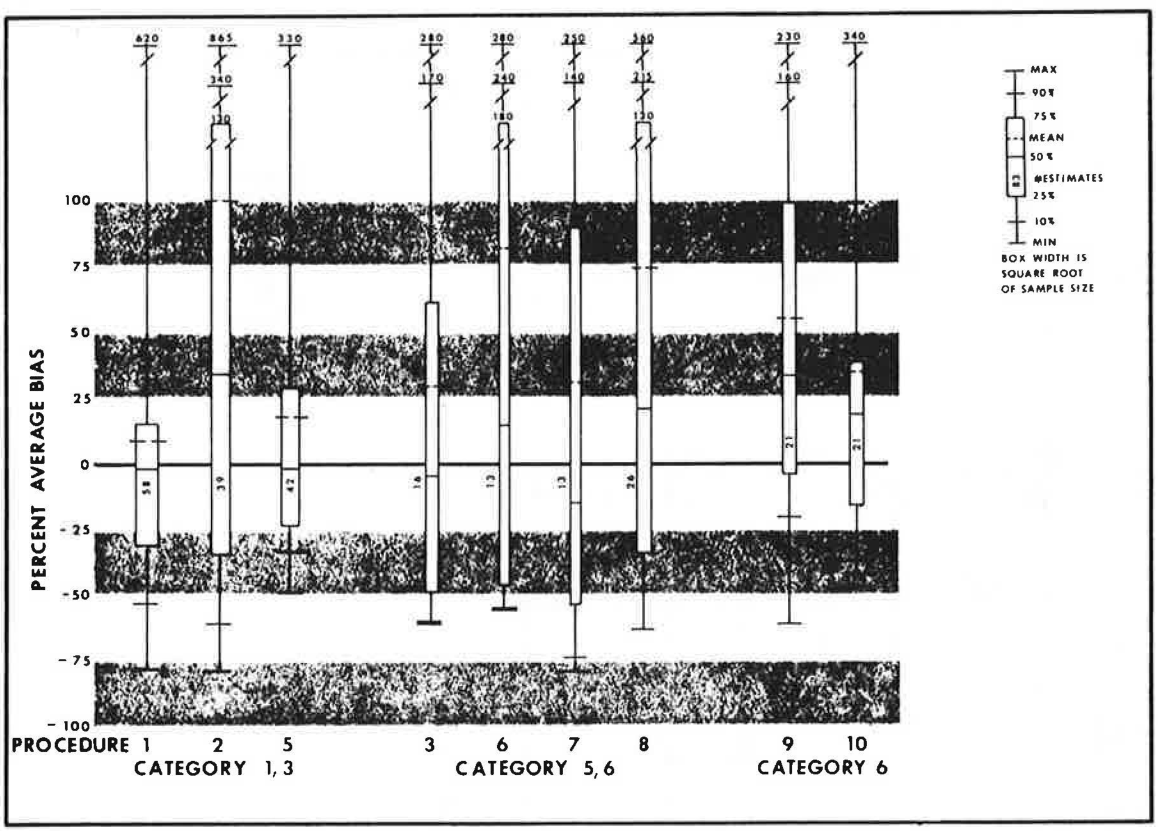


Figure 3. Total analysis: reproducibility.

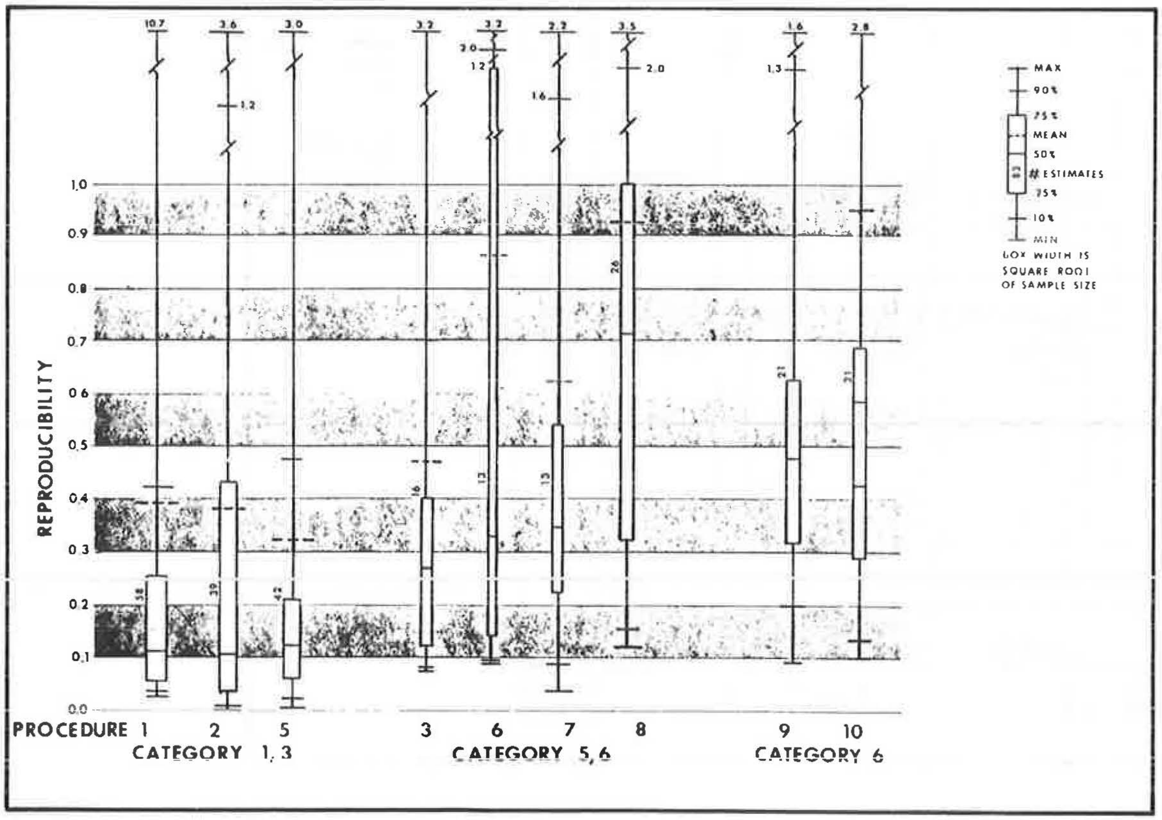
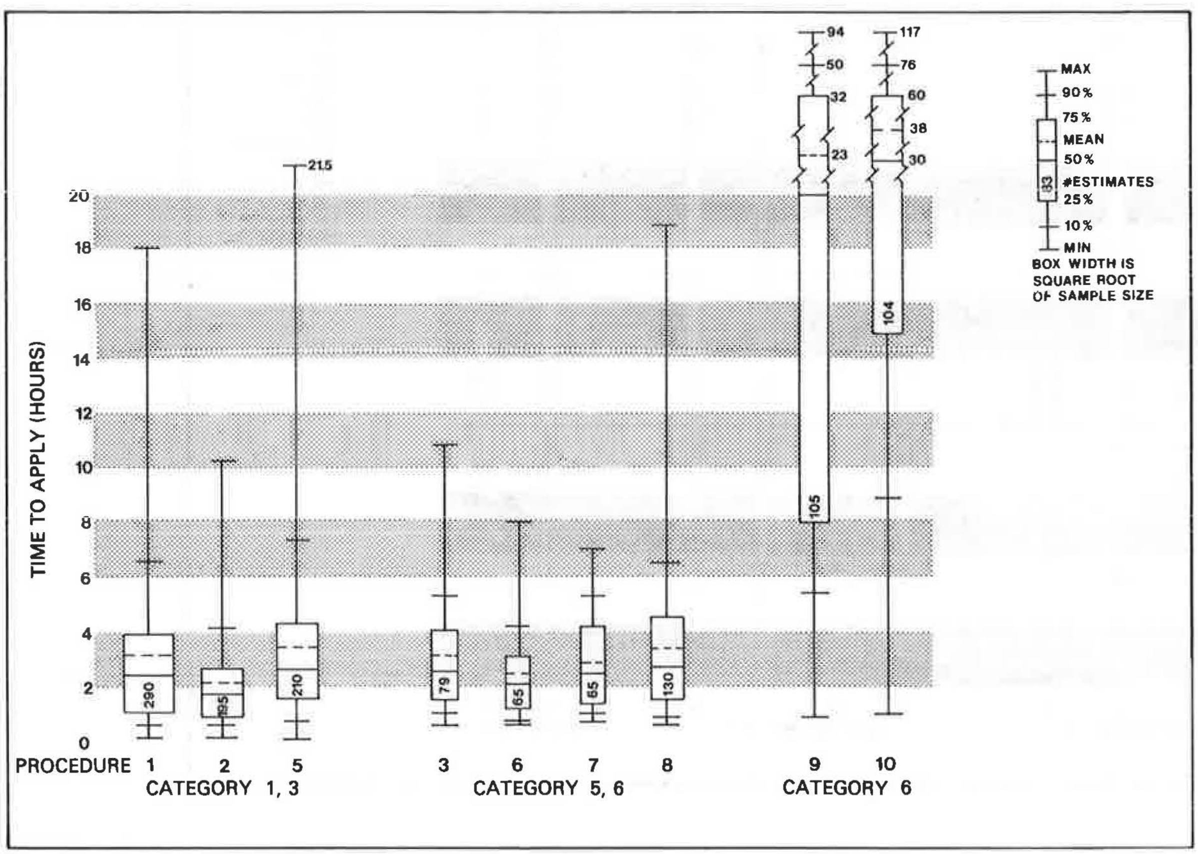


Figure 4. Total analysis: time to apply procedure.



DATA QUALITY EFFECTS

The following aspects of the quality of pilot test data were identified as having the potential to alter conclusions about procedure performance: (a) computational errors, (b) tester experience, (c) frequency of tester use of the procedure, (d) tester knowledge of the region, (e) a field visit by the tester, and (f) testing of regression-based procedures against the stations used to develop the procedures. Because the potential effects of these factors were different for the regression and rainrunoff procedures, the two types of procedures are discussed separately.

The statistical analyses by the Work Group were performed with unedited data. Only obvious coding errors and keypunch or other transcription errors were corrected. In the analysis of the data set, 50 extreme values identified as being in error were excluded (about 3 percent of the data base). These extreme values included errors in parameter determination as well as errors in solving the equations. This analysis did not change the conclusions from those obtained in analyzing the unedited data (1, pp. 114 and 265).

Regression-Based Procedures

Errors in estimating parameters and computational errors in solving the flow estimating equations, given the parameters, can be anticipated in dealing with as many numbers as were involved in the pilot test. The U.S. Geological Survey (USGS) State Equations and the Fletcher procedure can be checked rather simply for computational errors since there is a unique, direct solution for each set of parameters. These procedures were computerized and solved by using input parameters provided by the testers.

Nine percent of the 290 USGS State Equation applications and 11 percent of the 285 Fletcher applications were found to be in error. An error was considered to be a difference of at least 10 percent from the computer solution. These errors did not significantly affect the bias criterion (1, p. 113) and thus would not change conclusions about procedure performance. The USGS Index Flood methods were not checked, but there is no reason to believe that the error rate would be any different than for the USGS State Equations or the Fletcher procedure.

As shown in Figures 5 and 6, the performance of the USGS State Equations (procedure 1) and the Fletcher procedure (procedure 2) did not change significantly with experience. Average percentage deviation did increase with experience, but this was evidently due to a few unusually large estimates, as shown by the median values. It should be noted that the question about experience asked for "years of hydrologic experience". It did not specifically ask for years of experience in making flood-frequency estimates for ungaged watersheds.

As Table 3 (1, Table VIII-1, p. 79) indicates, the size of the test was such that persons unfamiliar with the procedure or the region and who did not visit the site did much of the testing. The pilot test was not specifically designed to evaluate the effect of tester background or field visits on procedure performance. However, this information was collected. Testers whose background met the following set of conditions were identified:

Factor	Case 1	Case 2	Case 3
Hydrologic experience (years)	>2	>2	>2
Knowledge of region	Some, much	None	Some, much
Frequency of use of procedure	Occasionally, frequently	Occasionally, frequently	Never

Case 1 identifies the tester who is potentially the most proficient--who has more than 2 years' experience, uses the procedure at least occasionally, and has at least some knowledge of the region. Case 2 identifies the tester who lacks only a knowledge of the region to be proficient. Case 3 isolates the frequency-of-use factor. The effect of a field visit was not investigated because only about 6 percent of the testers who applied the regression-based procedures made one. In addition, in these procedures it was not believed that a field visit would significantly affect the flow-frequency estimates.

To obtain information about the potential effects of tester background, flow-frequency estimates made by testers using the USGS State Equations were grouped as follows:

1. All testers,
2. Testers who met case 1 conditions and all other testers on the same watersheds,
3. Testers who met case 2 conditions and those testers on the same watersheds who met case 1 conditions, and
4. Testers who met case 3 conditions and those testers on the same watersheds who met case 1 conditions.

The box plots shown in Figure 7 compare the estimates for testers using the USGS State Equations. The number of watersheds for which the comparison can be made is shown. For example, there were 47 watersheds where at least one tester met case 1 conditions. On these 47 watersheds, 74 testers met the three case 1 conditions and 161 did not meet at least one of the conditions. There were 44 watersheds where at least one tester met case 2 conditions. On these 44 watersheds, 77 testers met case 2 conditions and 52 met case 1 conditions.

Comparisons within each case, between cases, and with the total analysis show no dramatic change in overall procedure performance between the potentially most proficient testers and the other groups. It is concluded that the results achieved by using the USGS State Equations are not improved by knowledge of the region or frequency of use of the procedure. This is as expected because all judgments are explicitly incorporated into the parameters of the equation. Although the Fletcher and Index Flood methods were not evaluated, the same results could be expected with those procedures.

The regression-based procedures were tested against gaging stations that were used to develop the prediction equations. This could affect bias but it would not affect reproducibility and application time because they are not compared with the gage estimate. The number of pilot test sites in a given state or region represented a small percentage of the stations used to develop the equations and thus was not expected to have a significant influence on a particular equation. The regression equations for Illinois and Ohio were recomputed without the watersheds used in the pilot test. New flow-frequency estimates were made by using watershed characteristics supplied by the tester. The conclusion was that the bias criterion variable and procedure comparisons in the pilot test were not affected by this factor (1, p. 118).

Figure 5. Tester experience: procedure 1.

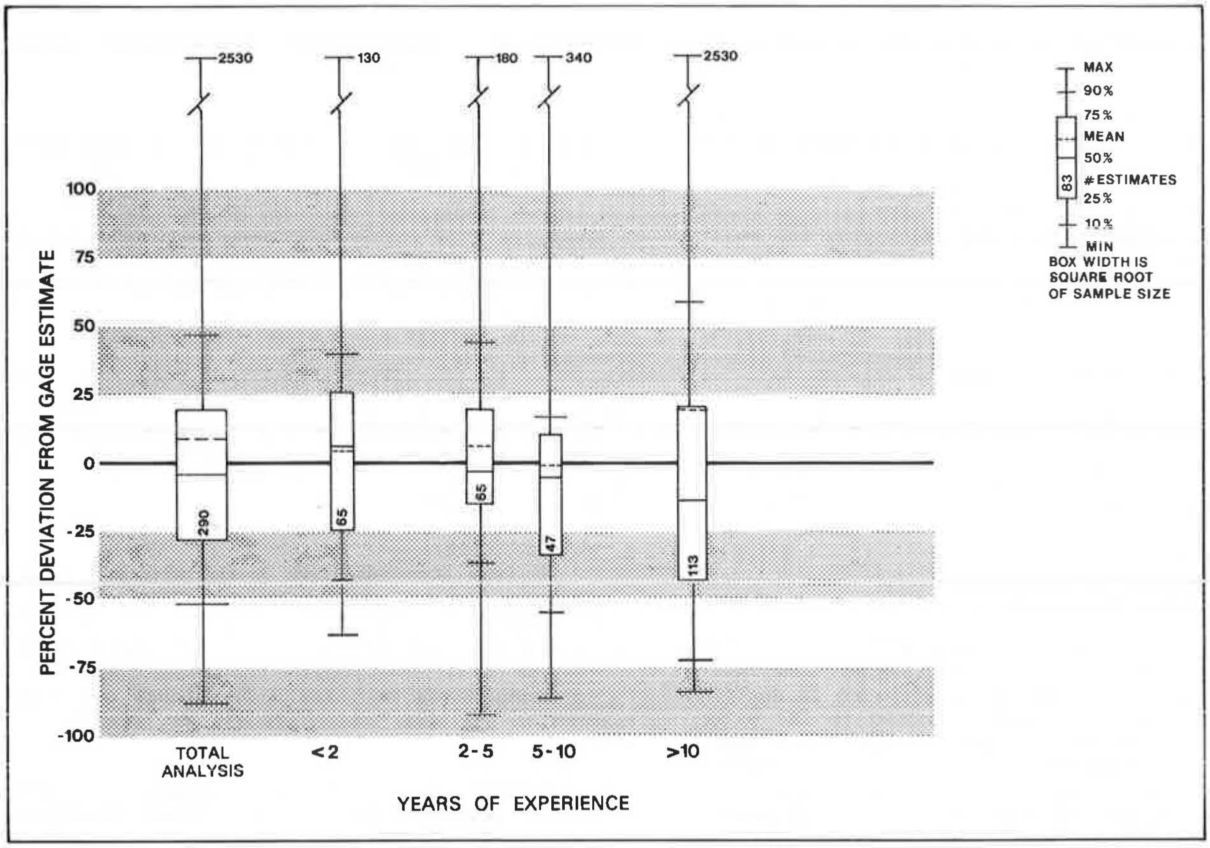
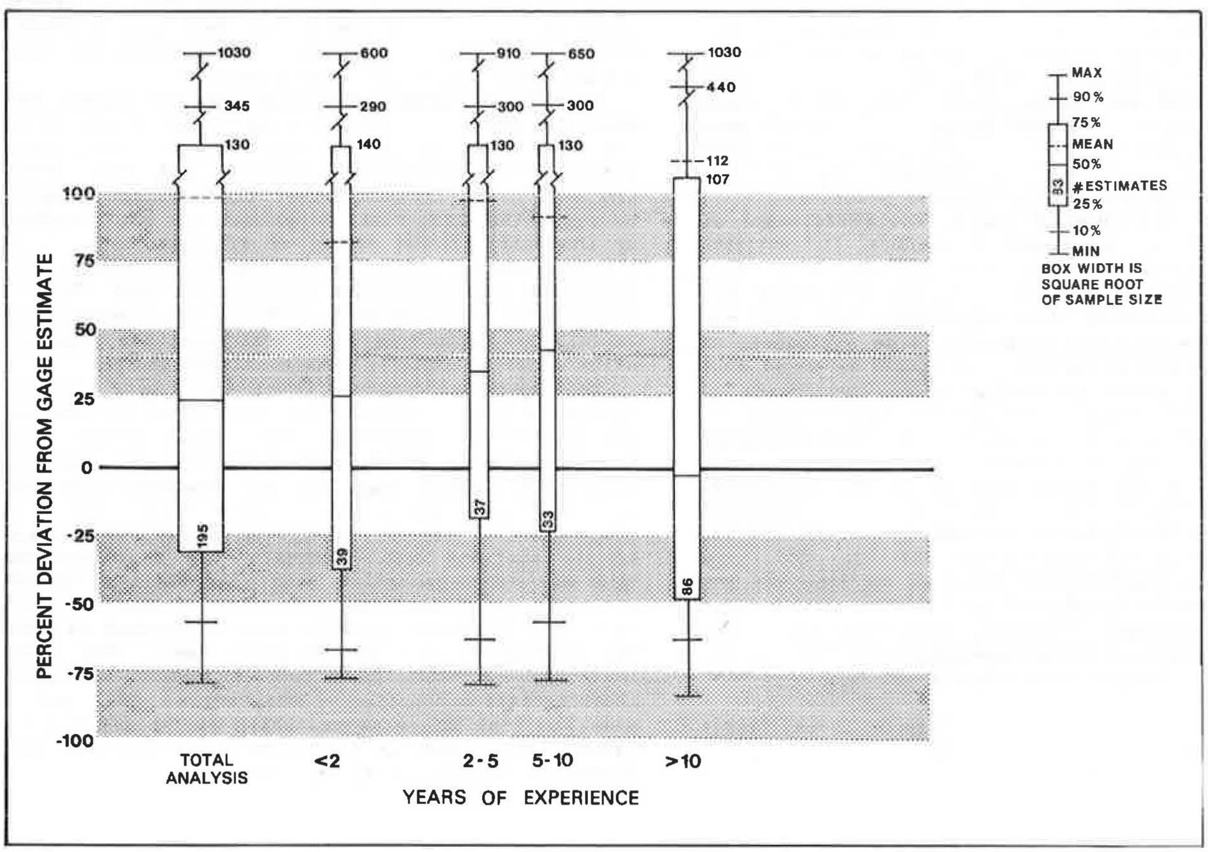


Figure 6. Tester experience: procedure 2.



Rain-Runoff Procedures

The rational formula estimates were checked for computational errors as were the USGS State Equations and the Fletcher procedure. Only 7 percent of the 130 applications were found to be in error. These errors did not significantly affect the bias criterion (1, p. 113).

It was more difficult to check the two TR-55 procedures because their application requires tester judgment. The problems associated with testing these procedures were subjectively evaluated by a person familiar with the procedures. This evaluation was based on a detailed examination of 65 applications of the TR-55 Charts procedure and 195 applications of the TR-55 Graph procedure to determine the 1-percent-chance flood. For this evaluation, input parameters given on the test record sheets were used to reconstruct the testers' peak estimates.

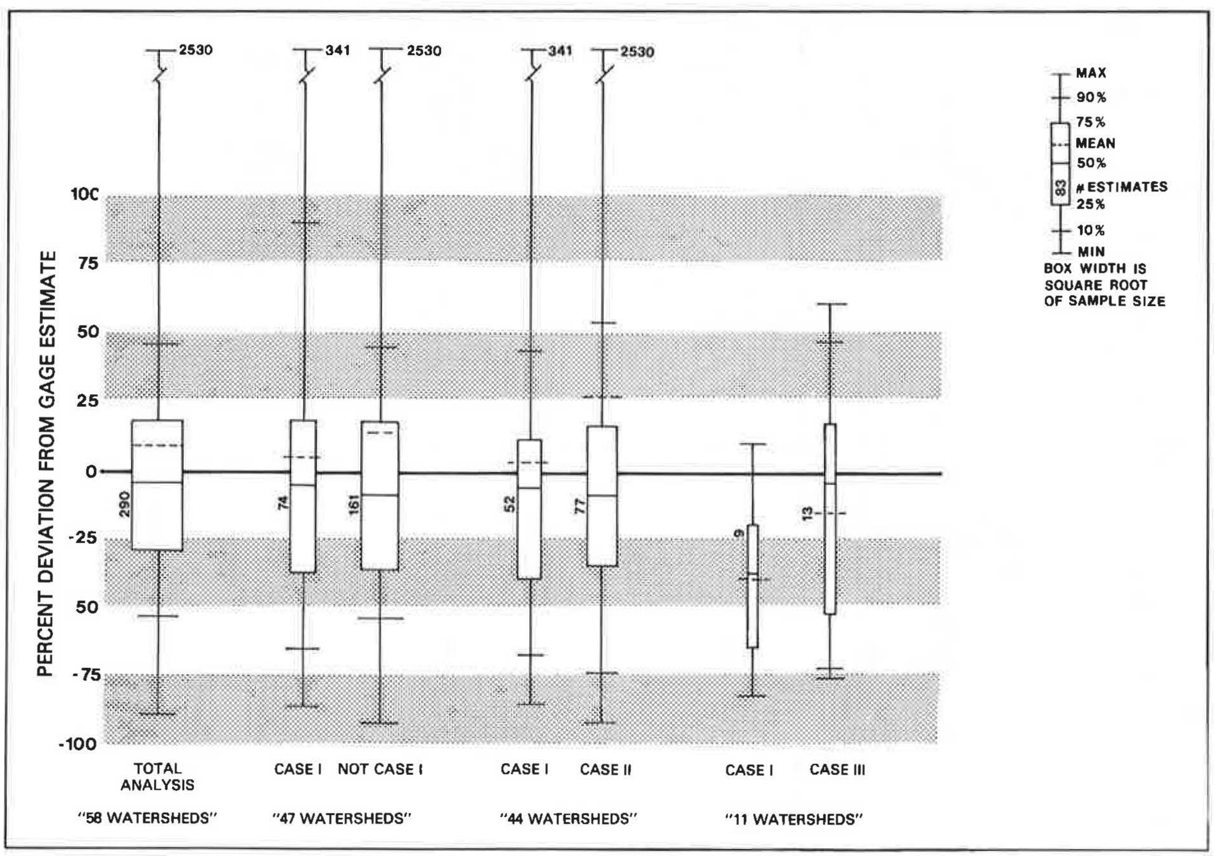
Two types of errors were identified: mathematical errors and improper use of the procedure. A result was classified as a mathematical error if the tester estimate could not be reconstructed within 5

Table 3. Tester background information related to use of various procedures.

	Testers (%)									
	Category 1 and 3 Procedures			Category 5 and 6 Procedures				Category 6 Procedures		
Factor	1	2	5	3	6	7	8	9	10	Avg
Hydrologic experience (years)										
0-2	22	24	28	17	17	18	20	21	12	22
2-5	22	23	24	17	20	25	19	27	26	22
5-10	17	14	14	19	17	12	16	20	23	16
>10	39	39	34	47	46	45	45	32	39	40
Knowledge of region										
None	68	70	71	63	65	69	63	68	68	68
Some	27	27	24	32	32	29	33	26	27	28
Much	5	3	5	5	3	2	4	6	5	4
Frequency of use of procedure										
Never	40	80	57	92	55	58	59	68	60	62
Occasionally	38	19	39	8	39	37	29	23	34	30
Frequently	22	1	4	0	6	5	12	9	6	8
Field inspection										
No	94	94	93	93	94	97	91	82	84	92
Yes	6	6	7	7	6	3	9	18	16	8

Note: Based on 1505 applications.

Figure 7. Other tester information: procedure 1.



percent. The results of this error analysis are as follows (1, pp. 113 and 114):

Problem	T-55 Charts (%)	T-55 Graph (%)
Procedures applied "correctly"	45	73
Shape factor applied "incorrectly"	22	5
Rainfall not converted to runoff	13	8
Multiple, mathematical, or other procedural errors	20	14

The box plots shown in Figure 8, which we developed, compare the applicable estimates for the 1-percent-chance flood obtained by using the TR-55 Charts (procedure 7) and the TR-55 Graph (procedure 8) for (a) the total analysis, (b) the procedure when applied correctly, and (c) the various incorrect applications identified by the Work Group. Surprisingly, procedure performance is not significantly different for the three different groups.

The other rain-runoff procedures, Reich, TR-20, and HEC-1, were not checked for computational errors.

As shown by the analyses in Figures 9-11, the performance of the rational and two TR-55 procedures varied somewhat with experience. The overall performance of the best-performing experience groups, however, was still relatively poor when compared with the USGS State Equations (procedure 1) and Index Flood methods (procedure 5) (Figure 1).

Because of the judgments involved in applying the rain-runoff procedures, it would be expected that the tester's familiarity with a procedure and knowledge of the region and whether or not a field visit was made would have an impact on flow-frequency estimates. The same method was used to evaluate the impacts of these factors on the results obtained with the rational formula and the two TR-55 procedures as was used for the USGS State Equations. The results are shown in Figures 12-14. Comparisons within each case, between cases, and with the total analysis show no dramatic change in overall procedure performance between the proficient testers and other groups. It is concluded that the flood-frequency estimates made with these procedures under the pilot test conditions were not significantly improved by knowledge of the region or familiarity with the procedure. The effect of a field visit on procedure performance was not evaluated because so few testers made such a visit.

Because the samples are small, the above comparisons were made by using all tester applications of these rain-runoff procedures. This included applications of the rational formula and the TR-55 Graph on watersheds outside the intended range of applicability and applications identified as incorrect. The following comparisons and reasoning supported use of all of the data. The same comparisons were made by using only those watersheds within the intended range of applicability of the rational formula and the TR-55 Graph. No difference was found. The relation between tester knowledge of the TR-55 procedures and their correct application as identified by the Work Group was examined. It showed that the percentage of correct and incorrect applications was about the same regardless of the tester knowledge of the procedure. As discussed earlier, procedure performance was found to be not significantly different between procedure applications identified as correct or incorrect by the work group.

It is concluded that, for both the regression and rain-runoff procedures, the differences in procedure

Figure 8. Total analysis versus correct analysis versus incorrect analysis: procedures 7 and 8.

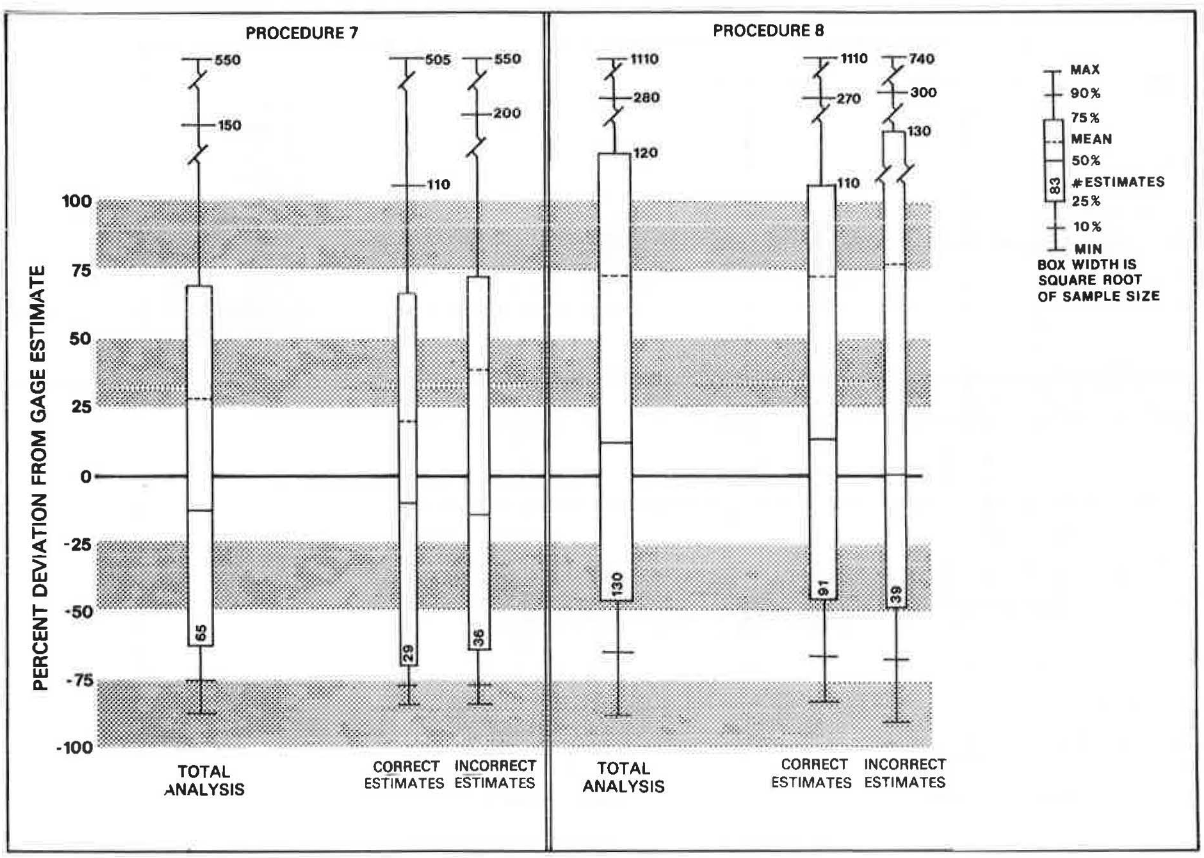


Figure 9. Tester experience: procedure 6.

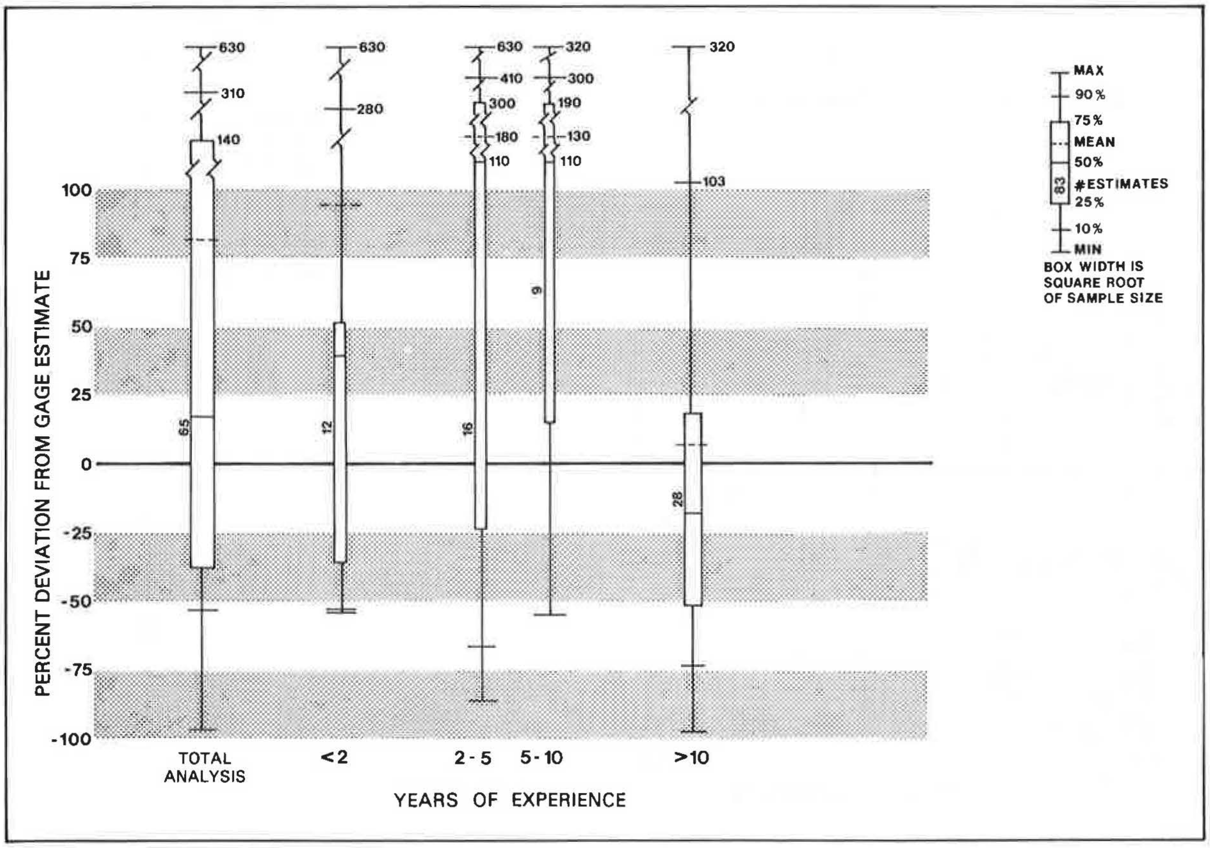


Figure 10. Tester experience: procedure 7.

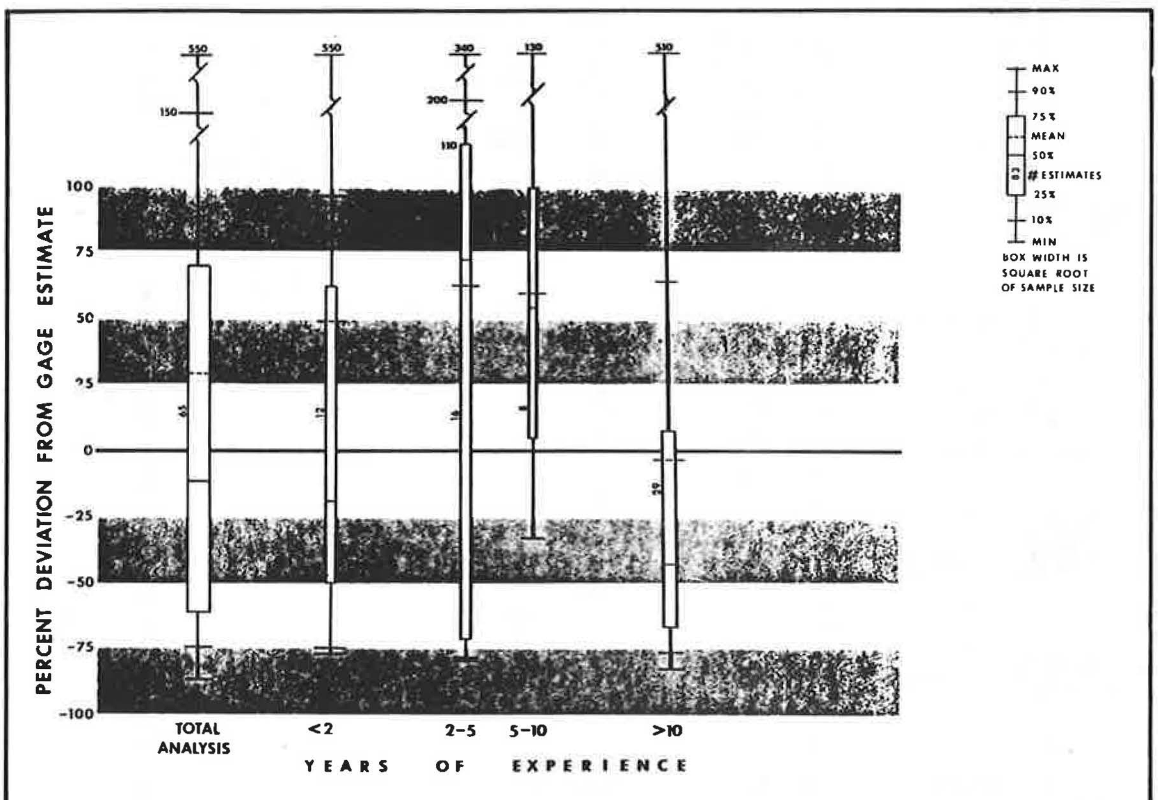


Figure 11. Tester experience: procedure 8.

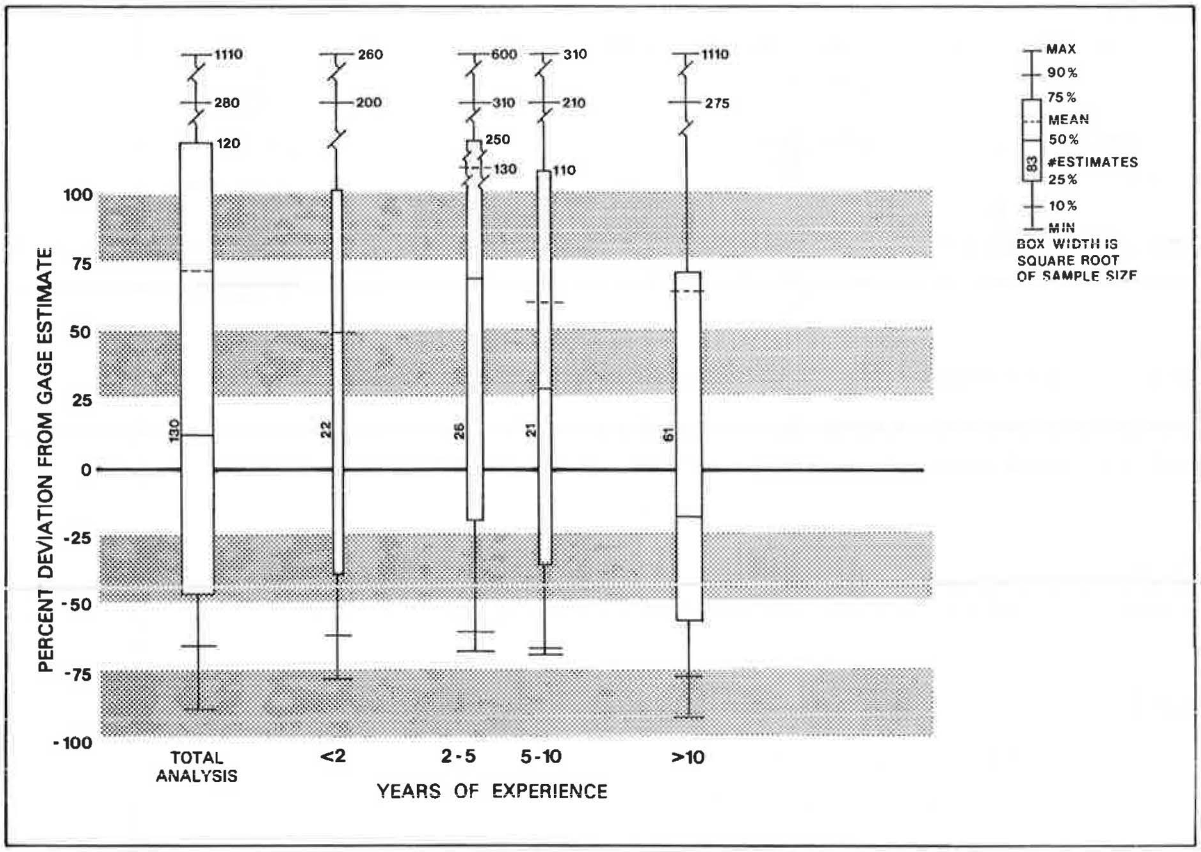


Figure 12. Other tester information: procedure 6.

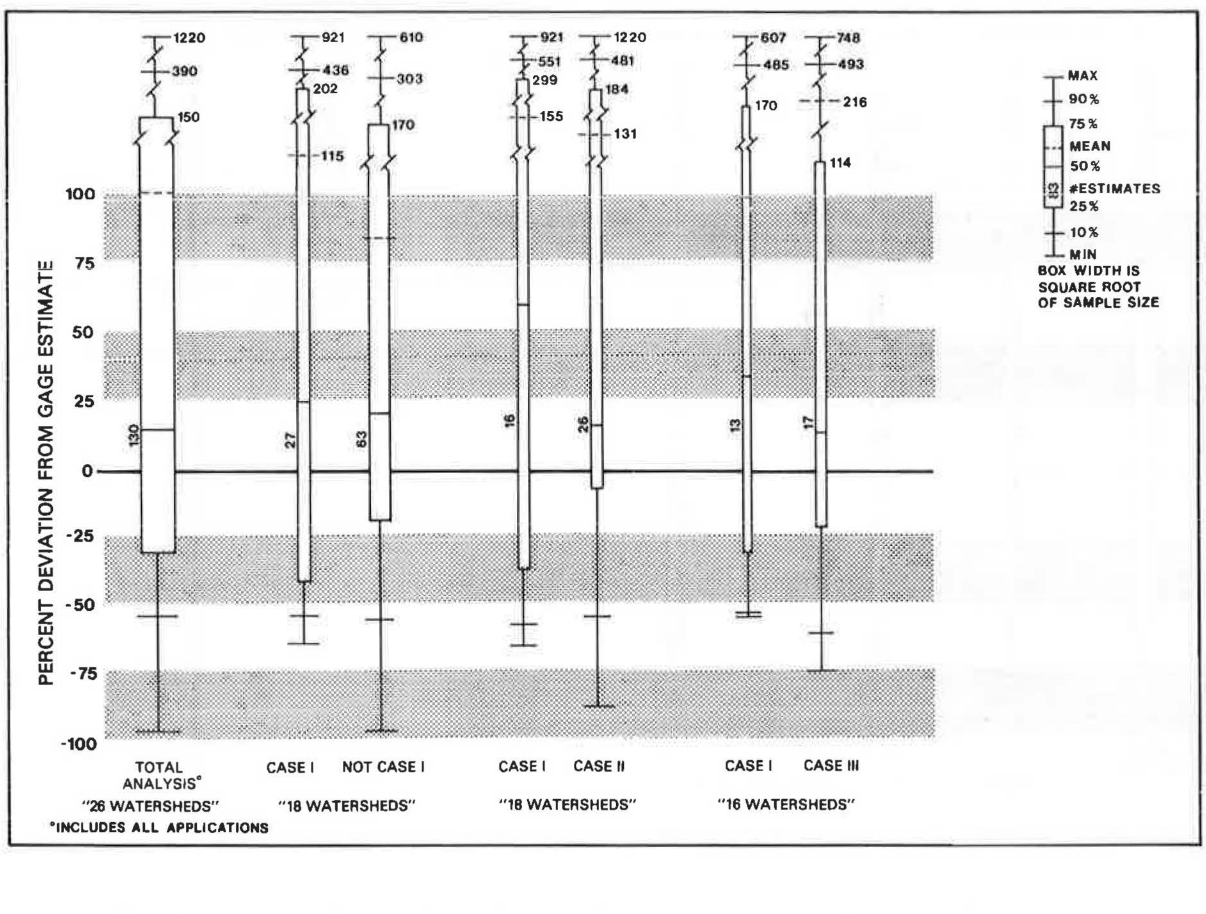


Figure 13. Other tester information: procedure 7.

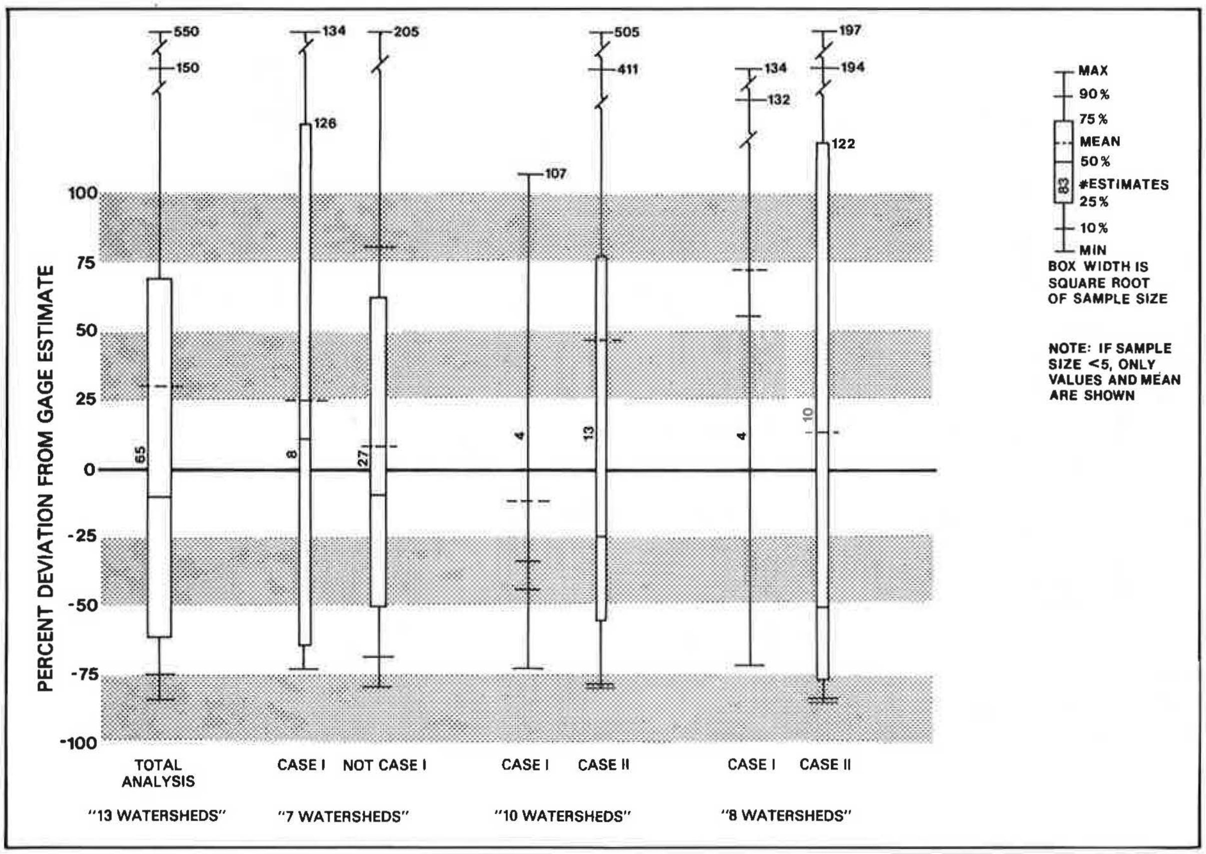


Figure 14. Other tester information: procedure 8.

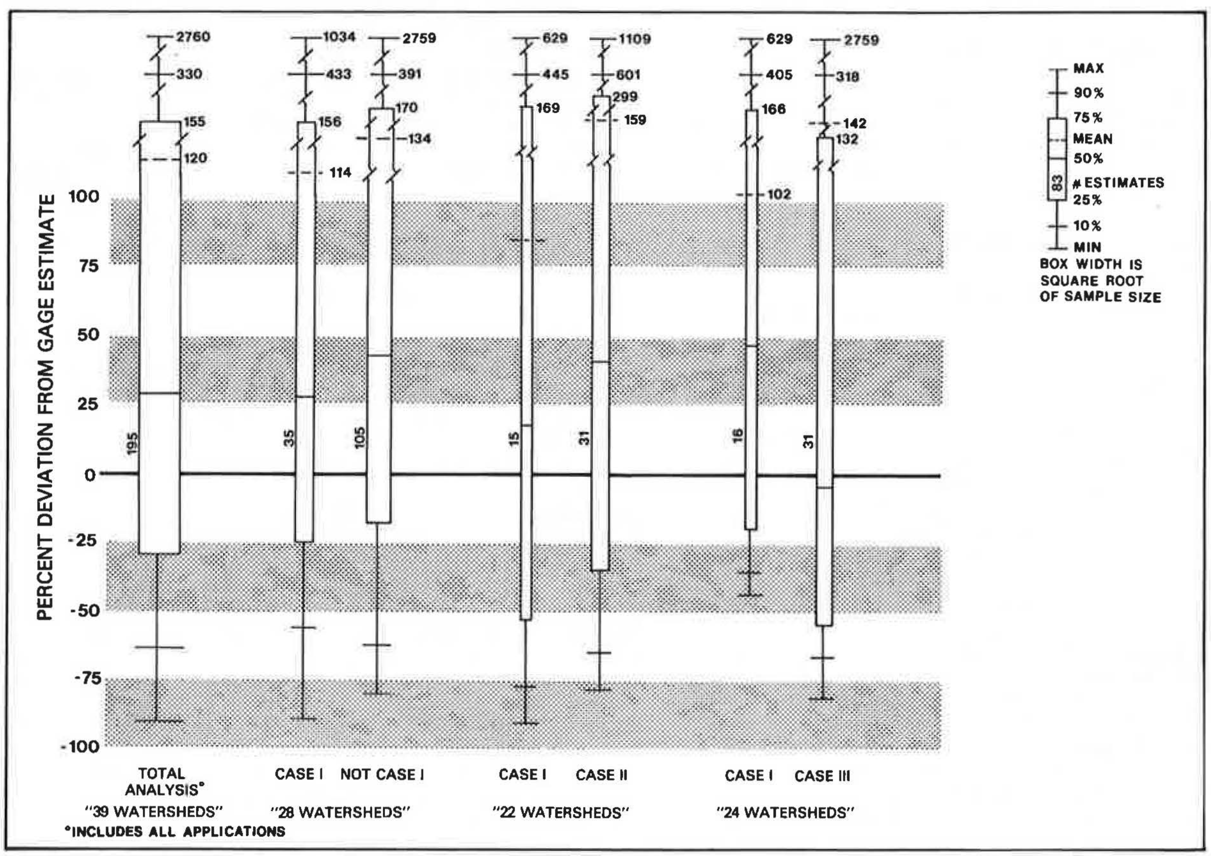
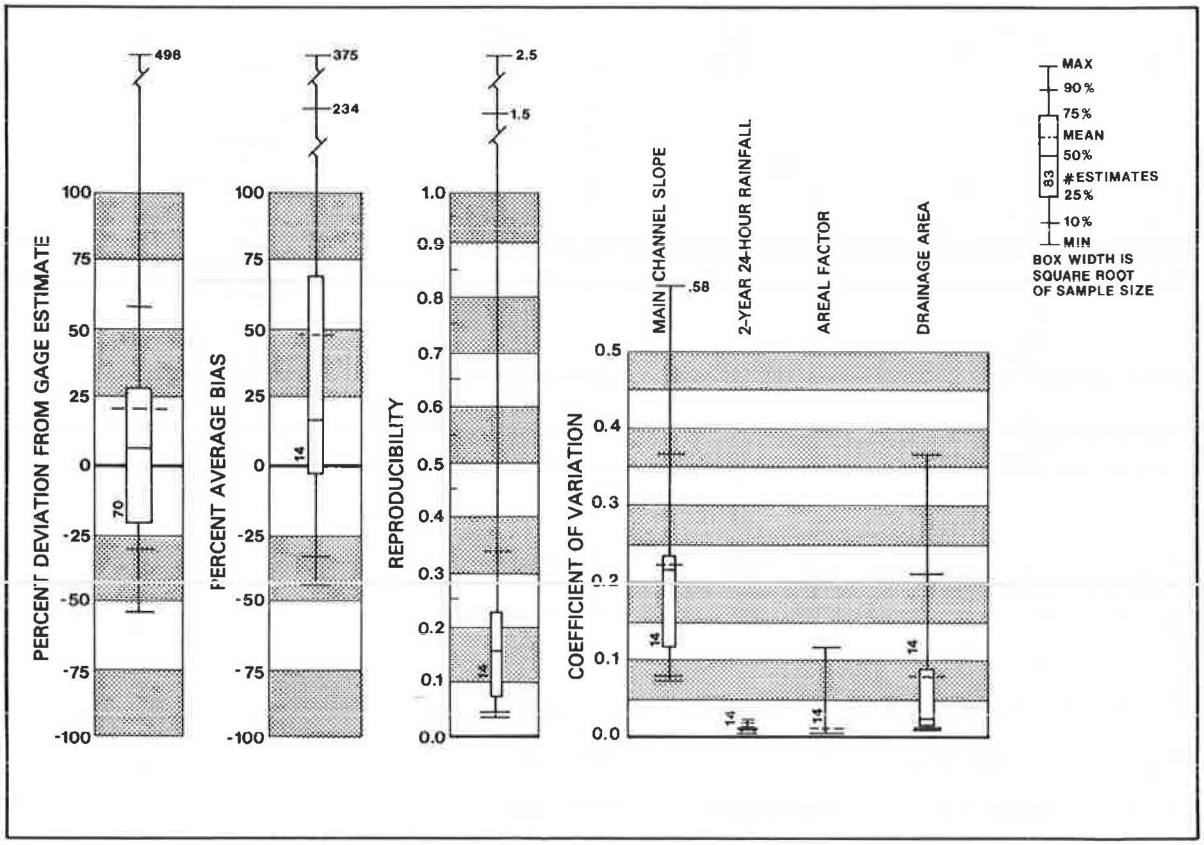


Figure 15. Procedure performance versus parameter variability: USGS State Equations (Illinois).



performance identified in the pilot test are not the result of data quality effects and can be expected to occur in practice in the Northwest and Midwest.

SOURCE OF PERFORMANCE DIFFERENCES

Having concluded that the differences in procedure performance identified by the pilot test are real, the next step is to identify what produced these differences and to determine whether the differences could be expected to occur in other regions of the country. The difference in performance could result from variations in the parameter estimates used to make the flow determinations, the sensitivity of flow determinations to parameter variability, and the basic assumptions inherent in procedure formulation.

Parameter Variability

The variability in the input parameter estimates provided by the testers was examined for all procedures tested except TR-20 (procedure 9) and HEC-1 (procedure 10). The coefficient of variation (standard deviation divided by the mean) based on the five parameter replicates for each watershed was used for parameter comparison. Parameter variability was found to vary generally according to the skill and judgment needed to estimate the parameter. Four parameter groups were identified. These were, in order of increasing variability, (a) those read directly from a map, graph, or table; (b) those measured from a topographic map; (c) those that were a combination of other parameters; and (d) those that required tester knowledge and judgment. There were variations within a group, depending on how explicitly a parameter and its determination were defined. For instance, variability in drainage area determinations, a well-known and well-defined parameter measured from maps, was as low as some parameters read directly from a map, such as rainfall intensity for a given duration.

Adjustment factors were also used in some procedures. Their impact on flow-frequency estimates depended on both their variability and whether the tester used the adjustment. In the pilot test, it was not possible to be sure whether the adjustment factors were used in the solutions, since this information was not specifically requested. Consequently, potential variation in procedure performance introduced by the adjustment factors could not be completely defined. Only the variation in the parameter itself was known.

A more complete discussion of the parameter variability is presented in the Work Group report (1, pp. 122-124 and Appendix 9). For this discussion, the question is whether the differences in procedure performance identified in the pilot test resulted, at least in part, from parameter variability.

Procedure Performance Compared with Parameter Variability

Figures 15-19 combine, in one graph, the box plots of procedure performance--percentage deviation from the gage estimate, bias, and reproducibility--and parameter variability for five procedures. These five procedures were selected because they are frequently used and afford a comparison among different types of procedure formulation. The USGS State Equations included a number of different equations that use a variety of parameters. Illinois was selected for the comparisons of Figure 15 because it provided the largest sample with common parameters.

Figure 16. Procedure performance versus parameter variability: Fletcher procedure.

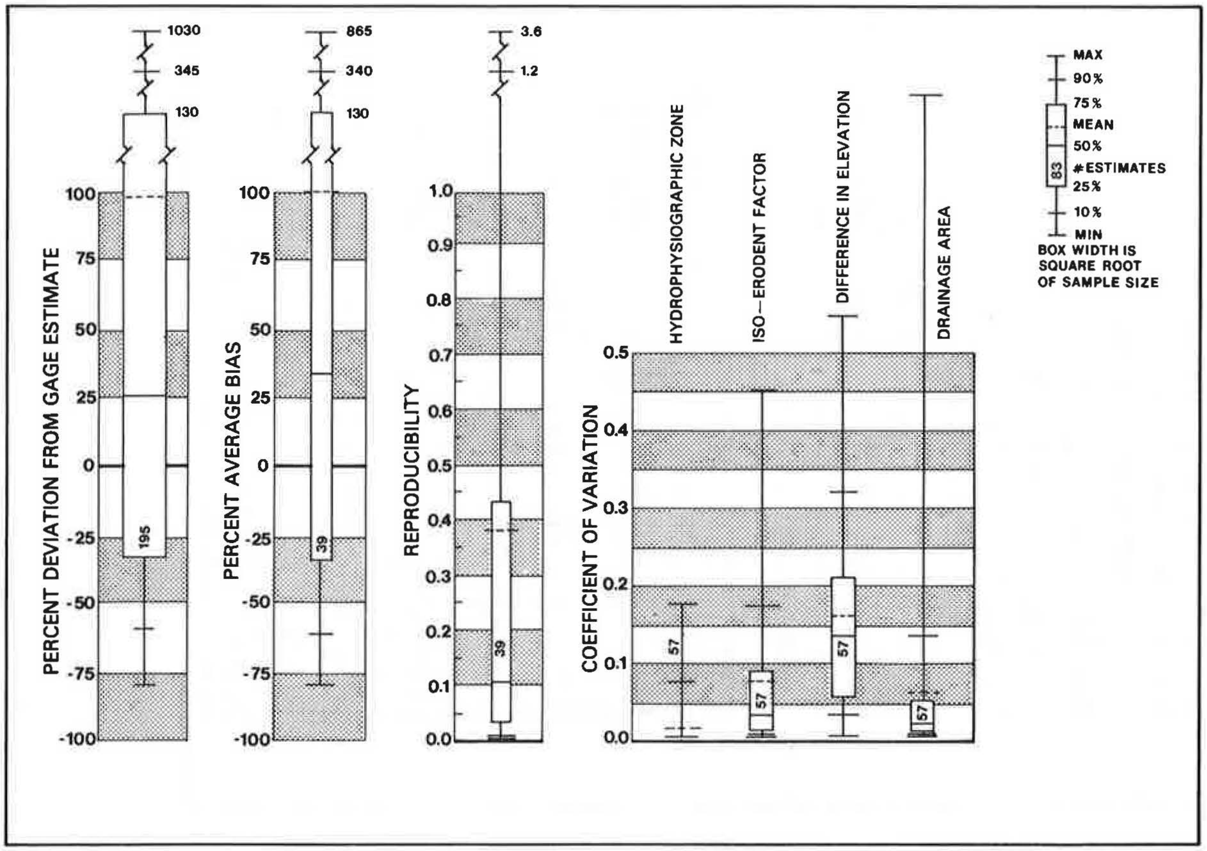


Figure 17. Procedure performance versus parameter variability: rational method.

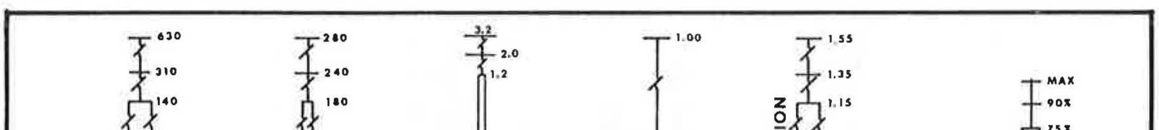


Figure 18. Procedure performance versus parameter variability: TR-55 Charts.

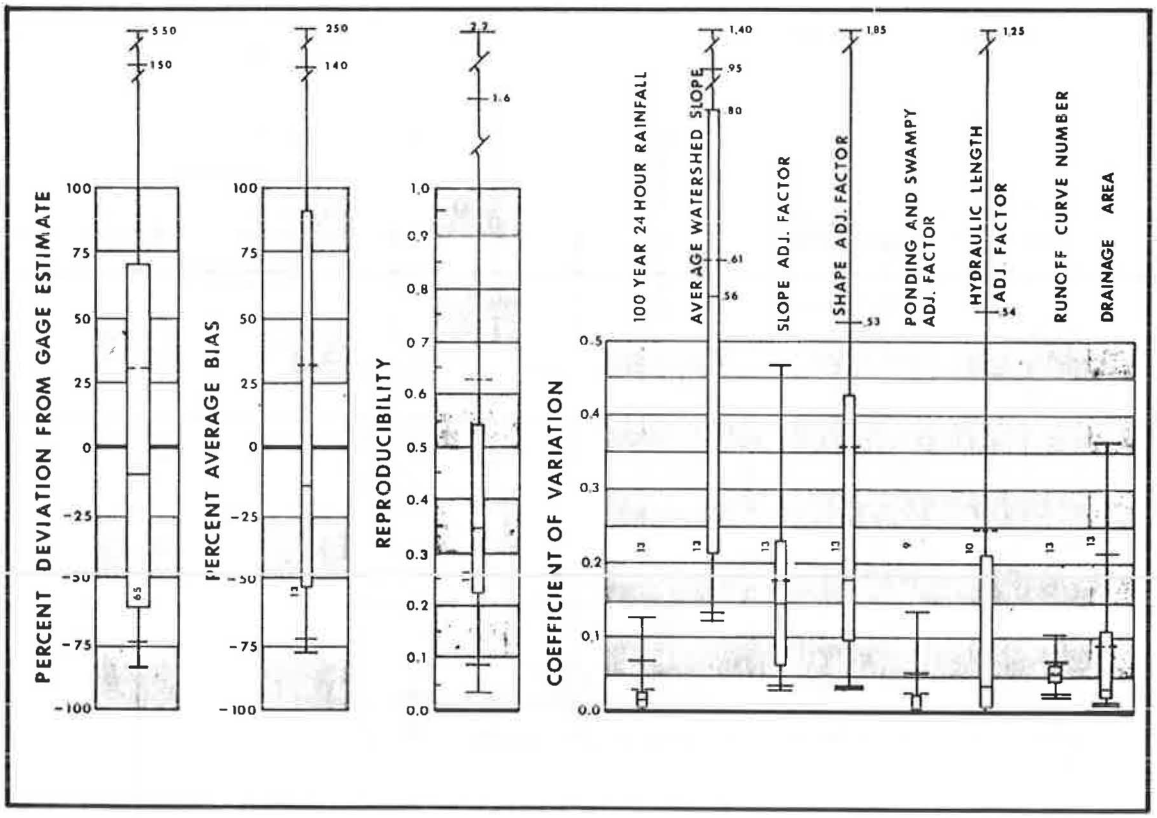
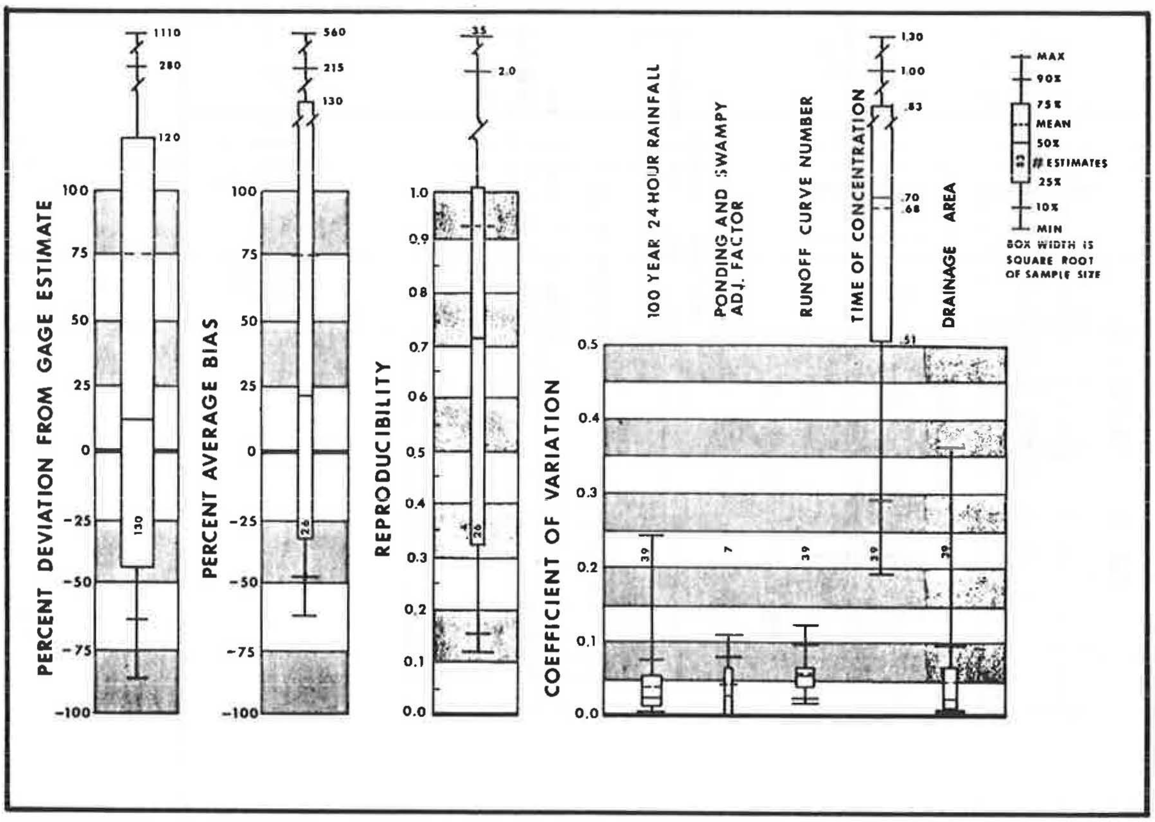


Figure 19. Procedure performance versus parameter variability: TR-55 Graph.



The other procedures, Fletcher (Figure 16), rational (Figure 17), TR-55 Charts (Figure 18), and TR-55 Graph (Figure 19), use the same parameters and adjustment factors for all situations.

It is concluded that, as expected, procedure performance is related to parameter variability. In general, the procedures that performed best used parameters that could be determined by different people relatively consistently. A notable exception is the Fletcher procedure (Figure 16), for which parameter variability is low and reproducibility is good but bias and overall performance are among the poorest of all procedures tested.

Sensitivity Analysis

The impact of parameter variability on flow estimates, and thus procedure performance, depends on the form of the prediction equation. When the equation is a product of parameters, there is a direct relation between the parameter change and flow change, which is a function of the parameter exponent. The percentage change in the flow estimate is approximately equal to the product of the percentage change in the parameter and its exponent, ragardless of the parameter value. In the rational formula all exponents are unity; thus, a 10 percent change in either C, i, or A will produce a 10 percent change in the flow estimate. In the case of the USGS State Equations, where the parameters are raised to a fractional power--for example, 0.7--a 10 percent parameter change will produce about a 7 percent change in the flow estimate.

In some procedures, the peak flow is a complex function of several parameters. For example, in the TR-55 Charts procedure the peak flow, in cubic feet per second per inch of runoff (q_p), is graphically defined as a function of the curve number (CN), drainage area (A), and average watershed slope (S) (20). In such cases, there is not a direct relation between the parameter change and flow change. The percentage change in the flow estimate depends on the parameter value as well as the percentage change in the parameter. With the TR-55 Charts procedure, a 10 percent increase in a curve number of 70 and 80 on a 1000-acre, moderately sloped watershed produces a 19 and 28 percent increase in q_p, respectively.

Tables 4-8 provide the results of our sensitivity analyses, which show the percentage change in the 1-percent-chance flood for a parameter change of ±10 percent. For those parameters not directly related to peak flow, sensitivity was defined in terms of a minimum, average, and maximum. The percentage changes in the flow estimate were calculated for a ±10 percent change across the range of parameter values encountered in the pilot test. The percentage changes in the flow estimate, both positive and negative, were then arrayed and the minimum, average, and maximum were selected.

An indication of the parameter variation encountered in the pilot test is also given in Tables 4-8. The percentage deviation of each parameter value from the watershed mean was determined, and all positive (increase) and negative (decrease) percentage deviations were then arrayed separately. The median values are the median of all positive percentage deviations and the median of all negative percentage deviations for the parameter. The maximum values are the maximum positive percentage deviation and maximum negative percentage deviation for the parameter. The change in the 1-percent-chance flood for a parameter change of ±25 percent is given in parentheses for those parameters whose median deviation in the pilot test was greater than ±10 percent.

A comparison of the sensitivity analyses in Tables 4-8 with the performance comparisons of Figures 15-19 helps in understanding variations in procedure performance. For instance, in the Fletcher procedure (Figure 16) the difference in elevation parameter showed medium variability (1, pp. 327-329). Flow determinations for zone 13, however, are not generally sensitive to this parameter (Table 5), so reproducibility is relatively good. Another procedure in which large parameter variability does not produce a commensurately poor reproducibility is the procedure that uses TR-55 Charts (Figure 18). Average watershed slope (S) showed high variability (1, pp. 324-325), but the flow determinations are not highly sensitive to this parameter (Table 7). Thus, the impact of the average watershed slope variability on flow determination is diminished. On the other hand, small changes in the curve number (CN), a consistently determined parameter (1, pp. 340), produce relatively large changes in flow estimates.

It is concluded that the differences in procedure performance observed in the pilot test result, at least in part, from the interrelation between parameter variability and the formulation of the prediction equation.

Assumptions

The regression-based procedures use statistical estimating procedures to relate flood peaks of selected frequencies directly to watershed and climatic variables. The rain-runoff procedures model the rain-runoff process in order to estimate a flood peak of a selected frequency. The pilot test results, as they relate to these two approaches, are discussed separately.

Regression-Based Procedures

The regression-based procedures are prediction equations calibrated to flood-frequency determinations at gaged locations by using statistical estimating procedures. Consequently, if the statistical estimating procedures are appropriately used and the parameters are well-defined, the resulting equations

Table 4. Sensitivity analysis: USGS State Equations (Illinois).

Parameter	Change in $Q_{0.01}$ (%): 10% Increase in Parameter	Change in $Q_{0.01}$ (%): 10% Decrease in Parameter	Parameter Variation Encountered in Pilot Test[a] (%): Median Decrease	Median Increase	Maximum Decrease	Maximum Increase
Area (A)	7.5	-7.7	-1	2	-44	48
Main channel slope (S)	5.0 (12.7)[b]	-5.3 (-13.8)[b]	-15	8	-89	75
Rainfall intensity index (I)	36.0	-40.0	-0.4	0.5	-3	3
Regional factor (AF)	10.0	-10.0	0	0	-5	21

Note: $Q_{0.01} = 152A^{0.762}S^{0.515}(I-2.5)^{0.386}AF$.

[a]Deviation from watershed mean. [b]Percentage change in 1-percent-chance discharge for ±25 percent change in parameter.

would be expected to produce accurate and reproducible estimates within the range of data used in the calibration.

The difference in accuracy between the USGS State Equations and the Fletcher procedure provides one of the most useful insights afforded by the pilot test. Both are regression-based procedures. The standard error of estimate for the Fletcher proce-

Table 5. Sensitivity analysis: Fletcher procedure.

Parameter	Change in $Q_{0.01}$ (%) 10% Increase in Parameter	Change in $Q_{0.01}$ (%) 10% Decrease in Parameter	Parameter Variation Encountered in Pilot Test[a] (%) Median Decrease	Median Increase	Maximum Decrease	Maximum Increase
Area (A)	6.8	-7.0	-1	2	-43	48
Iso-Erodent factor (R)	9.0	-9.0	-1	1	-15	51
Difference in elevation (DH)	0.1 (0.2)[b]	-0.1 (0.3)[b]	-10	6	-96	47
Storage correction multiplier (Sc)	10.0	-10.0				
Storage (S)	-1.1	1.1	0	0	0	53

Note: $Q_{0.01} = 1.64\ \hat{q}_{0.10}^{1.029}$, $\hat{q}_{0.10} = 6.18\ 115 A^{0.666\ 94} R^{0.874\ 34} DH^{0.010\ 23}$ Sc, and Sc = f(S).

[a]Deviation from watershed mean. [b]Percentage change in 1-percent-chance discharge for ±25 percent change in parameter.

Table 6. Sensitivity analysis: rational method.

Parameter	Change in $Q_{0.01}$ (%) 10% Increase in Parameter	Change in $Q_{0.01}$ (%) 10% Decrease in Parameter	Parameter Variation Encountered in Pilot Test[a] (%) Median Decrease	Median Increase	Maximum Decrease	Maximum Increase
Runoff coefficient (C)	10 (25)[b]	-10 (-25)[b]	-13	13	-53	41
Area (A)	10	-10	-1	1	-38	49
100-year rainfall intensity (i)	10 (25)[b]	-10 (-25)[b]	-22	23	-84	170
Time of concentration (T_c)						
Min	-6 (-15)[b]	7 (23)[b]				
Avg	-7 (-17)[b]	9 (26)[b]	-46	16	-97	277
Max	-8 (-18)[b]	10 (29)[b]				
Length (L)						
Min	-3	5				
Avg	-6	7				
Max	-8	8				
Slope (S)						
Min	2	-2				
Avg	3	-3				
Max	4	-5				

Notes: Q = CiA, i = f(T_c), and $T_c = 0.000\ 13 L^{0.77} S^{-0.385}$.
Min, avg, and max refer to minimum, average, and maximum percentage changes in 1-percent-chance flood across range of parameter values encountered in pilot test.

[a]Deviation from watershed mean. [b]Percentage change in 1-percent-chance discharge for ±25 percent change in parameter.

Table 7. Sensitivity analysis: TR-55 Charts.

Parameter	Change in $Q_{0.01}$ (%) 10% Increase in Parameter	Change in $Q_{0.01}$ (%) 10% Decrease in Parameter	Parameter Variation Encountered in Pilot Test[a] (%) Median Decrease	Median Increase	Maximum Decrease	Maximum Increase
24-h storm rainfall (P_{24H})						
Min	15	-10				
Avg	25	-20	-1	1	-23	7
Max	80	-40				
Curve number (CN)						
Min	20	-25				
Avg	50	-40	-4	4	-18	10
Max	95	-75				
Area (A)						
Min	5	-6				
Avg	6	-7	-3	2	-38	49
Max	8	-8				
Average watershed slope (S)						
Min	2 (4)[b]	-2 (-6)[b]				
Avg	3 (8)[b]	-4 (-10)[b]	-28	25	-100	242
Max	4 (9)[b]	-5 (-12)[b]				

Notes: $q = q_p \cdot Q \cdot SF \cdot$ adjustment factors, $q_p = f(P_{24H}, CN, A, S)$ (Figure D-2), $Q = f(P_{24H}, CN)$ (Table 2-1), and SF = f(S) (Table E-1).
Adjustment factors include ponding and swampy area factor (Table E2, E3, or E4) and watershed shape factor.
Min, avg, and max refer to minimum, average, and maximum percentage changes in 1-percent-chance flood across range of parameters encountered in pilot test.

[a]Deviation from watershed mean. [b]Percentage change in 1-percent-chance discharge for ±25 percent change in parameter.

Table 8. Sensitivity analysis: TR-55 Graph.

Parameter	Change in $Q_{0.01}$ (%)		Parameter Variations Encountered in Pilot Test[a] (%)			
			Median		Maximum	
	10% Increase in Parameter	10% Decrease in Parameter	Decrease	Increase	Decrease	Increase
Area (A)	10	-10	-2	1	-38	48
24-h storm rainfall (P_{24II})						
Min	15	-10				
Avg	25	-20	-2	2	-43	15
Max	80	-40				
Curve number (CN)						
Min	12	-12				
Avg	30	-30	-3	4	-19	22
Max	75	-60				
Time of concentration (t_c)						
Min	-3 (-8)[b]	4 (11)[b]				
Avg	-6 (-14)[b]	7 (19)[b]	-48	51	-90	226
Max	-8 (-16)[b]	8 (14)[b]				
Travel length (L)						
Min	-3	4				
Avg	-6	7				
Max	-8	8				
Average flow velocity (V_{avg})						
Min	4	-4				
Avg	6	-7				
Max	7	-8				
Watercourse slope (S)						
Min	2	-3				
Avg	3	-4				
Max	4	-5				

Notes: $q = q_p \cdot A \cdot Q$; $q_p = f(P_{24II}, CN, t_c)$ (Figure 5-2); $t_c = L/V_{avg}$, where $V_{avg} = f(S)$ (Figure 3-1); and $Q = f(P_{24II}, CN)$ (Table 2-1).
Min, avg, and max refer to minimum, average, and maximum percentage changes in 1-percent-chance flood across range of parameter values encountered in pilot test.

[a]Deviation from watershed mean. [b]Percentage change in 1-percent-chance discharge for ±25 percent change in parameter.

Table 9. Formulations of USGS State Equations and Fletcher procedure.

Item	USGS State Equations	Fletcher Procedure
Flood-frequency analysis	Log-Pearson Type III	Four-parameter polynomial
Annual peak flow record length (years)	≥6	≥20
Last data year	1971-1975	1968
Stream condition	Natural-flow streams	Natural-flow streams
Equation formulation	Regression analysis	Regression analysis
Hydrologic zones	1-12/state	24 nationwide and Puerto Rico
Number of gaging stations	149-450/state[a]	500 nationwide and Puerto Rico
Avg gage density per zone	37-152	20
Standard error of estimate (%)	9-150	15-42

[a]Includes synthetic estimates in one state developed from rainfall-runoff model.

dure is equal to or less than that for the USGS State Equations in most locations. The major differences are (a) the gage density used in the formulation, (b) the delineation of hydrologic zones, and (c) the method of defining flood probabilities from the gage records.

Table 9 provides a comparison of the formulations of the USGS State Equations and the Fletcher procedure (1, Appendix 1). It is concluded that differences in gage density and the delineation of hydrologic zones are the most important factors in the difference in procedure performance. The Fletcher procedure divides the nation into 24 large zones with about 20 gages/zone, whereas the USGS State Equations have several zones in most states and at least 37 gages/zone.

The flood-frequency estimate used to calibrate the Fletcher procedure was different from both that used to calibrate the USGS State Equations or Index Flood methods and that used by the Work Group to evaluate procedure performance. The Work Group analyses showed that conclusions about procedure performance were not altered by definition of the gage estimate over a fairly wide range of values (1, p. 111). This, however, is not the same as evaluating the effects on procedure performance of varying the gage estimate used to calibrate the procedure. Fletcher used a curve-fitting procedure that should not vary markedly from the log-Pearson procedure used by the USGS at low frequencies. The Fletcher procedure performed the same at all frequencies. Therefore, it is concluded that the fitting procedure had little effect on the performance of the Fletcher procedure.

Regression-based procedures perform relatively well, given an adequate data base. The pilot test data illustrate this point, but it is not possible to relate prediction accuracy to size of data base. Gage density criteria would depend on the range of conditions over which the equations would apply, the homogeneity of the zone, and the quality of the data used to develop the equations. The comparison between the standard error of estimate of the Fletcher procedure and USGS State Equations and their performance in the pilot test highlights the importance of understanding the difference between standard error of estimate and standard error of prediction.

Rain-Runoff Procedures

The assumptions inherent in the rain-runoff procedures tested are that (a) the translation of rainfall to precipitation excess is correctly modeled, (b) the precipitation excess is correctly translated into a discharge hydrograph, and (c) the computed peak discharge frequency is equal to the rain or storm frequency used in the computation.

It would be expected that flood-frequency estimates made by using the rain-runoff approach would increase in accuracy with increased accuracy of the watershed and climate modeling. The pilot test did not address this issue. The most sophisticated rain-runoff procedures are those in categories 7 and 8, which were not tested. The watershed models of procedures in these categories are normally calibrated to site data. The assumption of rain or storm frequency equaling flood frequency is removed by calculating a flood-frequency curve at the site, using observed or simulated storm rainfall to compute a record of maximum annual flood peaks. All of the rain-runoff procedures tested were empirical equations or single-storm-event procedures that used broadly based regional relations to define the watershed model and rain-runoff process. The effects of improvement in watershed modeling could have been compared if the watershed models in the TR-20 and HEC-1 procedures had been calibrated to site data.

The effect of the increasing sophistication of watershed modeling is partly addressed by comparing results from (a) the rational formula; (b) the TR-55 Charts, TR-55 Graph, and Reich procedures; and (c) the TR-20 and HEC-1 procedures. In effect, model sophistication increases from the rational to the more complex modeling procedures of TR-20 and HEC-1. Performance, however, did not change markedly, as shown by Figures 1-3. Thus, in our judgment, the use of more complex models to simply estimate peak flow frequencies is not warranted unless the accuracy achieved exceeds that of the other procedures tested when the watershed models are calibrated to site or regional data.

The question of whether a computed flow frequency is equal to the rain or storm frequency used in the computation is not new. It is an interesting issue that has attracted the attention of hydrologists for some time. One recent paper on this subject (23) concluded that a flood peak of a given frequency can be predicted from rainfall of the same frequency only if the correct antecedent moisture is used in calculating the runoff. One simple technique that can be used to evaluate the assumption is to compute a unit hydrograph based on gage records and use it to translate runoff from a storm of a given frequency into a flood hydrograph. The flood peaks for storms of selected frequency can be compared with the flood peak frequency curve determined from the gage record. This approach, in effect, separates the watershed modeling and storm assumptions of the category 6 procedures. We have tried this approach on a limited number of watersheds and found that the frequency curves computed from the storm rainfall and the gage record did not match. It is therefore concluded that the assumption that the computed peak discharge frequency equaled the rain or storm frequency used in the computation probably accounted, in part, for the poor performance of the rain-runoff procedures.

All rain-runoff procedures tested depended on broadly based regional relations to define the watershed response function. None had been developed by using all available data in a well-defined hydrologic region and the statistical techniques that are used in the regression-based procedures. It is concluded that parameter variability and sensitivity were so great that they overshadowed any potential accuracy of the rain-runoff approach to provide reliable estimates of peak flow frequencies. It would be interesting to see whether a rain-runoff estimating procedure based on a well-calibrated watershed response function that in turn was related to flood frequencies by using an appropriate climatic variable would perform better than the present regression-based procedures.

The rain-runoff procedures that were tested did not perform well, as shown by Figures 1-3. It is not possible to determine from the pilot test results how much was due to parameter variability and sensitivity and how much was due to the assumptions inherent in the procedures. The accuracy of the more sophisticated models of categories 7 and 8 and the TR-20 and HEC-1 procedures when calibrated to site data was not determined. All of these more sophisticated procedures (in most applications) require some gage data for calibration and considerable effort in application, as shown by Figure 4. Even if these procedures proved more accurate, their use would be restricted to those situations where the extra effort was warranted, such as where greater accuracy was desirable or modeling was needed to evaluate watershed changes or to design structures.

REGIONAL DIFFERENCES

An important remaining question is whether the differences in procedure performance identified by the pilot test reflect inherent differences that can be anticipated in the Tennessee Valley region or elsewhere in the nation. The pilot test showed only a slight tendency toward regional differences in two widely different regions, the Northwest and Midwest (1, Figures VIII-16 and VIII-17, pp. 106 and 107). The conclusions about procedure performance did not vary between the two regions (1, p. 105). This suggests that the conclusions are applicable to other regions. In addition, all of the procedures are calibrated to a particular region. The regression procedures are developed for specific hydrologic zones. The rain-runoff procedures use regional rainfall or storm data and, in the case of the TR-55 procedures, watershed soils and cover data.

It is concluded that the differences in performance result from procedure formulation. Thus, these same differences can be expected wherever the gage density for the regression-based procedure exceeds that used in the Fletcher formulation and approaches that used in the USGS State Equations.

The senior author has been making peak flow frequency estimates in the Tennessee Valley region for more than 25 years. During this time, a variety of procedures have been tried, including those in the pilot test applicable to the region except TR-20 and HEC-1. Although extensive formal comparisons were not made, the pilot test results are compatible with the author's experience and the many informal comparisons that have been made.

SUMMARY AND CONCLUSIONS

The objective of this study was to determine what are likely to be the most accurate and consistent procedures for determining peak flood-flow frequencies for ungaged watersheds in the Tennessee Valley region. This study was based on information developed during a pilot test conducted by the Work Group of the Hydrology Committee of the Water Resources Council. The pilot test is currently the most comprehensive and objective examination available of the performance of commonly used procedures for estimating peak flow frequencies at ungaged locations. Test results showed that there were significant differences in performance when procedures were evaluated by using the criteria of accuracy, reproducibility, and practicality. Although the pilot test evaluation was limited to the Midwest and

Northwest, it is concluded that the observed differences in performance result from fundamental differences in procedure formulation that can be expected to occur in the Tennessee Valley and in other regions as well.

The USGS State Equations and Index Flood methods were found to be the most accurate and reproducible procedures evaluated. These are regression-based procedures in which prediction equations are calibrated to flood-frequency determinations at gaged locations by using statistical estimating procedures. The most obvious reason for this superior performance is in the definition of the parameters and the formulation of the prediction equation. The procedures that performed best (a) used parameters that were well-defined and could be consistently determined, (b) were formulated so that the flood-frequency estimates were not sensitive to parameter variations, and (c) were calibrated to a large number of gage records in a relatively small, well-defined hydrologic region.

Based on the conclusions of this paper, the following criteria are recommended for evaluating an existing procedure or developing an improved procedure for use in a particular region:

1. The procedures (prediction equations) should be developed by using statistical regression and have low standard errors of estimate. They should be calibrated to flood-frequency estimates at gaged locations made by using procedures recommended by the Water Resources Council. This process should be followed for any category of procedure--regression or rain-runoff.
2. Although it is not possible to define an acceptable gage density from the available data, it is clear that equations should be developed for well-defined hydrologic zones with a gage density on the same order of magnitude as the USGS State Equations. Specific gage density criteria would depend on the range of conditions over which the equations would apply, the homogeneity within the zone, and the quality of the data used to develop the equations.
3. The parameters used in the prediction equations need to be uniquely defined and consistently measurable. Factors that require user judgments that cannot be consistently applied by a variety of users should be avoided.
4. Unless complex watershed modeling can be shown to improve accuracy, an application time of about 3 h should be sufficient when the objective is simply to estimate the peak discharge of a given frequency at a particular location.

The following additional comments and recommendations are offered:

1. Because of the effort involved, the use of the more complex single-storm-event watershed modeling procedures needs to be justified by special study requirements.
2. The accuracy of the complex watershed modeling procedures when the watershed model has been calibrated to site data needs to be evaluated.
3. Procedures in categories 7 and 8 should be tested as recommended by the Work Group. These are the most sophisticated of the rain-runoff procedures, and their potential accuracy needs to be defined.
4. It would be interesting to evaluate rain-runoff procedures that have been developed by a two-step process involving the calibration of watershed models to regional gage data, which, in turn, are calibrated to observed frequency curves by using climatic parameters.
5. Improved procedures are needed. Even with the best-performing procedures, only about 50 percent of the estimates were within ±25 percent of the gage estimate.
6. The full test recommended by the Work Group (1, Chapter IX) is needed to provide an authoritative basis for procedure selection for a national guide. The benefits of such testing and the resulting guide would exceed the testing costs by a considerable margin.

None of these conclusions, comments, or recommendations appear to be particularly startling. It is believed that a thorough literature search would identify papers supporting most of them. What is different is that, for the first time, there has been relatively extensive, although limited, testing of procedure performance under conditions approaching those encountered in practice. This permits a more objective comparison and evaluation of procedure performance.

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Abridgment

Magnitude and Frequency of Urban Floods in the United States

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A nationwide study of flood magnitude and frequency in urban areas was made for the purpose of reviewing available literature, compiling an urban flood data base, and developing methods of estimating urban flood-flow characteristics in ungaged areas. The literature review contains synopses of 128 recent publications related to urban flood flow. A data base of 269 gaged basins in 56 cities and 31 states, including Hawaii, contains a wide variety of topographic and climatic characteristics, land use variables, indices of urbanization, and flood-frequency estimates. Regression equations were developed that provided unbiased estimates of urban flood discharges for ungaged sites for recurrence intervals of 2, 5, 10, 25, 50, 100, and 500 years. Of primary importance in these equations is an independent estimate of the equivalent rural discharge for the ungaged basin. The equations essentially adjust the equivalent rural discharge to an urban condition. The primary adjustment factor, or index of urbanization, is the basin development factor. This factor is a measure of the extent of development of the drainage system in the basin and includes evaluations of storm drains (sewers), channel improvements, and curb-and-gutter streets. It offers a simple and effective way of accounting for drainage development and runoff response in urban areas. Other parameters in the equations include size of drainage area, channel slope, rainfall intensity, lake and reservoir storage, impervious area, and basin lag time.

With urban growth and development, there is an ever-increasing need for flood information and estimating techniques for use in areas where little or no data exist. In 1978, the Federal Highway Administration (FHWA) provided funds to the U.S. Geological Survey to make a nationwide study of urban flood frequency. The purposes of the study are to (a) review literature on urban flood studies; (b) compile a nationwide data base of flood-frequency characteristics, topographic and climatic characteristics, land use variables, and indices of urbanization for as many urbanized watersheds as possible; and (c) analyze the data for the purpose of defining estimating techniques that can be used in ungaged urban areas. This paper briefly describes the results of the study. A more detailed description of the study is provided elsewhere (1).

LITERATURE REVIEW

The first phase of the study was to search the available literature and compile a bibliography of reports that describe urban runoff, primarily those reports that relate to the magnitude and frequency of peak discharge. Shortly after the start of this review, it was learned that a similar literature review was being done by the Agricultural Research

Figure 1. Metropolitan areas included in study of nationwide urban flood frequency.

Service (ARS) of the U.S. Department of Agriculture; thereafter, the Geological Survey and ARS worked together and combined their reviews into a joint publication. The published report (2) contains synopses of 128 recent publications for urban flood-flow frequency.

DATA BASE

The second phase of the study was the compilation of a comprehensive data base for drainage basins affected by urbanization. Information obtained from the district offices of the Geological Survey revealed that almost 600 urbanized watersheds with at least three years of runoff data were available nationwide. Sites were selected for this study according to the following criteria:

1. The watersheds had to have at least 15 percent of their drainage area covered with commercial, industrial, or residential development.
2. Reliable flood-frequency data had to be available for the watershed. These could be based on actual peak flow records, if records were available for 10 or more years, or on synthesized data, if such data were based on a rainfall-runoff model specifically calibrated by using actual flood and rainfall data for that basin.
3. The period of actual flood data, or the period of calibration for synthesized data, was a period of relatively constant urbanization. This was the most difficult criterion to meet, and in some cases only part of a long record could be used. As a general guideline, "relatively constant urbanization" was defined as a change in development of less than 50 percent during the period of record.

An appraisal of all available watersheds resulted in a final list of 269 watersheds that met the selection criteria. These watersheds represent a broad spectrum of hydrologic conditions and metropolitan areas, from the East Coast to the West Coast and Hawaii. Watersheds are included for 31 states and 56 cities or metropolitan areas. Figure 1 shows the geographic distribution of the metropolitan areas.

The data compiled for each urban watershed include a comprehensive list of topographic and climatic variables, land use variables, indices of urbanization, and flood-frequency estimates. The data base is not provided in this paper due to space limitations, but a major part of the data base is presented in the report by Sauer, Thomas, Stricker, and Wilson (1).

Several parameters were evaluated for each basin in an attempt to measure the degree to which a basin has been urbanized. Among the parameters evaluated are the percentage of the basin occupied by impervious surfaces, population and population density determined from U.S. Bureau of the Census data for 1970, and the basin response time or lag time.

The most significant index of urbanization was a basin development factor (BDF) that provides a measure of the efficiency of the drainage system. This parameter, which proved to be highly significant in the regression equations, can be easily determined from drainage maps and field inspections of the drainage basin. The basin is first subdivided into thirds and within each third four aspects of the drainage system are evaluated and assigned a code. The four aspects are (a) channel improvements, (b) channel linings, (c) storm drains or sewers, and (d) curb-and-gutter streets. The code is assigned one if the aspect is present in at least 50 percent of that third of the basin and zero if it is present in less than 50 percent. The maximum value of BDF for a fully developed drainage system would be 12. Guidelines for determining the various drainage system codes are described more fully by Sauer, Thomas, Stricker, and Wilson (1).

Two primary sets of flood-frequency estimates, in cubic feet per second, for selected recurrence intervals, were defined for each station. One set represents an estimated flood-frequency relation for

Table 1. Regression equations.

Type	Equation	R^2	Standard Error of Regression	
			Log Units	Avg (%)
Seven parameter	$UQ2 = 2.35A^{0.41}SL^{0.17}(RI2 + 3)^{2.04}(ST + 8)^{-0.65}(13 - BDF)^{-0.32}IA^{0.15}RQ2^{0.47}$	0.93	0.1630	±38
	$UQ5 = 2.70A^{0.35}SL^{0.16}(RI2 + 3)^{1.86}(ST + 8)^{-0.59}(13 - BDF)^{-0.31}IA^{0.11}RQ5^{0.54}$	0.93	0.1584	±37
	$UQ10 = 2.99A^{0.32}SL^{0.15}(RI2 + 3)^{1.75}(ST + 8)^{-0.57}(13 - BDF)^{-0.30}IA^{0.09}RQ10^{0.58}$	0.93	0.1618	±38
	$UQ25 = 2.78A^{0.31}SL^{0.15}(RI2 + 3)^{1.76}(ST + 8)^{-0.55}(13 - BDF)^{-0.29}IA^{0.07}RQ25^{0.60}$	0.93	0.1705	±40
	$UQ50 = 2.67A^{0.29}SL^{0.15}(RI2 + 3)^{1.74}(ST + 8)^{-0.53}(13 - BDF)^{-0.28}IA^{0.06}RQ50^{0.62}$	0.92	0.1774	±42
	$UQ100 = 2.50A^{0.29}SL^{0.15}(RI2 + 3)^{1.76}(ST + 8)^{-0.52}(13 - BDF)^{-0.28}IA^{0.06}RQ100^{0.63}$	0.92	0.1860	±44
	$UQ500 = 2.27A^{0.29}SL^{0.16}(RI2 + 3)^{1.86}(ST + 8)^{-0.54}(13 - BDF)^{-0.27}IA^{0.05}RQ500^{0.63}$	0.90	0.2071	±49
Three parameter	$UQ2 = 13.2A^{0.21}(13 - BDF)^{-0.43}RQ2^{0.73}$	0.91	0.1797	±43
	$UQ5 = 10.6A^{0.17}(13 - BDF)^{-0.39}RQ5^{0.78}$	0.92	0.1705	±40
	$UQ10 = 9.51A^{0.16}(13 - BDF)^{-0.36}RQ10^{0.79}$	0.92	0.1720	±41
	$UQ25 = 8.68A^{0.15}(13 - BDF)^{-0.34}RQ25^{0.80}$	0.92	0.1802	±43
	$UQ50 = 8.04A^{0.15}(13 - BDF)^{-0.32}RQ50^{0.81}$	0.91	0.1865	±44
	$UQ100 = 7.70A^{0.15}(13 - BDF)^{-0.32}RQ100^{0.82}$	0.91	0.1949	±46
	$UQ500 = 7.47A^{0.16}(13 - BDF)^{-0.30}RQ500^{0.82}$	0.89	0.2170	±52

the urbanized basin during a period of constant urbanization, and the other represents the estimated relation for an equivalent rural basin. Flood-frequency data for equivalent rural conditions at each study basin were estimated from the applicable Geological Survey statewide flood-frequency reports. For each station, peak discharge was estimated for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence intervals by using log-Pearson Type III procedures as recommended by the Water Resources Council (3).

ESTIMATING PROCEDURES FOR UNGAGED URBAN SITES

The third phase of the study was to relate urban flood magnitude and frequency to watershed characteristics so that flood magnitude and frequency could be estimated for ungaged watersheds. Many attempts were made to derive a practical, easy-to-use method, most of which involved multiple linear regression of several dependent and independent variables. This paper describes the more significant results. The three sets of estimating equations are referred to as the seven-parameter, three-parameter, and seven-parameter alternative estimating equations.

Seven-Parameter Estimating Equations

Peak discharges for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year urban floods were related to seven independent variables by multiple linear regression techniques as shown by the equations given in Table 1. The significant variables account for the effect of basin size (A), channel slope (SL), basin rainfall (RI2), basin storage (ST), man-made changes to the drainage system (BDF), and impervious surfaces (IA). Regional runoff variations are accounted for in the equations through the use of the equivalent rural peak discharge (RQ). With regard to suitability and accuracy, these equations provide a good method for estimating the effects of urbanization on magnitude and frequency of peak discharge. From the 269 sites available for analysis, 55 were omitted because of known detention storage, 10 were omitted because detention storage effects were uncertain, and 5 were omitted because of missing data. Therefore, the equations are derived by using 199 sites.

The most significant variable in each of the equations is the equivalent rural discharge (RQ), which provides the key for explaining geographic variations of runoff experienced in various parts of the United States. Consequently, the equations can be used in urban areas throughout the United States with no expected geographic bias. As noted earlier, the equivalent rural discharge is estimated by using the applicable Geological Survey statewide flood-frequency report.

The second most significant variable is the basin development factor (BDF). This variable is somewhat subjective but seems very effective in explaining variations of urban peak discharges. BDF is used on a reverse scale (13 - BDF) in the equations because it was found that this greatly improved the linearity of the equation and reduced the standard error.

Contributing drainage area (A) was the third most significant variable in all equations. The high degree of significance of A implies that a given amount of urbanization will affect small basins differently than large basins. The other variables, slope (SL), rainfall intensity (RI2), storage (ST), and impervious area (IA), were all much less significant than RQ, BDF, and A, but overall offered enough improvement to warrant inclusion in the equations. SL is limited to an upper value of 70 ft/mile. For channels with slopes greater than 70 ft/mile, a value of 70 was used. This limitation was found to be effective in reducing the standard error of regression and is logical in that very steep slopes may not cause significant increases in peak discharge.

Three-Parameter Estimating Equations

Dropping the least significant variables from the seven-parameter equations increases the standard error of regression but also greatly reduces the amount of data and effort required for application. The three-parameter equations given in Table 1, which include only the independent variables RQ, BDF, and A, can be used to estimate urban peak discharges for ungaged sites. These equations were based on the same 199 sites used to derive the seven-parameter equations.

Seven-Parameter Alternative Estimating Equations

A third set of estimating equations, the seven-parameter alternative equations, was developed by including lag time (LT) as an independent variable. The alternative equations differ from the seven-parameter equations discussed earlier in that LT replaces storage (ST) as an independent variable. The standard errors of regression for these equations are less than for the seven-parameter equations, but this resulted because only 164 sites were used for calibration. The reduction in standard error was shown to be a function of the number of stations by using the same independent variable as in the seven-parameter equations in computing regression equations for the 164 stations. The shorter-record crest-stage stations with larger time-sampling errors were deleted from the 164 station equations, which probably contributed to the lower standard error.

The seven-parameter alternative equations are more difficult to apply than the equations in Table 1 because the variable LT is not easily determined and requires access to both rainfall and runoff hydrograph data applicable to the basin. The alternative equations have not been reproduced for this paper but are available in the report by Sauer, Thomas, Stricker, and Wilson (1).

Limitations of Significant Variables

The effective or usable range of basin and climatic variables to be used in the estimating equations described in this paper is given below:

Variable	Min	Max
A ($miles^2$)	0.2	100
SL (ft/mile)	3.0	70
RI2 (in)	0.2	2.8
ST (%)	0	11
BDF	0	12
IA (%)	3.0	50
LT (h)	0.2	45

If values outside these ranges are used, the standard error may be considerably higher than for sites where all variables are within the specified range. The maximum value of SL for use in the equations is 70 ft/mile, although numerous watersheds used in this study had SL values up to 500 ft/mile.

Effects of Detention Storage

If temporary in-channel storage, or detention storage, is significant, it will tend to reduce peak discharges. The estimating equations defined by this study were calibrated without including those stations known to be affected by temporary detention storage and therefore represent conditions relatively free of the effects of detention storage. The recommended way to determine the effect of detention storage in a specific watershed is through the use of reservoir and channel routing techniques, which is beyond the scope of this study.

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A copy of the complete data base for the study can be obtained by writing to Chief, Data Management Section, U.S. Geological Survey, Mail Stop 437, National Center, Reston, Virginia 22092.

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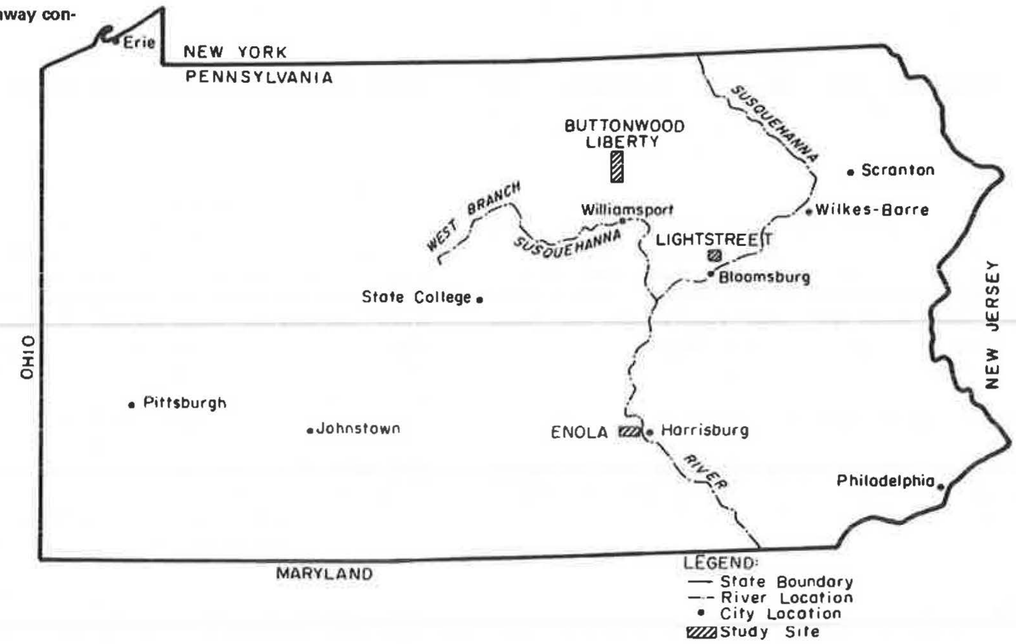
Comparison of Prediction Methods for Soil Erosion from Highway Construction Sites

ARTHUR C. MILLER, WILLIAM J. VEON, AND RONALD A. CHADDERTON

The disturbance of land by construction is almost invariably accompanied by sudden, sometimes drastic increases in the potential for soil erosion. The amount of sediment eroded and delivered to a stream should be minimized within practical economic limits. Prediction methods for soil erosion from highway construction sites are compared. All but one of the methods, a new rational method, are currently being used to predict soil erosion. The accuracy of the methods varied from 55 to 85 percent based on a mean error analysis. The best predictive method determined from the data analyzed was a new rational method.

The disturbance of land by construction is almost always accompanied by sudden, sometimes drastic increases in soil erosion. Erosion controls should be selected through a process of comparing the costs of controls at each site with the environmental, economic, and other benefits or forgone damages to be obtained in the local region. The first step in such a process, of course, should be the prediction of quantities of material to be eroded.

Figure 1. Location of highway construction sites.



Engineers must be able to predict the potential amount of sediment eroded from construction sites before they can intelligently design and implement erosion control measures. The intent of this paper is to critique and evaluate some of the sediment erosion prediction methods currently in use.

PREDICTING SOIL EROSION

There are four levels of sophistication that can be used to determine sediment yield:

1. Level 1 relations are prediction equations developed from regression analysis with average parametric values for input variables. The rational formula for determining runoff is such an equation, and the universal soil loss equation (USLE) for predicting sediment yield, developed by Wischmeier and Smith (1), is another.
2. Level 2 relations are similar to those in level 1, but the methods combine potential erosion with a routing procedure (delivery ratio) to predict the amount of sediment entering the stream system. The delivery ratios are typically developed by using regression analysis with measured data. An example of level 2 would be the Younkin equation presented in a later section of this paper.
3. Level 3 relations incorporate the unit hydrograph theory in hydrology and are appropriately called unit-sediment-graph (USG) methods. Many of the assumptions in the derivation of the unit hydrograph apply to the USG. The advantage of the USG is that it can be used in water-quality modeling where concentration of sediment is a significant indicator of pollution.
4. Level 4 uses a combination of equations to solve the dynamic soil erosion process. Many causal factors affect soil erosion. A particle is first detached from the surrounding soil by the impact of the rainfall energy or by the erosive properties of the overland flow. Once the soil particle has been detached, it is transported over the construction site by rainfall-runoff. The sediment is finally delivered to the stream system, where it may or may not pose an ecological problem. The methods that constitute level 4 all attempt to model analytically each of the important steps in the erosion process. Methods developed by Meyer and Wischmeier (2) and Simons and Li (3) are good examples of this modeling.

This paper concentrates only on the level 2 relations. The relations of level 1 are too simple to predict soil loss accurately; there are currently few or no data available to calibrate the prediction equations of levels 3 and 4 adequately, even though with combined research these procedures will be more usable in the near future and will be inherently superior to the level 2 relations described here for reasons in addition to their dimensional consistency.

ANALYSIS OF EXISTING PREDICTIVE METHODS

This analysis involves six existing relations for estimating construction sediment yield and one relation developed specifically for this study. The various equation terms and methods of determination are defined only once as each is first introduced in the analysis. Although the existing equation factors and coefficients were supposedly fixed by their original authors, modifications were made in some cases to achieve better results. When modifications were necessary, 80 percent of the total data was used in the calibration process, which left 20 percent of the data for testing and calculating the resulting relation.

SITE DESCRIPTIONS

The U.S. Geological Survey, in cooperation with the Pennsylvania Department of Transportation (DOT), has collected rainfall, stream-flow, suspended sediment, and turbidity data at several Pennsylvania sites downstream from highway construction (see Figure 1). One site, located near Enola in Cumberland County, consisted of five small adjacent drainage basins. Another site, located near Lightstreet in Columbia County, consisted of two subareas. The third site, located in the Buttonwood-Liberty area of Lycoming and Tioga Counties, consisted of four subbasins. All drainage areas were gaged for a minimum of 2.5 continuous years. Figure 1 shows the locations of the sites and their proximities to the larger urban centers of the state (4).

Table 1. Comparison of error parameters and equation significance terms for seven equations considered.

Equation		Error				Significance	
No.	Name	Avg Error (%)	95 Percent-of-Data Avg Error (%)	Median Error (%)	Portion of Estimate in Error by at Least 100 Percent (%)	R^2	F/F*
1a	Younkin	510	219	83	33	0.45	51.6/3.00
2a	Scott Run	499	166	91	35	-	-
3a	USLE 1	638	132	73	29	0.61	220/3.84
4a	USLE 2	500	193	76	28	0.002	0.27/3.92
5b	USLE 3	282	98	68	25	0.37	33.2/3.07
6	USLE 4	282	98	68	25	0.37	33.2/3.07
7	Rational model	119	74	55	22	0.84	156/2.45

DATA DESCRIPTION

The U.S. Geological Survey provided basic precipitation, stream-flow, sediment, and turbidity data for the various study areas. The precipitation information was obtained in the form of cumulative rainfall amount versus time plots as recorded by graphic analog rain gages. The stream-flow data were obtained in the form of water stage versus time plots as recorded by continuous strip-chart recorders. Pertinent stream-flow rating curves were also available so that the water stage values could be transformed into discharge values. Finally, plots of suspended sediment concentration versus time were obtained. The Geological Survey used automatic pendulum samplers to collect the sediment samples during storms. Samples were taken at predetermined time increments, usually every 15 min, and later they were analyzed to determine the sediment concentrations. Between storms, suspended sediment samples were collected intermittently by hand with U.S. DH-48 samplers. In addition to the hydrologic data, detailed construction data were incorporated into the data base.

DATA MANAGEMENT

Hydrologic data for about 25 years for the seven study areas (for both control and construction periods) were processed by the personnel of the Pennsylvania State University Hydrology Laboratory. The initial step in collapsing the data to usable form was isolating the "good" storm events. (A storm was defined, for this study, as the occurrence of at least 0.10 in of rainfall with a separation time of at least 5 h from any other rainfall event.) The events were then ranked according to the quality of the respective suspended sediment graphs, and only data that were considered consistent were used in the analysis (5).

All of the data were reduced and put on magnetic tape. The digitized information was then transferred to four sets, one each for precipitation, stream-flow, sediment, and construction information for each rainfall event.

EQUATIONS

Younkin

Younkin (6) developed the following equation to predict the suspended sediment loads in streams caused specifically by uncontrolled, rainfall-induced erosion from highway construction sites in Pennsylvania:

$$SY_T = [Cy\, R_S(\log A_C)^{2.80}(1.93)^D]/(P_L)^{0.66} \qquad (1)$$

where

SY_T = total sediment yield (tons),
Cy = equation constant with a value between 0.129 and 0.153 for the watersheds Younkin studied,
R_S = rainfall-erosion index on a per storm basis,
A_C = area under highway construction (acres),
D = average depth of highway cut and fill (yd), and
P_L = proximity factor.

Cy reflected the overland transport factors of slope gradient and natural gradient and natural ground cover as well as the erodibility of the basin soils. D was used to express the slope length and gradient of the exposed construction area. R_S, the rainfall factor, and A_C, the exposed-area term, were taken to be measures of the soil detachment phase of soil erosion. Finally, P_L represented the overland transport phase of the erosion process. It was defined as the ratio of the surface area between the upslope side of the construction area and the nearest stream to the total area exposed by construction up to the time of the storm in question.

For this study, Younkin's equation was modified by using regression analysis on the data base previously described. The parameters that Younkin originally defined were not changed, but the coefficients were calibrated to the new data. The resulting equation was as follows:

$$SY_T = 0.127(R_S)^{1.26}(\log A_C)^{1.27}(1.19)^D(P_L)^{1.90} \qquad (1a)$$

Equation 1a, though admittedly much different, was found to be statistically better than Equation 1 and was used in the comparisons presented in Table 1.

Scott Run

Guy, Vice, and Ferguson (7) studied the effects of highway construction on the sediment load carried by Scott Run in Fairfax, Virginia. After continued analysis, they concluded that the most accurate relation between causal factors and measured suspended sediment discharge was

$$SY_T = Q_{ST}T_RA_CK_S \qquad (2)$$

where

SY_T = suspended sediment discharge or sediment yield (tons),
Q_{ST} = mean storm-event sediment transport rate (tons/day/acre of highway construction),
T_R = duration of storm runoff (days), and
K_S = mean seasonal erodibility factor.

Equation 2 was recalibrated by using the new data base and the resulting equation became

$$SY_T = 0.17\, Q_{ST}T_RA_CK_S^{0.74} \qquad (2a)$$

Modified USLE 1

A representative version of Williams' modified USLE (8), assumed to have applicability to highway and other types of construction sites, was taken in this study to be

$$SY_T = a(Q \times q_p)^b\, \overline{K}_C\overline{LS}_C \quad (3)$$

where

- a,b = coefficients with values of 95 and 0.56, respectively, in Williams' study;
- Q = volume of direct runoff (acre-ft);
- q_p = peak flow rate (ft³/s);
- $\overline{K}_C$ = average soil erodibility factor for the construction site at the time of the storm (tons/acre/unit of erosion index); and
- $\overline{LS}_C$ = average slope length factor and slope gradient factor for the construction site at the time of the storm (L is the ratio of soil loss from a specific field slope length to that from a 72.6-ft length for the same soil type and percentage slope, and S is the ratio of soil loss from a specific field gradient to that from a 9 percent slope).

Equation 3 was calibrated for the data. The a and b coefficients were evaluated by a simple least-squares regression analysis that related the dependent variable ($SY_T/\overline{K}_C\overline{LS}_C$) to the independent variable (Qxq_p). A log-log transformation to linearize the model was necessary prior to the application of the regression routine. The modified equation became

$$SY_T = 0.10(Q \times q_p)^{0.68}\overline{K}_C\overline{LS}_C \quad (3a)$$

Note that the a coefficient was calibrated to be 0.10 versus Williams' reported coefficient of 95. The reason for this difference is the site dependency of regression equations. However, the b coefficient does offset the a value, and the difference is not as significant as it might appear.

Modified USLE 2

Holberger and Truett (9) adapted the USLE to the estimation of sediment yields from construction sites. To do this, they empirically fitted factors to the equation to account for the effects of intervening terrain between the construction area and the point of sediment measurement in a nearby watercourse. One factor was the average distance from the foot of the exposed area to the nearest perennial system, and the other parameter was the percentage of the drainage basin undergoing construction. The Holberger and Truett equation took the following form:

$$SY_T = R_S\overline{K}_C(\overline{L}_C)^d(\overline{S}_C)^e(Do)^f \quad (4)$$

where d, e, and f are constants and Do is a factor, considered to be a sediment "loading function" or delivery ratio term, that accounts for the effects of intervening terrain between the construction area and the point of interest in a nearby receptor stream. Equation 4 was calibrated for the data to be

$$SY_T = 0.10R_S\overline{K}_C\overline{LS}_C(Do)^{-0.13} \quad (4a)$$

Values of d and e = 1 and f = 0.13, plus a coefficient of 0.10, were needed.

Modified USLE 3

The USLE 3 and USLE 4 relations are both gross erosion-delivery ratio equations. USLE 3, the U.S. Soil Conservation Service version of the original USLE applicable to construction areas, is defined as

$$A = R\overline{K}_C\overline{LS}_C \quad (5)$$

where A is the average annual soil loss in tons per acre per year and all other parameters are evaluated on an annual basis. However, in this study the USLE parameters were analyzed on a per storm basis in the form

$$A_S = R_S\overline{K}_C\overline{LS}_C \quad (5a)$$

and the corresponding estimated construction sediment yield values for each storm were computed from the following equation:

$$SY_T = A_S \times DR \quad (5b)$$

where DR is the delivery ratio based on the USLE equation.

Modified USLE 4

Clyde and others (10) substituted an erosion control factor (VM) for the crop and management factors in the original USLE so that they could estimate soil loss from highway construction sites. The VM term described the effects of all erosion control measures that could be implemented for the soil surface as well as chemical treatments. The parameter did not, however, encompass the effects of structures such as berms, ditches, or ponds. The equation was of the following form:

$$A_S = R_S\overline{K}_C\overline{LS}_C(VM) \quad (6)$$

A relation between the computed delivery ratios and appropriate causal factors was needed to define the DR term in Equation 5b. Only factors related to the construction site were considered. Therefore, the hydrologic and physical parameters analyzed with respect to prediction of delivery ratios for the construction sites were total direct runoff; runoff duration; maximum 30-min rainfall intensity; seasonal relative rainfall factor; effective precipitation factor; peak flow rate; Williams' direct runoff peak flow rate term (Qxq_p); average stream flow; total construction area, cleared and grubbed area; area devoted to earth-moving activities; area devoted to final grading; total exposed construction area; percentage of area devoted to different construction activities; average depth of cut and fill; total overland flow area outside of, but directly draining from, the construction site; average slope of overland flow area; average overland flow distance between the construction site and the receiving stream; and month of occurrence of the event.

The various parameters were logarithmically transformed so that a linear relation could be obtained via multiple linear regression analysis. The most suitable combination of independent causal variables with respect to the dependent delivery ratio was given by

$$DR_3 = 42.8(Q \times q)^{0.41}/(Ao)^{1.13} \quad (6a)$$

where Ao is the off-site overland flow area in acres.

Rational Model

A rational model was constructed in the form of a gross-erosion/delivery-ratio relation. The gross-erosion part of the equation provided a measure of the expected total soil detachment and erosion within the highway construction right-of-way with reference to the toe of the cut and/or fill slopes. The delivery part of the equation provided a measure of the portion of the total erosion actually transported to the stream. The proposed equation took the following form:

$$SY_T = [a(\bar{K}_C)(\bar{S}_C^1)(A_E)^b(M)^c(P)^d(U)^e]/[(\bar{L}_C^1)(V)] \quad (7)$$

where a, b, c, d, and e are constants and

M = rainfall parameter,
P = seasonal parameter,
U = runoff parameter, and
V = proximity parameter.

The soil erodibility term ($\bar{K}_C$), average percentage slope ($\bar{S}_C^1$), and slope ($\bar{L}_C^1$) of the construction area and the proximity parameter were not fitted with coefficients because each of these factors had only three different values since the data were collected on only three distinct construction sites.

The data variation for these variables was not considered to be significant. However, the terms themselves were considered to be important and necessary in any sediment yield prediction equation and were thus incorporated into the dependent variable parameter of the proposed least-squares multiple regression relation.

The various factors composing Equation 7 were chosen to represent specific effects in the soil erosion process. The rainfall parameter is a measure of the power of a storm to detach soil particles. The $\bar{K}_C$ parameter is a measure of the susceptibility of a soil to detachment and erosion. The ($\bar{S}^1/\bar{L}^1$) ratio (topographic factor) is assumed to be a measure of the susceptibility of the reshaped highway right-of-way slopes to erosion in addition to being a measure of the sediment transport capabilities. The exposed construction area is a measure of the maximum possible erosion. The seasonal factor is a measure of the general variation to be expected in meteorological conditions, the seasonal variation in soil moisture, and the seasonal variation in available runoff. The runoff factor is a measure of the transport capabilities of the storm runoff. Finally, the proximity factor is assumed to be a measure of the effects of the intervening terrain between the construction site and the point of sediment measurement.

All of the parameters considered in Equation 7 are rational indicators of the various components of the soil erosion/sediment delivery process. The actual proportionalities of the factors with respect to sediment yield, as indicated in the equation, are also rational from the standpoint of expected tendencies. That is, higher soil erodibility values, steeper slope gradients, greater quantities of rainfall and runoff in the form of larger values of the rainfall and runoff factors, and larger exposed areas should all tend to be associated with greater quantities of soil erosion and sediment yield. By definition and actual derivation, higher soil erodibility values are synonymous with those soils that are more susceptible to erosion. Steeper slope gradients will act to accelerate the flow of runoff water more than flatter slope gradients; and thus, besides detachment of soil by raindrop impact, the faster-flowing waters will detach or erode additional soil particles. Greater quantities of rainfall and runoff will potentially provide for greater detachment and transport of soil particles. Finally, larger areas of exposed soil should naturally tend to allow for larger quantities of sediment yield.

On the other hand, larger slope lengths should be associated with smaller quantities of soil erosion and sediment yield. The larger the slope length, given the same quantity of runoff and the same slope gradient, the greater should be the potential for deposition due mainly to the loss of energy (friction loss) as the runoff water flows down the slope. Loss of energy due to friction or turbulence or any other means can be translated directly into less energy available for keeping soil particles in suspension and, consequently, a greater chance for deposition.

The proportionality of the proximity factor with respect to sediment yield will vary depending on which of the possible proximity terms is considered. The total overland flow area outside of, but directly draining, the construction site (Ao) and the average overland flow distance between the construction site and the receiving stream (Do) should be expected to be inversely proportional to sediment yield. That is, larger values of Ao and Do should be associated with smaller quantities of sediment yield for the same reason as given above for slope length. However, the average slope of the off-site overland flow area (So) should be expected to be directly proportional to sediment yield in that steeper slopes should allow a larger portion of the total suspended sediment to reach the stream, all other conditions being the same. The purpose of the seasonal factor in Equation 7 is to act as an adjustment variable (i.e., the parameter is used as a fitting coefficient) to account for a portion of the variation in the measured sediment yield data that the combination of the other factors could not otherwise account for. Thus, its relation to sediment yield, whether it be directly or inversely proportional, should be solely dictated by the way in which the factor can best reduce the remaining variability in the data once the other variables in the equation are considered.

Values for the Equation 7 coefficients (a, b, c, d, and e) were determined by a least-squares multiple regression analysis. As indicated previously, the $\bar{K}_C$, $\bar{S}_C^1$, $\bar{L}_C^1$, and V factors were incorporated into the dependent variable, which took the following form (prior to logarithmic transformation):

$$DV = [(SY_T)(\bar{L}_C^1)(V)]/[(\bar{K}_C)(\bar{S}_C^1)] \quad (8)$$

with the V variable represented by Ao or Do. The V variable was transferred to the denominator of Equation 8 when it was represented by the So factor. The independent variables were, then, the logarithmically transformed versions of the A_E, M, P, and U parameters.

DISCUSSION OF RESULTS

As anticipated, the rational model proved to be the best or most consistent estimator of sediment yield at highway construction sites. The six existing techniques for estimating sediment yield were generally found to be less adaptable to the available data. This was most likely due to the necessary use of average values in defining the physical parameters associated with the construction sites. Even though each of the six existing equations was recalibrated to allow the equation coefficients to adjust to the use of the maximum average values, the results were, overall, less than impressive.

As mentioned previously, the rational model was composed of a combination of factors that represented the effects of various physical properties and hydrologic phenomena with regard to the soil erosion process and that together seemed to explain the process most reasonably. Each of the other six existing equations was deficient in one or more of the parameters considered to be important. The Younkin equation was composed of a rainfall factor, an affected-area term, a slope parameter, and a proximity factor but was missing a runoff term, a soil erodibility parameter, a slope length factor, and a seasonal term when compared with Equation 7. The Younkin Cy coefficient was reported to reflect the overland transport factors of slope gradient and natural cover as well as the erodibility of the basin soils; however, it could also be interpreted to reflect the overland transport terms of the slope gradient and natural ground cover. The Younkin equation is still deficient in three parameters that are believed to be significant in explaining the soil erosion process.

The Scott Run equation (Equation 2) was composed of a runoff parameter, a time duration factor, an affected-area term, and a seasonal factor but was deficient in a rainfall parameter, a slope gradient term, a slope length factor, a proximity term, and a soil erodibility parameter. Although the Scott Run seasonal factor was reported to be a seasonal erodibility term, it really is not in the strictest sense but is a factor that more heavily weighted those times during the year when larger than average quantities of sediment were measured.

The modified USLE 1 (Equation 3) was composed of a runoff factor, a soil erodibility term, and a slope-gradient/slope-length parameter. The relation was missing a rainfall factor, an affected-area term, a seasonal parameter, and a proximity term. Although the runoff factor was reported to have adequately replaced both the rainfall parameters and the need for a delivery ratio sediment yield via the original USLE, it is considered to be incomplete in totally representing the hydrologic aspects of the detachment and transport process. The runoff term should be interpreted as being representative of the major portion of the transport phase and the runoff or scour portion of the detachment phase but not also representative of the rainfall impact portion of the detachment and transport phases. The impact of raindrops on the soil surface loosens the upper soil particles, making them susceptible to easier entrainment by runoff waters at the beginning of the storm. Since the soil particles are already loosened, less runoff energy is needed for scour and more is available for transport. Once runoff is fully established in the form of sheet flow, the raindrop impact energy is no longer totally expended in loosening soil particles, but some (or all) is imparted onto the sheet flow, depending on depth of flow and momentum of raindrops, which increases the available energy for transport and for scour. Thus, it seemed appropriate to consider a separate rainfall term that solely represented the rainfall energy.

The modified USLE 2 (Equation 4) was composed of a rainfall factor, a soil erodibility term, a slope gradient parameter, a slope length factor, a proximity term, and, indirectly, an affected-area parameter but was deficient in a runoff factor and a seasonal term when compared with Equation 7. The runoff factor was found to be the most significant of all predictor terms, and so exclusion of the parameter should and did lead to less-than-acceptable results.

USLE 3 and USLE 4 (Equations 5 and 6) were both composed of a rainfall factor, a runoff parameter, a soil erodibility term, a slope gradient factor, a slope length parameter, a proximity factor, and, indirectly, an affected-area term. USLE 4, in addition, had an erosion control factor that could not be evaluated in this study but could become a significant factor in future research, reflecting highway right-of-way soil surface condition. Therefore, only the seasonal parameter was missing from either of the equations when they were compared with Equation 7. Although the seasonal factor was the least significant of the force-fit model terms, its purpose was to act as an adjustment parameter in "fine-tuning" the equation. Thus, its exclusion, although not an overly serious omission, was reflected in the predictive power of the equation.

The rational model equation (Equation 7) not only was considered to be the most complete and rational of the equations analyzed but also proved to be the best relation to use in estimating measured construction sediment yield. Table 1 gives the various error parameters and significance data for each of the seven equations analyzed. The superiority of Equation 7 can be clearly established in Table 1 if comparison is made among the error parameters and the R^2 values. Only in the F/F* category did the force-fit model not provide the most significant values. Nevertheless, the F and F* values indicated that the relation was highly significant at the 95 percent confidence level.

CONCLUSIONS

Six existing equations used by engineers to predict soil loss from highway construction sites were compared with a seventh method developed with data gathered from three highway construction sites in Pennsylvania. The accuracy of all the equations is best illustrated in Table 1 by the average error prediction (column 1). The smallest average error is 119 percent and the largest is more than 600 percent. How good or bad are these errors? It really depends on how the result is intended to be used. There is nothing really wrong with the accuracy of these equations as long as the user is aware of the possible errors and the limits of the methods. In time, as additional data are gathered, level 3 and level 4 equations will become verified and it is hoped that they will eventually be implemented in most design situations.

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Drainage Control Through Vegetation and Soil Management

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A procedure is developed that promotes the use of soil infiltration capacity and available soil profile storage in the design of highway drainage systems. By considering a design volume represented by the soil profile storage, the dependence on constructed runoff detention basins or other drainage structures can be reduced. This design volume is selected as the antecedent available storage in the soil that produces the T-year runoff from the T-year design rainfall. Data requirements of the overall methodology are commonly available soils, vegetation, and climatic parameters. The influence of antecedent moisture on the relation between rainfall and runoff frequency was tested by using 5 years of daily soil moisture and hourly rainfall and 10 years of hourly runoff data from the Calhoun Experimental Forest near Union, South Carolina. Equations that estimate the design antecedent moisture and its associated storage for ungaged sites are developed. Vegetation and soil management techniques that increase the volume of soil profile storage and soil infiltration capacity are reviewed. In addition, the Calhoun soil moisture data are fitted to frequency distributions to assess the risk involved in using soils-based drainage designs.

Traditional drainage design is usually based on (a) an estimate of peak design storm runoff for a given area and (b) man-made facilities that can accommodate and transport these peak flows away from developed sites. In recent years, however, the trend has shifted toward using on-site or source control to reduce flow rates leaving developed areas and thus prevent increased risk of downstream flooding. This change in philosophy has resulted in part from the excessive cost of building detention facilities but mostly from the growing concern over the effects of storm runoff downstream.

Engineers who design urban drainage systems often choose to use paved, open drainage channels and curb and gutter because of their high efficiency and stability in transporting runoff. Unfortunately, the efficiency that makes paved channels and curb and gutter desirable for removing runoff can cause detrimental effects downstream, including increased potential for flooding, erosion of natural waterways, and sediment pollution. Consequently, grassed roadside ditches or swales, infiltration pits and trenches, and porous pavements have been suggested for use in urban drainage design. All of these facilities rely on the use of soil infiltration capacity and soil profile storage to reduce the volume of storm runoff. This paper concentrates on the development of a methodology that allows the water storage capabilities of the soil profile to be explicitly included in the design of on-site drainage systems for handling storm water.

RAINFALL FREQUENCY VERSUS RUNOFF FREQUENCY

In the design of facilities for managing storm water, it is common practice to assume that the peak discharge from some selected design storm has the same return period as the rainfall depth in some "critical" duration. However, numerous studies of watersheds have concluded that the return frequency of runoff produced by a given storm is not fixed but varies over a wide range and depends on antecedent conditions in the catchment (1).

The runoff response on natural watersheds is highly sensitive to antecedent soil moisture or surrogate measures of wetness, such as five-day antecedent precipitation. This means that the proper selection of antecedent moisture is necessary to produce the desired T-year design runoff from the T-year rainfall. In a study of the density function of the difference between gross rainfall and the antecedent soil moisture deficit, Beran and Sutcliffe (2) concluded that for a given location and season the mean soil moisture deficit produces the rainfall excess of T-year return period from the rainfall of the same return period.

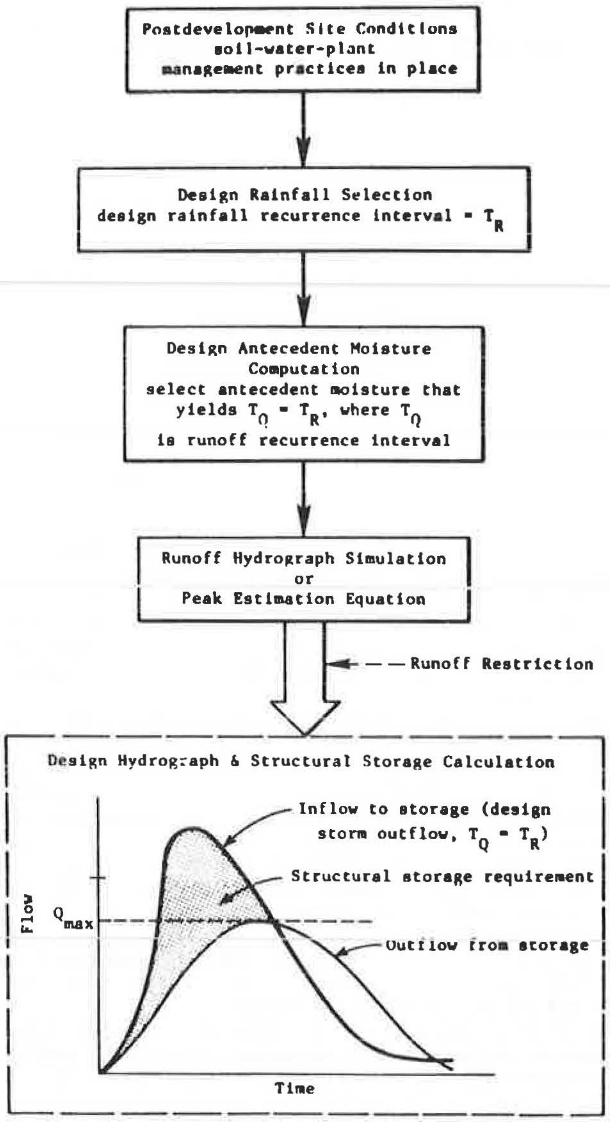
In critiquing a paper by Larson and Reich (3), Laurenson addressed the question, When the design storm-loss-rate unit hydrograph method of flood estimation is being used, what loss rate should be selected to produce equality of rainfall and runoff recurrence interval? He suggested that the correct value is the median of all values of loss rate that have been derived for the catchment.

CURRENT DESIGN PROCEDURE

Current drainage design practices can be summarized as follows:

1. Postdevelopment site conditions, such as slope, vegetation, and replacement of disturbed soils, are planned and minor attention is given to hydrologic impacts.
2. The runoff hydrograph or peak flow produced by some design rainfall of return frequency T_r is calculated. Antecedent soil moisture conditions are arbitrarily set, maybe at saturation, to yield a conservative runoff hydrograph of peak-flow estimate.
3. If no runoff restriction is in force, then outlet pipes from the site are sized to carry the predicted peak flow (Q_p). If restrictions are in force and they are exceeded by Q_p, then a detention structure with a controlled outlet must be sized so that Q_{max} allowed by the restrictions is not exceeded.

Figure 1. Flowchart of proposed drainage design procedure.



PROPOSED APPROACH

Two major changes in the current drainage design procedure are proposed. First, vegetation and soils management techniques are to be used to maintain or improve postdevelopment opportunity for on-site storage of storm water in the natural soil profile. This action will reduce postdevelopment runoff volumes and flow rates. Second, the design antecedent moisture and its associated storage will be calculated based on the soils and vegetation of the catchment, and they will be the proper moisture to produce the T-year flood from the T-year rainfall. Figure 1 shows these modifications in the context of the total drainage design procedure.

The findings of Beran and Sutcliffe (2) and others indicate that the seasonal, or perhaps annual, mean value of antecedent moisture should be selected to produce a runoff peak with a recurrence interval approximately equal to that of the design rainfall. The validity of their results was first investigated by using data collected for a forested watershed in South Carolina.

WATERSHED DESCRIPTION AND DATA BASE

The data used in this study were collected at the Calhoun Experimental Forest in South Carolina. A detailed description of the project is given by Metz and Douglass (4).

One of the catchments studied in the Calhoun project, referred to here as catchment 3, was selected for this investigation because data on antecedent soil moisture, peak stream flow, and rainfall were all available for it. The catchment had an area of 21.8 acres and was covered with a 20- to 26-year-old stand of loblolly pine. No soil moisture data were collected on the catchment, but it was located within 2 miles of instrumented pine plots that had very similar soils, cover, and rainfall. Antecedent soil moisture on the catchment was therefore assumed to be equal to that recorded for the instrumented loblolly pine plot. The daily soil moisture record extended from May 1950 through March 1956.

Hourly rainfall data were available for the period 1950-1961 from rain gages located in the Calhoun Experimental Forest. Information on the frequency of rainfall for the site was found in a technical memorandum of the National Oceanic and Atmospheric Administration (5) and was also determined from 23 years of records for a recording gage at nearby Lockhart, South Carolina.

Instantaneous flow records for runoff from catchment 3 were available for the 10-year period from 1951 to 1961.

The antecedent soil moisture values showed a strong seasonal trend under all cover conditions at the experimental plots. High moisture levels were observed in winter, when gentle storms of long duration are common and evapotranspiration rates are low. These are contrasted with summer conditions, when convectional storms of short duration and high intensity predominate and evapotranspiration rates are greatly increased by plant activity and high temperatures. Seasonal antecedent moisture histograms for barren, broomsedge, and loblolly pine plots are shown in Figure 2. Two seasons were assumed: summer, lasting from May 1 through October 14, and winter, from October 15 through April 30 each year. The histograms also indicate the seasonal mean antecedent moisture ($\overline{EMC}_a$) for each cover type.

Results for barren and broomsedge plots are included to show how, within a season, the frequency of dry antecedent conditions is increased with the increasing evapotranspiration capabilities of the cover type. Approximate maximum or potential rates of evapotranspiration (ET_p) under the three cover types shown in Figure 2 are given below:

Cover	ET_p (in/day)
Forest	0.20-0.30
Grass	0.15-0.25
Barren	0.10

ANALYSIS OF RAINFALL VERSUS RUNOFF FREQUENCY

Rainfall Frequency

Based on a duration of 1 h, which corresponds closely to the time of concentration of the watershed, the rainfall intensities for various recurrence intervals were obtained from the work of Frederick and others (5). Point rainfall values were selected because of the small size of catchment 3.

Stream-Flow Frequency

The 10-year stream-flow record restricted the fre-

Figure 2. Seasonal antecedent soil moisture histograms for various cover types at Calhoun Experimental Forest: (a) barren, October 1950 through December 1954; (b) broomsedge, October 1950 through April 1955; and (c) loblolly pine, October 1950 through April 1955.

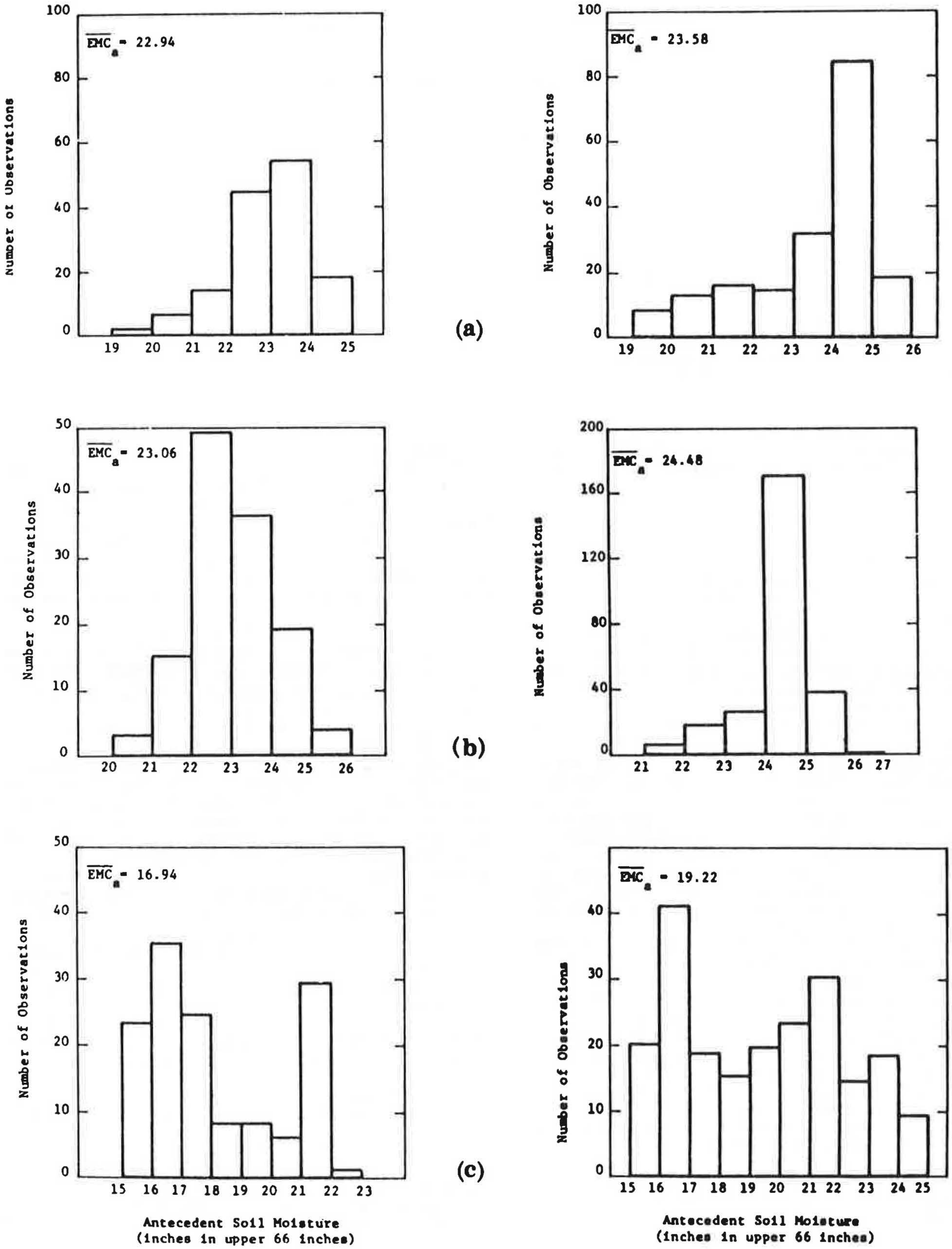


Table 1. Rainfall versus runoff frequency relations for catchment 3.

Storm No.	Date	Peak Flow (ft^3/s)	One-Hour Maximum Rain (in)	T_Q (years)	T_R (years)	T_{RW} (years)	$EMC_a - \overline{EMC_a}$ (in in upper 66 in)
1	12-20-51	4.55	0.40	1.4	<<1.0	1.1	-2.62
2	3-3-52	8.86	0.50	2.6	<<1.0	1.2	1.55
3	3-24-52	3.42	0.20	1.2	<<1.0	1.0	4.79
4	2-15-53	3.41	0.30	1.2	<<1.0	1.0	-0.62
5	2-20-53	2.91	0.40	1.1	<<1.0	1.1	2.68
6	1-16-54	8.77	0.80	2.6	<1.0	3.0	1.45
7	3-31-54	3.76	0.10	1.3	<<1.0	1.0	4.81
8	2-6-55	4.21	0.35	1.3	<<1.0	1.0	2.38
9	4-14-55	5.38	0.25	1.6	<<1.0	1.0	0.78
10	3-16-56	6.60	0.25	1.9	<<1.0	1.0	3.81
11	7-7-52	0.58	2.05	<<1.0	3.0	-	-
12	6-19-54	0.04	1.50	<<1.0	1.5	-	-
13	7-14-54	0.002	1.60	<<1.0	1.4	-	-
14	8-14-55	0.27	1.85	<<1.0	2.1	-	-
15	3-29-60	12.52	0.95	4.2	1.0	6	-
16	3-30-60	27.20	1.30	21.0	1.2	33	-
17	7-20-59	1.74	1.50	1.1	1.5	-	-
18	6-20-60	0.23	1.95	<<1.0	2.5	-	-
19	9-21-60	15.80	2.15	3.7	8.0	-	-

Note: T_Q = annual peak runoff recurrence interval, T_R = annual 1-h rain recurrence interval, T_{RW} = winter season 1-h rain recurrence interval, and ($EMC_a - \overline{EMC_a}$) = antecedent moisture differential.

quency analysis to recurrence intervals of 10 years or less. Frequency analyses were performed on the annual peaks by using the log-Pearson Type III analysis.

Analysis

The objective of the analysis of rainfall versus runoff frequency was to compare the recurrence interval of runoff with the recurrence interval of rainfall for selected storm events on catchment 3. The events initially selected for analysis were those that produced runoff peaks of at least one-year recurrence interval. Estimates of the peak-flow frequency for the selected events were based on analysis of the annual peaks.

Data were available on 12 winter storm events that met the previously mentioned criteria for the peak-flow recurrence interval, and antecedent moisture data were available for 10 of these. The results for these 12 storms are given in Table 1 (storms 1-10 and 15-16). It is readily apparent that each of the rainstorms had recurrence intervals less than the floods they produced. In addition, the results suggest that using the T-year rainfall in drainage design calculations may produce a flood peak with a recurrence interval greater than T, which would lead to overdesign.

Because all of these events occurred in the winter season, it was decided to select some summer events for analysis. Seven rainfall events of at least one-year recurrence interval were selected. The results are given in Table 1 (storms 11-14 and 17-19). For each of these storms, the assumption of equivalence of rainfall and runoff return frequency is rejected. This time, each of the rainfall events had a recurrence interval greater than that of the runoff peak it produced. For example, storm 19, with an 8-year recurrence interval, produced a runoff peak with a recurrence interval of less than 4 years.

The lack of equivalence of rainfall and runoff peak recurrence interval results from the seasonal nature of both rainfall intensity and antecedent moisture in catchment 3.

The occurrence of lower antecedent moisture levels in summer than in winter is shown in Figure 2. The information on rainfall and runoff peak frequency was derived from annual maximums and does not reflect the seasonal nature of either precipitation intensity or runoff peaks.

All of the high-runoff events in Table 1 occurred in the winter, as did 9 out of 10 observed annual peaks at catchment 3. A check of the stream-flow records at a U.S. Geological Survey gaging station on nearby Fairforest Creek revealed that 31 of 39 recorded annual peaks occurred in winter. These facts suggest that for catchment 3 the flood frequency based on annual peaks is equivalent to the winter season flood frequency.

In the last column of Table 1 the antecedent moisture differential for each storm is expressed as the difference between the antecedent moisture (EMC_a) and the winter season mean ($\overline{EMC_a}$) for a loblolly pine area (19.22 in).

Given that the peak flows occur in winter, the designer needs to know what recurrence interval of rainfall will produce the T-year (winter season) design runoff. Beran and Sutcliffe (2) found that for a given location and season the mean antecedent moisture produces equivalence between T_Q and T_R. Their results were tested for catchment 3 by computing the 1-h rainfall frequency for the winter season. This was accomplished by using 23 winter seasons of rainfall recorded at nearby Lockhart. The frequency curve for 1-h winter storms (based on NOAA data) is shown in Figure 3.

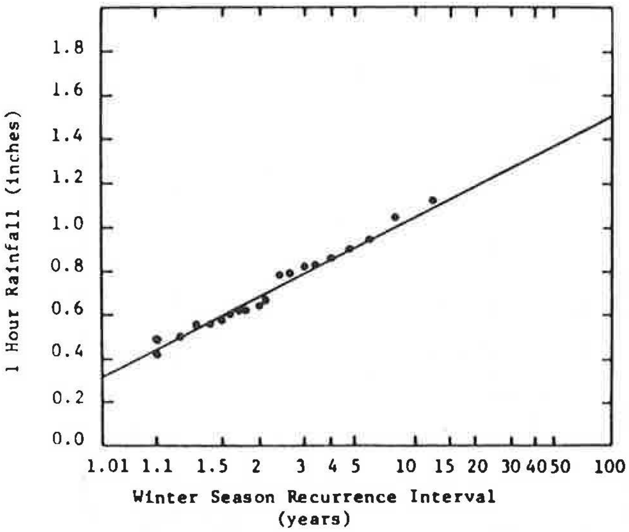
Winter season recurrence intervals for the winter storm events are given in Table 1. There is now a close relation between the winter values of T_R and the values of T_Q.

The analysis described above seems to verify the findings of Beran and Sutcliffe. The difference between rainfall and runoff peak recurrence interval for the 12 winter storms was reduced through the use of seasonal rather than annual frequency analysis. The last column of Table 1 indicates that the antecedent moisture did not differ more than 30 percent from the seasonal mean of 19.22 in for any of the 12 winter storms. The mean antecedent moisture appears to be a reasonable design assumption for producing the T-year runoff from the T-year rainfall, but additional storms on other catchments must be analyzed to further verify such an assumption.

Design Implications

The preceding analysis of rainfall, runoff peaks,

Figure 3. Rainfall frequency curve for 1-h winter season maximum storms at Lockart, South Carolina: 1951-1974.



and antecedent moisture on catchment 3 may be translated into the following general procedure for drainage design.

1. Analyze runoff data for the catchment region to determine whether runoff peaks occur mostly in one season.
2. Select some design runoff recurrence interval T.
3. If runoff peaks are seasonal, perform a frequency analysis on rainfall data for the season in which runoff peaks usually occur. If runoff is not seasonal, use frequencies based on annual maximums and skip step 4.
4. Use the results of step 3 to select T-year seasonal rainfall for design use.
5. When a runoff simulation model or a peak estimation equation requires an antecedent soil moisture assumption for the design rainfall, use the mean seasonal antecedent moisture for the runoff at the site in question. If runoff is not seasonal, use the mean annual antecedent moisture for design.

APPLICATION OF FINDINGS TO DRAINAGE DESIGN

Estimation of Seasonal Average Antecedent Moisture

Drainage design is often applied to catchments for which few or no soil moisture data are available. If antecedent moisture is to be routinely considered in drainage design in such catchments, then a technique is needed for estimating seasonal values of $\overline{EMC_a}$.

The Calhoun Forest has typical Piedmont soils and vegetative covers. The antecedent moisture information from the plots (Figure 2) is therefore assumed to be typical of that for the Piedmont region. The seasonal values of $\overline{EMC_a}$ from the various Calhoun plots were analyzed to provide estimates of $\overline{EMC_a}$ for catchments in the Piedmont region.

The seasonal value of $\overline{EMC_a}$ for each Calhoun Forest plot was normalized for application in Piedmont catchments by expressing $\overline{EMC_a}$ as a fraction of the field capacity (FIELDC) of the upper 66 in of soil on each plot. When a soil is at field capacity, all of its capillary pores are full. Much of this capillary water is available to plants for evapotranspiration, so that the fraction of FIELDC represented by $\overline{EMC_a}$/FIELDC for a particular season and cover type reflects the soil-drying potential of the cover type.

$\overline{EMC_a}$ is then computed by

$$\overline{EMC_a} = C \times \text{FIELDC} \tag{1}$$

where C is a multiplier that depends on cover type and season. From the Calhoun soil moisture data and the FIELDC as determined by Ramsey (6), values for C were obtained for various cover types and seasons in the Piedmont region. The C-values ranged from 0.70 for loblolly pine (summer) to 1.00 for broomsedge (winter).

This technique has yet to be verified. It should yield usable results for deep, well-drained Piedmont soils in locations that have a climate similar to that at the Calhoun Experimental Forest.

Available Storage

In any particular soil, the level of EMC_a determines the volume of empty pore space available to store any portion of the design rainfall that infiltrates. This volume of empty pore space is defined here as the antecedent available storage (AS_a). If EMC_a is at the wilting moisture, then the volume of AS_a in the soil profile is at a maximum for the soils and vegetative cover at the site. This maximum storage volume represents the total storage capacity (TSC) of the soil profile. When EMC_a is at some moisture above the wilting point, AS_a can be computed as

$$AS_a = TSC - EMC_a \tag{2}$$

or

$$as_a = tsc - emc_a \tag{3}$$

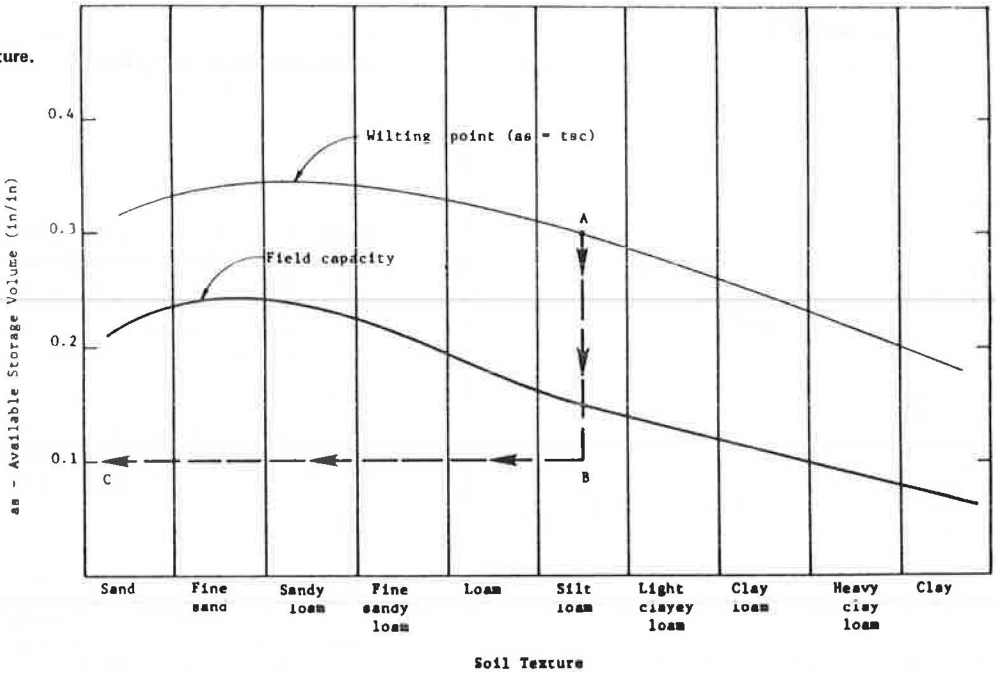
All upper-case terms are expressed in units of inches of water contained within a specified depth (e.g., the upper 66 in) of soil profile, whereas lower-case terms refer to inches of water per inch of soil.

England (10) used soil moisture tension and texture data compiled by Holtan and others (7) to estimate the volume of water held in excess of the wilting point (15-bar moisture retention) at saturation and at FIELDC for various soil texture classes. The volume of moisture held in excess of the wilting point will be referred to as M_w for units of total inches in the profile or m_w for units of inches per inch. England's results are shown in Figure 4. This figure can be used to obtain estimates for any combination of soil texture and level of moisture m_w. The curves were fitted by eye to England's data points.

The upper curve in Figure 4 represents the driest soil condition where available storage is at a maximum (as = tsc) for all texture classes. The available storage corresponding to higher levels of moisture is read from the vertical scale by moving vertically downward from the wilt line a distance equal to m_w and using the available storage scale.

For example, if it is desired to find the available storage provided by a silt loam with an m_w = 0.20 in/in, Figure 4 is entered at point A. Next, to account for the m_w of 0.20 in/in, move vertically downward from point A a distance of 0.20 in/in, as indicated by the ordinate scale, to point B. Finally, to find the corresponding level of available storage for m_w = 0.20 in/in, move horizon-

Figure 4. Variation in available storage at wilting point and field capacity as a function of soil texture.



tally to point C and read the answer: as = 0.1 in/in. This value is the average for the silt loam texture class. Because the wilting point for the silt loam ranges from 0.29 to 0.31 in/in, the estimate of available storage may range from 0.09 to 0.11 in/in.

A set of curves like those in Figure 4 could provide rapid estimates of TSC when the soil texture and depth are given. Such curves could also be used to find the average antecedent available storage ($\overline{as}_a$) for design use after $\overline{emc}_a$ has been determined and the wilting point moisture has been substracted to yield m_w.

In calculating TSC, EMC_a, or AS for drainage design, the depth of soil profile should be set at the rooting depth for the planned vegetative cover. This is because plant roots influence both the infiltration capacity and permeability during rain events and also regulate the antecedent moisture frequency, as indicated in Figure 2. Soil layers below the rooting depth are usually near capacity and provide insignificant storage during rain events.

Prediction of Design Rainfall Excess

The time distribution and magnitude of the design rainfall excess can be determined by using a modified ϕ-index technique. After the design storm excess rain hyetograph has been determined, then the runoff hydrograph can be calculated. Finally, with the design hydrograph known, it is possible to size required detention structures if Q_p is exceeded as it was in Figure 1.

Figure 5 shows the details of this modified, physically based ϕ-index. It traces the series of events for a hypothetical 10-year, 6-h rainfall on a site. AS_a for this hypothetical site is 1 in, and typical values have been assigned to other parameters. During the initial rain interval (t_i), all rainfall infiltrates at a rate equal to the rainfall rate. During the depression storage interval (t_d), the rainfall rate exceeds the infiltration rate (f_c) and all rainfall is captured in depression storage. After t_d, all rain in excess of f_c becomes runoff until, at the end of the soil storage interval, the soil profile is saturated. Once the soil profile is full, the rain can enter the soil only as fast as percolation and lateral drainage occur.

Rainfall and soil storage data for use in the modified ϕ-index approach are the design rainstorm and the average antecedent available storage, respectively. Other parameters such as interception storage, depression storage, and final infiltration rate must be evaluated by field experiments or estimated from the literature based on the soil-water-plant characteristics of the site.

SOIL AND VEGETATION MANAGEMENT PRACTICES

The management of soils and vegetation can maintain or improve soil storage capabilities in two ways: (a) maintaining high infiltration and percolation rates and (b) maintaining a high total storage capacity.

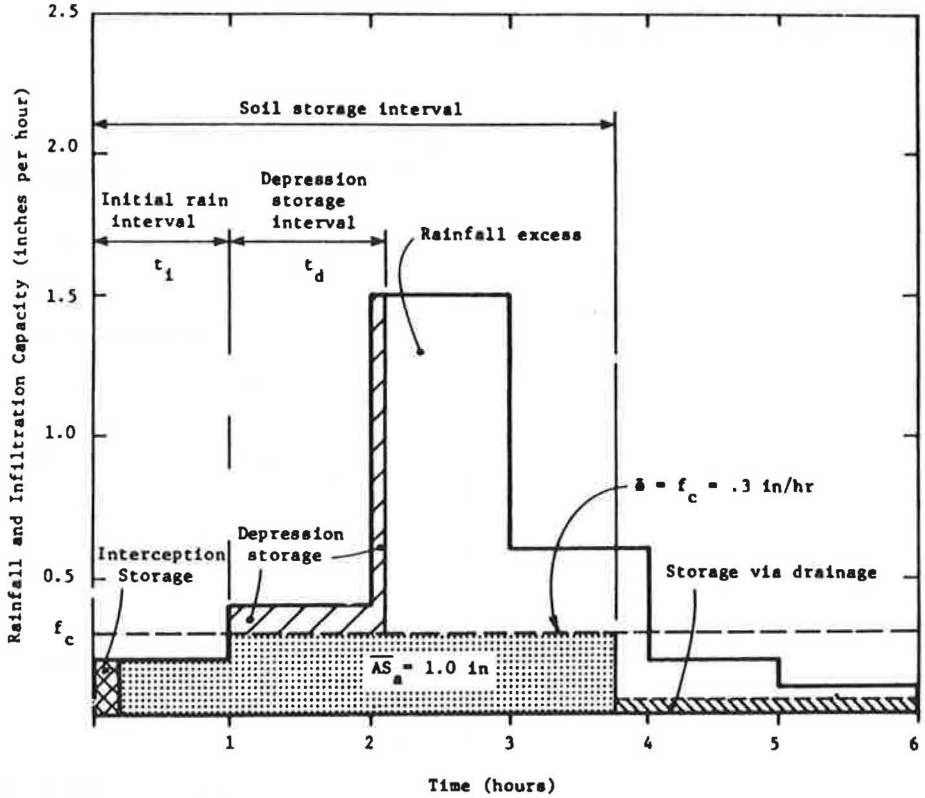
Infiltration Management

Practices that maintain a rough, open, stable structure at the soil surface and a high noncapillary porosity throughout the profile are the key to high infiltration capacities. Roughness refers to the microrelief that produces depression storage, whereas openness refers to the macroporosity visible at the soil surface (8). An open-surface structure allows water in while letting air out with very little pressure buildup. Structural stability prevents the surface sealing associated with the breakdown of surface aggregates and in-washing of fines. High noncapillary porosity permits fast percolation through the dominance of gravity drainage.

The soil surface should be left in a rough and uncompacted state as much as possible. A smooth, compact surface will not only impede infiltration but will also encourage high runoff velocities and result in erosion, washout of new vegetation or seeds, and the siltation of drainage works.

In backfilling, the most permeable, arable soil should be placed on top. This will ensure obtaining the highest infiltration capacities and allow for

Figure 5. Use of a physically based index to predict rainfall excess.



the establishment of vegetative cover.

Vegetation should be established as soon as possible. In addition to traditional erosion control and aesthetic value, a good cover shields the surface soil from direct impact of raindrops and thus preserves the open, rough surface structure created earlier. In addition, plant roots act to bind the surface structure and create deep macropores into less permeable layers. Plants also act as mulch formers, adding organic material to lighten and increase the noncapillary porosity of surface layers.

Special attention should be given to the maintenance of high infiltration capacities in upland areas. These areas have the driest soils, are seldom saturated, and can store large volumes of storm water.

Available Storage Management

At a given site, AS fluctuates daily with the soil moisture, which in turn is determined by infiltrated rainfall, gravity drainage, and losses due to evapotranspiration. AS is maximized when the soil moisture is minimized. Thus, the key objective in AS management is to drain the soil between rain events as quickly as possible through gravity drainage and evapotranspiration.

A high rate of gravity drainage or permeability is largely associated with a high noncapillary porosity. Where soils are not to be disturbed during development, no increase in existing permeability can be achieved. However, when earthwork is necessary, there is an opportunity for maintaining or increasing noncapillary porosity and, thereby, permeability.

An effective way to increase the noncapillary porosity of fine soils is to add organic matter. Organic matter binds small soil particles to form larger stable aggregates.

Excessive compaction destroys the noncapillary pores while usually having little effect on the capillary or plant-available porosity (9). Backfill soils should be only lightly compacted where bearing strength or slope stability is not a controlling factor. Care should be taken during construction to avoid compacting existing undisturbed soils.

The action of plant roots can greatly improve gravity drainage rates. As roots penetrate the soil, they often provide enough space for water to move alongside and downward into the profile. Root senescence and death leave a long, continuous noncapillary pore. During a rain event these extensive macropores provide access to a large wall area of relatively dry soil deep within the profile. In established vegetated areas, the noncapillary porosity caused by such root action is a major contributing factor to soil permeability.

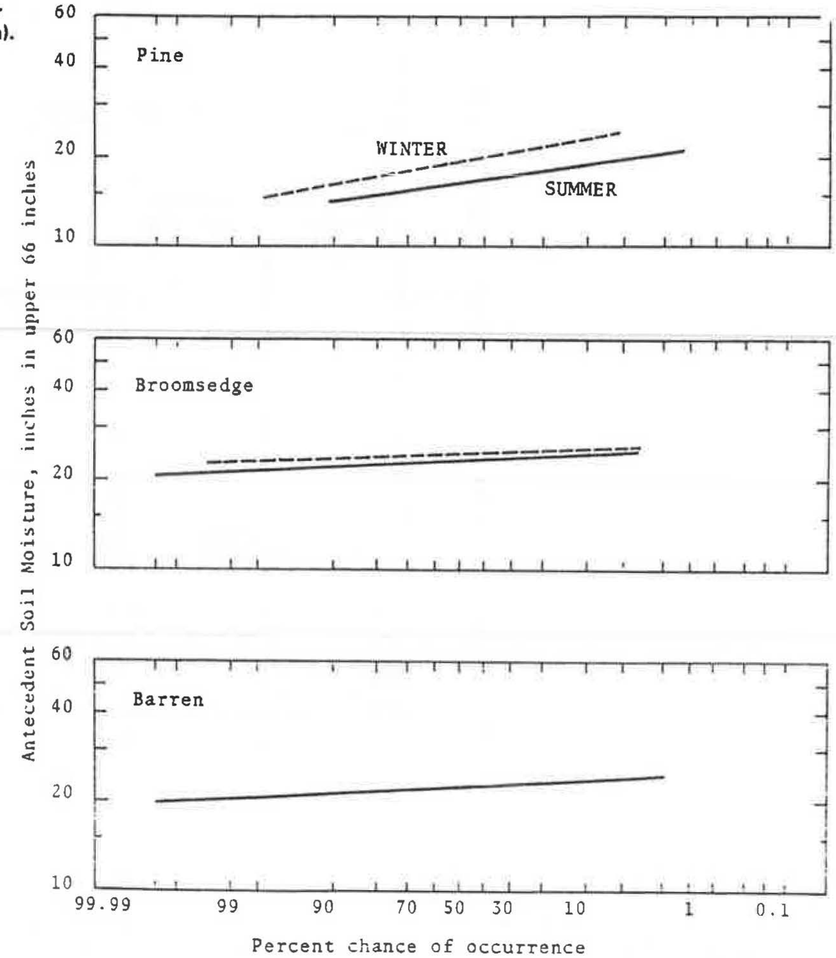
Plant species should be selected and placed in the watershed so as to withdraw soil water at the maximum feasible rate. Criteria for plant selection include the suitability of the soil, hardiness, rooting depth, evapotranspiration rate, maintenance, and the seasonal variability of the evapotranspiration rate. The best plant cover is one with good resistance to drought, a high evapotranspiration rate per unit depth throughout the year, a large total root depth, and inexpensive maintenance.

The placement of vegetation greatly affects its effectiveness as a soil moisture pump. Plants have access to more water and can create more new storage more rapidly when they are located in moist areas as opposed to dry areas. Such moist areas are usually located at low areas or near the toe of a slope. Field inspections and final grade plans should be used to detect or predict such moist areas, and water-using vegetation should be concentrated in these areas when the project is completed.

RISK ASSESSMENT

A constructed storage facility (e.g., a detention

Figure 6. Antecedent soil moisture probability analysis for various Calhoun Forest cover types (lognormal distribution).



basin) provides a fixed quantity of storage. This is in contrast to drainage designs, which incorporate the variable storage capability of the soil profile and thus involve risk. This risk is associated with the possibility that, when a design storm actually occurs, the antecedent soil moisture will be higher than the level assumed in the design.

The risk involved in such a soils-based drainage design can be calculated as the joint probability of the design storm event occurring simultaneously with various levels of antecedent moisture. The short period of record at catchment 3 makes the use of such a joint probability approach for that site impossible.

An alternative method of risk analysis requires the use of conditional probability concepts. The problem can then be expressed as follows: What is the probability that $\overline{EMC_a}$ will be exceeded, given that a rain event greater than or equal to the design storm occurs? In the symbols of probability analysis, it is desired to find the value of

$$P[X \geq x \mid Y \geq y]$$

where

X = antecedent soil moisture (in),
x = EMC_a (in),
Y = volume of the t-hour rain event (in), and
y = volume of the t-hour design rain event (in).

If X and Y are independent, a reasonable assumption for a given location and season is

$$P[X \geq x \mid Y \geq y] = P[X \geq x] \qquad (4)$$

The above equation indicates that the probability of $\overline{EMC_a}$ being equaled or exceeded prior to a design storm is $P[X \geq x]$. It is important to note that this equation holds for any design storm recurrence interval (T_R). Estimates of $P[X \geq x]$ for the three cover types at the Calhoun Experimental Forest were obtained by fitting observed antecedent soil moisture data to probability distributions. The data used in the analysis were antecedent moistures preceding the 30 largest rain events of the 4- to 5-year period of record for each season and type of vegetation. The results are shown in Figure 6.

The plots for antecedent moisture frequency indicate, for each season and type of vegetation, the probability that any given antecedent moisture will be equaled or exceeded. As an example, assume a design situation in which winter season soil moisture conditions are to be used and the vegetative cover is loblolly pine. If $\overline{EMC_a}$ is used for design, then Figure 2 indicates that $\overline{EMC_a}$ = 19.22 in for pine in winter. The middle plot in Figure 6 indicates that for an $\overline{EMC_a}$ = 19.22 in the chance of occurrence is approximately 52 percent. All cover types and seasons, except barren-winter, were found to be lognormally distributed by using the chi-square goodness-of-fit test at a 5 percent level of significance. The barren-winter results exhibited a large standard deviation and a severe skew that prevented a good fit to any distribution.

If one compares moisture frequency on a seasonal basis, Figure 6 indicates that the frequency of dry antecedent moisture is highest in summer (solid line) for pine and broomsedge. In comparing vegetative covers, the highest frequency of dry moisture conditions is maintained by pine followed by broomsedge and barren. Pine cover exhibits the greatest range of antecedent moisture. The greatest difference between winter and summer antecedent moisture for the full range of frequencies was observed under the pine cover.

The objective of drainage design is to size facilities to handle some T-year runoff peak. This study and others have shown that the best antecedent soil moisture assumption for predicting the T-year runoff peak from a T-year rainfall is seasonal antecedent moisture.

The risk analysis has shown, by using probability theory, that cover types that have high evapotranspiration rates (e.g., pine) can provide a drier antecedent soil moisture more frequently than can cover types that have lower evapotranspiration rates (e.g., broomsedge). In addition, seasonal antecedent soil moisture data probably fit a lognormal distribution for Piedmont cover types in other basins where soil and climatic conditions are similar to those found at the Calhoun Forest. Finally, the risk of experiencing a level of $EMC_a > \overline{EMC}_a$ when a design rainfall occurs does not represent a risk of a design failure but simply describes antecedent moisture probability.

CONCLUSIONS

1. Future drainage design work should incorporate source control of runoff through the management of postdevelopment soils and vegetation. This practice will reduce runoff peaks and thereby decrease the need for constructed detention facilities and other costly flood control measures.

2. The design storm concept of sizing drainage facilities is a valid technique only when used with hydrologic assumptions that will facilitate prediction of the T-year runoff peak from the T-year design rainfall. In locations where flood peaks are strongly seasonal, the seasonal rainfall frequency should be used for design. Flood peaks are seasonal if the annual frequency analysis yields the same frequency relation as one of the seasonal frequency analyses. If antecedent soil moisture is a parameter of the method for design peak-flow estimation and flood peaks in the watershed are seasonal, then the seasonal average antecedent moisture should be used for design. If flood peaks in the watershed are not typically seasonal, then the annual average antecedent moisture should be used in design.

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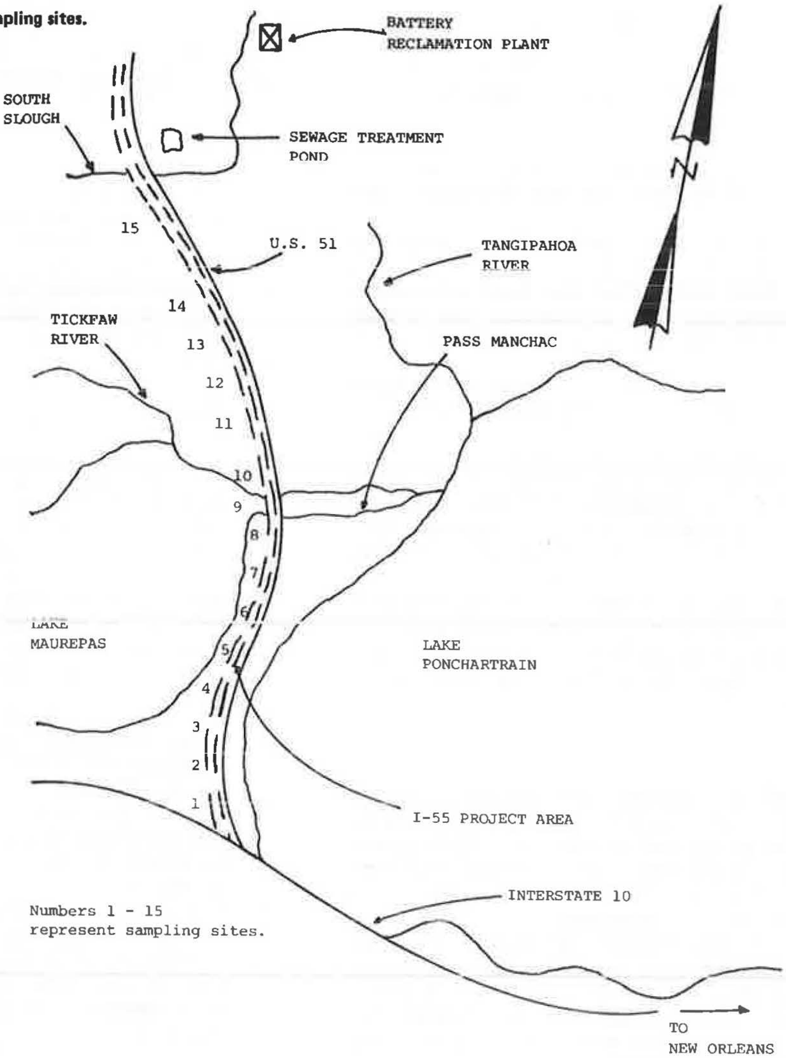
Effects of Dredged Highway Construction on Water Quality in a Louisiana Wetland

GEORGE H. CRAMER II AND WILLIAM C. HOPKINS, JR.

A research effort to determine, by physical and chemical means, the effect of current bridged highway construction techniques on water quality in a wetland is summarized. Selected water-quality parameters were monitored before, during, and after construction activities. The data show increases in turbidity and color during construction and a gradual returning to the preconstruction ambient in areas where construction was completed. Other parameters also followed this trend, but these changes were not as directly related to the construction activities as were turbidity and color. Local isolated activities other than highway construction were shown to produce more severe and longer-lasting effects on water quality. The information obtained may be useful in predicting the degree and duration of impacts of future construction projects on wetland environments.

The effects of highway construction on the water quality of wetland areas have been studied only to a limited degree. The apparent signs of water degradation, such as siltation and sedimentation, have

Figure 1. I-55 project area showing 15 sampling sites.



been seen many times in similar construction situations. The degree of degradation depends on construction techniques and watershed characteristics. Knowledge of the sedimentation process is necessary to assess the effects on the aquatic ecosystem. Chabreck (1) observed that sedimentation and the resulting turbidity depend on the vegetative cover and the soil type for a particular area. Hopkins (2) concluded that highway construction near watercourses should be watched very closely for silting and sedimentation.

The primary objectives of this research were as follows:

1. To provide a baseline or ambient condition for existing water quality.
2. To determine the changes in wetland water quality due to the dredging and construction of an elevated roadway.
3. To determine any residual effect on water quality due to the construction and the time rate of change caused by the construction.

STUDY PLAN

Description of Study Area

The area selected as a typical wetland was the new alignment for Interstate 55 beginning at the Interstate 10 junction north of LaPlace, Louisiana, and ending a few miles north of Pass Manchac between Lake Maurepas and Lake Pontchartrain (see Figure 1). This corridor offered an excellent opportunity for study because it contained areas not yet under construction, areas where construction was in process, and areas where construction was complete.

The area is located in the Mississippi River deltaic plain. Formation of the lakes occurred when two former deltas of the Mississippi River, St. Bernard and Cocodrie, filled in a formerly open bay with clay and silt from high water flows.

The wetlands of the area have undergone a number of changes in the past. In the late 1800s to early 1900s, logging and the Illinois Central Gulf Railroad posed two of the first man-made threats to this sensitive area. The railroad was built along the shores of Lake Pontchartrain and formed a barrier that limited the flushing action of the wetland ecosystem. Cypress logging activities from approximately 1910 to approximately 1935 left scars that can be seen even now when the area is viewed from the air. In 1954, the muck-fill construction of US-51 added to the problem of water movement within this marsh system. These alterations of the drainage patterns have contributed to the spread of the now overabundant water hyacinth (Eichhornia crassipes).

Table 1. Water-quality data.

		Turbidity			Color			Salinity			Dissolved Oxygen			pH		
Phase	Site	N	$\bar{x}$ (NTU)	σ (NTU)	N	$\bar{x}$ (PCU)	σ (PCU)	N	$\bar{x}$ (ppt)	σ (ppt)	N	$\bar{x}$ (ppm)	σ (ppm)	N	$\bar{x}$	σ
Preconstruction	1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2	17	15.82	7.61	16	100.94	29.56	16	0.43	0.27	16	3.91	1.48	16	7.09	0.28
	3	37	18.62	12.68	37	84.11	38.82	33	0.36	0.32	26	5.03	2.18	38	7.26	0.23
	4	40	15.30	18.00	40	96.73	41.41	33	0.44	0.54	27	3.84	1.84	40	7.05	0.30
	5	19	8.37	8.62	19	98.15	48.51	19	0.46	0.73	9	3.88	2.43	18	6.73	0.39
Construction	1	33	20.71	20.67	27	101.10	38.86	28	0.33	0.41	23	3.14	2.05	27	7.19	0.30
	2	23	15.57	11.28	23	79.22	31.09	17	0.81	1.35	13	5.53	2.30	23	7.29	0.28
	3	23	19.30	18.89	23	113.29	91.54	14	1.37	1.43	14	6.37	1.85	21	6.89	0.63
	4	17	18.66	16.04	16	190.63	89.40	11	0.50	0.39	10	3.81	2.72	16	6.20	0.57
	5	27	21.39	20.44	27	164.81	52.85	15	0.28	0.31	14	3.19	2.13	27	6.82	0.30
Postconstruction	1	60	6.17	6.11	59	75.29	23.29	49	0.63	0.35	49	4.88	2.82	59	7.21	0.36
	2	51	23.92	31.17	48	140.39	108.04	42	0.52	0.37	42	5.38	2.45	48	7.13	0.38
	3	33	37.24	36.95	32	119.47	79.07	32	0.94	0.62	33	7.15	2.22	33	7.37	0.29
	4	33	22.46	33.91	33	134.61	50.08	32	0.21	0.32	33	4.71	2.03	33	7.12	0.48
	5	27	15.11	18.33	26	133.15	36.77	27	0.09	0.22	28	4.48	2.43	27	6.82	0.30

Note: NTU = nephelometric turbidity units, PCU = platinum cobalt units, ppt = parts per thousand, and ppm = parts per million.

Performed Work

The construction technique used in this construction project consisted of using a dredge barge to dig an access canal between the existing borrow canal for US-51 and the US-51 roadway. This canal provided access for construction of the new bridged highway supported by concrete piles. In later years, the canal will provide access for maintenance of the structure as well as an area for a new aquatic habitat.

Two mitigation techniques were used on this construction project. The first technique required the spoil from the canal excavation to be placed in spoil areas so that water runoff from the dredging operation returned to the channel being excavated. The second technique was the use of earth plugs to minimize exchange of water along the new construction canal. By this means, increased sedimentation due to dredging was restricted to a specific area and not allowed to migrate the length of the new construction canal.

Hydrology

The major tidal flows and currents in the lakes are east-west in direction. Normal tidal fluctuations are 1-2 ft with storm tides of 3-4 ft. The borrow canal being studied generally travels in a north-south direction. Therefore, the tidal actions and currents from Lake Pontchartrain and Lake Maurepas exert a minimal effect on the study waterway. Data on the volume of water flowing through the Pontchartrain Basin and the study area are not readily available.

Sampling

Sampling sites were established in three different areas: one area not yet under construction, one area under construction, and one area where construction was completed. The number of sites originally selected for the monitoring study was 15 (Figure 1), but this was later reduced to 5 for better control. Sites that exhibited the stream characteristics most "typical" or representative of the area were chosen. Stream characteristics include depth, velocity of flow, stream bottom substrate, vegetation, and aquatic and wildlife habitat. Other factors used in site selection were the type of construction in the area and the accessibility of the site.

The sampling program was set up so that samples could be taken and processed within a one-week period. The sampling frequency was based on seasonal and climatic factors and construction activities. The schedule was adjusted for major events that affect runoff, such as heavy rains or unusual tides.

Construction

For the purposes of the study, "construction" was defined as any activity preparatory to or a part of the actual erection of the superstructure. This included clearing, grubbing, grading, filling, embankment development, and all structural work on the superstructure. It did not include finish work such as barrier rails, signs, and safety markers.

"Nonconstruction" was used to indicate that none of the above activities was in progress at the time of sampling. Due to variations in the work schedule based on the contractor's own time frame, the only separable parts of the construction activities were the dredging and structural work within these two categories; the various activities at each site were mixed up and followed no predictable sequence. Therefore, it is impossible to be any more specific about these activities.

Measured Parameters

The parameters selected for monitoring in this study were grouped into two categories. The first category included all parameters measured in the field study, such as temperature, salinity, pH, conductivity, and dissolved oxygen. The second category was the laboratory study in which turbidity, color, nutrients, and periodic oil and grease samples were evaluated. All laboratory tests were run at the Louisiana Department of Transportation and Development Materials Laboratory.

DATA ANALYSIS AND INTERPRETATION

The results of sampling and testing at each site for all parameters are summarized in Table 1. For each parameter, this table gives the total number of samples taken at each of the five final sites for the period of May 1975 to March 1980, the mean for each site, and the standard deviation of the samples taken during this period of time for each site. The data have been divided and analyzed in relation to preconstruction, construction, and postconstruction time frames. The variability of the data is expressed as sigma, the standard deviation of the observations. The magnitude of the standard deviation should be considered as a measure of the variability associated with material, sampling, and testing. The following is a brief description of the findings for each of the major parameters.

Turbidity

Turbidity is the term used to describe the degree of opaqueness produced in the water by suspended particulate matter. The major effects of high turbidity are (a) the quenching of light penetration, which inhibits photosynthesis and the production of oxygen by plants; (b) the building of zones of mud, silt, other sediments, and detritus; and (c) depletion of the dissolved oxygen as a result of respiration in the breaking down of suspended organic materials. The turbidity test data indicated that there were localized effects of construction on turbidity at site 1. The data also indicate that the turbidity began to decrease after completion of construction activities. Day and Boucher (3) indicated that this is the normal sequence of events with the parameter in relation to construction activities. There is a trend toward greater turbidity in the waters at sites 9 and 15. This is indicated by the preconstruction and postconstruction test data, which show overall increases in turbidity. At site 9, the most probable cause is the result of increased currents and tidal changes. At site 15, the increased turbidity is the result of off-site water pollution and low flows.

The effects of the construction were as predicted by Day and Boucher (3)--that is, minimal, controllable, and not of long duration.

Color

Color is defined as a quality of a visible phenomenon distinct from form and from light and shade. The standard test used in this project is the platinum cobalt spectrophotometric procedure of the U.S. Environmental Protection Agency (5). The color results indicated that there was an overall trend for the color to increase at the north end of the project. The data also indicate that construction activities at sites 12 and 15 increased the color content and, once the construction was completed, the trend was for the color to return to the ambient level.

Salinity

Salinity is a measure of the concentration of dissolved salts in water, expressed in parts per thousand. According to Cole (4), "Salinity affects the numbers and kinds of animals that can live in the area. Salinity also affects the amount of oxygen that can be dissolved in the water."

Significant changes in salinity occurred at sites 1, 9, 12, and 15. The data for sites 1 and 9 indicate that the salinity changes were due to natural phenomena rather than to construction activities. Sites 12 and 15 were subject to pollution from a used automobile battery plant. The effects of this are indicated by the decrease in salinity after the pollution of the project area by the battery junkyard was reduced.

Dissolved Oxygen

The amount of oxygen found in the water and available for use by aquatic flora and fauna is termed dissolved oxygen (DO). All of the significant test data indicate that the DO content was continuously increasing at sites 1, 5, and 9 throughout the duration of the project. The remainder of the sites showed no significant changes. The cause or causes of this were not determined in the study.

pH

The symbol pH represents the concentration of hydrogen ions (H^+)--i.e., the intensity of the acid. The pH of most inland waters varies from 6.0 to 9.0 depending on the amount and type of organic and/or mineral loads. The waters of the project area are subject to heavy organic loading from the marshes and swamp, which are the source of many organic acids.

The only effects of the construction were shown as a general trend to varying degrees at sites 1, 5, and 9. There the pH became more acidic during construction and then returned to the more alkaline ambient. Sites 12 and 15 showed the same tendency; however, because of pollution with battery acid during the project, the effect cannot be definitely attributed to the construction activities.

Solids

The solids category includes total, suspended, dissolved, and volatile solids. The tests on the various categories of solids produced few significant results. The parameter of volatile solids was deleted due to excess variability. The dissolved residue data indicated no significant change. The suspended residue showed increases at sites 12 and 15 during construction and a return toward the ambient after completion of construction.

The only significant change in total solids was a decrease at site 15 between the preconstruction data and postconstruction conditions. Although the data indicate that the decline in total solids continued through the construction activities, the initiation of the decline before and the continuation of it after construction make it doubtful that this improvement in water quality is related to the construction activities.

CONCLUSIONS

Review of all the data and external conditions affecting the various parameters leads to the identification of three highly significant trends within the study:

1. Elevated highway construction with environmental controls has minimal effects on the quality of the surrounding water.
2. Any effects produced by the construction tend to be temporary in nature and, once the construction is completed, the water quality tends to return toward the preconstruction ambient.
3. Local activities other than highway construction may produce greater and longer-lasting adverse effects on water quality.

It is recommended that future projects for this type of evaluation select research areas that absolutely minimize the influences of activities other than highway construction. These future projects should be designed very carefully to ensure a proper sampling program, a thorough evaluation of preconstruction (ambient) water-quality conditions, and a sufficient postconstruction period of evaluation to determine definitely whether the changes in water quality due to elevated highway construction are indeed reversible.

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Management of Drainage Systems from Highway Bridges for Pollution Control

YOUSEF A. YOUSEF, MARTIN P. WANIELISTA, HARVEY H. HARPER, AND JAMES E. CHRISTOPHER

Pollutants associated with runoff water from highway bridges were characterized and quantified. These pollutants are directly discharged through scupper drains to adjacent water bodies and floodplains or detained in ponds before being released to lakes and streams. Selected heavy metals, such as lead, zinc, copper, chromium, iron, nickel, and cadmium, were of particular concern because of their potential enrichment in biota. Results show significant differences in heavy metal concentrations between water samples from bridge runoff and adjacent streams. Heavy metals tend to concentrate in bottom sediments, floodplains, and adjacent soils. For example, bottom sediment samples from Lake Ivanhoe, north of Orlando, Florida, collected beneath bridges with scupper drains showed significantly higher concentrations of heavy metals than did samples collected beneath bridges without scupper drains. In addition, concentrations of heavy metals in the sediments of detention ponds receiving bridge drainage were higher than concentrations in sediments from adjacent lakes. It appears that management and careful design consideration of highway bridge drainage systems could result in significant reduction of the amount of pollutants released to adjacent water bodies.

In a 1979 National Cooperative Highway Research Program report, Shuldiner, Cope, and Newton (1) recognized the need for environmental impact assessment studies to satisfy guidelines for state and local agencies, U.S. Army Corps of Engineers permit procedures, and Section 4(F) of the U.S. Department of Transportation Act of 1966, as amended. They developed a user's manual and listed possible physical, chemical, and biological impacts from pile-supported roadway or bridging construction, maintenance, and use. These activities could result in major or variable impacts on turbidity, sedimentation, and chemical pollution. Biological responses may include changes in plant species composition, changes in primary and secondary productivity, and, in some cases, sudden mortality of aquatic species. The quantity and significance of these impacts have not been determined.

Many investigators, including Pitt and Amy (2), Sartor and Boyd (3), and Shaheen (4), had determined pollutant loadings, particularly lead (Pb), iron (Fe), zinc (Zn), cobalt (Co), chromium (Cr), nickel (Ni), and cadmium (Cd), associated with highway surfaces. Wanielista, Yousef, and Christopher (5) investigated pollutants from highway bridge runoff. Bell and Wanielista (6) investigated the transport of heavy metals by overland flow. They detected relatively high concentrations in adjacent soils.

This paper reports on pollutants detected in bridge drainage and associated impacts on the receiving land and/or freshwater environment.

RUNOFF FROM HIGHWAY BRIDGES

Lake Ivanhoe is a 125-acre freshwater lake located just north of the downtown section of the City of Orlando, Florida. A section of the central portion of the lake was filled in 1965 during the construction of Interstate 4, and the central island created was connected to the northern and southern shores by means of two bridges, as shown in Figure 1. The north bridge at Lake Ivanhoe consists of two sections, one for westbound traffic and one for eastbound traffic, each carrying three lanes of through traffic. Water on the bridge drains toward the adjacent land on either side since there are no scupper drains on the bridges. The south bridge also consists of two sections. Water is drained by a set of 4-in-diameter plastic pipe scupper drains set at 8-ft centers running along the eastern edge of the bridge. The average daily traffic (ADT) volume across Lake Ivanhoe, as provided by the Florida Department of Transportation (FDOT) during 1980, was approximately 45 000 to 50 000 eastbound and 48 000 to 58 000 westbound.

Samples were taken from beneath the northern bridges, from beneath two sets of scuppers on the southern bridges, and from the main body of the western portion of the lake, to serve as a control section. Direct runoff samples were collected directly from the scupper drains during three storm events in mid-August 1979. A total of 11 separate runoff samples were collected by taking samples from four different drains during each storm event.

The total concentrations of Zn, Pb, Ni, and Fe in runoff water averaged 4.7, 20.8, 3.5, and 12.6 times higher, respectively, than the average concentrations in Lake Ivanhoe, as given in Table 1. It is difficult to assess the relative impact of the bridge runoff due to a lack of specific information about the location and loadings from other sources and mixing zones within the lake.

Lead in the scupper drain runoff water was of special interest because it was detected in the highest concentrations of all the toxic heavy metals. The average lead concentration in the waters of Lake Ivanhoe (75 μg/L total Pb) and in the scupper drain runoff water (1558 μg/L total Pb) violates the maximum permissible concentration recommended by the Florida Department of Environmental Regulation. The rules specify that concen-

Figure 1. Sampling locations for bridge runoff at Lake Ivanhoe.

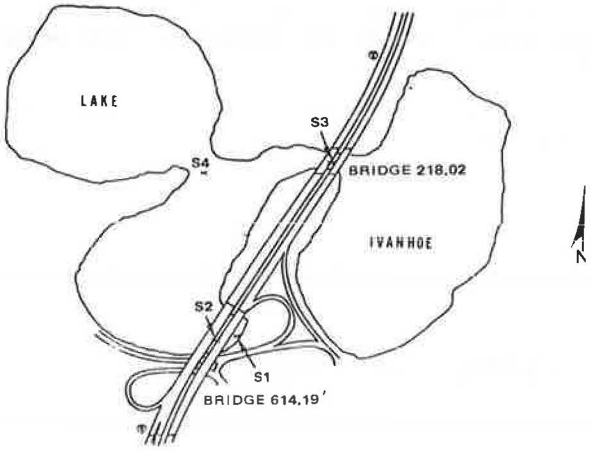

Table 1. Comparison of heavy metal concentrations in bridge runoff and Lake Ivanhoe water samples.

		Avg Concentration (μg/L)		Dissolved (%)		
Element	Form	Lake Ivanhoe	Bridge Runoff	Lake Ivanhoe	Bridge Runoff	Runoff/Lake Ratio
Zn	Total	104	498			4.7:1.0
	Dissolved	57	336	55	67	5.9:1.0
Pb	Total	75	1558			20.8:1.0
	Dissolved	55	187	73	12	3.4:1.0
Ni	Total	15	53			3.4:1.0
	Dissolved	9	49	60	92	5.4:1.0
Fe	Total	192	2427			12.6:1.0
	Dissolved	68	287	35	12	4.2:1.0

trations of Pb in all surface waters should not exceed 50 μg/L.

The heavy metals released into Lake Ivanhoe through the scupper drains can be estimated by assuming an average yearly rainfall of 50 in/year on the 73 440 ft^2 of bridge surface and the average concentrations determined in the runoff water from the south bridges during this study. The loadings based on these assumptions were 13.5 kg/year total and 1.6 kg/year dissolved lead, 4.3 kg/year total and 2.9 kg/year dissolved nickel, and 0.1 kg/year dissolved chromium. These loadings are probably conservative estimates of pollutants released to Lake Ivanhoe because dust fall and bulk precipitation have not been considered.

The fate of the heavy metals released into the waters of Lake Ivanhoe could not be completely predicted. Lead, which had the highest average concentration of the toxic heavy metals in the scupper runoff water, was shown to be primarily in the particulate form, 88 percent of the total lead. The exact fate of this particulate matter is not fully known; however, as reported by Olson and Skogerboe (7), lead compounds have generally been associated with the most dense fractions of the soil and should settle out of the water close to the point of release.

BOTTOM SEDIMENTS

Sediment samples were collected from several locations of each of the principal sampling sites in Lake Ivanhoe and measured for extractible metals. A statistical analysis was performed to compare the concentrations of heavy metals in the sediments collected underneath the south bridges with scupper drains and the north bridges without scupper drains. The results of the test analysis comparing the samples are presented in Table 2. The concentrations of Zn, Pb, Ni, and Fe were found to be significantly greater in the sediments underneath the south bridges with scuppers at the 99 percent confidence level. Lead concentrations were significantly different at the 99.9 percent confidence level and were more than three times higher in the sediments beneath the bridges with scuppers. Statistical analysis shows that there is significant heavy metal enrichment in the sediment underneath the bridges with scuppers. It supports the conclusion that heavy metals associated with particulates, especially lead, which was 88 percent in the particulate form in the runoff samples, will settle out and become immobilized in the sediments near the point of release.

RETENTION-DETENTION PONDS

The Maitland interchange north of Orlando was constructed in 1976. Maitland Boulevard crosses over I-4 by means of a bridge overpass created during construction of the interchange. The traffic lanes on the Interstate are separated by a 20-ft grassy median as they approach the interchange. The median widens to 44 ft through the interchange. Stormwater coming off the Interstate is delivered by overland flow over a good grass cover to storm-drain inlets or receiving waters. Three borrow pits that were dug to provide fill for the construction of the overpass remain in existence, serving as stormwater retention-detention facilities. Stormwater runoff from the Maitland Boulevard bridge crossing over I-4 is conveyed directly off the roadway surface through storm-water inlets to culverts that discharge directly into the ponds. The ponds are interconnected so that the two northernmost ponds flow into the southwest pond (referred to hereafter as the west pond) when they reach a certain design level. The water from the west pond flows over a wooden weir at its southern end, which is connected to Lake Lucien by means of a culvert and a short, densely vegetated ditch (see Figure 2). Runoff to the ponds is essentially all from the roadway environment, and flow to Lake Lucien is a combination of natural, highway, and citrus runoff. Lake Lucien is a 57-acre freshwater lake, and the lack of significant development on its shores has left it in a relatively clean condition.

The Maitland Boulevard bridge consists of two sections, one that carries two lanes of eastbound traffic plus one exit lane and another that carries two lanes of westbound traffic plus one exit lane. ADT on Maitland Boulevard was approximately 11 000 eastbound and 10 000 westbound during the early period. I-4 has three lanes of through traffic eastbound and westbound through the Maitland interchange. ADT on I-4 through the Maitland interchange was approximately 32 000 to 40 000 eastbound and westbound.

Three sets of samples were collected from the Maitland interchange site--one from the west pond, one opposite the outfall of the west pond into Lake Lucien, and one from the center of Lake Lucien to serve as a control.

The statistical comparison of the west pond and Lake Lucien (Table 3) showed that concentrations of dissolved and total lead, total chromium, and total iron were significantly greater in the west pond at the 95 percent confidence level. The west pond was also noted to be highly turbid, with many fine particulates in suspension, which would help to

Table 2. Significance of differences in heavy metal concentrations of bottom sediments from Lake Ivanhoe (t-test analysis).

	Mean Dry Weight (μg/g)		
Element	South Bridges With Scuppers[a]	North Bridges Without Scuppers[b]	Probability (%)
Zn	96.9	42.0	99.60
Pb	423.0	132.0	99.99
Cr	23.9	11.0	97.07
Ni	7.2	2.8	99.60
Cu	80.1	29.2	98.71
Fe	1689.0	643.0	99.85

[a]Eight samples were collected. [b]Seven samples were collected.

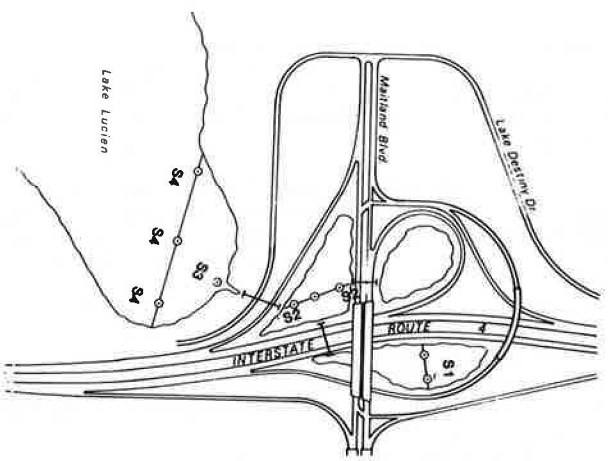

Figure 2. Sampling locations for bridge runoff at Maitland Interchange.

Table 3. Significance of differences in heavy metal concentrations in water samples from Maitland Interchange.

	Avg Concentration (μg/L)					
	Total		Dissolved		Probability (%)	
Element	Lake Lucien	West Pond	Lake Lucien	West Pond	Total	Dissolved
Zn	56	64	34	43	45.9	75.6
Pb	33	92	19	66	99.9	98.6
Cr	8.6	17	5.4	7	94.9	70.4
Ni	7.3	15	3.4	5	80.8	53.6
Cu	36	38	19	21	29.2	34.9
Fe	182	414	82	128	98.4	52.4

explain the higher total concentrations reported there.

A comparison of sediments from the west pond and sediments from the center of Lake Lucien (see Table 4) showed that concentrations of Pb, Cr, Ni, Cu, Fe, and Cd were 3-22 times greater in the sediments of the west pond. The analysis of Pb, Cr, Ni, Fe, and Cd showed significant difference at the >95 percent confidence level. A comparison between the west pond and the outfall area into Lake Lucien did not exhibit similar results (5). The sediments from the downstream area of the outfall were similar in concentration levels to those found in the west pond. These concentrations may be due in part to the high organic content of these sediments and flow leakage through the wooden slats of the exit weir from the west pond to the outfall area during periods of no flow over the weir. The use of a control structure that prevents leakage and the installation of underdrains or a simple filtration device to remove heavy metals by adsorption and filtration can significantly reduce the amount of heavy metals leaving the west pond.

Table 4. Significance of differences in heavy metal concentrations in bottom sediments from Maitland Interchange (t-test analysis).

	Mean Dry Weight (μg/g)		
Element	Lake Lucien	West Pond	Probability (%)
Zn	21.1	35.2	80.27
Pb	3.4	76.0	97.51
Cr	2.5	33.9	98.87
Ni	1.2	10.7	97.64
Cu	5.0	15.2	93.17
Fe	421.4	3264.7	98.62
Cd	0.1	0.7	96.05

FLOODPLAINS

Floodplains may filter out nutrients, heavy metals, sediments, and other pollutants and store them within the ecosystem. It is possible to use these floodplains for treatment of controlled highway bridge runoff without damaging their habitat. Odum (8) stated that wetlands are natural water management and treatment systems that operate on solar energy. The extent of their tolerance to heavy metals and highway-related activities is not well defined.

Soil and plant samples were collected from floodplains beneath highway bridges located at US-17-92 and Shingle Creek, US-192 and Shingle Creek, and I-4 and Padgett Creek in central Florida. These bridge areas receive direct runoff water through scupper drains or curb-and-gutter drainage systems. Control areas were selected upstream and downstream from the bridge areas. Preliminary results of heavy metal concentrations in soil and plant samples collected from bridge areas and control areas were analyzed.

The data indicated higher concentrations of heavy metals, particularly lead, in soil samples collected from bridge areas than were found in samples collected from control areas (see Table 5). Lead concentrations in soil samples from bridge areas were 78, 3.3, and 23.4 times higher than control samples from US-17-92 and Shingle Creek, US-192 and Shingle Creek, and I-4 and Padgett Creek, respectively. Plant samples exhibited similar trends, but the difference between plant samples from bridge areas and control areas was not as high as the difference in the soil samples. The work is continuing, and no obvious differences have been observed in the diversity of plants located in various areas surrounding the bridges. However, bioassay experiments in the laboratory tend to show increased plant productivity as a result of discharging runoff from highway bridges to adjacent streams (9).

CONCLUSIONS AND RECOMMENDATIONS

This paper has presented various examples of the management of bridge runoff. They include direct discharge of storm water through scupper drains to Lake Ivanhoe and I-4. Heavy metals, particularly lead and iron, in the scupper drain runoff were mainly in the particulate form, since an average of only 12 percent of the total concentration was in the dissolved form. These particulate fractions were most likely to settle out from the water column in the immediate vicinity of the point of release and become immobilized by the sediments. This may have resulted in the concentrations of Pb, Cr, Fe,

Table 5. Statistical analysis of extractable heavy metals in soil samples collected from floodplains receiving highway bridge runoff.

Location	Element	Control			Bridge Area		
		No. of Samples	Oven Dry Weight (μg/g) x	Oven Dry Weight (μg/g) σ	No. of Samples	Oven Dry Weight (μg/g) x	Oven Dry Weight (μg/g) σ
US-17-92 and Shingle Creek	Zn	9	2.1	0.8	14	46	40
	Fe	9	487	108	14	1213	689
	Pb	9	3.5	1.3	14	273	339
	Cr	9	1.3	0.7	14	5.7	4.3
US-192 and Shingle Creek	Zn	4	17	14	12	7.7	0.4
	Fe	4	302	94	12	607	164
	Pb	4	11	7.4	12	36	4.7
	Cr	4	1.7	2	12	3.6	3.1
I-4 and Padgett Creek	Zn	4	50	13	8	253	159
	Fe	4	3392	1485	8	5738	1202
	Pb	4	93	40	8	2174	2128
	Cr	4	16	6.8	8	17	4.0

Ni, and Zn in the sediments underneath the scupper drains being significantly higher than concentrations in the sediments collected from areas beneath the bridges without scupper drains.

Most of the heavy metals released to Lake Ivanhoe were concentrated in bottom sediments. On a mass balance basis, which estimated the average distribution of heavy metals per unit area among the dissolved fraction, the suspended fraction, and bottom sediments, it was shown that most of the heavy metals--typically 95-98 percent of the total--were associated with the bottom sediments (5). As a result, high concentrations of heavy metals in submerged plants and benthic organisms were detected.

Limited results from highway bridge sites that release runoff to floodplains demonstrated the capacity of soil to concentrate heavy metals. Floodplains are conjectured to be possible sinks of metals. The sorption-desorption capacities of floodplains are currently being investigated. It is also encouraging to notice no significant difference in plant diversity on soils located close to the bridge sites and control areas. Floodplains could become a feasible alternative to receive runoff from highway bridges.

The study conducted at the Maitland interchange was useful to illustrate the function of retention-detention ponds, which seemed to concentrate heavy metals associated with highway runoff and could be maintained to effectively minimize release of pollutants to adjacent Lake Lucien. Detention ponds retain an initial amount of runoff and release it into the lake at a controlled rate through the use of well-designed outlet structures. The outlet hydrograph depends on the type of control structure and the size of the detention pond. Detention reduces peak flows and suspended solids and can provide other pollutant removal efficiencies for storms of short duration (10,11).

When highways pass over bodies of water, especially land-locked impoundments where the effects of heavy metal pollution are more localized and where high traffic volumes are encountered, it is recommended that

1. The use of scupper drains in new construction be limited as much as possible,
2. Runoff from the bridge surface be directed off the bridge surface toward either side so that the runoff will experience the maximum overland flow to encourage percolation and removal of heavy metals by the soil before the runoff reaches the receiving body, and
3. Future research be conducted to determine the extent of the floodplain required adjacent to a bridge to create a desirable level of heavy metal removal.

Where detention-retention systems are used in conjunction with highways for the control and storage of runoff before the runoff is discharged into a receiving water body and where heavy metal removal is desired,

1. Control structures should be installed to ensure that heavy metals are not released to the receiving water body during periods of no flow by flow leakage between the ponds and the receiving waters,
2. Natural vegetation canals can be used to convey water from the detention-retention pond to the receiving water body to enhance additional settling out and adsorption of heavy metals before introduction of the runoff into the main body of the receiving water, and
3. Further research should be conducted to develop construction practices and management schemes for these detention-retention ponds to maximize removal of heavy metals, and consideration should be given to the types of sediments that afford the greatest degree of removal and the role and best types of plants that might be introduced to increase heavy metal removal.

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Field Instrumentation for Monitoring Water-Quality Effects of Storm-Water Runoff from Highways

THOMAS V. DUPUIS AND BYRON N. LORD

Storm-water runoff from operating highways can carry considerable quantities of pollutants, especially petroleum hydrocarbons and solids and metals, into the nation's receiving waters. The Federal Highway Administration, charged with the responsibility of protecting the environment from pollution from highway sources, has approached the problem in a multiphase research effort. The objective of the first phase was to identify and quantify the constituents of highway runoff. The next phase sought to identify sources and migration paths of these pollutants from the highways to receiving waters. The third phase, currently in progress, is analyzing actual impacts of highway runoff on receiving water. A wide variety of instrumentation has been used in the field monitoring portions of the investigation. Physical, chemical, and biological characterizations have been made. A general description of the types of equipment used in all three phases of the research program is presented. Included are brief observations on the effectiveness of certain types of specialized equipment not common to most water-quality surveys. It is hoped that this information will be of assistance to the highway community in planning and conducting environmental monitoring programs.

The highway system is a potential source of many possible pollutants to surrounding surface and subsurface waters. The National Environmental Policy Act of 1969 (NEPA) mandates that, for all federal projects that affect the environment, government agencies shall use a systematic, interdisciplinary approach that will ensure integrated use of the natural and social sciences and the environmental design arts in planning and decisionmaking. The Federal Water Pollution Control Act amendments of 1972 set a national goal of restoring and maintaining the chemical, physical, and biological integrity of the nation's water resources. In addition, many states either have already enacted or are in the process of enacting legislation similar to NEPA that may be more stringent than the federal laws in controlling various point and nonpoint discharges. Thus, consideration of the effects of a highway system on the environment plays an increasingly important role in the planning, design, construction, and operation of a transportation system.

Millions of roadway miles across the country pass over or near a variety of receiving waters. Thus, large volumes of highway storm-water runoff from highway right-of-way drainage areas are eventually discharged to a variety of large and small watersheds. The roadway contaminants contained in runoff might exert significant impact on receiving waters due to both chronic and acute loadings. However, there is currently very little information in the available literature on impacts on receiving water from highway runoff. Therefore, it is necessary to develop information on these impacts in order to properly address the need to protect the quality of receiving water from degradation.

The Federal Highway Administration, charged with responsibility for protecting the environment from pollution from operating highways, has approached the problem in a multiphase research effort that has the following objectives:

1. To identify and quantify the constituents of highway runoff,
2. To identify the sources and the migration paths of these pollutants from the highways to the receiving water,
3. To analyze the effects of these pollutants on receiving waters and on specific aquatic biota, and
4. To develop the necessary abatement and treatment methodology for objectionable constituents.

To date, studies designed to fulfill the first two research objectives have been effectively completed. A study of actual impacts on receiving water (objective 3) is currently under way. It should be noted that the scope of the research program is nationwide. Six sites located in different geographic regions of the contiguous United States were monitored in the phase 1 study. Four sites are to be used for both the phase 2 and phase 3 evaluations.

The objective of this paper is to describe instrumentation requirements for conduct of such a comprehensive field monitoring program. The scope is limited to types of equipment actually used in the program but includes physical, chemical, and biological techniques. To date, only lotic (flowing water) receiving water systems have been studied in the phase 3 research and only instrumentation pertinent to these types of systems is included in this discussion.

METEOROLOGICAL MONITORING

The meteorological parameters of most importance in this type of water-quality study include precipita-

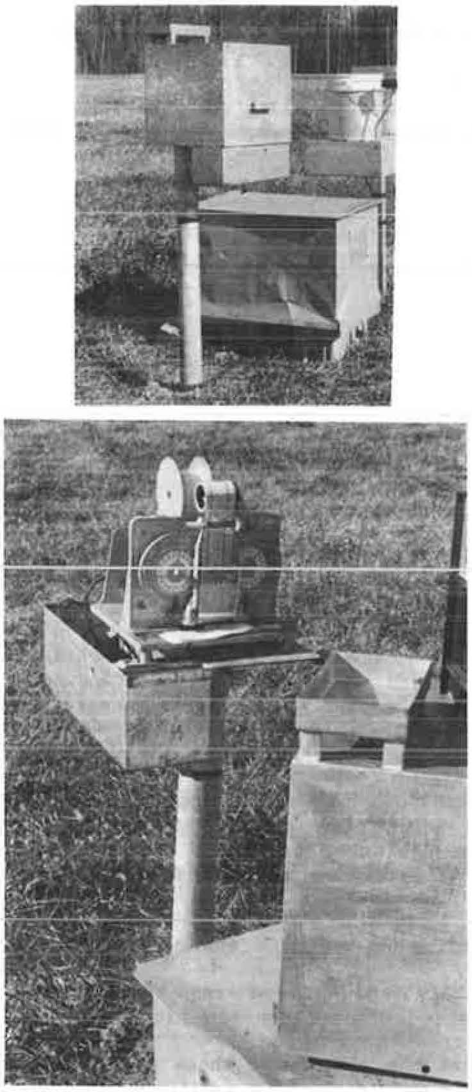


Figure 1. Precipitation measurement system.

tion (rain, sleet, hail, snow, etc.), wind speed and direction, air temperature, and atmospheric particulate deposition (dust fall).

Precipitation

The most common precipitation gage used in this program was a continuous-recording weighing gage. The major advantage of weighing gages is that continuous cumulative measurement is made so that time, quantity, and intensity can be determined for all types of precipitation. Evaporative losses for long-term charts (one week or more) can cause chart reading problems, but these losses are usually discernible.

Another system of precipitation measurement is being used by the U.S. Geological Survey (USGS) in North Carolina for one of the phase 3 sites. Rain is conducted from a collector tray to a standpipe (see Figure 1). A float and counterweight system translates the rainfall accumulation in the pipe to standard rainfall volume through the use of a Fischer-Porter automatic digital recorder. A small battery-powered pump, which is level actuated, periodically empties the standpipe of accumulated rainfall. This provides a digital punched tape

Figure 2. Wet-dry collector.

recording of rainfall volume per discrete time interval and greatly reduces manpower requirements for chart reading. Diurnal thermal expansion of metallic standpipes can produce minor apparent fluctuations in precipitation volume, but these can be readily accounted for as tapes are entered into the computer file. This system would not be appropriate in northern climates unless a heating device were incorporated into the collector tray and standpipe to melt snow and ice.

Dust Fall

Dust-fall buckets were used to provide estimates of background or highway right-of-way pollutant loadings due to atmospheric deposition. American Society of Testing and Materials (ASTM) specifications on the location and size of dust-fall buckets and analytic procedures (ASTM D1739-62) were followed. A standard tapered plastic (polyethylene) bucket, 8 in (20.3 cm) in diameter and 10 in (24.4 cm) deep and equipped with bird-ring support, was used. These buckets have held up well under all climatic conditions. In addition, atmospheric deposition was separated into precipitation-related and dry dust-fall components by using wet-dry collectors (see Figure 2). These units consist of two side-by-side polyethylene buckets 11.3 in (23.6 cm) in diameter and 9.1 in (23.2 cm) deep. One bucket serves as a wet collector and the other as a dry collector, and a mechanical cover shifts from one to the other depending on climatic conditions. A heated sensor detects rainfall (or any other form of precipitation) and activates the movement of the cover from wet to dry bucket and back again.

Wind Speed and Direction and Air Temperature

Supplemental meteorological measurements such as wind speed and direction were critical for the source and migration studies. Of importance to all phases of the research program was ambient air temperature, especially with respect to snowmelt runoff. All three measurements were made with a single Meteorology Research Institute instrument set at a height of 12 ft (3.66 m). A one-month strip chart provided continuous recording of wind speed and direction and air temperature.

Quantitative Highway Runoff Monitoring

Determination of the volume and intensity of stormwater runoff was obviously of critical importance for all phases of the research program. To achieve this objective, a variety of instrumentation was used. The two basic categories of flow instrumentation are (a) primary measurement devices and (b) level-recording devices.

Primary recording devices are calibrated flow restrictions that artificially control the liquid level or energy gradient of an open-channel flow system. The most common control devices are weirs and flumes. The selection of weir or flume, or a specific type of weir or flume, depended on the hydrologic characteristics of the basin, runoff measurement sensitivity requirements, and other specific study needs. For example, weirs are generally cheaper and easier to install than flumes but promote a relatively higher head loss, are not always self-cleaning, and their accuracy can be more easily affected by excessive approach velocities (1). More detailed discussions of the selection of primary measuring devices can be found in the general literature (1,2).

The second component is the instrument for recording liquid level. The primary control devices mentioned above are simply calibrated to provide volumetric flow rates as a function of liquid level. Some secondary instruments measure and record only liquid level. Others provide direct conversion of liquid level to flow rate, which is especially useful if automatic water sampling requires flow-integrated samples (discussed later in this paper).

Two general varieties of level-sensing devices were used in this study: (a) mechanical surface floats and (b) bubbler tubes. Surface floats are attached to a cable with counterweight. The liquid level is thereby recorded in conjunction with the angular position of a shaft. Bubbler tubes discharge compressed gas (air or nitrogen) into the flow stream at a fixed depth and gas flow rate. The pressure required to maintain a constant gas flow rate is proportional to liquid level. It is preferable to enclose both surface floats and bubbler tubes in attached stilling wells to dampen out minor perturbations in liquid level caused by turbulence.

Figure 3. Typical stream stilling well installation.

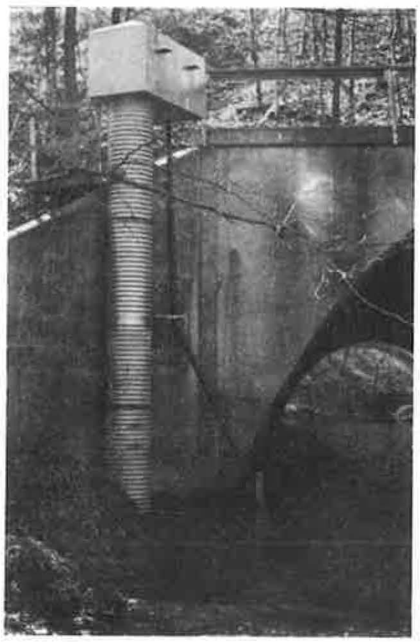
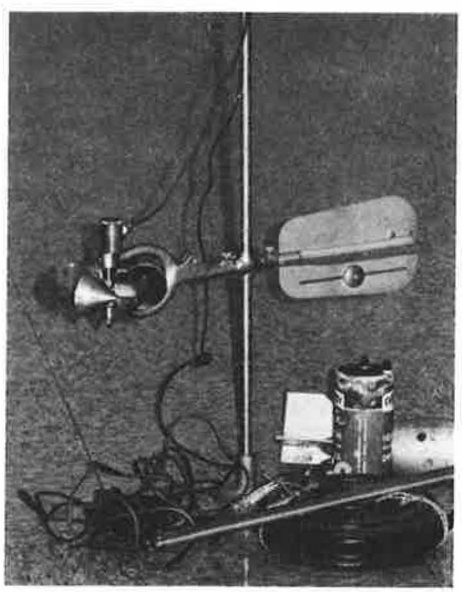


Figure 4. Gurley-type velocity meter.



HYDROLOGIC MONITORING OF RECEIVING WATER

For both lentic (standing water) and lotic receiving water systems, the most important measurement is liquid level. For lentic systems, liquid level combined with morphological characteristics and other water sources and sinks (surface and groundwater inflows, precipitation, and evapotranspiration) provides a complete water budget. For lotic systems, liquid level combined with periodic velocity measurement provides a stage-discharge curve that covers a wide range of volumetric flow rates. Mechanical surface floats enclosed in stilling wells with accompanying continuous level recorders were used exclusively for the measurement of receiving-water level in this study. Corrugated culvert sections were used for the construction of stilling wells (see Figure 3). Staff gages were securely mounted in the stream adjacent to each stilling well to calibrate the level recorders.

Gurley-type velocity meters were used for velocity measurement (see Figure 4). As the cups are rotated by liquid flow, electrical pulses are sent to a headset worn by field personnel. The number of clicks heard per unit of time can later be accurately related to flow velocity rating tables. Due to their simplicity of design, the meters have been quite effective in terms of both operational and maintenance reliability.

WATER-QUALITY SAMPLING

Sampling for Laboratory Chemical Analysis

Both automatic and manual water-quality sampling procedures were used. Automatic discrete sampling for both highway runoff and receiving water was done with Instrument Specialties Company (ISCO) samplers. These have the capability to sample in either a time or flow-volume-integrated mode. Of course, flow integration requires input from a flow measurement device that can directly relate liquid level to flow rate. The capability to sample in a flow-integrated mode greatly reduces manpower requirements for both sample compositing and chemical analyses. Other desirable features of the ISCO sampler are as follows:

1. It has weatherproof, corrosion-resistant construction.
2. It is capable of operating with either DC or AC power source.
3. It can take twenty-eight 500-mL or seven 2-L samples or several other combinations in between, in glass or plastic bottles.
4. Sampling frequency can be anywhere from 1 to 999 min or in a flow-integrated frequency mode.
5. Sample actuation can be controlled either by a level sensor-recorder or on the basis of flow volume.
6. Sample event marking is provided on corresponding flow charts to aid in sample compositing and hydrograph quality characterization.

Manual sampling devices consisted of Kemmerer and Zobell-type (3) water-quality and bacteriological samplers, respectively. These samplers allow discrete sampling at any desired depth. Also used were standard USGS depth-integrating suspended sediment samplers; i.e., liquid intake volume is proportional to flow velocity as the sampler is moved vertically through the water column (4). These sampling devices were quite simple, and few operational problems were encountered.

In Situ Measurement of Water Quality

In situ measurement of such water-quality parameters as pH, temperature, dissolved oxygen, and specific conductivity was done with common, commercially available instrumentation. Membrane-covered polarographic sensors were used for dissolved oxygen (DO) determinations (these are generally gold-silver or platinum-lead electrodes through which a small measured voltage is applied, and chemical reduction of oxygen passing through the membrane generates an electrical current at the anode). Potentiometric determination of pH is most common (a glass and calomel reference electrode is used, and the voltage across these electrodes is a measure of hydrogen ion concentration), and thermistors are generally used for temperature measurement. Conductivity measurements are generally made with Wheatstone bridge-type recorders and conductivity cells with platinum-coated electrodes firmly enclosed in plastic or glass insulation. All instruments proved quite reliable when properly calibrated with standard solutions or field titrations. DO measurement was difficult in very cold climates due to frozen membranes. Under these conditions, field titrations had to be performed.

SEDIMENT SAMPLING

Interactions between sediment and water have come to be recognized as crucial elements in determining the overall fate and impact of pollutants in receiving waters. Sediments can serve as either a source or a sink of most pollutants, depending on such factors as oxidation and reduction potentials, pH, and turbulence.

Consolidated sediments were sampled with core tubes and dredges. Core sampling was either performed manually or, if the sediments were soft enough, with a Jenkins-type automatic corer (5). This is a spring-loaded corer that, when sunk into soft sediments and activated with a messenger, simultaneously covers both ends of the tube. This allows withdrawal of a relatively undisturbed core. Disturbed sediment samples were obtained with an Ekman dredge, especially in portions of receiving waters with substrates that prohibited core penetration.

Sediments can act as significant DO sinks due to the decomposition of deposited organic matter. To determine whether highway runoff inputs affect the rate of sediment oxygen demand or to determine background oxygen sinks, special in situ chambers were constructed to measure these rates. These chambers were designed after those of Lucas and Thomas (6) and consisted of a 4.8-gal (18-L) plexiglass chamber that gave an exposed sediment surface area of 1.8 ft^2 (0.17 m^2). A DO probe was sealed in the chamber, and internal water recirculation was provided with a submersible pump. This allowed batch measurement of oxygen depletion (or accumulation) rates per unit surface area of sediment. This instrument can only be used effectively in relatively soft sediments due to sealing problems in rocky or gravelly substrate.

BIOLOGICAL MONITORING

The biological integrity of receiving water is perhaps the best indicator of the pollutional effect of storm-water runoff. Several different methods were used to collect various types of organisms from control and highway-runoff-influenced regions of the lotic systems studied to date. These included (a) Surber samplers, (b) drift nets, and (c) artificial substrates such as glass periphyton slides and Hester-Dendy samplers.

Surber samplers (see Figure 5) are designed to collect insects, larvae (macroinvertebrates), and other forms of aquatic benthic organisms from a known surface area [1 ft^2 (0.093 m^2)] of bottom substrate in shallow streams. An attached nylon or silk net with a mesh size of roughly 1000 μm collects organisms manually dislodged from the stream bottom.

Drift nets were used to quantify migrating organisms or organisms dislodged from bottom substrates. These nets had an upstream opening of 210 in^2 (1350 cm^2) and a mesh size of 363 μm. Nets are not in actual contact with the bottom as are the Surber samplers.

Artificial substrate samplers provide sites for organisms to colonize to minimize effects of different substrates on organism distribution. Two basic types were used. Periphyton (attached primary producers) samplers consisted of a box of six standard microscope slides (plain, 25x75 mm) attached near, but not in contact with, the stream bottom. Hester-Dendy samplers (see Figure 6) consisted of 14 round, tempered hardwood plates mounted in series with variable spaces. Each plate is smooth on one side and rough on the other and has a diameter of 3 in (7.5 cm) and an effective surface area of 1.4 ft^2 (0.13 m^2). These serve as efficient substrata for sampling of benthic macroinvertebrates.

SPECIAL INSTRUMENTATION FOR SOURCE AND MIGRATION STUDIES

Saltation Catchers

Saltation catchers were designed to capture "saltating" particles (movement by a series of jumps) from roadway surfaces. Measurement of roadside saltation dust provided a good qualitative estimate of the magnitude of nonsuspended particle transport compared with suspended dust transport (i.e., dust fall).

Each saltation catcher consisted of a dust-fall bucket fitted with a polyvinyl chloride pipe measur-

Figure 1. Steel noise barrier 1 m in height atop 3.7-m-high earth berm with 3:1 slope.

Figure 2. Concrete wall 0.8 m in height atop 2.2-m-high earth berm with 2:1 slope.

a 2-m-high wall would be offset by losing the extra 3-dB(A) insertion loss associated with earth berms.

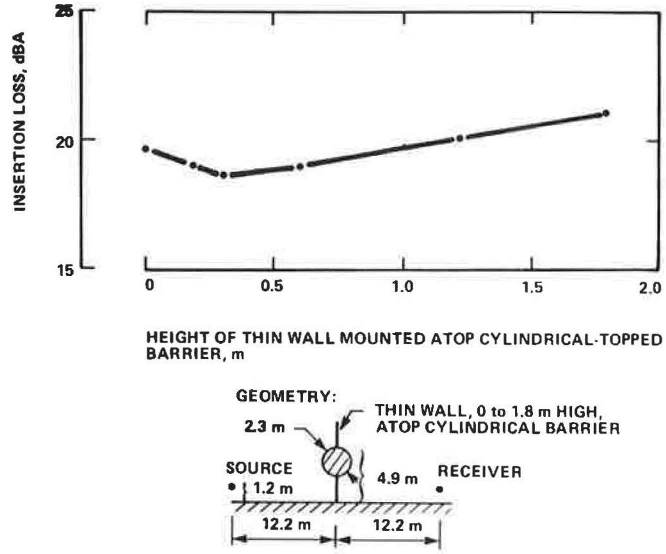
To investigate the advisability of building this wall-berm combination and to evaluate the relative acoustical performance of thin-walls and berms, a research study was undertaken that involved three phases: (a) a literature review of the effects of barrier shapes, (b) an evaluation of limited field measurements, and (c) a scale-model study. This work is part of a noise barrier research program in Ontario, the highlights of which were summarized by May (5).

REVIEW OF STUDIES CONCERNING BARRIER SHAPES

The effect of barrier shape on its performance has been studied extensively. The findings of the major studies reviewed, which compared sound attenuation by thin-walls with that achieved by wedges or berms of equal height, are summarized in Table 1 (6-19). The table does not include comparisons of thin-walls with other extraneous barrier shapes such as T-top (20) or thnadners (21) or with thin-walls that have sound-absorptive covering or sound-absorptive edges (22-24).

A review of the 13 studies listed in Table 1 does not provide an unequivocal answer to the question of whether earth berms are superior to thin-walls. Analytic investigations by several researchers concluded that thin-walls provide 1-3 dB(A) more attenuation than reflective wedges (8,17) and about the same, or slightly less, attenuation than absorptive wedges or berms (14,16). In general, scale-model studies (9,10,12,18) tend to agree with the conclusions based on analytic investigations. May and Osman (15) reported that thin-walls are less effective than wide barriers (2.4 m) with rectangular cross sections. However, the slope of the berm or wedge affects attenuation, and the vertical slope is most effective for reasons discussed later.

Figure 3. Effect of mounting thin-wall atop 4.9-m-high absorptive cylindrical-topped barrier with geometry as shown, receiver 1.2 m above ground, and grass-covered ground throughout.



Only one full-scale study directly comparing thin-walls and earth berms has been identified (11). According to that study, thin-walls are less effective than earth berms and wall-berm combinations by 2.4 and 1.3 dB(A), respectively. The finding that the wall-berm combination is more effective than the wall alone is somewhat difficult to explain, since the diffraction of sound occurs over identically shaped tops. In addition, other field studies have not found any measurable differences between the insertion loss provided by reflective thin-walls and thin-walls with a sound-absorptive covering (20,24). Additional studies comparing berms and thin-walls, conducted by the Ontario Ministry of Transportation and Communications (MTC), are discussed below.

FIELD MEASUREMENTS

Field measurements evaluating the performance of earth berms were conducted at two sites. At the first site, a 1.9-km-long highway noise barrier consisting of three sections--a concrete (reflective) wall, an earth berm, and a section combining the two (see Figure 2)--was evaluated. No difference in performance could be attributed to the different barrier shapes (25).

At the second location, sound-level measurements were conducted before the construction of an earth berm, after its construction, and, finally, after erection of a steel barrier on the top (Figure 1). The addition of the thin-wall atop the berm (see Figure 3) increased insertion loss roughly as expected from the increase in the total barrier height.

It should be recognized, however, that field evaluation of noise barrier performance is influenced by several weather-related factors and by other variables. It is difficult to measure and verify statistically a change of 1 or 2 dB(A) that occurs over the span of several months (26). This

Table 1. Review of acoustical studies on effects of barrier shapes.

Reference	Publication Date	Method of Investigation	Conclusion Regarding Effect of Barrier Shape for Structures of Equal Height
Delaney, Rennie, and Collins (6)	1972	Scale modeling, 1:30, point source (pneumatic jet)	Thin-walls are 2-3 dB(A) more effective than earth berms
Jonasson (7)	1972	Analytic investigation with some full-scale measurements	Generally there is no consistent difference between thin-walls and wedges covered by densely grown grass
Pierce (8)	1974	Analytic investigation	Thin-walls are about 1-3 dB(A) more effective than 3-sided (trapezoidal) barriers[a]
Porada (9)	1975	Scale modeling, 1:100 and 1:20, line source (air-jets)	Thin-walls are about 1-3 dB(A) more effective than wedges
Cann (10)	1975	Scale modeling, 1:80, point source (electric spark)	Thin-walls and earth berms are equally effective
Simpson (11)	1976	Field testing of completed barriers	Thin-walls were less effective than earth berms and berm-wall combinations by 2.4 and 1.3 dB(A), respectively
Ringheim (12)	1976	Scale modeling, 1:20, point source (pneumatic)	Good agreement with Pierce's method in that thin-walls are better than wedges[b]
Ivey and Russell (13)	1977	Scale modeling, 1:64, point source (electric spark)	There is no direct comparison of barrier shapes; however, Pierce's solution for wide rectangular walls (8) is supported by results of this study
Hayek and others (14)	1978	Analytic investigation	Thin-walls provide about 1 dB(A) less attenuation than absorptive cylindrical-topped wedges (berms)
May and Osman (15)	1980	Scale modeling, 1:16, point source (electric spark)	Thin-walls and wedges were not compared; thin-walls were 2.5 dB(A) less effective than wide (2.4-m) barriers with rectangular cross section
Nijs (16)	1980	Analytic investigation and scale modeling (unspecified type)	Thin-walls are 3-4 dB(A) better than acoustically hard wedges; however, if wedges are perfectly absorbing, they will not differ
Seznec (17)	1980	Analytic investigation	Thin-walls are 1-3 dB(A) more effective than 3-sided (trapezoidal) barriers
Lawther and others (18)	1980	Scale modeling, 1:5, line source (tone bursts using loudspeaker array)	Thin-walls are more effective than berms unless berms are covered by an absorptive material (grass), in which case they are equally effective

Note: Comparisons are restricted mainly to reflective thin-walls ⊥, wide barriers such as wedges Λ, and berms ⌒. Thin-walls with sound-absorptive covering or sound-absorptive edges are not included.
[a]Based on Seznec's substitution (17) into Pierce's formula (8).
[b]Similar results were obtained for trapezoidal barriers by Ringheim (19).

is one of the reasons why the scale-model study was undertaken.

SCALE-MODEL STUDY

Under the aforementioned circumstances, acoustical scale modeling was an indispensable tool. A multiplicity of interacting factors made it difficult to obtain an exact analytic answer to the question posed, and full-scale experiments were too expensive. In particular, the diffraction by absorptive wedges on finite impedance ground is influenced by the interaction of the following factors:

1. Source-barrier-receiver geometry;
2. Interference (and reflections) due to ground cover that may not be uniform;
3. Scattering and absorption losses on the barrier top; and
4. Finite barrier thickness or double-edge diffraction.

Scale modeling also permitted rapid and inexpensive situational changes (barrier height and shape and ground cover) and their rapid evaluation. Because the study was conducted indoors, weather-related factors were eliminated, which enhanced repeatability and the accuracy of the results.

The scale-model study was conducted at the MTC scale-modeling facility. The equipment and materials used were described by Osman (27,28) and were used extensively before (15). Similar equipment has been used by other researchers (10,13). Briefly, the acoustical hardware consisted of a high-voltage spark as a noise source, a 0.13-in microphone receiver, filtering and processing instrumentation, and an oscilloscope.

The model scale was 1:16 and was applied to both spatial variables and the A-weighted traffic noise spectra (27). In addition, the sound-absorptive properties of the model materials (e.g., grassland, pavement, and surfaces of reflective barriers) were tested to ensure that they appropriately modeled the real-life situations on the 1:16 scale. All dimensions and frequencies quoted in this paper are full-scale equivalents. Therefore, the model dimensions were 16 times smaller and the model frequencies 16 times higher than the full-scale values.

The source-barrier-receiver geometry (see Figure 4) and materials were selected to model typical highway situations (in plan view, the line between the source and the receivers is perpendicular to the barrier alignment). The model berms had a slope of 2:1 and were 1.2 m wide across the top. The flat tops with sharp edges on the sides were rounded to reflect the shape of actual earth berms. To simulate grass cover, the model berms, constructed of plywood, were completely covered with an appropriate, acoustically soft material (fiberboard). In contrast to the acoustically soft material used on the berms, acoustically hard material (painted plywood) was used to model the surfaces of thin-walls.

For simplicity, only a single point source was used in the study. This should not limit the validity of the results, since the relative effect of barrier shape is indicated by a point source even if the absolute insertion loss values may not be (15).

The height of the point source above ground was modeled at 1.2 and 2.4 m. The source height of 2.4 m is usually associated with the source height of heavy (diesel) trucks (4); the 1.2-m height may be considered to be an equivalent source height for highway traffic flow containing about 10 percent trucks.

The basic barrier configurations evaluated are given in Table 2. Most of these barrier shapes were tested for the three basic geometries defined in Figure 4. However, for illustrative purposes, only a sample of the results is presented here.

Figure 5. Stream macroinvertebrate sampler (Surber).

Figure 6. Hester-Dendy macroinvertebrate sampler.

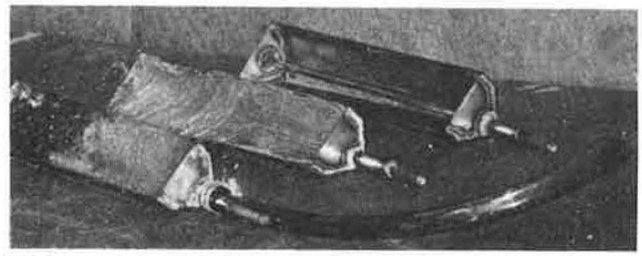
ing 28.5 in (72.4 cm) high and 6 in (15.24 cm) in diameter. This pipe has a vertical sampling slot 19 in (48.26 cm) high and 1.0 in (2.54 cm) wide that faces the roadway and captures the saltating particles, which have a height interval of 12-30 in (30-76 cm). Figure 7 shows a typical saltation catcher. The capture efficiency of this device is approximately 50 percent (7).

Zero-Tension Lysimeters

Zero-tension lysimeters measure groundwater percolation out of the major rooting zone (usually the top soil layer). They were used in the source and migration studies to estimate the loss of various chemical constituents from the highway system due to groundwater percolation. Figure 8 shows a lysimeter system. Plastic 1-gal (3.7-L) collection bottles store water that has percolated through the rooting zone into special stainless steel troughs. These troughs have a collection surface 12 in (30.5 cm) long and 2.1 in (5.4 cm) wide. Within the trough are two parallel stainless steel rods in slight contact with an overlying mesh screen, which has a thin layer of fiber wool to keep soil from clogging the screen. The rods in contact with the mesh

Figure 7. Saltation catcher.

Figure 8. Zero-tension lysimeter.



screen negate the surface tension of water percolating through the screen and provide effective capillary drainage.

Overall, lysimeters appeared to yield a high recovery efficiency provided that they were installed properly (i.e., if there was good contact between the lysimeter and the overlying soil layer) in well-drained soil. Using measured flows to convert sample percentages to areal mass loadings provided reasonable estimates of pollutant migration.

Sweeping and Flushing Studies

Sweeping and flushing studies were performed to quantify the accumulation and distribution of pollutants on the highway surface, seasonal variations in the surface pollutant load, and the particle size distribution of accumulated solids. This information was needed to establish the relations between pollutant deposition on the highway surface (including deposition from vehicles, the atmosphere, and highway maintenance activities), the highway surface load, and removal processes (including runoff, blowoff, and, where performed, highway sweeping). Test sections 50 ft in length in the distress median and travel lanes were swept and flushed to characterize the highway surface pollutant load. Test sections with visual accumulations of dust and dirt were first swept. All test sections were then wet-vacuumed with a standard carpet cleaning rinse and vacuum machines. Dry samples (swept) and liquid samples (wet-vacuumed) were analyzed separately for pollutant concentration. To date, quite good agreement has been observed between surface loads determined by sweeping and flushing and those calculated in deposition-removal mass balances. In addition, tests have shown that the rinse and vacuum machines do not add contaminants through the spraying system.

CONCLUSIONS

This paper documents instrumentation requirements for a comprehensive, multifaceted evaluation of water-quality impacts and spans several phases of investigation. The selection of instrumentation requirements obviously depends on both the required complexity of the monitoring program and available funding. As part of the research program described in this paper, separate report volumes have been, or will be, prepared concerning procedural guidelines for water-quality impact assessment and detailed monitoring guides for conduct of field programs. These manuals are designed to serve the needs of highway department personnel by providing simple and straightforward procedures in design, planning, conduct, and evaluation of proposed sampling programs and water-quality investigations.

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Are Earth Berms Acoustically Better Than Thin-Wall Barriers?

J. J. HAJEK

The two most common highway noise barrier structures are earth berms and thin-walls. Yet the relative acoustical performance of these barriers is not well understood. Previous analytic, scale-model, and full-scale studies, comparing the acoustical effectiveness of thin-walls with that of berms and wedges, are reviewed. Additional data obtained by full-scale measurements, and in particular by a 1:16 scale-model study, are presented. The source-barrier-receiver geometry and model materials used were selected to simulate typical highway situations. Preliminary results indicate that, contrary to a recommendation in the Federal Highway Administration Highway Traffic Noise Prediction Model, thin-wall barriers and earth berms of the same height are about equally effective in reducing noise. In addition, the acoustical effectiveness of combining a wall with an earth berm was found to be quite similar to that of using thin-wall barriers alone. The practice of erecting relatively low walls on top of earth berms was found to be acoustically sound.

Reflective thin-walls, earth berms, and combinations of the two, are the most common highway noise barriers. Their relative nonacoustical aspects, such as cost, maintenance, right-of-way requirements, and aesthetics, are well understood (1), but their relative acoustical performance is not so clear. Whereas some highway noise prediction methods assume that they perform equally (2,3), the widely used Federal Highway Administration (FHWA) Highway Traffic Noise Prediction Model (4) asserts that earth berms provide 3 dB(A) higher insertion loss than do thin-walls of the same height. This difference in acoustical performance has been attributed to absorption or edge effects.

The higher insertion loss assumed for earth berms could lead to an important consequence: If the shape of the earth berms (presumably the cause of the increase in the insertion loss) is changed by erecting a thin-wall on its top, the 3-dB(A) benefit provided by the berm top may be lost. Figures 1 and 2 show two wall-berm combinations. Such combinations are quite common in many states. Relatively low walls have been added to improve performance in comparison with earth berms alone. But do they?

This concern is illustrated in Figure 3, which is based on our results from scale-model testing. Details of the scale-model testing, such as instrumentation, methodology, and additional results, are discussed later in this paper. For now, Figure 3 is intended only to illustrate the effect of mounting a thin-wall atop a barrier with an absorptive top.

According to Figure 3, mounting a thin-wall atop a highly absorptive barrier can actually reduce insertion loss. Only after the thin-wall is raised to the height of 1.2 m is the reduction in the insertion loss--caused by violating the absorptive cylindrical shape--recovered by the increase in barrier height. The question arises, Can the same phenomenon occur if a thin-wall barrier is erected atop an earth berm?

This question has become acute in Ontario since a proposal was made to build a thin-wall, approximately 2 m in height, atop an existing 3-m-high earth berm. The berm is already providing some insertion loss [about 6 dB(A)], so the rate of increase in the insertion loss with additional barrier height would be about 1.5 dB(A)/m. However, the desired 3-dB(A) increase in the insertion loss expected from adding

Figure 4. Source-barrier-receiver geometry: grass-covered ground shown by hatched area and hard ground by heavy line.

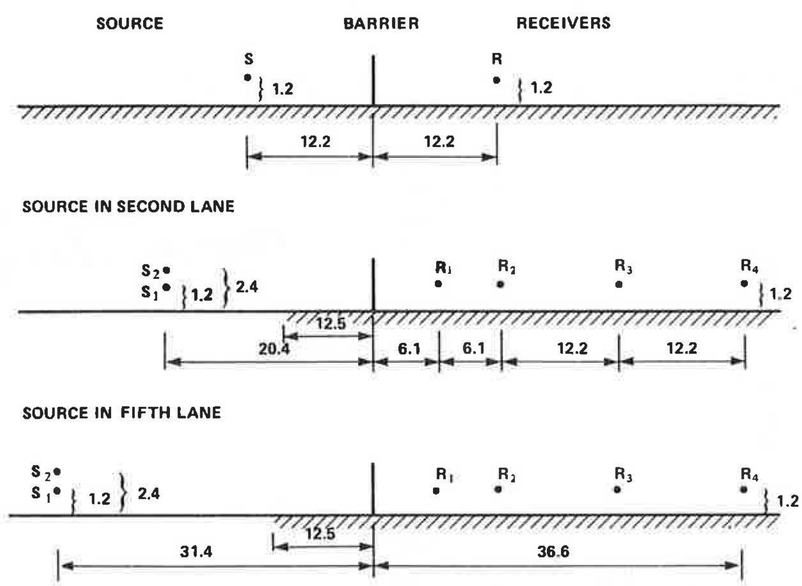


Table 2. Barrier shapes evaluated.

Barrier Height (m)	Type
3	Conventional barrier with vertical, reflective-surfaced walls 0.16 m thick
	Earth berm (rounded)
	Earth berm with distinct edges on top
4.9	Conventional barrier with vertical, reflective-surfaced walls 0.16 m thick
	Earth berm (rounded)
0.3-1.8	Conventional barrier atop 3-m-high earth berm
	Conventional barrier atop 3-m-high berm with sound-absorptive top

Barrier Height of 4.9 m

The acoustical performance of three different 4.9-m-high barriers--namely, a conventional thin-wall barrier, an earth berm, and a wall-berm combination--is compared in Figure 5. The source is in the fifth lane, 2.4 m aboveground, as detailed in Figure 4.

The insertion loss (i.e., the difference in sound level between the situations with and without the barrier, with no change in ground cover and source-receiver geometry) obtained for the three barrier shapes was quite similar; the lowest overall insertion loss was measured for the earth berm. The lower insertion loss provided by the earth berm in comparison with that of the thin-wall of equal height has been reported earlier (6) (Table 1) and can be tentatively attributed to two factors:

1. Sound waves diffracted into the shadow zone can also reach a receiver by reflection from the ground (29). In the case of earth berms, diffracted waves may also be reflected from the slope of the berm in the shadow zone.
2. Tilting the slope of a wedge while keeping its top at the same position alters its insertion loss because the position of the image source, with respect to the slope, shifts. As the wedge is spread out more (i.e., as the angle of tilt increases), the position of the image source shifts toward the base of the wedge and thus sound levels in the shadow zone increase. This is shown schematically in Figure 6.

The negative effect of these two factors on insertion loss is mitigated by the sound-absorptive properties of the berm surfaces and by the scattering and absorption losses taking place along the berm top.

No systematic difference between the conventional thin-wall barrier and the wall-berm combination was observed.

Barrier Height of 3 m

Insertion losses measured for 3-m-high barriers--a conventional barrier, an earth berm, and an earth berm with an "artificially" high sound-absorptive top--are shown in Figures 7 and 8. The two berms were identical except for a urethane foam used on the top of the absorptive berm. As mentioned before, the earth berm was completely covered with a special fiberboard material to simulate grass cover.

The berm with the sound-absorptive top was a somewhat "artificial" structure because the sound-absorptive property of the top (which had a noise reduction coefficient of 0.75) would be difficult to duplicate in the field. This structure was evaluated mainly to test whether and how the performance of a berm can be improved by using an absorptive material on its top.

The results in Figures 7 and 8 are based on the source height modeled 1.2 and 2.4 m aboveground, respectively. Both figures show that the 3-m-high conventional thin-wall barrier again slightly outperforms its earth berm counterpart. The replacement of the grass-covered top by the more absorptive top improved the berm performance by about 2 dB(A) for the source-barrier-receiver geometries used. This suggests that the absorptive material on the barrier top may be a more important influence of diffraction than the barrier shape.

Wall-Berm Combination

The effect of mounting a thin-wall conventional barrier atop an earth berm is shown in Figure 9. The height of the conventional barrier ranged from 0.3 to 1.8 m; the source-barrier-receiver geometry used is shown schematically in Figure 9 and is detailed in Figure 4.

Figure 5. Comparison of different 4.9-m-high barriers with source in fifth lane 2.4 m above ground.

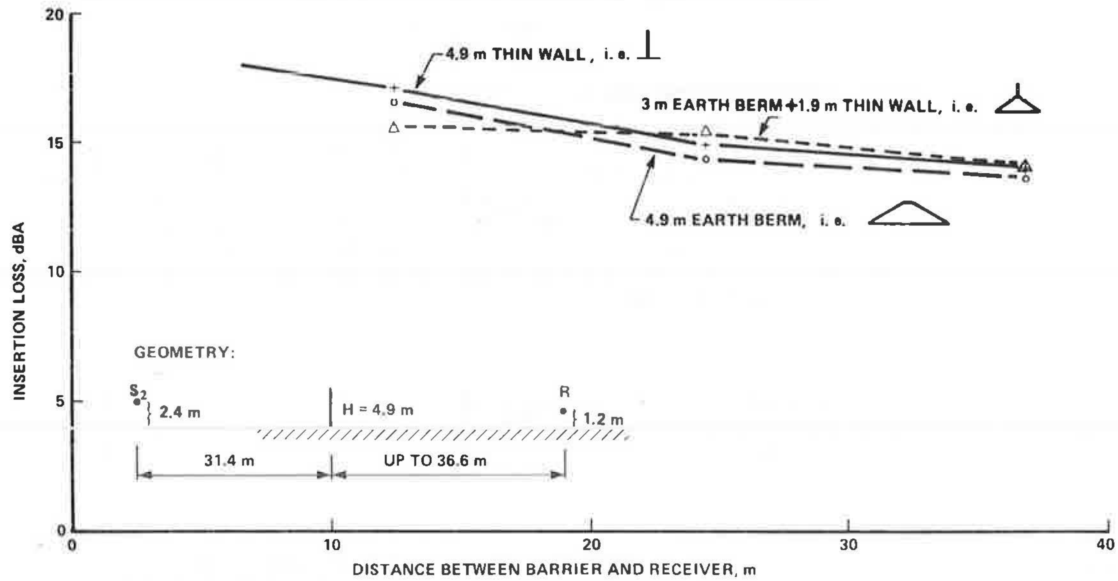


Figure 6. Effect of tilting the slope of a wedge: (left) small angle of tilt and (right) large angle of tilt.

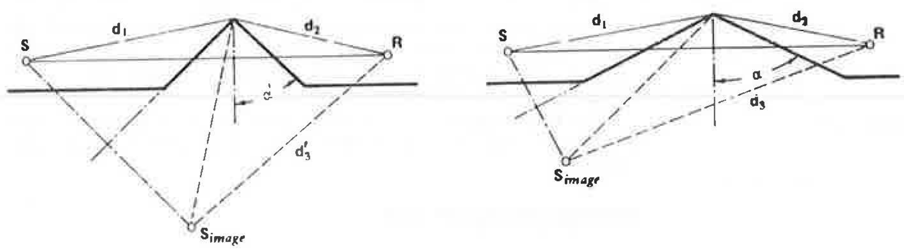


Figure 7. Comparison of different 3-m-high barriers with source in second lane 1.2 m above ground.

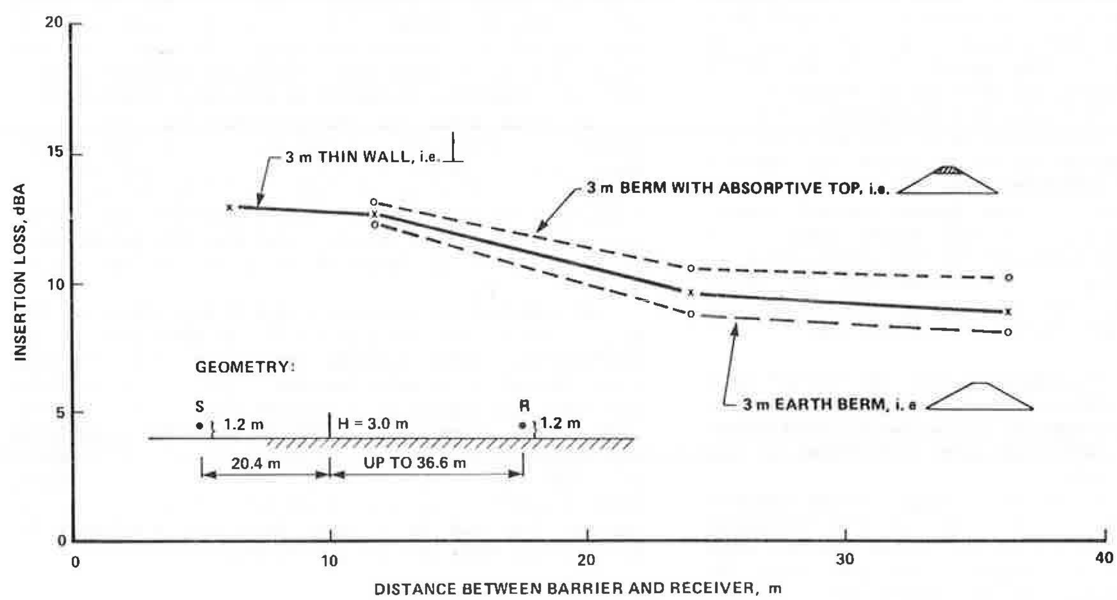


Figure 8. Comparison of different 3-m-high barriers with source in second lane 2.4 m above ground.

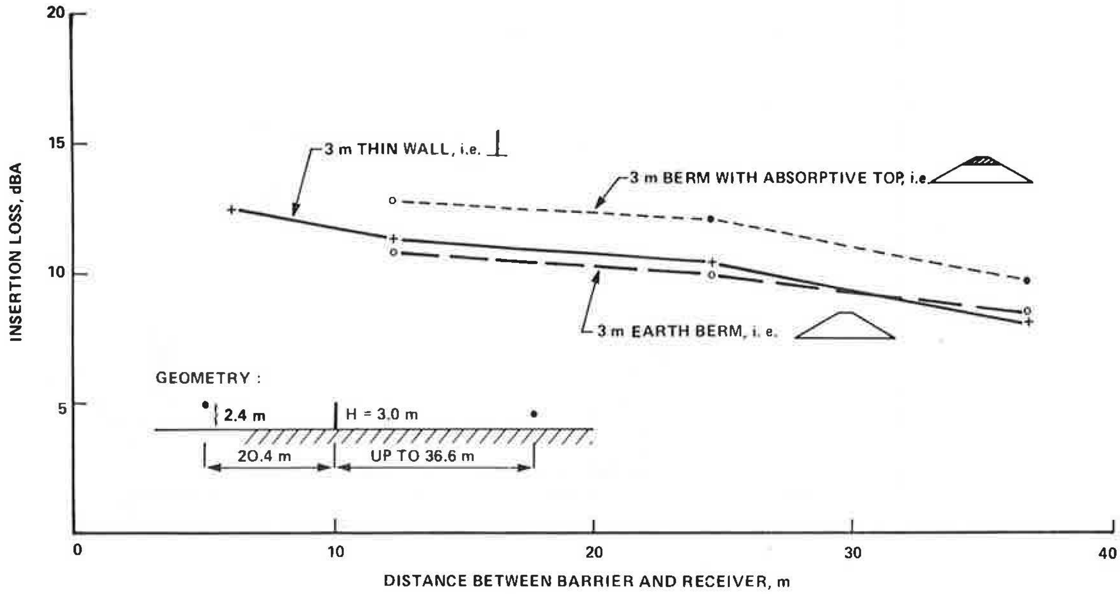


Figure 9. Effect of mounting thin-wall atop 3-m-high earth berm.

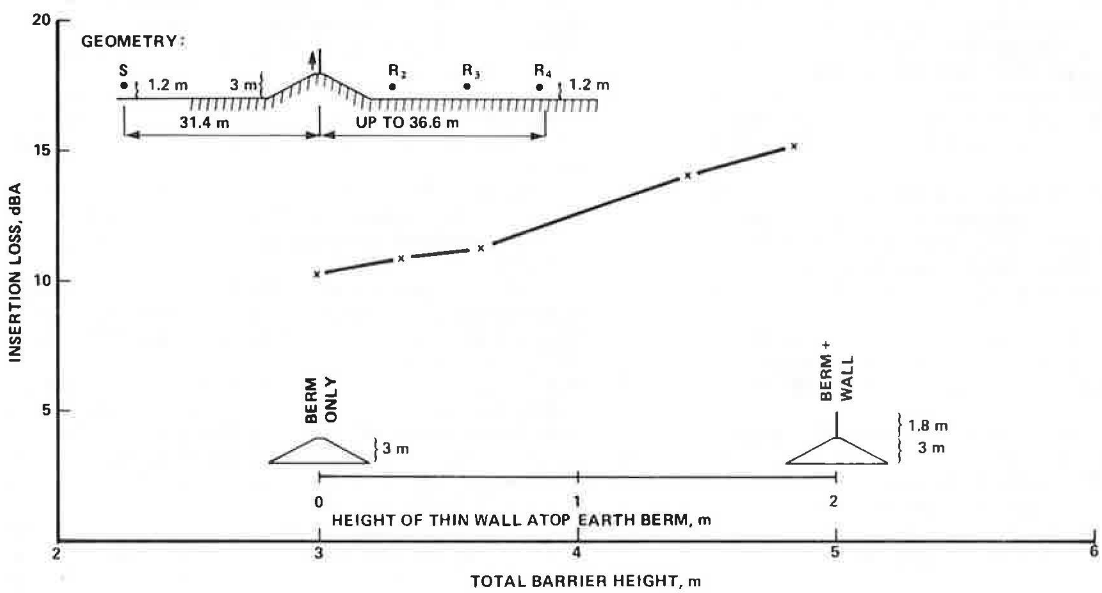


Figure 9 shows that insertion loss increased with the increased height of the thin-wall barrier atop the earth berm. The insertion loss shown is average for 12.2, 24.4, and 36.6 m behind the barrier (the source is in the fifth lane, as shown in Figure 4). The rate of increase in insertion loss was not quite uniform, being somewhat lower initially. Nevertheless, the erection of a thin-wall atop an earth berm consistently improved the insertion loss of the earth berm alone.

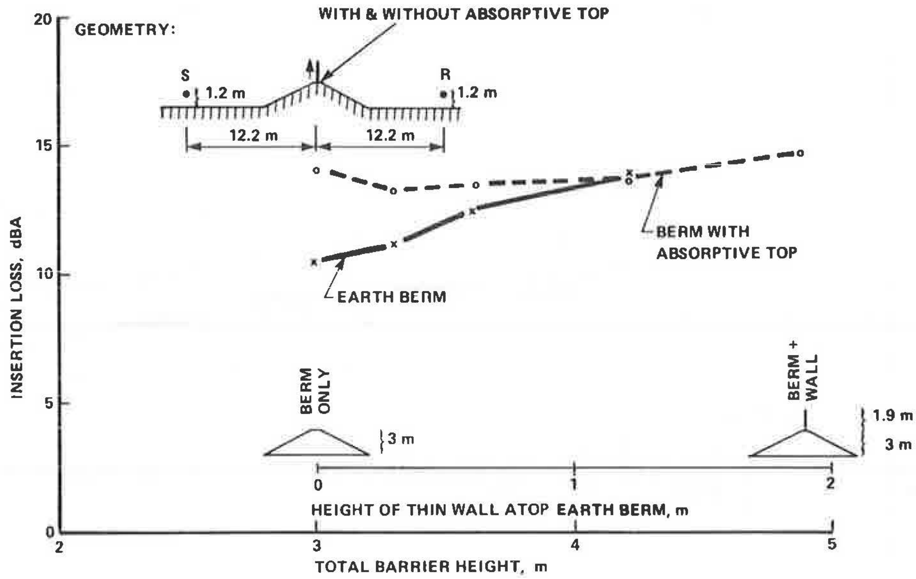
A different picture emerges if the thin-wall is mounted atop the berm with the sound-absorptive top as in Figure 10. (Source and receiver are 1.2 m above grass-covered ground, and the receiver is 12.2 m behind the barrier.) For the geometry used, this structure provides about 3 dB(A) higher insertion loss than its earth berm counterpart. Mounting a thin-wall atop the absorptive-topped berm does not initially increase insertion loss, since the beneficial effect of the absorptive top is lost and is not fully recovered by the increase in the total barrier height. However, as the height of the thin-wall increases to about 1.2 m, the effect of the absorptive top diminishes and the combination of the wall and the absorptive-topped berm and the combined wall and earth berm perform equally.

CONCLUSIONS

The following conclusions, based on the data presented in this paper, are intended mainly to stimulate interest in the relative acoustical performance of the two most common barrier shapes: reflective thin-walls and earth berms.

1. Reflective thin-walls, earth berms, and the

Figure 10. Effect of mounting thin-wall atop earth berm with sound-absorptive top.



combination of the two are about equally effective (in terms of insertion loss) provided the berm is covered by grass or similar material. Actually, for the majority of source-barrier-receiver geometries investigated in the scale-model study, a slightly lower insertion loss [usually less than 1 dB(A)] was measured for earth berms than for thin-walls of the same height. This difference may not have practical significance since it also depends on the sound-absorptive properties of the material used to model the grass-covered ground.

2. The acoustical performance of an earth berm can be increased by placing sound-absorptive material on its top. On the other hand, placing bicycle paths or walkways on earth berms that serve as noise barriers would make the top reflective and should be avoided.

3. The erection of relatively low thin-walls atop earth berms is acoustically justified since it increases the insertion loss beyond that of earth berms alone.

4. Research on the effect of barrier shapes should be continued, and full-scale testing should be emphasized. An improvement of several dB(A) attributable to barrier shape may be considered significant since the insertion loss provided by barriers in the field is usually in the 5- to 10-dB(A) range.

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Quality Control for Environmental Measurements

EARL SHIRLEY

A general overview of the quality assurance program for environmental measurements practiced by the California Department of Transportation is presented to illustrate current practice. The discussion, which is general rather than detailed, places the program in perspective and concentrates on equipment used to measure noise and air pollutants and the associated instrumentation and procedures for calibration. A quality assurance program is necessary to ensure the validity and reliability of environmental measurements. Traceability of instrument calibration to an authority such as the National Bureau of Standards is important. The program involves fairly complex instrumentation systems and requires expert technical personnel and good documentation.

One of the fundamental responsibilities of management is the establishment of a continuing program to ensure the reliability and validity of any measured test value. The California Department of Transportation (Caltrans) has been following such a program for a number of years to provide assurance that test data involving materials such as asphalt, soils, and concrete are valid. To achieve this, the department has been participating in national programs sponsored by organizations such as the American Society for Testing and Materials, the Materials Reference Laboratory of the American Association of State Highway and Transportation Officials, and the Cement and Concrete Reference Laboratory of the National Bureau of Standards (NBS) and has been carrying out its own quality control program.

The addition of environmental testing responsibilities to Caltrans' normal duties brought about a need for a quality assurance program (QAP) in those areas also. Specifically involved were test data relating to air quality, water quality, and noise and vibration. Some of the benefits that would result from such a program were seen to be

1. Increased confidence in decisions based on environmental data;
2. A solid, defensible position in the event of litigation involving environmental data;
3. Uniformity in techniques and procedures for the use of instruments and their calibration and for data analysis; and
4. Unqualified acceptance of Caltrans test results by other organizations.

With the need for a QAP identified, it was necessary to decide on the program type and scope that would best fit Caltrans needs. Three basic alternatives were examined:

1. Develop a full "standards laboratory" capability in-house,
2. Make use of equipment manufacturers' regional service centers, or
3. Develop an in-house capability similar to that of a manufacturer's regional service center.

The first alternative was judged to be too costly. For example, the noise portion would require either the rental or the construction of an anechoic chamber. It was also felt that full-scale testing of environmental measurement equipment in accordance with American National Standards Institute, U.S. Environmental Protection Agency (EPA), and NBS procedures was neither cost effective nor necessary for Caltrans operations.

The second alternative, based on previous experience, would lead to long "turn-around" times (up to three months) and tend to discourage regular calibration. In addition, since most of the regular project-related environmental analyses are done by personnel in the 11 Caltrans districts, this alternative would not free headquarters personnel from duties in coordination, documentation, field review, and training.

The third alternative was felt to be the most suitable solution. Much of the instrumentation necessary for such a program was already on hand. Other advantages included information exchange, corrective action on equipment and test procedures, and, in some cases, concurrent certification of testing technicians.

There were two general goals: (a) compliance with state and federal mandates for environmental testing and (b) ensuring the reliability and validity of environmental test results. The following objectives would lead to attainment of those goals:

1. Develop procedures for administration and documentation and define areas of responsibility,
2. Develop procedures for maintaining environmental measuring equipment in calibration and ensure that the instruments used for the calibration are traceable to NBS,
3. Develop procedures for certifying and auditing personnel assigned to perform field and laboratory environmental measurements, and
4. Frame a QAP to include and to implement the above procedures and provide the necessary support in terms of competent technical assistance and adequate calibration instruments.

Keeping these goals and objectives in mind, Caltrans developed QAPs for noise (1), air quality (2), and water quality (3). This paper summarizes the equipment and procedures used in the noise and air quality programs.

ELEMENTS OF A QAP

Framing a QAP to include the necessary administrative and technical procedures led to selection of the following program elements:

1. Program administration includes coordination; scheduling; preparation and transmittal of quality control samples; collection and analysis of data and reporting results; dissemination of instructions, information, and changes in policy and procedures; identification and correction of systematic deviations, bias, and erratic results; preparation of test methods; and equipment purchase and repair.
2. Training, although an administrative function, is treated as a separate element because of its importance. Training covers sampling, field testing, equipment calibration, equipment storage, laboratory testing, and record keeping. It consists of formal in-house courses, on-the-job training, academic courses, and manufacturers' seminars.
3. Instrument calibration involves establishing calibration standards traceable to NBS, determining equipment characteristics to be tested and applicable tolerances, and determining an optimum calibration interval. It is discussed in some detail for both noise and air quality instrumentation in the following sections of this paper.
4. Operator certification and procedural audits recognize that, in addition to calibrated instruments, valid environmental measurements depend to a great degree on the competence of the people who operate the instruments and analyze the data and the degree to which they follow established procedures. Although training is available, it is difficult to establish a policy for periodic retraining. As a result, inexperienced or "rusty" technicians can be involved in environmental measurement and analysis. Due to these inadequacies, it is necessary to evaluate and review the actual measurement procedures on an annual basis.
5. Documentation is an essential feature of a QAP. The bulk of the documentation concerns equipment calibration and is detailed and technical in nature. Documentation for the procedural audit consists of a simple performance certificate.

INSTRUMENT CALIBRATION

Noise

The basic goals of instrument calibration were to detect serious malfunctions or deficiencies in District equipment and to establish calibration documentation attesting to the long-term stability of the equipment. Of the many tests that are commonly performed on noise-measuring equipment, the following were determined to be necessary in terms of disclosing basic equipment malfunctions yet simple enough to be incorporated in a portable calibration program:

1. Calibrators--(a) Calibrator output level and (b) calibrator output frequency;
2. Sound-level meters (SLMs)--(a) Meter scale linearity, (b) 10-dB step attenuator accuracy, (c) A-weighting, (d) internal noise, and (e) SLM output voltage; and
3. Graphic level recorders (GLRs)--(a) Accuracy of response, (b) overshoot-undershoot and creep, (c) chart speed, and (d) writing speed.

With the specific tests decided on, it was necessary to select calibration instrumentation to perform the tests. Table 1 lists this equipment, and Table 2 relates the equipment to the test and gives the allowable tolerances.

Traceability to NBS is achieved by periodically submitting the specially designated "laboratory standard" microphone to NBS for calibration. On its return, this "calibrated" microphone is used to define the sensitivity of another microphone used in the calibration of Caltrans district calibrators. This microphone is referred to as a "working standard" microphone. This program of sending a laboratory standard microphone directly to NBS provides the highest level of NBS traceability (see Figure 1).

If continual calibration capability is to be maintained, it is necessary to have two laboratory standard microphones for cross calibration so that, when one is at NBS for calibration (turnaround time is about three to five weeks), the other is available for use. The Caltrans Transportation Laboratory (TransLab) purchased two 1-in laboratory standard microphones at a cost of about $700 each. Because they could be used on existing Translab equipment, the total cost of setting up a laboratory standard measuring system was minimized. The total cost of all equipment required for this type of system would be about $6000.

Another key instrument for which NBS traceability is necessary is the standard voltmeter used by TransLab in this program. This meter must be a true RMS digital type and is sent annually to its manufacturer for calibration against standards directly traceable to NBS. The instrument is a portable AC/DC unit with an accuracy of 0.05 dB or better and was approved by NBS for our purposes.

The other TransLab standard for which periodic calibration is required is the standard frequency counter. There are two existing standards that provide limited degrees of calibration: (a) tuning forks maintained by the electrical section of TransLab and certified by their manufacturer as to their exact frequency and (b) a 1000-Hz pure tone provided by the telephone company and accessible by dialing.

Although the above references can be used for periodic checks of the frequency counter at a limited number of frequencies, a thorough NBS-traceable calibration of the counter should be performed at least every two years. This is available from the manufacturer. Two years is considered a reasonable interval because the instrument is highly accurate and stable.

The calibration data for the NBS-calibrated laboratory standard microphones are used in adjusting the input sensitivity of the laboratory standard SLM/preamp/microphone system to estabalish a "true-reading" system. The working standard system is then compared with the laboratory standard system by using a pistonphone transfer standard. The pistonphone is a highly precise mechanical device that generates a pure tone of 250 Hz at a sound pressure level of 125 dB and has an accuracy of ±0.2 dB.

Table 1. Calibration equipment for noise instruments.

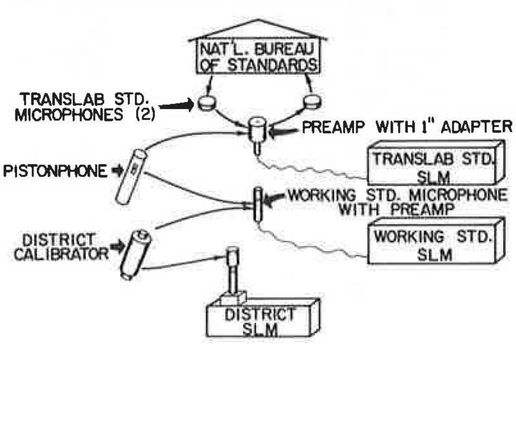
Category	Equipment No.	Equipment
Laboratory standard	1	SLM/preamp/microphone system
	2	Frequency counter
	3	Digital voltmeter
	4	Pistonphone transfer standard
Working standard	5	SLM/preamp/microphone system
Ancillary equipment	6	Acoustical signal generator (signal generator, amplifier, attenuator set, "slave" calibrator)
	7	Dummy microphone
	8	Stopwatch and ruler

Table 2. Tests and equipment for noise instrument calibration.

Test Item	Equipment No.[a]	Tolerance
Calibrator		
Output level	5	±0.5 dB
Output frequency(s)	2,5	±0.2 dB @ 1000 Hz
Sound-level meter		
Meter scale linearity	2,6	±0.4 dB @ 250, 500, and 1000 Hz
10-dB step attenuator accuracy	6	±0.5 dB @ 250, 500, and 1000 Hz
A-weighting	6	Depends on octave band
Internal noise	7	<30 dB
SLM output voltage	3,6	±0.4 dB for 5-dB steps @ 250, 500, and 1000 Hz
Graphic level recorder		
Accuracy of response	6	±0.5 dB
Overshoot-undershoot and creep	–	±1.0 dB (20-dB step)
Chart speed	8	±10 percent between runs
Writing speed	8	

[a]From Table 1.

Figure 1. NBS traceability: noise instruments.



Tests of SLM characteristics are made electroacoustically; that is to say, the SLM is subjected to acoustical input signals so that the microphone, as well as the meter electronics, is tested. It would be possible to remove the SLM microphone and input electrical signals directly to the SLM, but separate microphone tests would then have to be conducted.

Ideally, electroacoustical tests on SLMs are done in a reflector-free environment such as an anechoic chamber. Because of the trouble and expense involved in constructing one of these chambers, and because of the nature of the Caltrans calibration program, a relatively simple system was devised that would allow a controllable acoustical signal to be input to an SLM. This system requires the use of what will be called a "slave" calibrator, which is nothing more than a calibrator driven externally by a signal generator. The calibrator in this situation is simply serving as a transducer or loudspeaker, converting an electrical signal into an acoustical one. An infinite variation of signal amplitudes and frequencies can therefore be obtained by adjusting the signal generator.

The output frequencies of the signal generator are measured by the standard frequency counter. Placed in series with this system is an attenuator box capable of providing 10- and 1-dB step changes in signal level. Also in series with this array is an amplifier for increasing the output range of the system (see Figure 2).

Such a system as this can only be useful if its acoustical output can be accurately characterized with respect to frequency response. Here again, the laboratory standard microphone is required. By mounting the "slave" calibrator on the standard microphone and "sweeping" the frequency spectrum on the signal generator controls, the frequency response of the slave system can be exactly defined. It must be stressed that this slave system should not be used unless frequency response data are first obtained by using a standard microphone.

Air Quality

During the early years of air pollution monitoring, the methods of analysis were heavily based on wet chemical tests and were usually performed by chemical laboratory personnel. It is understandable that many early calibration methods and standards were derived from involved chemical reactions. Many relied on color changes of solutions exposed to air samples by fritted glass bubblers. The color changes were read by photometers at specific wavelengths characteristic of the reaction being studied. Calibration was achieved by mixing the pollutant in question in known quantities with the solution and documenting the color change. These basic methods were complex, time consuming, and very susceptible to conditions that existed when the mixing and testing were performed. They were also highly dependent on the quality of the chemicals and personnel involved.

New developments in instrumentation methods, coupled with the uncertainties involved in chemical analysis and the discrepancies between tests performed by different monitoring groups, brought new means of analysis and calibration to the forefront. The time period from the early 1970s to the present has seen the development of permeation tubes, flow dilution systems, especially lined cylinders, standard reference materials (SRMs) supplied by NBS, and the sensitive new gas phase reaction and optical method instruments.

These new instruments use the following methods of measurement:

1. Nondispersive infrared absorption for carbon monoxide (CO) measurement,
2. Chemiluminescence for ozone (O_3) and nitric oxide (NO_x) measurement,
3. Ultraviolet light absorption for O_3 measurement, and
4. Gas chromatography with flame photometric and flame ionization detection for sulfur dioxide (SO_2) and total hydrocarbon (THC) measurement, respectively.

These new detection methods brought with them sensitivities of 0.5 ppm or less and accuracies of 1-2 percent. This stretched the limits of past calibration methods and emphasized the need for standardization of procedures, tightening of tolerances on span gases, documentation of instrument performance, and in-use surveys of precision and linearity.

Calibration of these instruments, not including some of their internal preparatory calibration procedures, was restricted to certain tests, and appropriate calibration instrumentation was selected to perform these tests. Table 3 lists the equipment, and Table 4 relates the equipment to the individual test.

Traceability to NBS is achieved through the use of SRMs as laboratory standards (see Figure 3). These are in the form of bottled gases with precisely measured concentrations of the pollutants of interest. An exception is ozone. The laboratory standard, which in this case is also a working standard, is an ozone comparator that is verified quarterly with a reference photometer in the California Air Resources Board (CARB) Laboratory.

Two other working standards receive regular verification by CARB. The gas calibration system receives a check of the flow dilution settings and is supplied with a verified multigas cylinder that contains high concentrations of CO, NO_x, THC, and methane (CH_4). The dry gas flow meter is calibrated by use of a positive displacement instrument with a mercury-sealed piston. In addition, the digital multimeter is calibrated by NBS-traceable standards in the Caltrans electronics laboratory.

The calibration program for district air quality instrumentation begins with reference analyzers that are maintained in the Caltrans central laboratory under controlled environmental (temperature and humidity) conditions. These instruments are calibrated on the NBS SRMs. Individual cylinders of gases with pollutant concentrations determined by these analyzers are then used as transfer standards to calibrate upscale readings on field analyzers. Linearity checks are made with the gas calibration system.

Quality control is enhanced by a periodic interlaboratory testing effort that uses "unknowns" prepared by the central laboratory.

Figure 2. Calibration signal production.

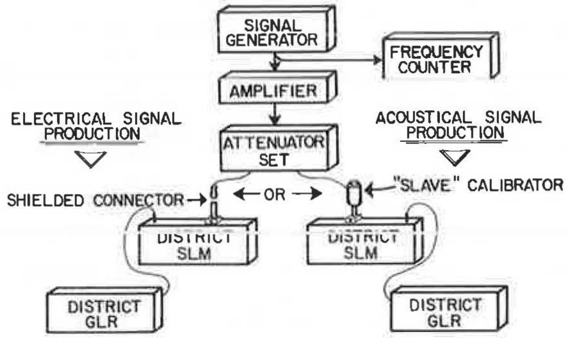


Table 3. Calibration equipment for air quality instruments.

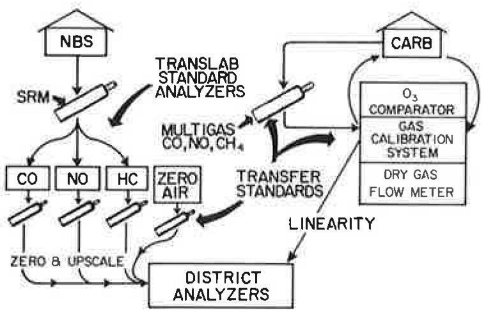
Category	Equipment No.	Equipment
Laboratory standard	1	NBS SRM for CO
	2	NBS SRM for NO_x
	3	NBS SRM for CH_4
	4	Reference analyzers (CO, NO_x, HC, O_3)
	5	Zero air system
Working standard	6	Transfer gas cylinders (CO, NO_x, CH_4, zero air)
	7	Ozone comparator
	8	Gas calibration system (includes multigas cylinder)
	9	Digital multimeter
	10	Dry gas flow meter
	11	Bubble flow meter
Ancillary equipment	12	Digital temperature sensor
	13	Strip chart recorder
	14	Stopwatch

Table 4. Tests and equipment for air quality instrument calibration.

Analyzer	Method	Calibration Test	Equipment[a]
CO	Nondispersive infrared absorption	Instrument bias	9, 13
		Zero reading	6, 8, 9, 13
		Upscale reading (span)	6, 9, 13
		Sample flow	10, 14
		Output linearity	8, 9, 13
O_3	Ultraviolet absorption	Zero reading	6, 8, 9, 13
		Catalytic O_3 scrubber	7, 9
		Upscale reading (span)	7, 9, 13
		Sample flow	10, 14
		Output linearity	7, 9, 13
NO_x	Chemiluminescence	Zero reading (including dark current)	6, 8, 9, 13
		Upscale reading (span)	6, 9, 13
		NO_2 - NO converter	7, 9, 12, 13
		Sample flow	10, 14
		Output linearity	7, 9, 13
HC	Gas chromatography with flame ionization detection	Flow rates (all gases)	11, 14
		Function and gate timing	13, 14
		Zero reading	6, 8, 9, 13
		Upscale reading (span)	6, 8, 9, 13
		Output linearity	7, 9, 13

[a] Refers to designation in Table 3.

Figure 3. NBS traceability: air quality instruments.



PROGRAM EXPERIENCE

Calibration of noise-measuring equipment, prior to the formal QAP, had been left up to the Caltrans districts. Appropriate intervals had been recommended, but practice varied considerably. There are 41 SLMs, 16 GLRs, 45 microphones, 7 analyzers, and 41 calibrators involved in the program. When the QAP was initiated, it was found that two years had elapsed since some instruments were calibrated. Fortunately, only about 6 percent of the instruments were out of specified tolerances. About 15 percent of the instruments showed problems that needed correction.

The program for air quality measurements interfaced with a previous program carried out by the California Air and Industrial Hygiene Laboratory. This meant that the equipment was in good calibration when the program started. Calibration, however, had been based on one or two upscale points, and it was found that linearity adjustments were necessary for the lower-scale values. Seventeen CO analyzers and eight each for O_3, HC, and NO_x are calibrated in this program. The hydrocarbon analyzers are more troublesome than the rest because of their complexity.

The procedures developed for the program seem adequate. Having the equipment adjusted and put in good repair as part of the calibration process is a great advantage. It is too early to draw conclusions about long-term instrument stability, but as we accumulate calibration history it will be one of the things that will affect calibration frequency.

The need for environmental measurement has diminished along with capital improvements, and Caltrans is doing much less today than a few years ago. As long as any measurements are made, however, a QAP will continue to be a necessary part of our work.

ACKNOWLEDGMENT

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Quick Fix for Washington Metro Brake Squeal

GUMMULURU N. SASTRY AND EDGAR C. GREEN, JR.

When the Washington Metropolitan Area Transit Authority (WMATA) made some changes in the brake system configuration of pads and discs to increase the wear life of the brake system in the fall of 1980, the quality, duration, and frequency of brake squeal increased dramatically, causing citywide complaints and adverse publicity. A quick fix had to be found within a few weeks to mitigate this squeal and avoid further losses of ridership. A test program was designed to find a solution simple enough to incorporate into the system immediately. Several configurations of brake pads were tested and a "quiet pad" that does not generate the annoying squeal was found among those pads made available to WMATA. Abex 1389b pads were retrofitted to the system, and the squeal disappeared.

When the Washington, D.C., Metro system was first opened to the public in 1976, quietness, speed, and comfort were the trademarks of the system and were highly praised by the public. But a letter to the editor of the Washington Post published on January 5, 1981, read as follows: "As a visiting New Yorker, I find the noise levels of arriving Metro trains intolerable. As a worker a year ago in Washington I knew this was not the case. Trains arrived with some quietness....Why has this been allowed to go unchecked? Where are the environmentalists?....Are D.C. travelers so immune to this insane decibel level that they ignore it?"

This rider's observation of the increased noise level in the Washington, D.C., Metro system was one of the more mildly worded of the widespread adverse reactions. Meanwhile, Washington Metropolitan Area Transit Authority (WMATA) environmentalists were aware of this annoying problem and were frantically working to alleviate the noise.

WMATA cars were designed with disc brakes as the primary friction braking system. There have been several investigations and tests over the past few years aimed at improving brake pad and disc life and mitigating the noise problem. When WMATA made some changes in the brake system configuration of pads and discs in the fall of 1980 in order to increase brake pad and disc life, the quality, duration, and frequency of the squeal changed dramatically, causing citywide complaints and adverse publicity. This prompted WMATA immediately to look into the brake squeal problem and to adopt a multifaceted approach. De Leuw, Cather and Company, the general engineering consultant to WMATA, was charged to find what was called a "quick fix" to the brake squeal problem. A solution had to be found within three or four weeks through diagnosis and identification of major noise contributors by field surveys and investigations. The solution had to be simple enough to be incorporated immediately into the existing braking system.

APPROACH TO THE PROBLEM

A brief survey of recent literature revealed that there was no simple explanation of the mechanism that produces squeal. Some of the significant factors are pad coefficient of friction, temperature, humidity, mass of brake assembly, and stiffness of brake components. It is also believed that squeal is affected by the geometric relation of the piston, the pad, and the disc, which may be subject to "geometrically induced or kinematic constraint instability" (1). A recent study by Ronk and Staino (2) indicates that "the squeal instability region is largely dependent upon the contact point position of the pad with the disc, disc stiffness normal to its plane of rotation, Young's modulus of pad material caliper system stiffness normal to the disc, and caliper mass."

It was evident from the start that a number of investigations incorporating several parameters were needed to define the Metro squeal problem and develop solutions. Our approach, however, was limited by the following constraints:

1. From the start, the Metro system has shown intermittent brake squeal at low speeds.
2. Squeal duration and frequency changed dramatically when WMATA attempted to prolong the wear life of pads and discs by experimenting with different friction materials.
3. Time constraints prevented major changes to the calipers or discs of the brake assemblies. Therefore, any solution requiring design changes in the braking system was ruled out in the exercise.

To satisfy all of these parameters, the pragmatic approach was to concentrate on the friction pads. The following guidelines were established at the beginning:

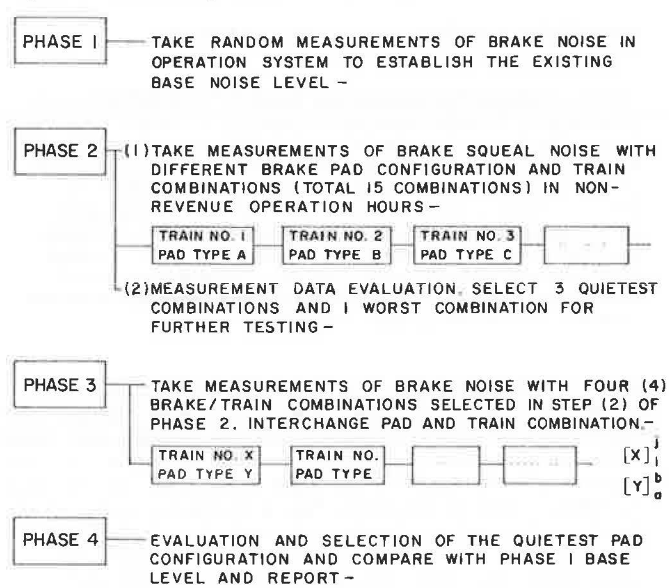
1. Test the effectiveness of a 0.125- or 0.25-in-thick resilient material installed between the pads and back plates and
2. Test various brake pads readily available on the market that met strict WMATA deceleration specifications for crush loads under wet and dry conditions.

The decision to concentrate on the pads alone was based on the theory that the disc and the total brake assembly act much as a bell ringing under the action of a clapper, i.e., the pad. It is known that a bell emits different frequencies when different clappers are used. The aim was to find a pad (the clapper) that produced the least squeal of low frequency because low-frequency squeals are not as objectionable to the human ear as high-frequency pitches. Table 1 identifies the various pads that were tested during the four-phase program (see Figure 1). Phase 1 established a base noise level by measuring the brake squeal of existing six-car trains during normal revenue hours. Phases 2 and 3 involved the testing of several brake pad configurations installed on two-car test trains operated during nonrevenue hours. Pads were ranked according

Table 1. Brake pad configurations and test trains used for brake squeal measurements.

Car No.	Brake Pad Configuration
1084/85	Knorr disc; Knorr pad (881)
1046/47	Abex disc; Knorr pad (881), backing plates with FOL 8; bonding 303 IMP
1040/41	Abex disc; Knorr pad (881); B1 dynamic brake
1050/51	Abex disc; Abex pad (45109)
1104/05	
1228/29	
1264/65	Abex disc; Raybestos pad without backing plates
1242/43	Abex disc; Knorr pad (402)
1282/83	Abex disc; Knorr pad (881); factory-bonded noise abatement material
1084/85	Knorr disc; Knorr pad (881); laminated inserts
1026/27	Abex disc; Knorr pad (881) with link isolators
1104/05	Abex disc; Abex pad (45109)
1118/19	Abex disc; Knorr pad (881), eight-segment
1032/33	Abex disc; Knorr pad (881)
1176/77	Knorr disc; Knorr pad (881), used pads cut with additional slot to form eight segments; one-half wear
1118/19	Abex disc; Knorr pad (881), eight-segment, new pads with two-day revenue service
1032/33	Knorr disc; Knorr pad (881)
1118/19	Knorr disc; multisegment (eight) Knorr pad, backing plate standard noise abatement material
1032/33	Abex disc; single-slot Knorr pad (881), backing plate standard noise abatement material
1280/81	Abex disc; outside brake Knorr pad (881), inside brake single-slot Knorr pad (881) (standard)
1286/87	New York Air Brake System
1190/91	Knorr disc; Knorr pad, standard
1168/69	Abex disc; Abex pad 1389b, single-slot
1076/77	Abex disc; Raybestos pad
1068/69	Abex disc; Abex pad 1389b
1076/77	Abex disc; Knorr pad (881-1)
1032/33	Abex disc; Knorr pad (881), single-slot
1076/77	Abex disc; Knorr pad (881-1), three-slot without backing
1150/51	Abex disc; Knorr pad (881-1), one diagonal, three-slot without backing
1122/23	Abex disc; Knorr pad (881-1) with backing plate
1076/77	Abex disc; Knorr (881-1) standard pad, three-slot; crush load
1168/69	Knorr disc; Abex pad 1389b, single-slot
1122/23	Knorr (881) standard pads, outside pad with backing plate (0.125 in)
1192/93	Inside pad Knorr (881) standard, outside pad Abex 1389b
1076/77	Abex disc; Abex pad 1389b; 33 000-lb crush load
1084/85	Knorr disc; Abex pad 1389b
1036/37	Abex disc; Abex pad 1389b with backing plate
1064/65	Abex disc; Abex pad 1389b

Figure 1. Metro brake squeal short-term test program.



to their quietness, and the three quietest pads were selected for further testing. Phase 4 consisted of testing the quietest pads installed on six-car revenue trains and comparing them with the base noise levels established during phase 1.

INSTRUMENTATION AND MEASUREMENT TECHNIQUES

Measurements of Metro brake squeal were conducted at Union Station near the Brentwood Yard on the Red Line. Instruments were installed in the middle of the center platform, about 15 ft from either edge, so that when the test train stopped the middle of the train would be near the microphones.

Two sets of instrumentation were used. The first set consisted of a B&K 4156 0.5-in condenser microphone, a B&K 2619 preamplifier, and a NAGRA IV SJ tape recorder. The microphone, equipped with a wind screen, was mounted on a tripod about 5 ft from the ground and connected to channel 1 of the NAGRA tape recorder through the preamplifier. The channel 1 output in turn was connected to the channel 2 input with an additional 10-dB attenuation set on the front panel of the tape recorder. The reason for 10-dB additional attenuation on channel 2 was to preserve any overshoots on channel 1.

The second set of instruments consisted of a B&K 2209 precision sound-level meter with a B&K 4131 1-in microphone and a B&K 2306 level recorder. The sound-level meter with a wind screen was mounted on a tripod about 5 ft above the ground. The output from the sound-level meter was connected to the level recorder. All of the instruments were calibrated with a B&K 4230, both before and after each set of measurements. A 1-min calibration signal was recorded on each tape. The maximum sound level observed on the sound-level meter for each run was logged by an assistant. A voice commentary describing each run was also recorded on the tape. All instruments functioned satisfactorily throughout the three-week measurement schedule.

The measurement procedure involved the following: A two-car test train was fitted with a selected brake pad and disc configuration. For example, a test train comprising cars 1282 and 1283 was fitted with Abex discs and a Knorr pad 881 with factory-bonded noise abatement material (Table 1). The test train ran from Brentwood Yard and came to a full stop at Union Station in automatic mode, continued to the next station (Judiciary Square), reversed direction, and came to a full stop at Union Station. It returned to Brentwood Yard and then repeated the same run. Each stop at Union Station was considered one run. The tape recorder started recording the run as soon as the train was in sight at the station portal. Approximately 20-30 runs for each test train were recorded to obtain consistent squeal readings. The trains were usually empty, but sev- eral tests were conducted with 33 000-lb ballast loaded on each car (equal to a crush load) for selected brake pad configurations.

The recorded magnetic tapes were sent to Wilson, Ihrig and Associates, Inc., to obtain one-third octave band plots.

RESULTS

In phase 1, brake squeal measurements included the establishment of existing squeal levels during normal revenue service. A total of 79 readings were taken with the B&K 2209 sound-level meter equipped with a B&K 4131 1-in microphone mounted on a tripod 5 ft above ground in Union Station. Of the 79 readings, 18 were discarded because two trains, inbound and outbound, stopped in the station simultaneously. The average squeal level of the remaining 61 readings was 98 ± 3.5 dB(A); the lowest was 88 dB(A) and the highest 104 dB(A). Eighteen readings were exactly 98 dB(A). Average duration of the squeal was found to be 12 s [duration was defined as the length of time a squeal exceeds 80 dB(A)]. This noise level, which a local newspaper compared to the noise of a jackhammer, indicates the severity of the problem.

American Public Transit Association (APTA) Guidelines for Design of Rapid Transit Facilities, published in January 1979 by the Rail Transit Committee, established a level of 80 dB(A) for platform noise produced by trains entering or leaving stations. Although this criterion does not deal specifically with brake squeals, it was considered applicable and was used for the purpose of this paper. Therefore, any brake pad configuration with a squeal lower then 80 dB(A) was considered a "quiet" pad.

It is beyond the scope of this paper to discuss the measurements and analysis of each brake pad configuration. However, Table 2 summarizes a sample of squeal levels recorded during several runs of three different brake pad configurations. Figures 2-4 show the one-third octave band spectra of brake squeal, including the duration of some squeals (shown at bottom left of the figures).

In general, the analysis of the squeal levels and the noise spectra indicated the following:

1. The instantaneous spectrum of squeal shows significant peaks at 375, 500, 2500, 3100, 3800, and 5800 Hz (some of these peaks can be seen in Figure 2).
2. It appears that the squeal generally originates within the frequency range of 350-750 Hz and that the disc subsequently vibrates at its higher modes.
3. The New York Air Brake System brakes generated no significant squeal during the 10 runs conducted; they were tested no further because a separate testing program was planned for this at a later date.
4. The squeals generated by the Abex disc and the Knorr pads were generally in the range of 2800-3000 Hz, although higher modes were also common. The Knorr pads were tested in different configurations; some were slotted at various locations, and others

Table 2. Brake squeals for several runs of three brake pad configurations.

		Brake Squeal Level [dB(A)]		
Run No.	Direction	Cars 1032/33, Abex Disc and Knorr 881 Pad	Cars 1068/69, Abex Disc and Abex Pad 1389B	Cars 1076/77, Abex Disc and Abex Pad 1389B, Crush Load
1	Inbound	86	76	75
2	Outbound	97	78	77
3	Inbound	81	72	81
4	Outbound	86	79	80
5	Inbound	92	80	79
6	Outbound	92	78	78
7	Inbound	94	77	79
8	Outbound	90	76	76
9	Inbound	88	77	79
10	Outbound	89	75	76
11	Inbound	94	78	78
12	Outbound	90	76	75
13	Inbound	95	78	79
14	Outbound	90	76	77
15	Inbound	94	79	78
16	Outbound	89	75	76
17	Inbound	96	78	78
18	Outbound	90	76	76
19	Inbound	96	78	78
20	Outbound	90	77	76

Figure 2. Test results for cars 1282/83: Abex disc and Knorr pad (881) with factory-bonded noise abatement material.

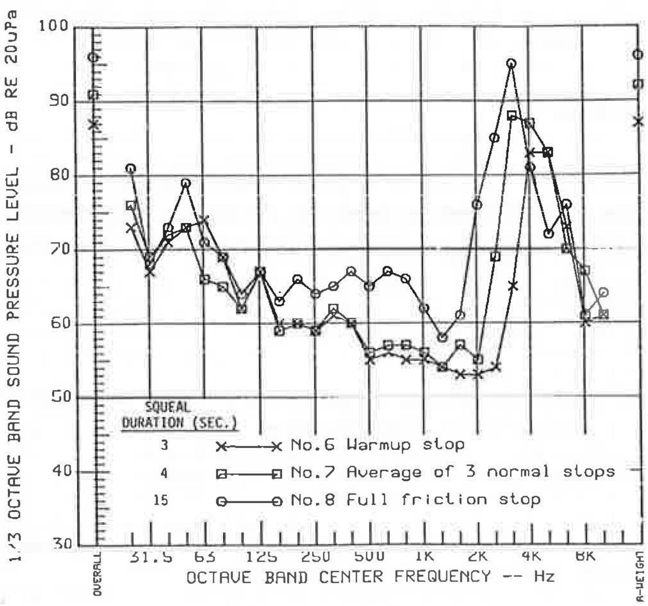
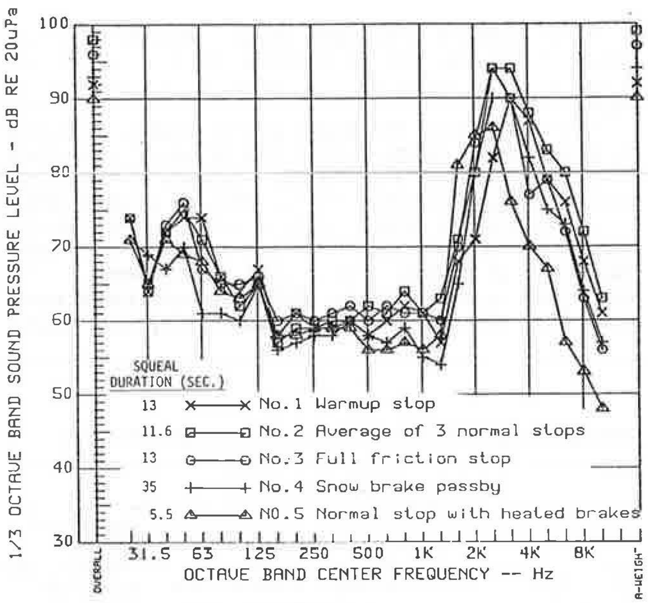


Figure 3. Test results for cars 1084/85: Knorr disc and Knorr pad (881) with laminated insert.

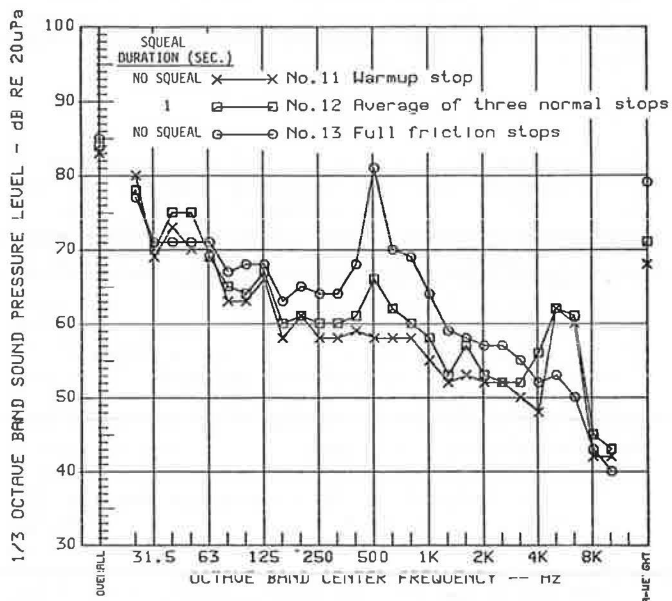


were equipped with special noise abatement plates installed between pad and backing plate. In every case, no significant improvements were recorded.

5. The noise spectrum of the Abex disc and Abex pads generally indicates a squeal peak at 500 Hz. Squeals of higher modes in the spectrum were not evident.

6. Because of procurement difficulties, only the Abex 1389b pads could be tested extensively. Nearly 160 tests were conducted. Various cars were used, empty or crush loaded, equipped with either Abex or Knorr discs but all using Abex 1389b pads. Only 32

Figure 4. Test results for cars 1104/05: Abex disc and Abex pad (45109).



runs generated squeals, and of these only 6 produced squeal exceeding 80 dB(A).

It was concluded that the Abex 1389b pads were suppressing the higher modes of disc vibration, thus eliminating most of the unpleasant squeals. They rarely produced squeals above established acceptance criteria of 80 dB(A) and even then only at frequencies of 500-750 Hz. The duration of the squeals was also very limited (approximately 1 or 2 s).

Thus, the Abex 1389b pad was deemed the candidate most likely to eliminate the annoying brake squeal problem, and all WMATA cars were subsequently retrofitted with these pads. Several follow-up tests conducted during revenue service consistently confirmed the results.

ACKNOWLEDGMENT

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