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SECTION 6. MODELING

# EFFECT OF EROSION ON PRODUCTIVITY -- EPIC WATER EROSION MODEL

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## INTRODUCTION

The Erosion-Productivity Impact Calculator (EPIC) model (Williams et al., 1984) was developed to assess the effect of soil erosion on soil productivity. EPIC is composed of physically based components for simulating erosion, plant growth, and related processes and economic components for assessing the cost of erosion, determining optimal management strategies, etc. The physical processes involved are simulated simultaneously and realistically using readily available inputs. Since erosion can be a relatively slow process, EPIC is capable of simulating hundreds of years if necessary. The model is generally applicable, computationally efficient, and capable of computing the effects of management changes on outputs.

There are nine major components of the EPIC model--hydrology, weather, erosion, nutrients, plant growth, soil temperature, tillage, economics, and plant environment control. In keeping with the theme of this Conference, only the details of the water erosion component will be described. A description of the entire EPIC model was presented previously (Williams et al., 1984).

The EPIC model was used to analyze the relationship between erosion and productivity as part of the Soil and Water Resoruces Conservation Act (RCA) analysis for 1985. EPIC provided erosion-productivity relationships (E/P) for about 900 benchmark soils and 500,000 crop/tillage/conservation strategies. Details of the methods used to develop the E/P were presented previously (Williams et al., 1985). An example application of the E/P methodology is given here.

## THE EPIC WATER EROSION MODEL

The EPIC water erosion model simulates erosion caused by rainfall and runoff and by irrigation (sprinkler and furrow).

### Rainfall/Runoff

To simulate rainfall/runoff erosion, EPIC contains three equations--the USLE (Wischmeier and Smith, 1978), the MUSLE (Williams, 1975), and the Onstad-Foster modification of the USLE (Onstad and Foster, 1975). Only one of the equations (user specified) interacts with other EPIC components. The three equations are identical except for their energy components. The USLE depends strictly upon rainfall as an indicator of erosive energy. The MUSLE uses only runoff variables to simulate erosion and sediment yield. Runoff variables increased the prediction accuracy, eliminated the need for a delivery ratio (used with USLE to estimate sediment yield), and provided an equation designed for single storm application instead of the USLE annual estimates. The Onstad-Foster equation contains a combination of the USLE and MUSLE energy factors. Since the Onstad-Foster equation was used as the driving equation in the 1985 RCA analysis, it is the only one of the three equations described in this section. The Onstad-Foster equation is

$$Y = \{0.646\ EI + 0.45\ (Q)\ (q_p)^{0.333}\}\ (K)\ (CE)\ (PE)\ (LS),\ Q > 0 \qquad (1)$$

$$Y = 0\ , \quad Q \leq 0$$

where Y is the sediment yield in t $ha^{-1}$, EI is the rainfall energy factor in metric units, Q is the runoff volume in mm, $q_p$ is the peak runoff rate in mm $h^{-1}$, K is the soil erodibility factor, CE is the crop management factor, PE is the erosion control practice factor, and LS is the slope length and steepness factor. The PE value is determined initially by considering the conservation practices to be applied. The value of LS is calculated with the equation (Wischmeier and Smith, 1978)

$$LS = \left(\frac{\lambda}{22.1}\right)^{\xi} (65.41\ S^2 + 4.56\ S + .065) \qquad (2)$$

where S is the land surface slope in m $m^{-1}$, $\lambda$ is the slope length in m, and $\xi$ is a parameter dependent upon slope. The value of $\xi$ varies with slope and is estimated with the equation

$$\xi = 0.3\ S / \{S + \exp(-1.47 - 61.09\ S)\} + 0.2 \qquad (3)$$

The crop management factor is evaluated for all days when runoff occurs using the equation

$$CE = \exp\{(\ln 0.8 - \ln CE_{mnj})\ \exp(-0.00115\ CV) + \ln CE_{mnj}\} \qquad (4)$$

where $CE_{mn,j}$ is the minimum value of the crop management factor for crop j and CV is the ground cover composed of crop residue and growing biomass.

The soil erodibility factor K, is evaluated for the top soil layer at the start of each year of simulation using the equation

$$K = 0.2 + 0.3\ \exp\{-0.0256\ SAN\ (1 - SIL / 100)\}\ \left(\frac{SIL}{CLA + SIL}\right)^{0.3}$$

$$\left(1 - \frac{0.25\ C}{C + \exp(3.72 - 2.95\ C)}\right) \left(1 - \frac{0.7\ SN1}{SN1 + \exp(-5.51 + 22.9\ SN1)}\right) \qquad (5)$$

where SN1 = 1 - SAN /100 , and SAN, SIL, CLA, and C are the sand, silt, clay, and organic carbon content of the soil in %. Equation 5 allows K to vary from about 0.1 to 0.5. The first term gives low K values for soils with high coarse sand content and high values for soils with little sand. The fine sand content is estimated as the product of sand and silt divided by 100. The expression for coarse sand in the first term is simply the difference between sand and the estimated fine sand. The second term reduces K for soils that have high clay to silt ratios. The third term reduces K for soils with high organic carbon content. The fourth term reduces K further for soils with extremely high sand content (SAN > 70%).

The runoff model supplies estimates of Q and $q_p$. To estimate the daily rainfall energy in the absence of time-distributed rainfall, it is assumed that the rainfall rate is exponentially distributed.

$$r = r_p \exp(-t / k) \qquad (6)$$

where r is the rainfall rate at time t in mm $h^{-1}$, $r_p$ is the peak rainfall rate in mm $h^{-1}$, and k is the decay constant in h. Equation 6 contains no assumption about the sequence of rainfall rates (time distribution). The USLE energy equation in metric units is

$$E = \Delta R \left(12.1 + 8.9 \log \frac{R}{\Delta t}\right) \qquad (7)$$

where $\Delta R$ is a rainfall amount in mm during a time interval $\Delta t$ in h. The energy equation can be expressed analytically as

$$E = 12.1 \int_0^\infty r\, dt + 8.9 \int_0^\infty r \log r\, dt \qquad (8)$$

Substituting Eq. 6 into Eq. 8 and integrating gives the equation for estimating daily rainfall energy

$$E = R\{12.1 + 8.9 (\log r_p - 0.434)\} \qquad (9)$$

where R is the daily rainfall amount in mm. The rainfall energy factor, EI, is obtained by multiplying Eq. 9 by the maximum 0.5 h rainfall intensity and converting to the proper units.

$$EI = R\{12.1 + 8.9 (\log r_p - 0.434)\} (r_{.5}) / 1000 \qquad (10)$$

To compute values for $r_p$, Eq. 6 is integrated to give

$$R = (r_p)(k) \qquad (11)$$

and

$$R_t = R\{1 - \exp(-t/k)\} \tag{12}$$

To provide a convenient means of estimating $R_{.5}$ stochastically, the $\alpha$ ratio is introduced.

$$\alpha_{.5} = \frac{R_{.5}}{R} \tag{13}$$

It is much simpler to generate values of $\alpha_{.5}$ than $R_{.5}$ because $0 < \alpha_{.5} < 1$ and because $R_{.5}$ is related to R. To determine the value of $r_p$, Eqs. 13 and 11 are substituted into Eq. 12 to give

$$r_p = -2\, R \ln(1 - \alpha_{.5}) \tag{14}$$

Since rainfall rates vary seasonally, $\alpha_{.5}$ is evaluated for each month using Weather Service information (U.S. Department of Commerce, 1979). The frequency of the maximum 0.5 h rainfall amount is estimated using the equation

$$F = \frac{1}{2\tau} \tag{15}$$

where F is the frequency of occurrence of the largest event of a total of $\tau$ events. The total number of events for each month is the product of the number of years of record and the average number of rainfall events for the month. To estimate the mean value of $\alpha_{.5}$, it is necessary to estimate the mean value of $R_{.5}$. The value of $R_{.5}$ can be computed easily if it is assumed that the maximum 0.5 h rainfall amounts are exponentially distributed. From the exponential distribution, the expression for the mean 0.5 h rainfall amount is

$$\bar{R}_{.5,j} = \frac{R_{.5F,j}}{-\ln F_j} \tag{16}$$

where $R_{.5,j}$ is the mean maximum 0.5 h rainfall amount, $R_{.5F,j}$ is the maximum 0.5 h rainfall amount for frequency F, and subscript j refers to the month. The mean $\alpha_{.5}$ is computed with the equation

$$\alpha_{.5j} = \frac{\bar{R}_{.5,j}}{\bar{R}_j} \tag{17}$$

where R is the mean amount of rainfall for each event (average monthly rainfall/average number of days of rainfall) and subscript j refers to the month. Daily values of $\alpha_{.5}$ are generated from a two-parameter gamma distribution. The base of the gamma distribution is established by examining upper and lower limits of $\alpha_{.5}$. The lower limit determined by a uniform rainfall rate gives $\alpha_{.5}$ equal .5/24 or 0.0208. The upper limit of $\alpha_{.5}$ is set by considering a large rainfall event. If the event is large, it is highly unlikey that all the rainfall would occur in 0.5 h ($\alpha$ = 1). The upper limit of $\alpha_{.5}$ can be estimated by substituting a high value for $r_p$ (250 mm is generally near the upper limit of rainfall intensity) into Eq. 12.

$$\alpha_{.5u} = 1 - \exp(-125 / R) \qquad (18)$$

where $\alpha_{.5}$ is the upper limit of $\alpha_{.5}$. The peak of the $\alpha_{.5}$ gamma distribution can be computed using the equation

$$\alpha_{.5P,j} = \frac{\alpha_{.5,j} (\nu - 1)}{\nu} \qquad (19)$$

where $\alpha_{.5P,j}$ is the $\alpha_{.5}$ value at the peak of the gamma distribution and $\nu$ is the gamma distribution shape parameter (a value of 10 is generally satisfactory).

## Irrigation

Erosion caused by applying irrigation water in furrows is estimated with the Modified Universal Soil Loss Equation (MUSLE) (Williams, 1975).

$$Y = 11.8 \, (Q \bullet q_p)^{0.56} \, (K) \, (CE) \, (PE) \, (LS) \qquad (20)$$

where CE, the crop management factor, has a constant value of 0.5. The volume of runoff is estimated using the equation

$$Q = (EIR) \, (AIR) \qquad (21)$$

where AIR is the volume of irrigation water applied in mm and EIR is the runoff ratio. The peak runoff rate is estimated for each furrow using Manning's Equation and assuming the flow depth is 0.75 of the ridge height and that the furrow has a triangular shape. If irrigation water is applied to land without furrows, the peak runoff rate is assumed to be 0.00189 $m^3 \, s^{-1}$ per m of field width.

## Estimating the E/P

A method, based on a concept introduced by Perrens (1983), was developed to transform EPIC output into an E/P (Williams et al., 1985). Two long-term ($\approx$100 years) EPIC simulations are required to apply the method to a particular soil at a selected location. The two runs are designed to envelop the range of conservation strategies that could be applied to the land. The first run represents perfect conservation by preventing erosion (the water erosion control practice factor (PE) and the wind erosion climatic factor (WC) are set to zero). Maximum erosion rates are simulated in the second run by assuming no conservation practices (PE = 1) and assigning the appropriate value to WC. Except for erosion, the two runs are identical (weather, initial conditions, crop rotations, etc.). Annual crop yields from the two runs are used to compute an erosion-productivity index (EPI). Annual EPI values are calculated by dividing simulated crop yield with maximum erosion by simulated crop yield with zero erosion. The E/P is obtained by relating the EPI to accumulated erosion. Details of the development of the curve fitting scheme were described previously (Williams, 1985). The necessary equations are

$$EPI = 1. - \frac{2E\,\{E + b_1 \exp(-b_2 E)\} - E^2\,\{1. - b_1\, b_2 \exp(-b_2 E)\}}{\{E + b_1 \exp(-b_2\, E)\}^2} \tag{22}$$

and

$$E = \frac{(T)\,(Y)}{Y_e} \tag{23}$$

where T is time from the start of the simulation in years, Y is the average annual erosion rate for a particular conservation strategy in t $ha^{-1}$, $Y_e$ is the rate for zero conservation, E is an accumulated erosion equivalent in t $ha^{-1}$, and $b_1$ and $b_2$ are shape parameters. The values of $b_1$ and $b_2$ are determined to guarantee a fit of the mid and end points of the accumulated EPI data. Thus, EPI can be estimated for any conservation strategy from the E/P (Eq. 22) if the average annual erosion rate is known.

To apply the E/P it is necessary to estimate both wind and water erosion for the management strategies considered. A short-cut procedure based on EPIC wind and water erosion models was developed (Williams et al., 1985) for this purpose.

## Example Problem

To demonstrate the application of the E/P technique, an example problem is presented. A hypothetical test site was assumed to be near Lafayette, IN, and to contain Miami silt loam soil. The slope length and steepness were assumed to be 100 m and 5%, respectively. Continuous corn with an annual fertilizer rate of 100 kg/ha N and 20 kg/ha P was the assumed management strategy. Two EPIC simulations of 100 years each (one with maximum and one with zero erosion) were performed to obtain data for calculating EPI values. Figure 1 shows the annual EPI values (crop yield with erosion/crop yield zero erosion) plotted with the ● symbol. The predicted EPI values plotted in Figure 1 with the * symbol were obtained from a simultaneous solution of Eqs. 22 and 23.

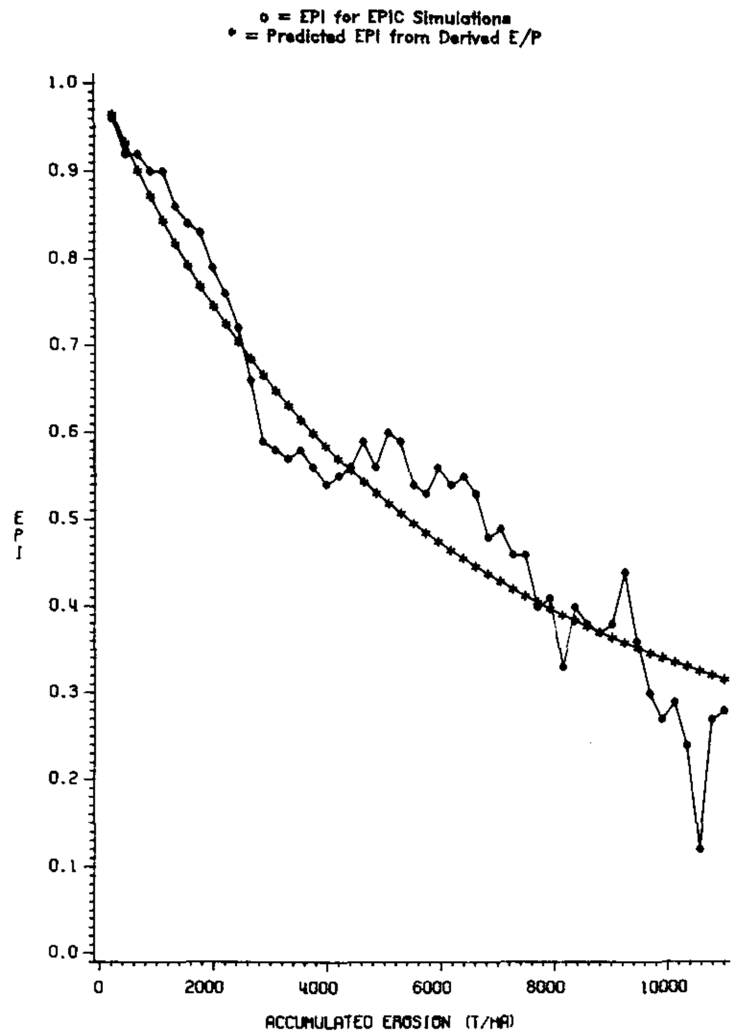


Fig. 1. Erosion-productivity relationship (E/P) for Miami Soil near Lafayette, IN.

The E/P appears to be a reasonable representation of what could occur during 100 years of cultivation without conservation on a Miami soil. Productivity drops rapidly during the early years as the favorable top soil is eroded. After the top soil (≈130 mm is eroded, the texture changes gradually from 55% silt, 19% clay to 45% silt, 30% clay, the available water capacity decreases from 0.18 to 0.16 m $m^{-1}$, bulk density increases from 1.45 to 1.6 t $m^{-3}$, and organic C decreases from 1.02% to 0.5%. These less favorable conditions exist to a depth of 640 mm. Beyond 640 mm, the soil is unfavorable for crop production. The available water capacity is ≈0.075 m $m^{-1}$, bulk density is 1.8 t $m^{-3}$, and organic C is ≈0.2%. As shown by the E/P, production begins to level off as the accumulated erosion approaches 6000 t $ha^{-1}$ (≈360 mm of erosion). The tillage mixing effect smooths the transition between soil layers and prevents direct comparison of the E/P and the soil profile. If soil layers were removed instantaneously, the E/P should reflect soil property differences more distinctly.

## SUMMARY AND CONCLUSIONS

A method for estimating the relationship between erosion and soil productivity (E/P) was presented. Also, details of the EPIC water erosion model were given. The EPIC water erosion model contains three equations for estimating erosion--the USLE, the MUSLE, and the Onstad-Foster Modification of the USLE. Only one of the equations (user specified) interacts with other EPIC components. The Onstad-Foster equation was selected as the driving equation for the 1985 RCA analysis.

The EPIC water erosion model estimates erosion for all days that have rainfall. The crop management factor is a function of the minimum CE value for the crop and the ground cover (residue and growing biomass). The soil erodibility is computed annually as a function of soil texture and organic C content. Rainfall energy and intensity are estimated stochastically from daily rainfall to provide input to the USLE rainfall energy equation. There is also a provision for using MUSLE to estimate erosion caused by irrigation.

EPIC simulated erosion rates and crop yields are used to develop the E/P. Two long-term EPIC simulations are performed to establish the E/P for a particular soil and location. Once the E/P is established, any management strategy can be evaluated quickly if the average annual erosion rate is known. Short-cut methods for estimating annual erosion based on the EPIC erosion model are useful in applying the E/P.

The E/P provides a means for determining the cost of erosion and for evaluating various conservation strategies. An example application of the methodology was the 1985 RCA analysis.

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# ACCELERATED EROSION RISKS VS WATERSHED CONDITION

Credits

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## ABSTRACT

We present here one method of displaying the interrelationships of ground cover, runoff, and return period in arid and semiarid landscapes. Similar approaches are possible for forested ecosystems, utilizing the factors that index the hydrologic processes controlling watershed condition in those areas. A single value watershed condition threshold, sensitive by ecosystem, is displayed within the context of the relationship between ground cover and the risk of experiencing a storm which will cause accelerated erosion. Reducing ground cover increases the risk of accelerated erosion. The rate of increasing risk varies with ecosystem, with resilient environments having a slow rate of increase, and fragile environments having a rapid rate of increasing risk. The resultant method provides a way to assess watershed condition by comparing existing ground cover in relation to the potential ground cover the watershed can support, and that level of ground cover needed to limit the risk of triggering accelerated erosion and upward channel migration. In addition, it allows practitioners to discern the decision space available and assess the sensitivity of environments they manage.

## THE NEED FOR A METHOD TO EVALUATE THE RISK OF ACCELERATED EROSION

The term "watershed condition" describes the state, or "health", of a watershed. It effectively integrates a number of resource factors such as flow regime, sediment and nutrient output, and site productivity. Typically, a watershed attribute affected by management activities which, in turn, affects the above factors, is utilized to "index" watershed condition. The concept can span a wide variety of ecosystems. However, the "index" must change to reflect those hydrologic factors that maintain the ecosystem in dynamic equilibrium. In ecosystems from semidesert grasslands up to Ponderosa pine, ground cover is an appropriate index. Grazing has historically been the predominate means of disturbance in these areas, and early research demonstrated that reducing ground cover can lower infiltration rates, increase overland flow and decrease surface resistance to erosion. Such changes can lead to increased storm response, on-site erosion, and to the formation and upward migration of rills and gullies (Bailey and Copeland, 1961; Colman, 1953; Morston, 1952; Meeuwig, 1960; Rich, 1961).

The above sequence of events was not uncommon on Western rangelands (Bailey, Forsling, and Becraft, 1934; Cooperrider and Hendricks, 1937). To avoid similar occurrences, a "preventative" approach to assessing watershed condition was needed. This would enable a manager to identify the risk of accelerated erosion before experiencing watershed and downstream damages.

General guidelines developed to prevent accelerated erosion indicated that at least 60 to 70 percent ground cover was needed to maintain good watershed condition (Coleman, 1953; Marston, 1952; Packer, 1951). This single point threshold was developed for mountainous rangelands where an undisturbed watershed can produce ground cover approaching 100 percent. More arid ecosystems cannot support such biomass production and potential levels of ground cover may be much less than the levels identified for high mountain rangelands. These ecosystems have naturally adapted through physical adjustments, such as higher channel density and armoring, to greater levels of overland and peak channel flow. For the vegetative threshold concept to have wide applicability, it must be sensitive to the potential level of ground cover a particular ecosystem can naturally attain.

Point threshold techniques of evaluating watershed condition provide a simple target, easily discerned by land managers. However, in an era of resource optimization, managers need an evaluation system which displays how incremental changes in ground cover will affect the risk of accelerated erosion and allows for a more effective method of balancing resource outputs, capital investments, and opportunity costs.

## A SINGLE POINT THRESHOLD CONCEPT

Any landscape has a maximum, or potential, average ground cover it can naturally achieve. Potential ground cover on undisturbed soils provides the highest infiltration capacities, absorbing the maximum quantity of precipitation, making it available for transpiration or ground water recharge. Reduction of ground cover increases the risk of accelerated erosion from overland flow in two ways:

1. The available erosive energy of surface runoff increases due to reduced infiltration, and

2. Surface resistance to this erosive force decreases through reduced energy dissipation by plants and litter.

The net effect of reductions in cover decreases the drainage area or slope distance required for a channel to form, causing the channel network to extend headward through gully erosion. This leads to a shorter time of concentration, yielding larger peak flows and additional channel scouring. If unchecked, this process ultimately will severely degrade watershed condition to the point where the functional relationship between runoff and sediment output changes and the landscape can no longer attain the former potential level of ground cover.

Maxwell and Solomon (Maxwell et. al., 1985) previously developed a point-value threshold technique for use in arid and semiarid ecosystems. The method consists of computing a lower limit for groundcover. Reducing ground cover below this level exposes the landscape to a greater risk of permanently altering hydrologic function and degrading other resource values.

To calculate how changes in ground cover affect the slope distance required for a channel to form, Maxwell and Solomon (1983), utilized Horton's (1945), "Belt of No Erosion" (B), where:

$$B = \frac{62.4 \tan(a)^{0.50}}{\sin(a)^{1.67}} \cdot \frac{r^{1.67}}{ec} \qquad (1)$$

where B is in feet, a = mean land slope (degrees), r = surface resistance to erosion (lb/sq. ft), e = surface runoff rate (in/hr) and c = a roughness factor which expresses the tendancy of the surface to concentrate runoff. Solomon (1983), developed a rainfall-runoff model for "e" that is sensitive to ground cover, and jointly with Maxwell, developed, from previous studies, relationships between ground cover, "r" and "c" (Maxwell and Solomon, 1983).

Having developed a model of fluvial response sensitive to changes in ground cover, the next step was to link this with a model of risk based on (1) a landscape's ability to recover from destabilizing runoff events and (2) the probability of the occurrence of such events.

Watershed Disruption and Recovery

Even with systems in dynamic equilibrium, periodic storms occur with sufficient erosive force to overcome the resistance of the landscape to erosion, destabilizing it through gully, channel, or mass erosion (Wolman and Miller, 1960; Wolman and Gerson, 1978). With ground cover at potential, such an event may have a recurrence interval from a few years to more than 100 years, depending on the ecosystem. In most landscapes, the intervening mean "rest period" between destabilizing events usually allows sufficient time for the land and channels to fully recover (Kelsey, 1982; Rice, 1982). A landscape's resilience (stability) depends on its inertia (resistance to change) and elasticity (recovery rate), which both vary with climate and topography (Westman, 1978).

The complex relationship between climate and geomorphic processes precludes description by any simple model. Scant existing data suggests, however, that both landscape inertia and elasticity coarsely relate to the area's moisture and temperature regime as defined by effective precipitation; Kelsey, 1982; Leopold, Wolman, and Miller, 1964; Rice, 1982; Schumm,1977; Thornthwaite and Mather, 1955; Wolman and Gerson, 1978). Effective precipitation is the annual sum of the monthly balances of precipitation minus potential evapotranspiration. It serves as a proxy for water available for transpiration, and provides a rough index of the total amount of biomass (and hence potential ground cover) an ecosystem can support, and the rate of which the ecosystem can produce biomass. As ecosystems become progressively more arid, the potential ground cover decreases commensurately. Infiltration rates decrease, overland flow and peak flow rates increase, and the energy available to erode both the soil surface and the channels increases. Thus, the natural ability of the ecosystem to withstand disruption (its initeria) decreases. Storms with smaller return intervals can cause accelerated erosion. Similarily, more arid eocsystems have insufficient moisture to rapidly produce vegetation and associated litter. Such ecosystems are less elastic and take longer to recover from a disruptive event. These relationships are depicted in Figure 1. On landscapes represented by lower values of effective precipitation, destabilizing events occur more frequently, and the associated recovery period increases.

Figure 1 also depicts the hypothesized applicable range for utilizing ground cover to index watershed condition. The lower limit of -30 inches of effective precipitation roughly corresponds to the semidesert grasslands. The upper limit of 0 inches corresponds to Ponderosa pine forests. Below -30 inches, potential ground cover is so little that incremental changes in cover no longer significantly influence hydrologic response. In heavily stocked Ponderosa pine forests and above, where potential ground cover approaches 100 percent, overland flow occurs infrequently and other factors dominate hydrologic processes. The methods presented in Maxwell et al., (1985) allow computation of a threshold ground cover value. This percent ground cover represents the "tolerance, or threshold ground cover" needed to reduce the risk of accelerated erosion and the subsequent triggering of continuous watershed damage.

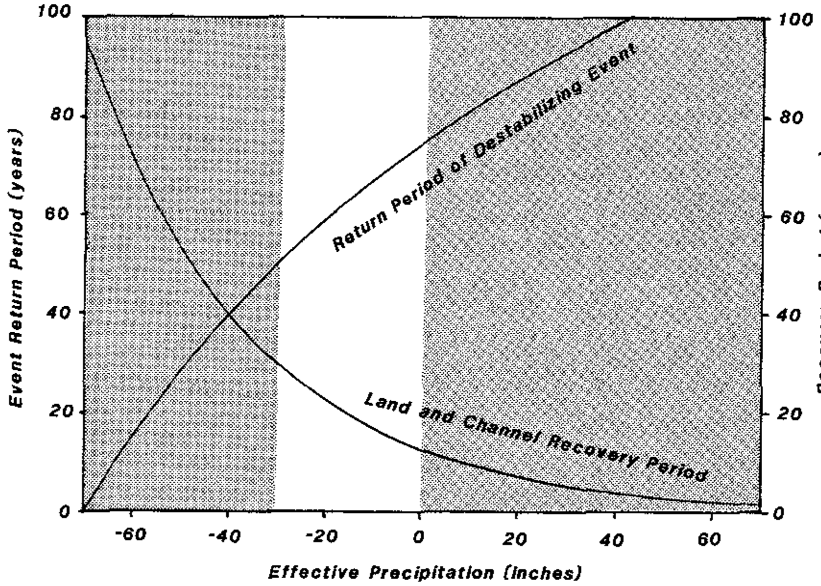


*Figure 1. Relationship between return period of destabilizing events and period necessary for full land and channel recovery as indexed by effective precipitation. Unshaded areas represent estimated applicable range of ground cover as a valid index of watershed condition.*

## CONTINUUM APPROACH TO THRESHOLD CONCEPT

While the single point threshold is a simple tool that managers can easily apply, it does not allow them to visualize the dynamics of the hydrologic system. Figure 2 presents the instantaneous peak overland flow runoff rate (z-axis), as a function of both ground cover (x-axis), and return period (y-axis). The figure represents the modeled response from a one-half hour storm where total rainfall input is a function of return period. Runoff is calculated using an overload flow model (Solomon, 1983). The soil parameters used in this example are typical of a clay loam with five percent rock cover. The surface represents runoff from a 35 percent slope. Successive simulations produced a matrix of maximum runoff rates with varying ground covers and return period storms.

These "data" points were then used to develop an equation for the runoff surface. Manual curve fitting techniques (Jensen and Homeyer, 1971, Jensen, 1973) were utilized. A third parameter included in the equation of the surface is effective precipitation. This allowed for the generation of a family of runoff surfaces, each representing a different ecosystem, e.g., semidesert grassland, pinyon-juniper, or Ponderosa pine.

Although the surface fitting approach was somewhat trial and error, the basic curve forms through the xz and yz planes were well defined from the simulations produced by the model. The general sigmoid shape of the ground cover-runoff relationship (xz-axis) reflects the modeled response from the overland flow model. Within the model exists a relationship describing the maximum infiltration rate as a function of ground cover (Fletcher, 1960; Smith and Leopold 1941; Woodward and Craddock, 1945). This relationship is sigmoid in shape, and principally controls the shape of the resultant runoff-ground cover curves.

Displaying The Risk of Accelerated Erosion

Figure 2 allows a manager to visualize the relationship between groundcover and runoff and gain an understanding of the sensitivity of a particular ecosystem to changes in ground cover. Figure 2 shows the modeled runoff relationship for a Ponderosa pine site while Figure 3 presents the same modeled relationship for a chaparral/grassland site.

Figure 3 is simply a truncation of Figure 2. At an effective precipitation of -20 inches, potential ground cover is an estimated 16 percent. The obvious difference between the two Figures is the restricted opportunity to alter on-site storm runoff in the more arid ecosystems. Here, vegetation plays a decreasing role in regulating storm response. The potential ground cover is so low that changes produce little effect on infiltration.

Managers cannot use the modeled runoff relationships, such as Figures 2 and 3, directly to assess the risk of accelerated erosion because the risk is not only a function of energy (expressed as runoff) but also of resistance to the erosive forces. Figure 4 represents the risk of incurring an event which may trigger accelerated erosion. This probability of landscape disruption in any given year is shown on the y-axis, with ground cover on the x axis.

The curve on the right, depicting zero inches of effective precipitation, might represent a Ponderosa pine site. The curve on the left, at -15 inches effective precipitation, represents a moist chaparral/grassland site. Potential and threshold ground cover are shown on each curve. The curve was derived by finding that ground cover at various return interval (exceedance probability) storms which yielded the same "Belt of No Erosion" as at potential and threshold ground cover. Reductions in ground cover increase the proportion of overland flow and decrease the surface resistance to erosion. Thus, storms of lesser magnitude can yield the same "Belt of no Erosion" value as that calculated for larger, less frequent storms under denser ground cover conditions. Notice that even at potential ground cover, there is some risk of receiving a storm which will disrupt the dynamic equilibrium of a watershed. The area between potential and threshold ground cover represents the decision space for a prudent land manager. Even though the risk of a disruptive event increases with reductions of ground cover from potential, the recovery period is less than the return interval of damaging events until the threshold is reached. Thus, the probability of incurring permanent watershed damage brought about by successive disruptive events is quite low. On average, the watershed will fully recover before another disruptive event occurs. At ground cover levels below the threshold, the probability of receiving a damaging event increases rapidly. More importantly, however, the probability of continuous watershed damge becomes most likely. Below threshold ground cover, the recovery period exceeds the

$$RUNOFF = ((PRO * TOP - BASE) * e^{-\left|\frac{COV}{100(1-I)}\right|^{n}}$$

where

$PRO = 0.136 \ln RP - 0.00045\ EP * \ln RP + 0.00197\ EP + 0.0946$

$TOP = 0.137\ (EP + 20)^{0.85} + 6.95$

$BASE = 1.03\ (TOP * PRO) - 5.77$

$N = 0.54\ (TOP - BASE) + 5.00$

$I = -0.054\ (TOP - BASE) + 0.80$

and

RUNOFF = Peak overland runoff rate (inches/hour)

EP = Effective Precipitation (0 to -20 inches)

RP = Return Period (years)

COV = Ground Cover (percent)

e = Base of natural logarithm

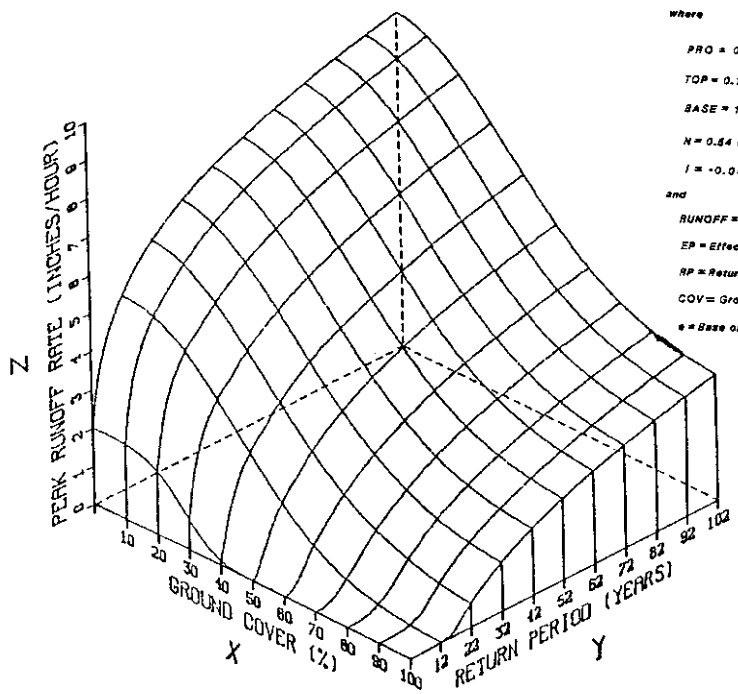


*Figure 2. Instantaneous peak rate of overland flow versus ground cover versus return period for areas having zero inches of effective precipitation (Ponderosa pine).*

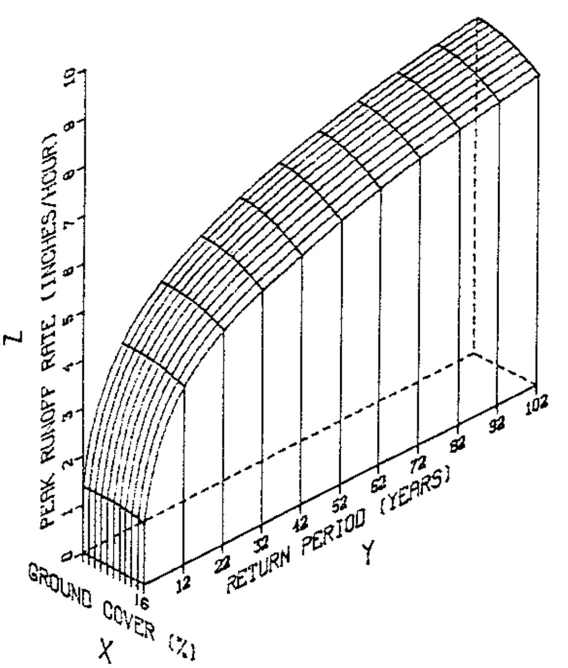


*Figure 3. Instantaneous peak rate of overland flow versus ground cover versus return period for areas with -20 inches of effective precipitation (chaparral/grassland site).*

return interval of disruptive events. The watershed will likely be in a distressed state when the next disruptive event occurs. Natural recovery has not progressed to completion and storms of relatively low magnitude can cause further disruption and accelerated erosion.

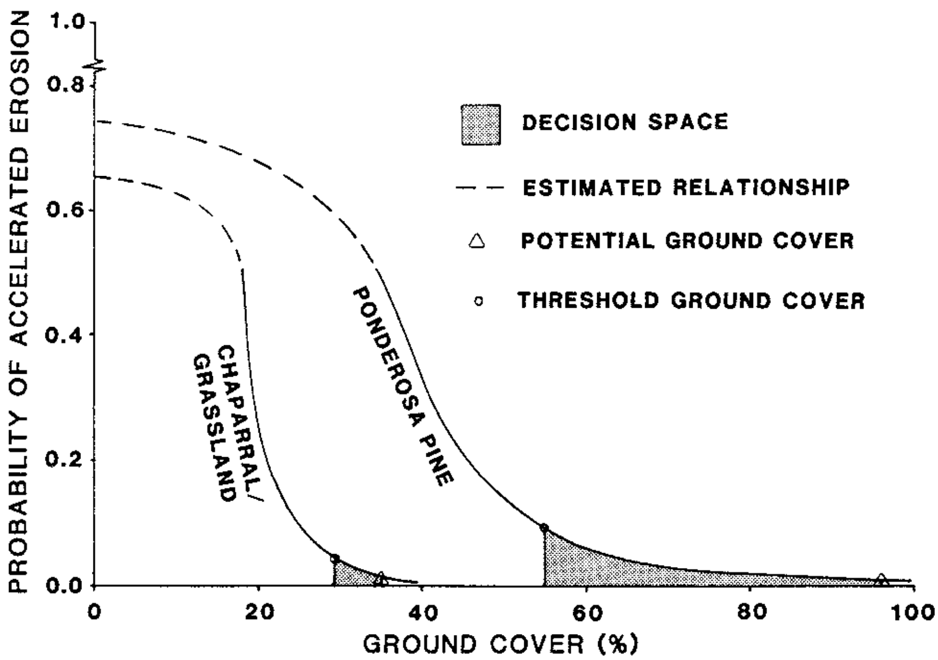


*Figure 4. Probability of accelerated erosion versus ground cover for a Ponderosa pine site and a chaparral/grassland site.*

Figure 4 depicts a sigmoid relationship between the risk of landscape disruption and ground cover. At ground cover levels approaching zero, vegetation plays an insignificant role in controlling hydrologic response. However, as ground cover increases, so does its role. Incremental changes in ground cover cause proportionate decreases in overland flow rates and increases in surface resistance. However, as ground cover levels approach potential, the effects of incremental increases in cover on infiltration rates and resistance begin to diminish. The system approaches the limit that ground cover can exert within that ecosystem. At potential ground cover, an undisturbed ecosystem exhibits resilience via excess capacity and alternate mechanisms with which it can infiltrate precipitation and dissipate erosive energy.

Managers can relate changes in ground cover to changes in the probability of disruption directly from Figure 4. For example, on the Ponderosa pine curve, at 30 percent ground cover there is a 60 percent chance of receiving a disruptive event in any given year. Increasing cover to 85 percent reduces that probability to just 2 percent. Figure 4 also reveals the relative sensitivity of ecosystems to accelerated erosion and to impairment of hydrologic function. Notice that, for the Ponderosa pine site, there is a span of 40 percent between threshold and potential ground cover, and the slope between these points is commensurately shallow. These factors reflect the high resiliency within this ecosystem. The relationships are much different for the chapparral/grassland site. Here the difference between threshold and potential ground cover is only 7 percent and the slope of the entire curve is much steeper. A prudent land manager has little room to manuver in terms of manipulating ground cover.

## Applications

The central application of the relationships and figures presented here allows practitioners and decision makers to quickly visualize the dynamics of runoff, accelerated erosion, ground cover and risk. Using this technique enables managers to identify situations where present management creates a sizable risk of incurring future watershed damage. By identifing such situations, and appropriately altering management activities, managers can prevent watershed damage, negating the need for capital investments to perform rehabilitation. Other possible applications exist. Knowing existing ground cover, threshold cover, and the relationship between cover and risk of accelerated erosion will allow managers to apply a rational process in developing a priority list for watershed restoration investments. It will also allow for the development of a target ground cover unique to each watershed to gage the success of restoration efforts.

Defining the relationship between runoff, ground cover and return interval, as in Figure 2, allows for more efficient design of small flood control structures. Such structures are commonly designed without considering ground cover improvements as a way to moderate peak flows. Figure 2 permits the designer to evaluate the most cost effective combination of structure heights and ground cover improvements.

Ultimately these techniques will allow for more sophisticated resource allocation and planning. Functionalizing ground cover-runoff-return interval relationships along with adjunctive relationships, such as ground cover-biomass, and ground cover-flood damage, permits finding that ground cover which provides maximum resource benefits. Presently, watershed condition indices function only as constraints that help define the decision space. The system of evaluating watershed condition presented here is still in the developmental stage. To develop a system that practioneers can easily apply over large areas, a number of simplifing assumptions were made. Numerous field trials must take place to assess its reliability. Further research must help define and refine these relationships, along with validating potential ground cover levels for various arid and semiarid ecosystems.

## SUMMARY AND CONCLUSION

Watershed condition is an umbrella concept that can integrate many resource values and apply to a wide range of ecosystems. The index used to measure watershed condition must reflect the controlling hydrologic processes that management practices impact. Typically, a watershed at potential allows a maximum amount of precipitation to infiltrate the soil. Delivery pathways to channels are at a maximum length, and soil moisture regimes promote landform stability.[1] These conditions provide the highest base flows and rates of biomass production and the lowest flood peaks, mass wasting and sediment production rates. Based on the concepts of landscape inertia and resilience, one can define a threshold unique to a given watershed wherein incremental changes in the index will produce successively greater negative impacts on watershed values, and where the risk of sustained watershed impairment becomes

1/ Not all of these attributes are mutually compatable in every case, e.g., maximum infiltration of precipitation versus landform stability.

great. The decision space for a prudent land manager lies between the potential and threshold values.

The concept of man-caused accelerated erosion presented here for arid and semiarid landscapes allows defining a threshold level of ground cover below which the risk of accelerated erosion, and continuous impairment of hydrologic function becomes unacceptable. Displaying the ground cover-runoff-return interval relationship and the risk of watershed damage as a function of ground cover allows the land manager to quickly discern the risks and tradeoffs involved in manipulating ground cover. It also provides a way of visually displaying the sensitivity of different ecosystems to altered ground cover. The techniques utilized here have other potential applications, such as ranking watershed restoration needs, providing a target cover level for restoration projects, for flood control design, and resource planning and allocation.

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# ESTIMATING SEDIMENT DISCHARGE IN CANALS IN COLORADO

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## ABSTRACT

Suspended-sediment concentrations were measured at the South Platte River and at four sites on each of three nearby irrigation canals of the South Platte River in northeastern Colorado during 1982-83. Suspended-sediment discharge was calculated and log linear-regression equations were developed at each site to estimate suspended-sediment discharge when only water discharge is available. Analysis showed that the best estimate of suspended-sediment discharge in the canals was made using the water discharge at each canal site and the water discharge in the South Platte River.

Measured suspended-sediment concentrations of the South Platte River were compared to measured suspended-sediment concentrations at the upstream measurement site of each canal. The correlation coefficients indicated that suspended-sediment concentrations in the South Platte River are highly related to the suspended-sediment concentrations in the three canals and affect the suspended-sediment inflow to the canals.

## INTRODUCTION

Suspended-sediment concentrations were measured at the streamflow-gaging station 06758500 South Platte River near Weldona and at four sites each on the Fort Morgan Canal (FM1-FM4), the Upper Platte and Beaver Canal and Ditch (UP1-UP4), and the Lower Platte and Beaver Canal and Ditch (LP1-LP4) (fig. 1). These sites were used because they are located in the vicinity of the proposed Narrows Reservoir and could be affected by its construction. The data were collected in cooperation with the U.S. Bureau of Reclamation to serve as baseline data for comparison with data collected during and after construction of the proposed reservoir. A major concern with the construction of the proposed Narrows Reservoir is the potential effect on the canals of conveying water released with low suspended-sediment concentrations. The fine-sized particles are necessary in the canals because, as they settle out, they effectively seal the canal and reduce leakage to the ground-water system. Impoundment and subsequent release of the water from the reservoir could result in significantly lower suspended-sediment concentrations in the river immediately downstream from the dam. As a result, water low in suspended-sediment concentrations is released to the downstream user, possibly leading to degradation of the channels of the South Platte River and irrigation canals.

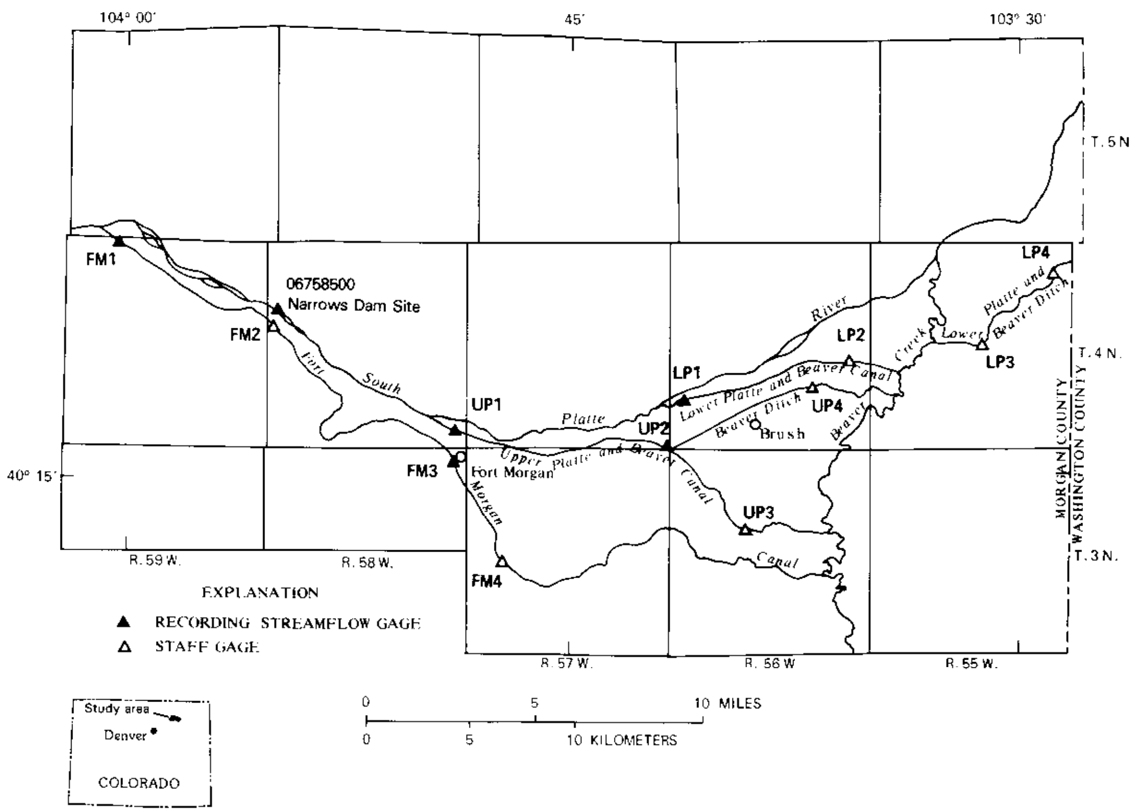


Figure 1. Location of study area and suspended-sediment sampling sites.

## SOUTH PLATTE RIVER-CANAL RELATIONS

Daily suspended-sediment data were collected at all measurement sites. The suspended-sediment concentration was used to calculate the suspended-sediment discharge at each canal site with the equation:

$$Q_s = Q_c C_c K,$$

where $Q_s$ = suspended-sediment discharge, in tons per day;

$Q_c$ = canal water discharge, in cubic feet per second;

$C_c$ = canal suspended-sediment concentration, in milligrams per liter; and

$K$ = conversion factor = 0.0027.

Regression relations were then developed for each canal site between the logarithms of suspended-sediment discharge and logarithm of water discharge at the canal site. The correlation coefficients and standard errors were poor. To improve the equations, the logarithms of the suspended-sediment discharge in the canal were regressed against the logarithms of the South Platte River water discharge and the canal water discharge. Further improvement in the correlation coefficients and the standard errors occurred when the logarithm of the South Platte River water discharge was added to the equations (table 1).

The multiple-regression equations in table 1 suggest that suspended-sediment concentrations in the canals are dependent upon the suspended-sediment concentrations and water discharge in the South Platte River. Water discharge in the canals is often independent of water discharge in the South Platte River because the canals carry an amount of water that has been allocated and does not totally depend on the natural runoff. The suspended-sediment concentrations and thus the suspended-sediment discharges will be higher in the canals when the water discharge in the South Platte River is higher because the concentration and discharge of suspended sediment in the South Platte River increase with water discharge.

To further examine the potential problem of releasing relatively clear water into the canals, relations between suspended-sediment concentrations in the canals and suspended-sediment concentrations in the South Platte River were determined. Logarithms of the suspended-sediment concentrations measured at the upstream site of each canal, FM1 (Fort Morgan Canal), UP1 (Upper Platte and Beaver Canal), and LP1 (Lower Platte and Beaver Canal), were regressed against the logarithms of the suspended-sediment concentrations measured in the South Platte River.

Regression equations presented in table 2 indicate that suspended-sediment concentrations in the canals are closely related to the suspended-sediment concentrations in the river. The correlation coefficients and the standard errors for the Upper Platte and Beaver Canal and for the Lower Platte and Beaver Canal indicate a good relation between suspended-sediment concentrations in the canals to the suspended-sediment concentration in the South Platte River. The correlation coefficient is lower and the standard error is higher in the relation for the Fort Morgan Canal indicating that the relation is weaker. The intake for the Fort Morgan Canal is upstream from the gage site on the South Platte River, and it appears there was less suspended sediment moving in the river at the Fort Morgan Canal intake site than further downstream. Because the concentration in the canals and the river are closely related, lower suspended-sediment concentrations in the South Platte River may cause a reduction in suspended-sediment concentration in the three irrigation canals.

Table 1.--*Summary of multiple-regression relations for 1982-83 suspended-sediment data for all canal sites*

[n, number of data points used in regression analysis; r, correlation coefficient; se, standard error, in percent; $Q_s$, suspended-sediment load, in tons per day; $Q_c$, canal discharge in cubic feet per second; $Q_{sp}$, South Platte River discharge, in cubic feet per second]

| Site (reference number) | n | Regression equation | r | se | Range of canal water discharge | |
|---|---|---|---|---|---|---|
| | | | | | Minimum | Maximum |
| FM1 | 179 | $Q_s = 0.0005\ Q_c^{1.68} Q_{sp}^{0.485}$ | 0.91 | 50 | 31 | 253 |
| FM2 | 180 | $Q_s = 0.0001\ Q_c^{1.87} Q_{sp}^{0.555}$ | .94 | 46 | 15 | 214 |
| FM3 | 185 | $Q_s = 0.00007\ Q_c^{2.00} Q_{sp}^{0.532}$ | .91 | 76 | 16 | 193 |
| FM4 | 21 | $Q_s = 0.0016\ Q_c^{2.03} Q_{sp}^{0.168}$ | .92 | 73 | 17 | 149 |
| UP1 | 182 | $Q_s = 0.0001\ Q_c^{1.78} Q_{sp}^{0.629}$ | .95 | 34 | 36 | 230 |
| UP2 | 138 | $Q_s = 0.00005\ Q_c^{2.15} Q_{sp}^{0.521}$ | .87 | 62 | 37 | 178 |
| UP3 | 147 | $Q_s = 0.0050\ Q_c^{1.46} Q_{sp}^{0.403}$ | .78 | 84 | 0.1 | 28 |
| UP4 | 142 | $Q_s = 0.00002\ Q_c^{2.86} Q_{sp}^{0.524}$ | .85 | 77 | 10 | 56 |
| LP1 | 196 | $Q_s = 0.00004\ Q_c^{2.15} Q_{sp}^{0.539}$ | .96 | 33 | 24 | 183 |
| LP2 | 146 | $Q_s = 0.00003\ Q_c^{2.25} Q_{sp}^{0.526}$ | .95 | 34 | 20 | 159 |
| LP3 | 191 | $Q_s = 0.000002\ Q_c^{2.59} Q_{sp}^{0.776}$ | .90 | 65 | 20 | 114 |
| LP4 | 187 | $Q_s = 0.000008\ Q_c^{2.43} Q_{sp}^{0.734}$ | .90 | 67 | 16 | 99 |

Table 2.--*Summary of regression relations for suspended sediment between streamflow-gaging station 06758500 South Platte River near Weldona and the upstream measurement site on each canal*

[n, number of data points used in regression analysis; r, correlation coefficient; se, standard error, in percent; $C_c$, sediment concentration of the canal, in milligrams per liter; $C_{sp}$, sediment concentration of the South Platte River, in milligrams per liter]

| Site (reference number) | n | Regression equation | r | se |
|---|---|---|---|---|
| FM1 | 185 | $C_c = 4.69\ C_{sp}^{0.598}$ | 0.63 | 61 |
| UP1 | 188 | $C_c = 4.21\ C_{sp}^{0.636}$ | .83 | 46 |
| LP1 | 198 | $C_c = 3.76\ C_{sp}^{0.673}$ | .87 | 43 |

## SUMMARY

Analysis of measured suspended-sediment data collected on the South Platte River and four sites each on the Fort Morgan Canal, the Upper Platte and Beaver Canal and Ditch, and the Lower Platte and Beaver Canal and Ditch indicate a close relation between suspended sediment in the the river and in the three canals. Suspended-sediment discharge at the canal sites, although dependent on the canal water discharge, were also dependent on the South Platte River water discharge. The addition of river water discharge to the equations, which estimate suspended-sediment discharge in the canals, improved the correlation coefficients and reduced the standard errors. This occurs because the suspended-sediment concentrations in the canals are dependent upon the suspended-sediment concentrations in the river, and suspended-sediment concentrations in the river are related to river water discharge.

Suspended-sediment discharge in the canals may increase if the river sediment concentration increases and the canal water discharge remains constant. These relations also indicate that if the impoundment and release of river water with a reduced sediment supply occurs, there will be a reduction in sediment supply to the canals which may decrease the efficiency of the canals to transport water.

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# SEDIMENT TRANSPORT CAPACITY OF FLOW IN RILLS AND SMALL CHANNELS

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## ABSTRACT

A method of modifying currently used sediment transport equations to take into consideration differences in the availability of various particle size classes of sediment was developed using the concept of an effective transport efficiency. The modifications were based on theory and tested with field and laboratory measurements of the transport of various sizes of sediment in different sized channels. A comparison of test results with values predicted by the unmodified transport equations indicates that the latter can seriously overestimate the amount of sediment transported in small channels. The proposed modifications greatly increased the accuracy with which the transport capacity for smaller sized particles was estimated.

## INTRODUCTION

Most current sediment yield models use some form of the universal soil loss equation (USLE) to estimate upland erosion. The USLE estimates the amount of detached soil particles that are made available for transport from the upland areas of watersheds. Soil particles, after being detached by rainfall or runoff, are transported relatively short distances by splash action and overland flow to rills. From there, a routing equation is generally used to transport the sediment through the watershed channel system to the outlet. The amount of sediment actually leaving the watershed depends upon the transport capacity of the flow in the rills and channels and the physical characteristics of the sediment being transported. Many studies have been conducted to relate streamflow and sediment discharge and are summarized by Simons and Senturk (1977). Most of these studies, however, deal with transport capacity in noncohesive, alluvial channels. These conditions differ greatly from that of flow in small rills and channels that predominate in upland areas of watersheds. There is no assurance that the open channel flow equations are valid for the conditions of extremely shallow flow depths and small flow rates found in upland rills. Factors such as surface energy, gravitational effects, and vertical forces all affect sediment transport under these conditions. Alonzo et al. (1981) evaluated nine of the more commonly used sediment transport formulas for their accuracy for use in watershed modeling of upland areas. They found that no one formula satisfactorily described the sediment transport characteristics for all flow conditions likely to be encountered in the field. The objective of this study is to determine flow transport efficiencies of various sediment size classes based on flow characteristics in upland rills and channels.

## THEORY

Most sediment transport equations available for estimating the transport capacity of flow in small channels are based on the assumption of an adequate supply of relatively uniform and noncohesive sediment particles. Few of these equations are able to deal adequately with nonuniform sediment of varying densities. In considering nonuniform sediment, only the largest particles are usually present in sufficient quantity to satisfy the transport capacity of flow in a rill or channel. Less often is there a sufficient amount of the smaller particles, especially clay size particles, to satisfy the transport capacity of the flow. This paper discusses a method to modify a commonly used transport equation to consider differences in the availability of various eroded particle size-classes in order to estimate the sediment load.

The Bagnold suspended sediment transport equation, (Bagnold, 1966) is based on the concept of energy balance, is applicable to fully turbulent flow, and involves the basic relationships between work, the energy expenditure of the stream and the quantity of the sediment transported.

The Bagnold equation is expressed as

$$g_s = k\tau v^2/v_{ss} \qquad [1]$$

where

- $g_s$ is the suspended sediment transport capacity in kg/sec-m,
- $\tau$ is the shear stress in kg/m$^2$
- $v$ is the transport velocity of the suspended load in m/sec.
- $v_{ss}$ is the particle fall velocity in m/sec, and
- $k$ is a transport capacity factor

The transport capacity factor is normally calculated as

$$k = (1-e_b)\ e_s[\gamma_w/(\gamma_s-\gamma_w)] \qquad [2]$$

where

- $e_b$ is the bedload transport efficiency,
- $e_s$ is the suspended load transport efficiency,
- $\gamma_s$ is the specific weight of the sediment in kg/m$^3$, and
- $\gamma_w$ is the specific weight of water in kg/m$^3$.

The bedload transported in a channel is described by DuBoys (DuBoys, 1879) as

$$g_b = c_s\ \tau(\tau-\tau_c)\ b\gamma_s \qquad [3]$$

where

- $g_b$ is the amount of sediment transported in the bedload in kg/sec-m,
- $\tau$ is the shear stress on the bed in kg/m$^2$,
- $\tau_c$ is the critical shear stress in kg/m$^2$ at which $g_b$ becomes zero (assumed to be 0.07296 kg/m$^2$)1/,
- $b$ is the width of the channel bottom in m, and
- $c_s$ is a constant depending on sediment size.

While it is recognized that the value of $\tau_c$ will vary with the size and density of sediment particles, experimental results have indicated that the value shown can be used satisfactorily for a wide range of conditions. The value of $c_s$ varies according to the equation (extrapolated from Straub, 1935)

1/ Little, W. C. Unpublished data, 1967.

$$c_s = 0.658x10^{-3}\ D_s^{-0.7524} \qquad [4]$$

where $D_s$ = particle diameter in mm. Equation 3 was derived assuming steady flow conditions and thus may not be valid for conditions of unsteady, nonuniform flow. The total sediment load for a channel, $g_t$, is the sum of the bedload and the suspended load.

From equation [2], the value of k assumes that the product of $(1-e_b)e_s$ is constant for all particle sizes. From flume studies, the combined efficiency term, $(1-e_b)e_s$, has been found to be approximately 0.01 (Simons and Senturk, 1977). However, this value was determined from studies using mostly cohesionless sand grains (Bagnold, 1956). It was assumed that a supply of particles was sufficient for each size class to satisfy the calculated transport capacity. Under these conditions, the assumption of a constant k in equation [1] is probably valid. However, rill flow from cohesive soils is generally much below its sediment-carrying capacity (Meyer et al., 1975). This may be because either the erosion rate on the cohesive soil does not produce enough sediment to fill the transport capacity or most of the detached soil particles available for transport in the rill system consist mainly of aggregated material (Young, 1980). In the latter case, relatively few of the finer sized particles, especially clay, are available for transport as individual particles. Therefore, the value of k will not be constant for all particle sizes but will vary with the availability of the material of each size class and the density of the material.

In reality, the amount of soil particles of each size class that becomes available for transport in a rill system is dependent on a number of different factors, including the rate of soil detachment in the rills and interrill areas, the movement of interrill detached particles into rills by splash action and overland flow, the resistance of soil aggregates to breakdown under the forces of raindrop impact and flowing water, and various topographic and vegetative factors. These all combine to form a complex system comprising the erosion process. Current knowledge is insufficient to adequately describe all of the relationships and interactions involved, and thus, it becomes necessary to develop empirical relationships relating to sediment transport to fill the immediate needs of erosion modelers. In the future, as research sheds more light on the exact nature of the physical processes involved, more physically based relationships will replace the empirical relationships currently being developed.

Because the suspended sediment transport capacity, $g_s$, for a particular particle size varies inversely as the settling velocity, $v_{ss}$, of that particle (eqn. [1]) and the bedload transport capacity varies inversely as the particle diameter (eqn. [3] and [4]), the calculated total transport capacity, $g_t$, for larger particles is much less than for smaller particles which have very slow settling velocities. Thus, for sand particles and large aggregates, the supply of sediment particles may be sufficient to satisfy the transport capacity of the flow. However, for smaller particles, the transport capacity is usually quite high and the supply of available particles is usually not sufficient to satisfy it. In that case, the actual sediment transport will be operating at less than 100 percent efficiency. If k in equation 2 is allowed to equal $0.01\ \gamma_w/(\gamma_s-\gamma_w)$, (Simons and Senturk, 1977), then it will vary only with particle density but the total sediment transport must be expressed as

$$g_t = \Sigma \eta_i (g_{si} + g_{bi}) \quad [5]$$

where

- $g_t$ is the actual amount of sediment transported in kg/sec-m,
- $\eta$ is an effective transport efficiency related to particle size, and
- $i$ refers to various particle size classes.

The effective transport efficiency in this case is a term reflecting the combined bedload and suspended load transport efficiency. Because transport capacity increases with decreasing particle size and because the availability of small particles is often limiting in cohesive soils, the effective transport efficiency, $\eta$, is less for smaller particles.

It is reasonable to assume that, for shallow flows in rills and small channels, the equations for open channel flow can be modified to include gravitational and/or surface energy effects (Young and Mutchler, 1969). Thus, sediment transport efficiency may be related to the relative magnitude of the sedimentation characteristics of a particle to the inertial forces of the flow, which can be expressed as a dimensionless ratio of the inertia forces to the gravitational forces. This ratio is an entrainment function (Shields, 1936), $E_{fi}$, which is the Froude number for a particular particle size calculated by

$$E_{fi} = \frac{\tau}{D_{si}(\gamma_s - \gamma_w)} \quad [6]$$

where $D_s$ is the particle diameter in m and

$$\tau = \gamma RS \quad [7]$$

where $R$ is the hydraulic radius of the channel in m, and
$S$ is the slope of the channel bottom in m/m.

Since the value of $E_f$ increases as the sediment particle size and density decreases, the effective sediment transport efficiency, $\eta$, should vary inversely with $E_f$.

## PROCEDURE

The effective transport efficiency for each particle size class depends on the availability and the sedimentation characteristics of that size particle. In order to determine the efficiency with which particles of different sizes are transported in field situations, data from several different field studies and one lab study were examined. In each case, sediment loads and flow characteristics were measured from different sized rills or furrows. The studies included 12 data sets from five soils -- eight from a Barnes loam (Udic Haploboroll) (Young and Mutchler, 1969; Young and Wiersma, 1973), one from a Russell silt loam (Typic Hapludalf) (Meyer et al., 1975), one from a Fox loam (Typic Hapludalf) (Meyer et al., 1970), one from a Crofton silt loam (Typic Ustorthent) (Young and Wiersma, 1973), and one from a Collins silt loam (Aquic Udifluvent) (Mutchler and McGregor, 1983).

## RESULTS AND DISCUSSION

The soil characteristics, channel geometry, runoff rates, and sediment loads are listed in Table 1. Particle size distributions of the undispersed sediment were estimated from the primary particle size distribution of the matrix soil given in Table 1 and the relationships developed by Foster et al. (1985).

Table 1. Data sets used.

| Soil | Soil Texture | | | Runoff Characteristics | | | | Channel Characteristics | |
|---|---|---|---|---|---|---|---|---|---|
| | sa | si | cl | Flow Rate | Vel | Flow Depth | Sediment Load, q | Width | Slope |
| | ---- % ---- | | | kg/sec-m | m/sec | --- mm --- | kg/sec-m | m | m/m |
| Fox loam (Typic Hapludalf) | 22 | 57 | 21 | 0.13 | 0.14 | 0.93 | 0.009 | 3.700 | 0.150 |
| Collins silt loam (Aquic Udifluvent) | 17 | 63 | 20 | 0.24 | 0.04 | 6.50 | 0.004 | 3.700 | 0.001 |
| Crofton silt loam (Typic Ustorthent) | 16 | 56 | 26 | 1.94 | 0.37 | 5.50 | 0.008 | 0.042 | 0.090 |
| Russel silt loam (Typic Hapludalf) | 24 | 59 | 17 | 0.33 | 0.12 | 5.50 | 0.023 | 0.044 | 0.060 |
| Barnes loam (Udic Haploboroll) | 48 | 34 | 18 | 1.44 | 0.30 | 5.00 | 0.011 | 0.041 | 0.060 |
| | | | | 4.82 | 0.12 | 5.76 | 0.012 | 0.087 | 0.090 |
| | | | | 7.92 | 0.23 | 6.70 | 0.049 | 0.050 | 0.053 |
| | | | | 14.00 | 0.11 | 10.43 | 0.050 | 0.036 | 0.052 |
| | | | | 10.22 | 0.25 | 14.31 | 0.133 | 0.050 | 0.056 |
| | | | | 30.05 | 0.11 | 13.00 | 0.032 | 0.026 | 0.046 |
| | | | | 9.19 | 0.27 | 6.89 | 0.060 | 0.105 | 0.056 |
| | | | | 13.08 | 0.27 | 9.56 | 0.114 | 0.071 | 0.057 |

Suspended loads were estimated using equation [1] and bedloads were estimated by equation [3]. Total sediment load was estimated as the sum of the suspended load and bedload. An entrainment function, $E_f$, was calculated for each set of flow conditions and particle size class using equation [6] and is shown in Table 2. Effective transport efficiency, $\eta_i$, for each particle size class was determined by dividing the measured sediment load of a particular particle size group by the estimated total sediment load for that group.

Table 2. Entrainment function values by particle size class.

| | Sand | Large Agg | Small Agg | Silt | Clay |
|---|---|---|---|---|---|
| Fox l. | 0.421 | 0.463 | 4.964 | 8.424 | 43.438 |
| Collins si.l. | .020 | .022 | .231 | .392 | 2.019 |
| Crofton si.l. | 1.205 | 1.326 | 14.206 | 24.107 | 124.301 |
| Russel si.l. | .484 | .533 | 5.707 | 9.685 | 49.938 |
| Barnes l. | 1.115 | 1.226 | 13.139 | 22.297 | 114.969 |
| | .458 | .503 | 5.393 | 9.152 | 47.188 |
| | .509 | .560 | 6.000 | 10.182 | 52.500 |
| | .767 | .843 | 9.036 | 15.333 | 79.062 |
| | .921 | 1.013 | 10.857 | 18.424 | 95.000 |
| | .652 | .717 | 7.679 | 13.030 | 67.188 |
| | .579 | .637 | 6.821 | 11.576 | 59.688 |
| | .797 | .877 | 9.393 | 15.939 | 82.188 |

A simple regression of transport efficiency, $\eta_i$, on the entrainment function, $E_{fi}$, using seven of the data sets -- four from the Barnes soil, one from the Collins, and one from the Crofton -- yielded a power function

$$\eta_i = 0.322 E_{fi}^{-1.768} \qquad [8]$$

$$r^2 = 0.82$$

$$n = 35$$

The relationship and data are shown in Fig. 1. This relationship was tested using the remaining five data sets. Figure 2 shows the results of the test. A simple regression of the calculated sediment load, $g_i$, on the modified measured sediment load, $q_i/\eta_i$ gave a correlation coefficient of 0.85.

From this, it is apparent that an entrainment function reflecting relative gravitational and inertial effects can be used with the sediment transport equations of Bagnold and DuBoys to account for efficiency of transport of different size particles according to equations [5] and [8]. Values obtained provide a more realistic and accurate representation of the sediment load in the small rills and channels normally found in the upland areas of watersheds. If effective transport efficiency is omitted, the amount of sediment transported can be seriously overestimated, especially for smaller sized particles.

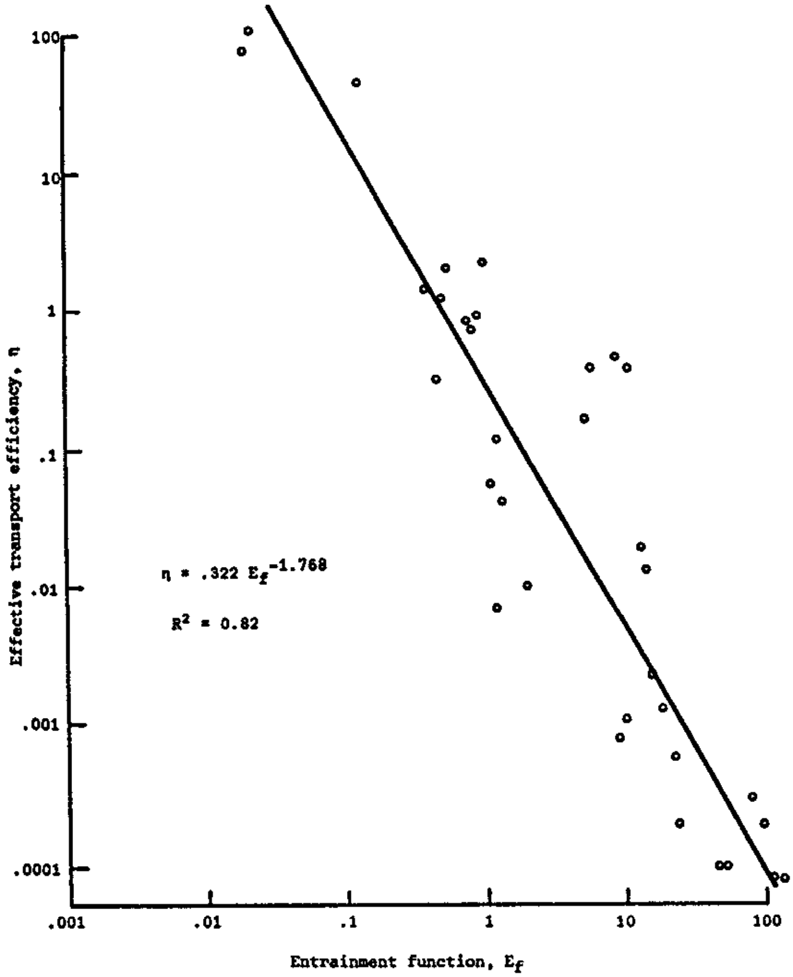


Fig. 1. Effective transport efficiency, versus entrainment function, $E_f$.

There are some situations where this relationship should be used with caution. For example, in very coarse textured soils which are relatively noncohesive, rill flow can have a large transport capacity for smaller sized particles, but the particles may not exist in sufficient quantity in the soil to fill this capacity. In that case, the resulting effective transport efficiency may be much lower than that estimated by equation [8].

In the other extreme, where flow rates are very low, such as where land slopes are very flat, calculated transport capacity will be very small and the supply of all size particles may be sufficient to satisfy it. The actual transport efficiency then may approach 100% for all size classes of sediment, including the very fine particles. In this case, the values estimated by equation [8] will be too low. In most cases, however, the values of calculated with equation [8] will be realistic and should provide improved estimates of sediment loads in small channels.

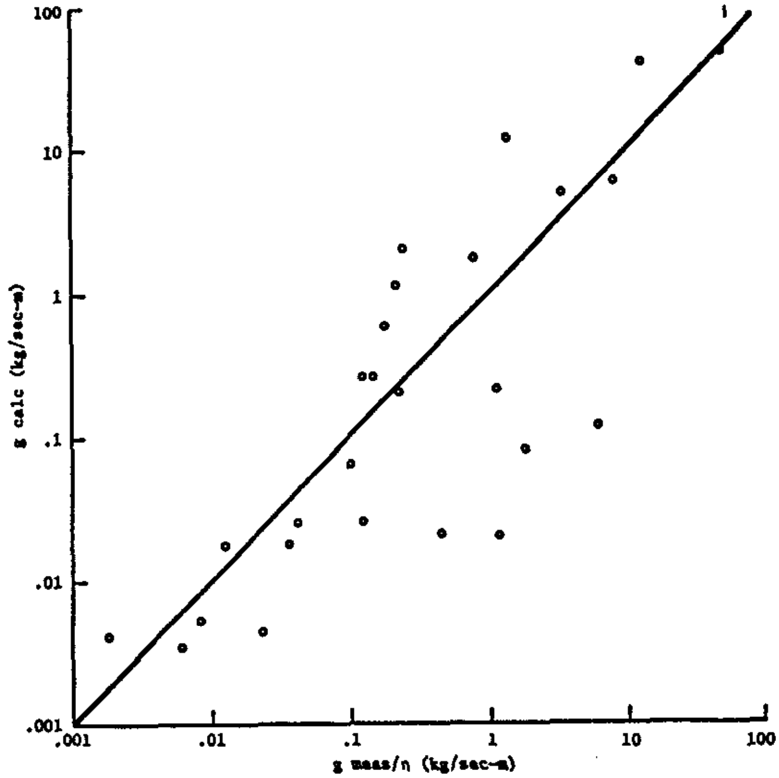


Fig. 2. Modified measured sediment transport versus calculated sediment transport for five different flow conditions.

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# SEDIMENT ROUTING-AN ELECTRONIC SPREADSHEET TEMPLATE

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## ABSTRACT

Sediment routing is a mathematical modeling technique that simulates deposition and transport of sediment within the stream channel and on the flood plain. Sediment routing involves numerous calculations. It is often necessary to redo these calculations to fine tune the system. An electronic spreadsheet template was devised to save time in these repetitive calculations.

## INTRODUCTION

The United States Department of Agriculture (USDA) Soil Conservation Service (SCS) is involved in many watershed projects where an estimate of the volume of sediment produced by a watershed is required for designing flood retarding structures, debris basins, or channel structures. Engineers need to know the volume and texture of sediment that will be moved through or stored in project structures. Economists use the volumes to calculate the dollar cost of damages that could be reduced. Data on volume and texture of the sediment is also needed to evaluate the downstream impacts of a project. Such data is provided now by sediment routing, a mathematical modeling technique that simulates deposition and transport of sediment within the stream channel and on the flood plain. Under this modeling procedure, the sediment must be routed downstream through the project area taking into account areas of deposition due to overbank flow, slow flow areas, or ponding. These areas are determined by field examination of the stream channel and the flood plain. New sources of sediment along the way must be taken into account. The calculations are performed several times in order to have the model more closely represent the actual conditions found in the watershed.

The addition and subtraction of sediment in the stream can be thought of as a sediment budget. Since it is a budget, the calculations can be done on an electronic spreadsheet using a mini or micro-computer. An electronic spreadsheet is a program that allows the user to interrelate different rows and columns of numbers. A template is a specific application of the electronic spreadsheet. The sediment routing template was designed to represent only a portion of the stream. The portion of the stream being modeled would have input from the upstream reach, and the possibility of two additional inputs and three additional outputs before the downstream end of the section of stream being modeled is reached (Figure 1). The output from the just calculated section would be reentered into the program to calculate the next section, and so on until the outlet of the project area is reached.

## TEMPLATE INPUTS

Several parameters must be calculated before this template can be used. Each input is the combined total from several different types of erosion, i.e., sheet and rill, mass wasting, streambank, land area (urban), road, gully, and other (mining, construction, etc.). In most cases, the average annual sediment yield for each process is used. The sediment yield of a watershed is the amount of material passing from the drainage basin at some selected boundary. The sediment yield can be determined from reservoir sediment surveys, United States Geological Survey (USGS) and State suspended sediment records, or the ratio method using known data from a similar nearby watershed. The average annual sediment yield is calculated for the beginning point of the first reach. This includes sediment derived from all types of erosion. Sheet and rill erosion is calculated using Wischmeir's Universal Soil Loss Equation (USLE) (USDA, 1978) (Wischmeir, 1958). Streambank and gully erosion are calculated from field measurements using the Direct Volume Method (area of erosion times a lateral recession rate) and expanded for the whole watershed. Gross erosion is the sum total of all the different types of erosion. A sediment delivery ratio (SDR) for each type of erosion must also be determined. The sediment delivery ratio is defined as the sediment yield divided by the gross erosion for the watershed. This represents the efficiency of the watershed to move particles from the erosion source to the point where the SDR was measured. A sediment delivery ratio can be determined using Flaxman's Slope Continuity Procedure (1974), by dividing a known sediment yield by your estimate of the gross erosion for the watershed, or using judgment. The amount of sediment entering the stream channel is calculated by multiplying the erosion by the sediment delivery ratio for that particular type of erosion. The volume of sediment is usually converted to tons of sediment using published or tested density values. Each category of erosion or sedimentation (input or output) requires the percentages of fines, sand, and gravel constituting the sediment. Gravel ranges from 4.76 to 76 mm in diameter, and sand ranges from 0.074 to 4.76 mm in diameter. Fines are smaller then 0.074 mm (USDA, 1971). This is important because the different grain sizes behave differently in water. The grain size percentages are derived from particle size distributions found in soil survey reports. If no published soils data are available, representative soil samples are collected and analyzed. The template is partially reprinted in Appendix A. Appendix B lists the formulas and labels used for each cell of the template.

For most projects, sediment yield for individual storm events are required. Sediment yield has been shown to be proportional to runoff (McGuiness et al, 1971). Jimmy Williams' (1975a) Modified Universal Soil Loss Equation (MUSLE) is used to predict the sediment yield from sheet and rill erosion for individual storm events. An explanation of this procedure can be found in SCS Geology Note No. 2 (1980). Sediment yields for at least three individual storms should be calculated. The average annual sediment yield can then be determined by measuring the area under a curve of storm frequency versus sediment yield. This answer should compare within 10 percent of yield calculations using the USLE and SDR. Adjustments of the SDR or the erosion rates are made if the MUSLE estimates are not within 10 percent of the original estimate.

The MUSLE only calculates the amount of sediment transported as suspended load in a stream. It usually consists of all the fines and a percentage of the sands. To calculate the total sediment load, the MUSLE sediment yield is divided by the percent of sediment in the suspended load. This percent is usually 100 percent of the fines and 10 to 50 percent of the sands. This total sediment load is then reapportioned using the grain size category percentages.

There is no established procedure to determine sediment yield from individual storms for other types of erosion than sheet and rill. It is recognized that the more severe the storm, the greater the sediment yield. To increase the sediment yield for different frequency storms, a ratio of the storm runoff (peak flow in cubic feet per second (cfs) or volume in acre-feet to the average annual runoff (2-year, 6-hour storm) is used. The average annual sediment yield is multiplied by the runoff ratio to determine the sediment yield for that frequency storm. Ratios are adjusted if the average annual sediment yield determined from the area under the curve of storm frequency versus sediment yield is not within 10 percent of the original estimate.

## TEMPLATE OUTPUTS

There are four possible outputs per section in the template: three diversion or overbank flow conditions and one channel deposition. Each output, except for channel deposition, requires the percentage of total sediment available to be removed from the system and the percentage of channel flow that is available for overbank flow. These data are needed because of the vertical distribution of the different sediment grain sizes in a stream (Figure 2). For overbank flow, the amount of sediment delivered to the flood plain depends upon the amount of sediment in the overbank flow portion of the flow and the length of time that overbank flow can occur. Overbank flow usually occurs during the peak of the runoff. Technical judgment must be used to determine the percentage of the total sediment load in the portion of flow flooding over the channel banks. The amount of sediment deposited on the flood plain depends on the velocity of the overland flow, and the trap efficiency of the land surface. Areas of slow flow will deposit all sand and a percentage of the fines if overflow velocity is less than 2.5 feet per second. For higher velocities, the possibility of flood plain scour must be examined. Areas of ponding will usually deposit almost 100 percent of the sediment carried in the flood water. Field observations or records of past flood damages will aid in deciding the proportions of volumes of sediment deposition between various deposition areas. The volumes of sediment deposited in the channel are estimated by field observation of existing areas of deposition, and by observing the grain-size composition of the deposits.

## SUMMARY

The use of an electronic spreadsheet program for sediment routing significantly reduces design time. For example, one iteration on a stream with nine reaches would require 30 hours to perform all the calculations by hand. The electronic spreadsheet template can do all these calculations in about 2.5 hours. The spreadsheet program does the required calculations quicker and with less chance for a mathematical error. All these calculations need to be repeated if the initial distribution of sediment deposition in the channel or on the flood plain does not match observed or recorded distributions. If the sediment must be rerouted to better fit the alternative conditions, only a few adjustments in the input data are required in order to produce the new output on the computer instead of doing all the calculations again by hand.

The process of determining sediment yield by the routing techniques described in this paper require judgment. The results can be calibrated from field observations of past floods and in some cases by reservoir sediment or suspended load data. Thus the calculated tons of sediment may be imprecise. However, the model does indicate the sedimentation processes that can occur in the project area, where sedimentation might occur, and the relative severity of problems caused by sedimentation.

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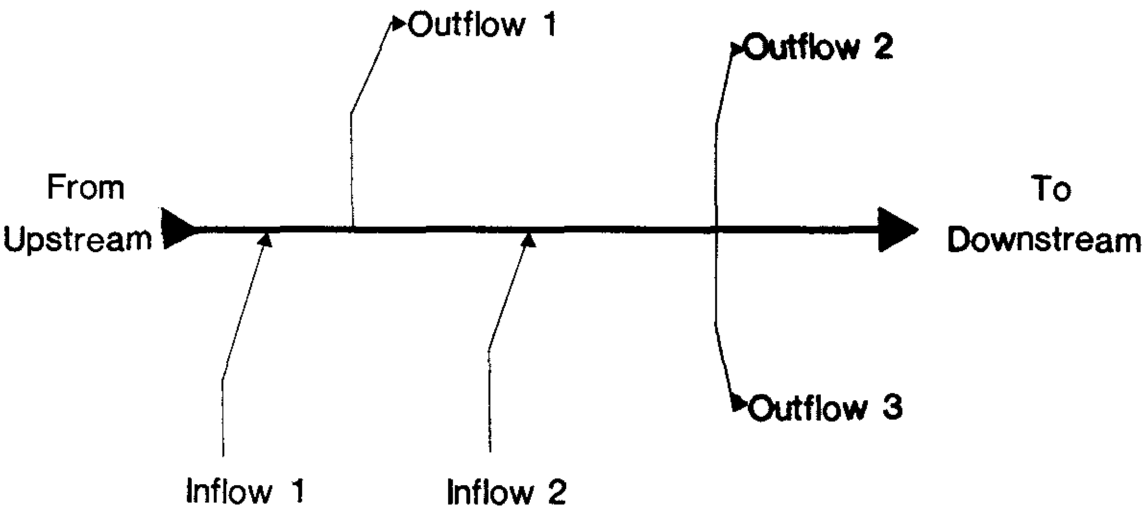
## Fig 1 Reach Schematic



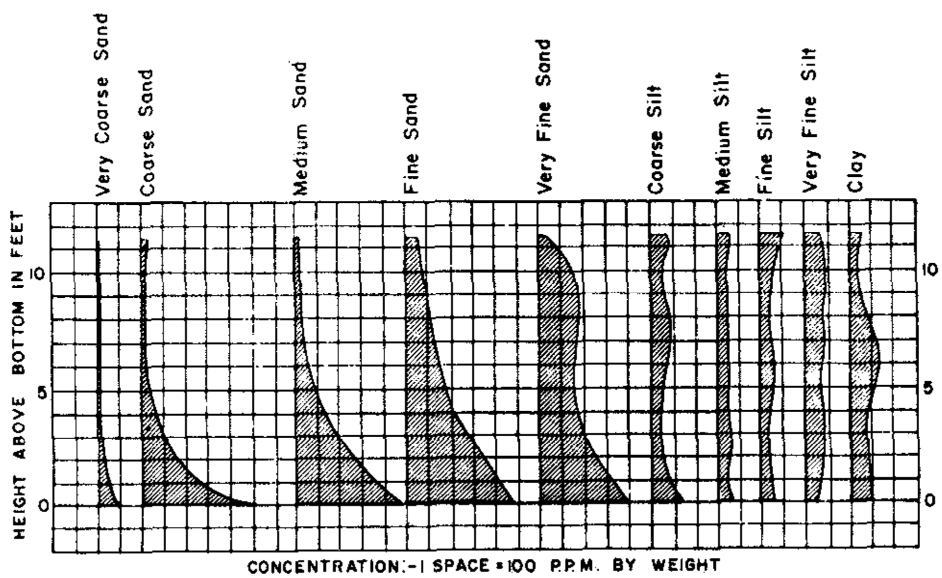




Figure 2.—Vertical distribution of sediment in Missouri River at Kansas City, Mo. From Federal Inter-Agency River Basin Committee (1963, p. 28).

# APPENDIX A

EOF(Input) = F161 EOF(Output) = A161 - J211
Program Documentation: Goto A213 EOF = J230

SEDIMENT TRANSPORT ROUTING SPREADSHEET


Please notify the author of any modifications needed or done.


ENTER THE FOLLOWING:
REACH ? ----> ?

| | |
|---|---|
| YEAR STORM | ? |
| RATIO Storm to Avg. Annual Storm | ? |
| MUSLE at Initial Input Point | ? |
| Avg. Annual Sediment Yield | ? |
| | |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |
| Percentage of Sand in Wash Load | ? |

********************** INPUT PARAMETERS **********************

| | |
|---|---|
| Input 1: MUSLE | ? |
| SDR: MUSLE | ? |
| Percentage of Fines | ? |
| Percentage of Gravel | ? |
| Percentage of Sand in Wash Load | ? |
| | |
| Input 1: Streambank - Annual | ? |
| SDR: Streambank | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |
| | |
| Input 1: Erosion From Roads | |
| Length of Roads (miles) | ? |
| Road Erosion Rate (t/mi) | ? |
| SDR: Roads | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |
| | |
| Input 1: Erosion from Other Sources | |
| Total Erosion From Other Sources(tons) | ? |
| SDR: Other Sources | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |

| | |
|---|---|
| Input 1: LANDSLIDE - ANNUAL | ? |
| SDR: LANDSLIDE | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |
| | |
| Input 1: Land Area Contribution | |
| Area (Acres) | ? |
| Avg. Annual Sediment Yield(t/ac) | ? |
| SDR: Land Area | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |
| | |
| Input 1: Erosion From Gullies | |
| Total Gully Erosion (tons) | ? |
| SDR: Gullies | ? |
| Percentage of Fines in Sediment | ? |
| Percentage of Sand in Sediment | ? |
| Percentage of Gravel in Sediment | ? |

********************** OUTPUT PARAMETERS **********************

| | | | |
|---|---|---|---|
| Output #1 % Sediment Available | ? | Output #2 % Sediment Available | ? |
| Output #1 % Overbank Flow | ? | Output #2 % Overbank Flow | ? |
| Percentage of Fines | ? | Percentage of Fines | ? |
| Percentage of Sand | ? | Percentage of Sand | ? |
| Percentage of Gravel | ? | Percentage of Gravel | ? |
| Output #3 % Sediment Available | ? | % Deposition in Main Channel | ? |
| Output #3 % Overbank Flow | ? | Percentage of Fines in Deposition | ? |
| Percentage of Fines | ? | Percentage of Sand in Deposition | ? |
| Percentage of Sand | ? | Percentage of Gravel in Deposition | ? |
| Percentage of Gravel | ? | | |

********************** RESULTS OF CALCULATIONS **********************

| DESCRIPTION | FINES | TONS OF SAND | GRAVEL | | |
|---|---|---|---|---|---|
| Initial Input | - | - | - | | |
| Sediment Yield | 0 | 0 | 0 | | |
| Total to Downstream | - | - | - | = | - |
| CONTRIBUTION: INPUT 1 | | | | | |
| Sheet & Rill | - | - | - | | |
| Landslide Erosion | 0 | 0 | 0 | | |
| Streambank Erosion | 0 | 0 | 0 | | |
| Land Area | 0 | 0 | 0 | | |
| Roads | 0 | 0 | 0 | | |
| Gullies | 0 | 0 | 0 | | |
| Other Sources | 0 | 0 | 0 | | |
| Subtotal | - | - | - | = | - |
| OUTPUT #2 | - | - | - | = | - |
| OUTPUT #3 | - | - | - | = | - |
| DEPOSITION (Main Channel) | - | - | - | = | - |
| (% of Grain Sizes)/100 | - | - | - | | |
| TOTAL INPUT | - | - | - | = | - |
| TOTAL OUTPUT | - | - | - | = | - |

*************************************************************

PROGRAM DOCUMENTATION

This program is designed as a template for an electronic spreadsheet. The template simulates a general version of a stream channel with several variations. In order to run the complete stream channel in question, the template must be used several times using the answers from the upstream reach as input data for the next reach.

Directions

1. Input is required wherever there is a "?".
2. Percentages are to be entered in decimal format, ie. 70% = 0.7.
3. If an option is not used in a particular reach, enter zero.
4. ERROR appears in results of calculations because values have not been entered yet.
5. Printouts require a 15 inch printer to print in regular print. Use compressed print for a printer with an 8 inch platen.

## PROGRAM LISTING

A listing of the cell entries follows. The letter-number combination after the greater than sign (>) is the cell location number. The letter is the column location and the number is the row location. Thus >B2: is second column, second row. The quotation mark (") designates that the cell contents are a label. These do not have to be entered unless the cell contents start with something other than a letter. Mathematical formulas start with a plus or minus sign, parenthesis, or an ampersand (@).

```
>A1:"EOF(Input) =
>C1:"EOF(Output)
>E1:"J211
>C2:"Goto A213
>B5:"TRANSPORT
>A6:"    Copyrigh
>A7:"    All righ
>A8:"Please notif
>D8:"fications ne
>A9:"Permission I
>D9:"s program to
>A10:"personnel of
>D10:"ion Service
>G10:"s.
>D13:"REACH
>G13:"    ?
>A16:"RATIO Storm
>E16:"    ?
>C17:"oint
>B18:"Sediment Yie
>A20:"Percentage o
>E20:"    ?
>C21:"diment
>B22:"f Gravel in
>A23:"Percentage o
>E23:"    ?
>C26:"*************
>H26:"*************
>A28:"Input 1:
>A29:"SDR: MUSLE
>B30:"f Fines
>B31:"f Sand
>B32:"f Gravel
>B33:"f Sand in Wa
>A35:"Input 1:
>E35:"    ?
>E36:"    ?
>C37:"ediment
>B38:"f Sand In Se
>A39:"Percentage o
>E39:"    ?
>C41:" Annual
>B42:"ank
>B43:"f Fines in S
>A44:"Percentage o
>E44:"    ?
>C45:"Sediment
>B47:"Land Area Co
>E48:"    ?
>C49:"Id
>A50:"SDR: Land Ar
>B51:"f Fines in S
>A52:"Percentage o
>E52:"    ?
>C53:"Sediment
>B55:"Erosion From
>B56:"ads (Miles)
>B57:" Rate
>A58:"SDR: Roads
>B59:"f Fines in S
>A60:"Percentage o
>E60:"    ?
>C61:"Sediment
>B67:"Erosion From
>B68:"Erosion
>A69:"SDR: Gullies
>B70:"f Fines in S
>A71:"Percentage o
>E71:"    ?
>B1:" F161
>D1:" = A161  -
>A2:"Program Docu
>D2:"EOF = J230
>C5:" ROUTING
>B6:"+ 1985  Robin
>B7:"ts reserved.
>B8:"y the author
>E8:"eded or done
>B9:"s hereby gra
>E9:" be copied a
>B10:" the USDA So
>E10:"or other Fed
>A12:"ENTER THE FO
>E13:"    ?
>A15:"YEAR STORM
>B16:"to Avg. Annu
>A17:"MUSLE at Inl
>E17:"    ?
>C18:"Id
>B20:"f Fines in S
>A21:"Percentage o
>E21:"    ?
>C22:"Sediment
>B23:"f Sand In Wa
>A26:"*************
>E26:"INPUT PARAME
>I26:"*************
>B28:"MUSLE
>E29:"    ?
>E30:"    ?
>E31:"    ?
>E32:"    ?
>C33:"sh Load
>B35:"Landslide -
>A36:"SDR: Landsli
>A37:"Percentage o
>E37:"    ?
>C38:"diment
>B39:"f Gravel in
>A41:"Input 1:
>E41:"    ?
>E42:"    ?
>C43:"ediment
>B44:"f Sand in Se
>A45:"Percentage o
>E45:"    ?
>C47:"ntribution
>A49:"Avg. Annual
>D49:"(t/ac)
>B50:"ea>E50:"    ?
>C51:"ediment
>B52:"f Sand in Se
>A53:"Percentage o
>E53:"    ?
>C55:" Roads
>E56:"    ?
>C57:"(t/mi)
>E58:"    ?
>C59:"ediment
>B60:"f Sand in Se
>A61:"Percentage o
>E61:"    ?
>C67:" Gullies
>C68:"(tons)
>E69:"    ?
>C70:"ediment
>B71:"f Sand in Se
>A72:"Percentage o
>B2:"mentation:
>A5:"SEDIMENT
>D5:"SPREADSHEET
>C6:" S. White
>C8:" of any modi
>F8:".
>C9:"nted for thi
>F9:"nd used by
>C10:"il Conservat
>F10:"eral agencie
>B12:"LLOWING:
>F13:"---->
>E15:"    ?
>C16:"al Storm
>B17:"tial Input P
>A18:"Avg. Annual
>E18:"    ?
>C20:"ediment
>B21:"f Sand in Se
>A22:"Percentage o
>E22:"    ?
>C23:"sh Load
>B26:"*************
>F26:"TERS
>J26:"*************
>E28:"    ?
>A30:"Percentage o
>A31:"Percentage o
>A32:"Percentage o
>A33:"Percentage o
>E33:"    ?
>C35:"Annual
>B36:"de
>B37:"f Fines in S
>A38:"Percentage o
>E38:"    ?
>C39:"Sediment
>B41:"Streambank -
>A42:"SDR: Streamb
>A43:"Percentage o
>E43:"    ?
>C44:"diment
>B45:"f Gravel in
>A47:"Input 1:
>A48:"Area (Acres)
>B49:"Sediment Yie
>E49:"    ?
>A51:"Percentage o
>E51:"    ?
>C52:"diment
>B53:"f Gravel in
>A55:"Input 1:
>A56:"Length of Ro
>A57:"Road Erosion
>E57:"    ?
>A59:"Percentage o
>E59:"    ?
>C60:"diment
>B61:"f Gravel in
>A67:"Input 1:
>A68:"Total Gully
>E68:"    ?
>A70:"Percentage o
>E70:"    ?
>C71:"diment
>B72:"f Gravel in
>C72:"Sediment
>B74:"Erosion from
>A75:"Total Erosio
>D75:"(tons)
>B76:"ources
>B77:"f Fines In S
>A78:"Percentage o
>E78:"    ?
>C79:"Sediment
>B81:"MUSLE
>E82:"    ?
>C83:"ediment
>B84:"f Sand in Se
>A85:"Percentage o
>E85:"    ?
>C86:"sh Load
>B88:"Landslide -
>A89:"SDR: Landsli
>A90:"Percentage o
>E90:"    ?
>C91:"diment
>B92:"f Gravel In
>A94:"Input 2:
>E94:"    ?
>E95:"    ?
>C96:"ediment
>B97:"f Sand In Se
>A98:"Percentage o
>E98:"    ?
>C100:"ntribution
>A102:"Avg. Annual
>D102:"(t/ac)
>B103:"ea
>B104:"f Fines In S
>A105:"Percentage o
>E105:"    ?
>C106:"Sediment
>B108:"Erosion From
>B109:"ads (Miles)
>B110:" Rate
>A111:"SDR: Roads
>B112:"f Fines In S
>A113:"Percentage o
>E113:"    ?
>C114:"Sediment
>B116:"Erosion From
>B117:"Erosion
>A118:"SDR: Gullies
>B119:"f Fines In S
>A120:"Percentage o
>E120:"    ?
>C121:"Sediment
>B123:"Erosion from
>A124:"Total Erosio
>D124:"(tons)
>B125:"ources
>B126:"f Fines In S
>A127:"Percentage o
>E127:"    ?
>C128:"Sediment
>B135:"*************
>F135:"ETERS
>J135:"*************
>C137:" Available
>B138:"  $ Overbank
>A139:"Percentage o
>A140:"Percentage o
>A141:"Percentage o
>A143:"Output #2
>E72:"    ?
>C74:" Other Sourc
>B75:"n From Other
>E75:"    ?
>E76:"    ?
>C77:"ediment
>B78:"f Sand In Se
>A79:"Percentage o
>E79:"    ?
>E81:"    ?
>A83:"Percentage o
>E83:"    ?
>C84:"diment
>B85:"f Gravel In
>A86:"Percentage o
>E86:"    ?
>C88:"Annual
>B89:"de
>B90:"f Fines In S
>A91:"Percentage o
>E91:"    ?
>C92:"Sediment
>B94:"Streambank -
>A95:"SDR: Streamb
>A96:"Percentage o
>E96:"    ?
>C97:"diment
>B98:"f Gravel In
>A100:"Input 2:
>A101:"Area (Acres)
>B102:"Sediment Yie
>E102:"    ?
>E103:"    ?
>C104:"ediment
>B105:"f Sand In Se
>A106:"Percentage o
>E106:"    ?
>C108:" Roads
>E109:"    ?
>C110:"(t/mi)
>E111:"    ?
>C112:"ediment
>B113:"f Sand In Se
>A114:"Percentage o
>E114:"    ?
>C116:" Gullies
>C117:"(tons)
>E118:"    ?
>C119:"ediment
>B120:"f Sand In Se
>A121:"Percentage o
>E121:"    ?
>C123:" Other Sourc
>B124:"n From Other
>E124:"    ?
>E125:"    ?
>C126:"ediment
>B127:"f Sand in Se
>A128:"Percentage o
>E128:"    ?
>C135:"*************
>H135:"*************
>A137:"Output #1
>E137:"    ?
>C138:" Flow
>B139:"f Fines
>B140:"f Sand
>B141:"f Gravel
>B143:"  $ Sediment
>A74:"Input 1:
>D74:"es
>C75:" Sources
>A76:"SDR: Other S
>A77:"Percentage o
>E77:"    ?
>C78:"diment
>B79:"f Gravel in
>A81:"Input 2:
>A82:"SDR: MUSLE
>B83:"f Fines In S
>A84:"Percentage o
>E84:"    ?
>C85:"Sediment
>B86:"f Sand In Wa
>A88:"Input 2:
>E88:"    ?
>E89:"    ?
>C90:"ediment
>B91:"f Sand in Se
>A92:"Percentage o
>E92:"    ?
>C94:" Annual
>B95:"ank
>B96:"f Fines In S
>A97:"Percentage o
>E97:"    ?
>C98:"Sediment
>B100:"Land Area Co
>E101:"    ?
>C102:"Id
>A103:"SDR: Land Ar
>A104:"Percentage o
>E104:"    ?
>C105:"diment
>B106:"f Gravel in
>A108:"Input 2:
>A109:"Length of Ro
>A110:"Road Erosion
>E110:"    ?
>A112:"Percentage o
>E112:"    ?
>C113:"diment
>B114:"f Gravel in
>A116:"Input 2:
>A117:"Total Gully
>E117:"    ?
>A119:"Percentage o
>E119:"    ?
>C120:"diment
>B121:"f Gravel In
>A123:"Input 2:
>D123:"es
>C124:" Sources
>A125:"SDR: Other S
>A126:"Percentage o
>E126:"    ?
>C127:"diment
>B128:"f Gravel in
>A135:"*************
>E135:"OUTPUT PARAM
>I135:"*************
>B137:"  $ Sediment
>A138:"Output #1
>E138:"    ?
>E139:"    ?
>E140:"    ?
>E141:"    ?
>C143:" Available
```

```
>E143:"    ?        >A144:"Output #2      >B144:" $ Overbank
>C144:" Flow        >E144:"    ?          >A145:"Percentage o
>B145:"f Fines      >E145:"    ?          >A146:"Percentage o
>B146:"f Sand       >E146:"    ?          >A147:"Percentage o
>B147:"f Gravel     >E147:"    ?          >A149:"Output #3
>B149:" $ Sediment >C149:" Available      >E149:"    ?
>A150:"Output #3    >B150:" $ Overbank    >C150:" Flow
>E150:"    ?        >A151:"Percentage o   >B151:"f Fines
>E151:"    ?        >A152:"Percentage o   >B152:"f Sand
>E152:"    ?        >A153:"Percentage o   >B153:"f Gravel
>E153:"    ?        >A155:"$ Deposition   >B155:" In Main Cha
>C155:"nnel         >E155:"    ?          >A156:"Percentage o
>B156:"f Fines In D >C156:"eposition      >E156:"    ?
>A157:"Percentage o >B157:"f Sand In De   >C157:"position
>E157:"    ?        >A158:"Percentage o   >B158:"f Gravel In
>C158:"Deposition   >E158:"    ?          >A162:"************
>B162:"************ >C162:"************   >E162:"RESULTS OF C
>F162:"ALCULATIONS  >H162:"************   >I162:"************
>J162:"************ >F164:"  TONS OF      >A165:"DESCRIPTION
>D165:"   FINES     >F165:"   SAND        >H165:"   GRAVEL
>A167:"Initial Inpu >B167:"+ >D167:(E17/(E20+(E21*E23)))*E20
>F167:(E17/(E20+(E21*E23)))*E21
>H167:(E17/(E20+(E21*E23)))*E22 >A168:"Sediment >B168:"Yield
>D168:(E18*E16)*E43 >F168:(E18*E16)*E44   >H168:(E18*E16)*E45
>D169:"------------ >F169:"------------   >H169:"------------
>A170:"Total to Dow >B170:"nstream        >D170:+D167+D168
>F170:+F167+F168    >H170:+H167+H168      >I170:"    =
>J170:+D170+F170+H170                     >A172:"CONTRIBUTION
>B172:": INPUT 1    >A173:"Sheet & Rill
>D173:(E28/(E30+(E31*E33)))*E30*E29
>F173:(E28/(E30+(E31*E33)))*E31*E29
>H173:(E28/(E30+(E31*E33)))*E32*E29
>A174:"Landslide  >B174:" Erosion    >D174:(E35*E37)*E16*E36
>F174:(E35*E38)*E16*E36                  >H174:(E35*E39)*E16*E36
>A175:"Streambank E  >B175:"rosion   >D175:(E41*E43)*E16*E42
>F175:(E41*E44)*E16*E42                  >H175:(E41*E45)*E16*0
>A176:"Land Area                    >D176:(E48*E49)*E16*E51*E50
>F176:(E48*E49)*E16*E52*E50         >H176:(E48*E49)*E16*E53*45
>A177:"Roads                        >D177:(E56*E57)*E58*E59
>F177:(E56*E57)*E58*E60             >H177:(E56*E57)*E58*E61
>A178:"Gullies   >D178:(E68*E70)*E69    >F178:(E68*E71)*E69
>H178:(E68*E72)*E69  >A179:"Other Source   >B179:"s
>D179:(E75*E76)*E77  >F179:(E75*E76)*E78 >H179:(E75*E76)*E79
>D180:"------------  >F180:"------------ >H180:"------------
>C181:"Subtotal      >D181:@SUM(D173...D179)+D170
>F181:@SUM(F173...F179)+F170     >H181:@SUM(H173...H179)+H170
>I181:"    =         >J181:+D181+F181+H181   >A183:"OUTPUT #1
>D183:-(D181*E137*E138)*E139     >F183:-(F181*E137*E138)*E140
>H183:-(H181*E137*E138)*E141     >I183:"    =
>J183:+D183+F183+H183  >A185:"CONTRIBUTION  >B185:": INPUT 2
>A186:"Sheet & Rill     >D186:(E81/(E83+(E84*E86)))*E83*E82
>F186:(E81/(E83+(E84*E86)))*E84*E82
>H186:(E81/(E83+(E84*E86)))*E85*E82        >A187:"Landslide Er
>B187:"osion                          >D187:(E88*E90)*E16*E89
>F187:(E88*E91)*E16*E89               >H187:(E88*E92)*E16*E89
>A188:"Streambank E   >B188:"rosion   >D188:(E94*E96)*E16*E95
>F188:(E94*E97)*E16*E95               >H188:(E94*E98)*E16*E89
>A189:"Land Area               >D189:(E101*E102)*E16*E104*E103
>F189:(E101*E102)*E16*E105*E103
>H189:(E101*E102)*E16*E106*E103        >A190:"Roads
>D190:(E109*E110)*E111*E112       >F190:(E109*E110)*E111*E113
>H190:(E109*E110)*E111*E114          >A191:"Gullies
>D191:(E117*E119)*E118               >F191:(E117*E120)*E118
>H191:(E117*E121)*E13          >A192:"Other Source      >B192:"s
>D192:(E124*E125)*E126         >F192:(E124*E125)*E127
>H192:(E124*E125)*E128         >D193:"------------
>F193:"------------            >H193:"------------
>C194:"Subtotal                >D194:@SUM(D186...D192)
>F194:@SUM(F186...F192)        >H194:@SUM(H186...H192)
>I194:"    =                   >J194:+D194+F194+H194
>A196:"OUTPUT #2       >D196:-(D181-D183+D194)*E145*E143*E144
>F196:-(F181-F183+F194)*E146*E143*E144
>H196:-(H181-H183+H194)*E147*E143*E144
>I196:"    =       >J196:+D196+F196+H196        >A198:"OUTPUT #3
>D198:-(D181-D183+D194)*E151*E149*E150
>F198:-(F181-F183+F194)*E152*E149*E150
>H198:-(H181-H183+H194)*E153*E149*E150       >I198:"    =
>J198:+D198+F198+H198                      >A200:"DEPOSITION I
>B200:"N MAIN CHANN                        >C200:"EL
>D200:-(D181-D183+D194-D196-D198)*E156*E155
>F200:-(F181-F183+F194-F196-F198)*E157*E147
>H200:-(H181-H183+H194-H196-H198)*E158*E147
>I200:"    =              >J200:+D200+F200+H200
>A202:"TO DOWNSTREA    >B202:"M
>D202:+D181+D183+D194+D196+D198+D200
>F202:+F181+F183+F194+F196+F198+F200
>H202:+H181+H183+H194+H196+H198+H200     >I202:"    =
>J202:@SUM(D202+F202+H202)                >A204:"($ofGrain
>B204:"Sizes)/100   >D204:+D202/J202     >F204:+F202/J202
>H204:+H202/J202    >A207:"TOTAL INPUT   >D207:+D170+D181+D194
>F207:+F170+F181+F194    >H207:+H170+H181+H194   >I207:"    =
>J207:+D207+F207+H207    >A208:"TOTAL OUTPUT
>D208:(D183+D196+D198+D200)        >F208:(F183+F196+F198+F200)
>H208:(H183+H196+H198+H200)        >I208:"    =
>J208:+D208+F208+H208              >A212:"************
>B212:"************                >C212:"************
>D212:"************                >E212:"************
>F212:"************                >G212:"************
>H212:"************                >I212:"************
>J212:"************                >D214:"PROGRAM DOCU
>E214:"MENTATION    >A216:"This program    >B216:" is designed
>C216:" as a templa  >D216:"te for an el >E216:"ectronic spr
>F216:"eadsheet.  T  >G216:"he template  >H216:"simulates
>A217:"a general ve  >B217:"rsion of a s >C217:"tream channe
>D217:"l with sever  >E217:"al variation >F217:"s.  In order
>G217:" to run the   >H217:"complete     >A218:"stream chann
>B218:"el in questi  >C218:"on, the temp >D218:"late must be
>E218:" used severa  >F218:"l times usin >G218:"g the answer
>H218:"s from the    >A219:"upstream rea >B219:"ch as input
>C219:"data for the  >D219:" next reach. >D221:"Directions
>B222:"1.  Input is  >C222:" required wh >D222:"erever there
>E222:" is a '?'.    >B223:"2.  Percenta >C223:"ges are to b
>D223:"e entered in  >E223:" decimal for >F223:"mat, ie. 70$
>G223:" = 0.7.       >B224:"3.  If an op >C224:"tion is not
>D224:"used in a pa  >E224:"rticular rea >F224:"ch, enter ze
>G224:"ro (0).       >B225:"4.  ERROR ap >C225:"pears in the
>D225:" results of   >E225:"calculations >F225:" because val
>G225:"ues have not  >H225:" been entere >I225:"d yet.
>B226:"5.  Printout  >C226:"s require a  >D226:"15 inch print
>E226:"ter to print  >F226:" In regular  >G226:"print.  Use
>H226:"compressed p  >I226:"rint          >B227:"    for a pr
>C227:"inter with a  >D227:"n 8 inch pla >E227:"ten.
```

# VERIFICATION OF THE PRMS SEDIMENT-DISCHARGE MODEL

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## ABSTRACT

The Precipitation-Runoff Modeling System is a modular design modeling system designed to evaluate the impacts of various combinations of land use on basin hydrology. A subroutine, Unit Discharge, calculates water discharges and suspended-sediment concentrations for storms. A modified Unit Discharge subroutine was calibrated and verified using rainfall, water discharge, and suspended-sediment-concentration data collected during 21 storms at a 4.2-acre surface coal mine that was being reclaimed. The mine is located in south-western Pennsylvania, in Fayette County. Eleven storms were used for calibration and ten storms were used for verification of the subroutine. For the 11 storms used for calibration the standard error of estimate for water discharge was 0.0378 log units or about 9 percent, and the standard error of estimate for sediment load was 0.100 log units or about 23 percent. For the 10 storms used for verification the standard error of estimate of water discharge was 0.133 log units or about 31 percent, and the standard error of estimate of sediment load was 0.229 log units or about 55 percent. The particle-size distribution of the suspended sediment discharged from the Fayette County site was determined on samples collected on September 2, 1981—a storm used for verification, and on August 8, 1982—a storm used for calibration. The particle-size distribution of suspended sediment calculated by the model was within about 10 percent of the measured values.

## INTRODUCTION

### Purpose and Scope

This report describes the results of a study to calibrate and verify a modified version of the PRMS UNITD subroutine using rainfall, water discharge, and suspended-sediment data collected during 21 storms from July 26, 1981, through September 3, 1982, at a 4.2-acre surface coal mine that was being reclaimed. The particle-size distribution of suspended sediment measured in samples collected above and below the sediment-control pond was compared with the particle-size distribution predicted by the model for the same samples. The trap efficiency of the sediment-control pond measured during four of the storms was compared with the trap efficiency computed by the model for those storms.

### Description of the Study Site

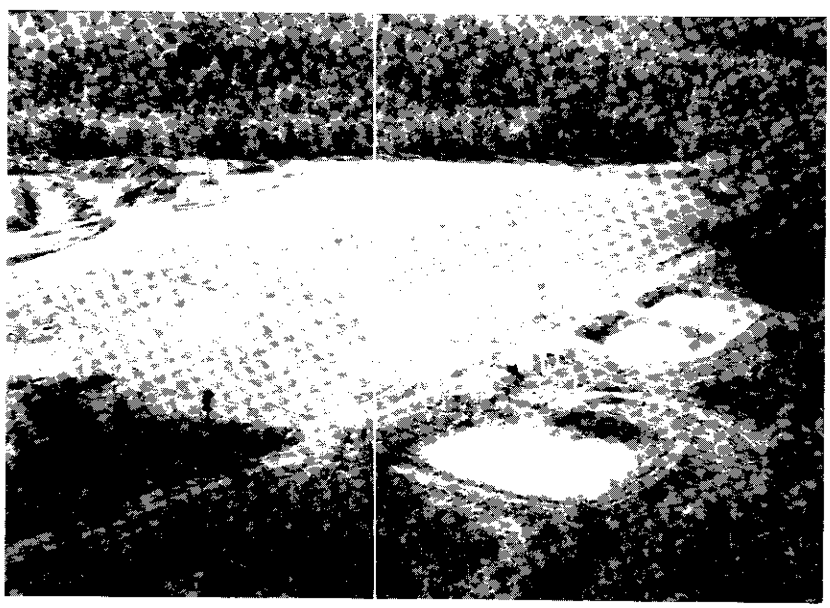
#### Prior to Mining

The area studied is in Fayette County about 2 miles south of Farmington, Pennsylvania. The area was forested prior to mining, slopes were generally 12 to 30 percent and most of the soils had an organic layer of leaf litter and humus over the surface. The top soils were dark grayish brown channery silt loams and were as thick as 7 inches. The subsoils were yellowish brown silty clay loams and channery silty clay loams and were 24 to 36 inches thick. Soils were classified as Gilpin (U.S. Department of Agriculture, 1973), moderately erodible, very stony loams, belonging to the Gilpin-Wharton-Ernest association.

During Mining and Reclamation

The area was surface mined using the block-cut method. The first step in block-cut mining is to construct sediment-control ponds and diversion channels. The first cut is then made at one end of the mine site, perpendicular to the contour. The top soil and rock from the first cut are hauled to a site adjacent to the location of the last planned cut. After the coal has been removed from the first cut, the second cut is started. Once again the soil is hauled to the approximate location of the last planned cut, but the rock overburden is turned into the first cut and graded to approximately the original contour, and coal is removed from the second cut. Soil from the third cut is placed on the area of the first cut, which is then graded, and seeded. This process of mining with concurrent reclamation continues until mining is completed.

At this site, mining began in May 1979, shortly after the sediment-control pond and diversion channels had been constructed. Mining of the study site progressed rapidly until October 1980. From October 1980 through September 1982, coal was sold from a stockpile and no new cuts were made because of a reduced demand for coal. Figure 1 shows the coal mine shortly after mining was discontinued in October 1980.



EXPLANATION

A Reclaimed slope

B South diversion

C Ponds for treating acid water

D Gaging station

E Sediment-control pond

Figure 1.--The mine site at the time data-collection equipment was installed in July 1981 (view looking south).

Although mining ceased in the fall of 1980, reclamation on the area drained by the south diversion (fig. 1) continued. The diversion channel occupies an area of 0.3 acres and drains an additional 3.9 acres. The average land slope is 17 percent, and the average slope of the diversion channel is 3 percent. Soil had been spread on the entire 4.2-acre area when data collection began in July 1981, but the area was not seeded until October 1981. The October seeding did not germinate and only about 10 percent of the surface area was covered with vegetation in the spring of 1982. On July 1, 1982, the area was reseeded and, by September 1982, about 70 percent of the area was covered with vegetation. The particle size composition of the surface following reclamation was 43 percent sand or larger particles, 31 percent silt, and 26 percent clay.

### Sediment-Control Pond

The sediment-control pond was constructed in April 1979, with a 3-inch valve to control water levels between the permanent pool elevation and the elevation of the principal spillway. The principal spillway is a drop inlet with a diameter of 24 inches located 5.3 feet above the 3-inch valve. An emergency spillway was built at an elevation 3 feet above the top of the principal spillway, and 8.3 feet above the 3-inch valve. Figure 1 shows the pond when the water level was at the level of the 3-inch valve.

## DATA-COLLECTION METHODS

A rain gage, flow-control weir, stage recorder, and an automatic suspended-sediment sampler were installed at outlet of the southern diversion channel (fig. 1) and a flow-control weir, stage recorder, and an automatic suspended-sediment sampler were installed at the outlet of the sediment-control pond. Data collection began in July 1981 and continued through September 1982. During that time data were collected during 21 storms at the outlet from the south diversion channel. Discharge from the sediment control pond occurred for only 3 of the 21 storms. Data were collected for all three of these events; however, one of these storms resulted in two separate peak water discharges about 3 hours apart. This storm was treated as two separate storms during the analysis of sediment-pond trap efficiency.

## DESCRIPTION OF MODEL

The Precipitation-Runoff Modeling System (PRMS) is a modular design modeling system designed to evaluate the impacts of various combinations of land use on basin hydrology (Leavesley and others, 1983). Land use combinations are evaluated by partitioning a drainage basin into units of similar land use called Hydrologic Response Units (HRU's). The PRMS model uses precipitation and snow melt data to calculate infiltration, precipitation excess, and the quantity of soil dislodged from each HRU. The quantity of soil dislodged from each HRU is computed using an erosion mechanics approach, which incorporates both rainfall detachment and overland flow detachment.

Precipitation excess and ground-water discharge are routed as streamflow. Suspended sediment is routed as a conservative substance with the water discharge. All calculations are made by the Unit Discharge subroutine

(UNITD). After water-discharge and suspended-sediment concentration are calculated for each HRU, the model combines the outputs from the HRUs and uses routing techniques to calculate the resultant water discharge and suspended-sediment concentration at the drainage-basin outlet.

## Modifications to UNITD

The modified version of UNITD discussed in this paper operates in the interactive mode on the U.S. Geological Survey PRIME mini-computer system. At present, it does not run as a working subroutine with PRMS. Suspended-sediment computation routines were expanded to calculate the erosion, deposition, and transport of eight sizes of sediment independently. The eight sizes include fine and medimum clay, coarse clay, four sizes of silt, fine sand, and medium sand. Precipitation intensity, particle-size distribution, erodibility, and exposure of the surface soils are used to calculate the quantity of each of eight particle sizes of soil dislodged and placed in suspension by the impact of rainfall. The particle-size distribution and erodibility were used because the they are readily available from most County Soil Surveys. The eight size fractions of sediment are routed independently and deposition of each size is calculated for the area of sheet flow on the basis of the particle settling velocity and the depth of flow. Sediment remaining in suspension is routed down a network of rills and additional sediment put in suspension by erosion in rills is calculated.

UNITD was also modified to route the water and sediment through a sediment-control pond. Each of the eight particle sizes of sediment are routed through the pond independently, and sediment deposition is calculated based on the settling velocity of the sediment and the depth of water in the pond. Total sediment deposition and trap efficiency of the sediment-control pond are determined. The maximum length of record that the model will calculate for any storm is 1,440 minutes (24 hours), regardless of the amount of data tabulated in the rainfall, water-discharge, and suspended-sediment-concentration file.

## Parameters Used in Model

Parameters used to run the model can be divided into two groups. One group of parameters are constant for an area regardless of the season or the time between storms, and one group changes from storm to storm. Parameters that are constant for all storms are the length and width of the area, the length and width of the diversion terrace, the size of the sediment-control pond, the soil erodibility, and the saturated hydraulic conductivity (KSAT).

Parameters that change from storm to storm include the exposed soil (as vegetation grows the amount of soil exposed decreases), water in the subsurface available for seepage, and the precipitation required to saturate the surface soil. PRMS normally accounts for changes in subsurface water and the the water required to saturate the surface soils, but the values must be supplied for each storm when this modified version is used. The time between storms and the recent precipitaion can be used as a guide to select appropiate values.

Constant Parameters

Since the area above the diversion terrace was reclaimed and reseeded at the same time, the area was considered as one HRU. The width of the HRU was 430 feet and the length was 427 feet, these are average values because the model only handles rectangular HRU's. The saturated hydraulic conductivity (KSAT) was 0.035 inch per hour. The length of the diversion terrace was 450 feet and bottom width was about 3 feet. The sediment pond was entered into the program as being 6 feet deep, 37 feet wide, and 70 feet long. The pond was actually larger but, because only 37 percent of the drainage area to the pond was measured and sampled by the station on the southern diversion terrace, the entered pond size was proportional to the percentage of the drainage area that was gaged. A settling factor of 100 times the normal particle-settling velocity was selected because the pond contained water with a high aluminum concentration of 1.45 mg/L. The erodibility of the soil in the area was 0.48 (Wischmeier and Smith, 1958 and U.S. Department of Agriculture, 1973).

Variable parameters

Table 1 lists values of the parameters that were adjusted to obtain the best fit between the simulated and observed water discharge and suspended-sediment concentration for the storms used for calibration. The percent of the soil exposed ranged from 5 to 100 percent, and the water required to saturate the surface soils ranged from 0.10 to 0.30 inches.

Table 1.--Parameters that were adjusted to produce the best fit between simulated and measured water discharge and suspended-sediment concentration for the calibration storms at the surface coal mine

| | Date of Storm | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1981 | | | | | 1982 | | | | | |
| Parameter | July 26 | July 28 | Aug. 15AM | Aug. 31 | Apr. 3 | May 22 | June 28 | Aug. 8 | Aug. 25AM | Sep. 2AM | Sep. 3 |
| Area of exposed soil (percent) | 65 | 25 | 61 | 25 | 60 | 100 | 40 | 16 | 6 | 11 | 5 |
| Subsurface water available as seepage (inches) | .45 | .05 | .10 | .05 | .10 | .10 | .10 | .10 | .10 | .10 | .1 |
| Precipitation required to saturate surface soil (inches) | .11 | .17 | .17 | .28 | .16 | .14 | .30 | .31 | .21 | .10 | .1 |
| | Storm data used as a guide to adjust storm variables for model | | | | | | | | | | |
| Days since last storm | 6.0 | 1.2 | 18. | 1.5 | 3.0 | .75 | 4. | 3.5 | 3.0 | 8. | .2 |
| Precipitation the past 7 days | 1.53 | .82 | 0. | .18 | .35 | .60 | .15 | 2.15 | .70 | 0. | .5 |

Table 2 lists the values of the parameters for the verification storms. The values were based on the date of the storm, the days since the last storm and the precipitation during the past 7 days. As an example, the verification storm that occurred on June 29, 1982 occurred 1 day after a calibration storm. A value of 40 was used for the percentage of soil exposed when the June 28 storm was run, and a value of 16 was used for the calibration storm on August 8; therefor, a value of 38 was selected for the verification storm on June 29. The June 29 storm began one day since the last storm, and it had rained 0.75 inches during the past 7 days, so that moderate values were selected for the amount of subsurface water and precipitation required to saturate the surface soil.

Table 2.--Parameters that were selected for verification of UNITD at the surface coal mine

| | Date of storm | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1981 | | | 1982 | | | | | | |
| Parameter | July 27 | Aug. 15PM | Sep. 2 | Apr. 6 | May 24 | June 16 | June 29 | Aug. 17 | Aug. 25PM | Sept. (total) |
| Area of exposed soil (percent) | 45 | 45 | 25 | 60 | 80 | 50 | 38 | 10 | 6 | 6 |
| Subsurface water available as seepage (inches) | .25 | .10 | .10 | .10 | .10 | .10 | .10 | .10 | .10 | .10 |
| Precipitation required to saturate surface soil (inches) | .09 | .08 | .08 | .13 | .16 | .25 | .16 | .30 | .10 | .08 |
| | Storm data used as a guide to select variables for the verification storms | | | | | | | | | |
| Days since last storm | .3 | .25 | .5 | 2.6 | 1.0 | 3.5 | 1.0 | 6.0 | .25 | 8. |
| Precipitation the past 7 days | .89 | .15 | 1.00 | .85 | .87 | 1.10 | .75 | .95 | .45 | 0. |

## RESULTS OF CALIBRATION AND VERIFICATION TRIALS

### Water and Suspended-Sediment Discharge

The measured and calculated water and suspended-sediment discharge, the peak water discharge, and the peak suspended-sediment concentration for the 11 storms used for calibration are listed in table 3. If a storm had more than one period of intense precipitation, the peak water discharges and peak suspended-sediment concentrations for each are listed. Measured water discharge for the 11 storms ranged from 0.0016 to 0.096 $ft^3$/s-days (138 to 8,290 $ft^3$). The measured suspended-sediment discharge ranged from 0.053 to 3.9 tons, and the measured peak water discharges and peak sediment concentrations ranged from 0.19 to 9.8 $ft^3$/s and from 4,100 to 38,360 mg/L, respectively. The calculated values were about the same as the measured values (table 3). One exception is peak sediment concentrations and sediment load on June 28, 1982. The storm had two peak water discharges of 1.1 and 5.0 ft$^3$/s; the measured sediment concentrations during the two peaks were 22,000 and 11,000 mg/L, respectively, and the measured sediment discharge for the entire storm was 1.4 tons. The model calculated peak sediment concentrations of 12,500 and 18,000 mg/L and a sediment discharge for the entire storm of 2.3 tons. Adjusting model parameters to decrease sediment discharge would have reduced both peak sediment concentrations, increasing the relative error for the first peak but reducing the error for the second peak. The model does not predict lower sediment concentrations for the second peak because the quantity of sediment available for transport in the model is too large or because there is some decrease in soil erodibility that is not accounted for by the model.

Table 3.--Water and suspended-sediment discharge, peak water discharge, and peak suspended-sediment concentration measured and computed by UNITD for the calibration storms at the surface coal mine

| Date | Water discharge (ft$^3$/s-days) | | Suspended-sediment discharge (tons) | | Peak water discharge (ft$^3$/s) | | Peak sediment concentration (mg/L) | |
|---|---|---|---|---|---|---|---|---|
| | Measured | Calculated | Measured | Calculated | Measured | Calculated | Measured | Calculated |
| July 26, 1981 | 0.042 | 0.044 | 3.9 | 3.7 | 8.8 | 8.0 | 38,360 | 18,000 |
| | | | | | - | - | 36,700 | 29,500 |
| 28 | .021 | .025 | .20 | .27 | 1.2 | .91 | 8,610 | 4,700 |
| | | | | | .58 | .51 | 4,870 | 4,020 |
| August 15 | .0092 | .0092 | .30 | .30 | .66 | .60 | 17,600 | 12,900 |
| 31 | .018 | .018 | .20 | .27 | 4.2 | 4.2 | 19,800 | 14,400 |
| April 3, 1982 | .0054 | .0054 | .065 | .066 | .19 | .21 | 10,700 | 5,290 |
| May 22 | .0016 | .0018 | .053 | .058 | .19 | .09 | 19,800 | 12,500 |
| June 28 | .062 | .063 | 1.4 | 2.3 | 1.1 | .81 | 22,000 | 12,500 |
| | | | | | 5.0 | 6.8 | 11,000 | 18,000 |
| August 8 | .096 | .096 | 2.1 | 2.5 | 9.8 | 11. | 18,200 | 13,200 |
| 25 | .030 | .026 | .13 | .12 | 1.4 | 1.8 | 4,100 | 2,030 |
| Sept 2 | .021 | .021 | .29 | .29 | 2.5 | 3.2 | 10,800 | 5,700 |
| 3 | .031 | .029 | .34 | .27 | 1.7 | 3.6 | 9,160 | 4,370 |

The measured and calculated water and suspended-sediment discharge, the peak water discharge, and the peak suspended-sediment concentration for the 10 storms used for verification are listed in table 4. If a storm had more than one period of intense precipitation, the peak water discharges and peak suspended-sediment concentrations for each are listed. Measured water discharge for the 10 storms ranged from 0.010 to 0.11 ft$^3$/s-days (864 to 9,500 ft$^3$). The measured suspended-sediment discharge ranged from 0.10 to 5.5 tons, and the measured peak water discharges and peak sediment concentrations ranged from 0.54 to 7.3 ft$^3$/s and from 1,240 to 40,700 mg/L, respectively. The calculated values were generally within 50 percent of the measured values, although some differences were as large as 200 percent.

Table 4.--Water and suspended-sediment discharge, peak water discharge, and peak suspended-sediment concentration measured and computed by UNITD for the verification storms at the surface coal mine

| Date | Water discharge ($ft^3$/s-days) | | Suspended-sediment discharge (tons) | | Peak water discharge ($ft^3$/s) | | Peak sediment concentration (mg/L) | |
|---|---|---|---|---|---|---|---|---|
| | Measured | Calculated | Measured | Calculated | Measured | Calculated | Measured | Calculated |
| July 27, 1981 | 0.030 | 0.034 | 0.50 | 1.2 | 4.5 | 2.5 | 6,800 | 16,400 |
| August 15(P.M.) | .070 | .067 | 1.7 | 2.5 | 2.7 | 3.4 | 16,800 | 16,700 |
| | | | | | 2.6 | 2.8 | 10,600 | 13,000 |
| | | | | | 2.5 | 4.2 | 13,400 | 15,700 |
| September 2 | .063 | .067 | 1.2 | 1.6 | 2.7 | 4.1 | 7,980 | 11,500 |
| | | | | | 3.3 | 3.4 | 9,100 | 8,860 |
| April 6, 1982 | .030 | .032 | .67 | .94 | 1.6 | 2.2 | 10,500 | 15,800 |
| May 24 | .020 | .018 | .60 | 1.1 | 1.5 | 1.7 | 18,800 | 27,600 |
| June 16 | .11 | .11 | 5.5 | 5.0 | 3.8 | 1.5 | 40,700 | 24,800 |
| | | | | | 7.3 | 8.5 | 30,000 | 26,400 |
| | | | | | .54 | .82 | 4,900 | 7,700 |
| | | | | | .75 | 1.2 | 12,000 | 9,100 |
| 29 | .081 | .084 | 1.3 | 2.6 | - | - | 5,000 | 5,500 |
| | | | | | 4.7 | 3.1 | 11,600 | 15,300 |
| | | | | | 2.8 | 2.8 | 6,600 | 10,700 |
| | | | | | 2.5 | 2.4 | 3,300 | 10,300 |
| August 17 | .010 | .018 | .10 | .12 | .68 | 1.2 | 8,900 | 3,110 |
| | | | | | .10 | .30 | 1,240 | 1,650 |
| 25 | .043 | .044 | .62 | .50 | 3.6 | 7.3 | 8,000 | 5,800 |
| September 2 | .071 | .12 | .63 | .95 | 3.7 | 4.6 | 7,000 | 4,200 |
| | | | | | 2.2 | 3.3 | 4,140 | 2,950 |
| | | | | | 3.0 | 7.2 | 9,460 | 4,800 |

For the 11 storms used for calibration the standard error of estimate for water discharge was 0.038 log units or about 9 percent, and the standard error of estimate for sediment load was 0.100 log units or about 23 percent. For the 10 storms used for verification the standard error of estimate for water discharge was 0.133 log units or about 31 percent, and the standard error of estimate for sediment load was 0.229 log units or about 55 percent. Table 5 lists the standard errors of estimate for the storms used for calibration and verification. The table also lists the standard deviations and the correlation coefficients of least squares regression equations where X is the value measured at the site and Y is the value predicted by UNITD.

Table 5.--Results of calibration and verification of PRMS UNITD for the reclaimed surface coal mine in Fayette County Pennsylvania

| Parameter | Number of obser-vations | Standard error of estimate (log units) | Least squares regression equation ($Y=aX^b$) Where; Y=values predicted by UNITD X=values measured at site: a | b | Standard deviation (log units) | Correlation coefficient |
|---|---|---|---|---|---|---|
| | | | Calibration storms | | | |
| Water discharge ($ft^3$/s-days) | 11 | 0.038 | 0.917 | 0.974 | 0.034 | 0.998 |
| Sediment discharge (tons) | 11 | .100 | 1.06 | 1.02 | .098 | .989 |
| Peak water discharge ($ft^3$/s) | 13 | .164 | .959 | 1.13 | .146 | .976 |
| Peak sediment concentration (mg/L) | 14 | .252 | .574 | 1.02 | .150 | .898 |
| | | | Verification storms | | | |
| Water discharge ($ft^3$/s-days) | 10 | .133 | .742 | .863 | .106 | .943 |
| Sediment discharge (tons) | 10 | .229 | 1.40 | .944 | .152 | .946 |
| Peak water discharge ($ft^3$/s) | 21 | .232 | 1.53 | .717 | .171 | .871 |
| Peak sediment concentration (mg/L) | 22 | .229 | 7.64 | .782 | .216 | .766 |

## Particle-Size Distribution

The particle-size distribution of the suspended sediment discharged from the mine site was determined for samples collected on September 2, 1981, during a storm used for verification; and on August 8, 1982, during a storm used for calibration. Table 6 lists the measured values and the values computed by the model for the two storms. The table also lists the particle-size analyses for three samples of the discharge from the sediment control pond. One of the samples was collected on September 2, 1981, and the other two were collected on August 8, 1982. The mean particle-size distribution of the sediment discharge from the sediment-control pond as calculated by UNITD also is listed. Standard errors were not calculated for the data on table 6, but the errors appear to average about 10 percent.

Table 6. Measured and computed particle size analysis of suspended-sediment samples collected from the surface coal mine

| | Suspended-sediment discharge from the diversion terrace | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Measured | | | | Calculated | | | |
| | Sediment concentration Mg/L | Percent sand | silt | clay | Sediment concentration Mg/L | Percent sand | silt | clay |
| Sept. 2, 1981 | 8,370 | 11 | 50 | 39 | 8,860 | 2 | 46 | 52 |
| Aug. 8, 1982 | 14,700 | 2 | 56 | 42 | 9,170 | 4 | 48 | 48 |
| | 13,500 | 4 | 52 | 44 | 12,000 | 5 | 50 | 45 |
| | 1,570 | 1 | 31 | 68 | 4,000 | 1 | 39 | 60 |

| | Suspended-sediment discharge from the sediment-control pond | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Measured | | | | Calculated | | | |
| | Sediment concentration Mg/L | Percent sand | silt | clay | Sediment[1] concentration Mg/L | Percent[2] sand | silt | clay |
| Sep. 2, 1981 | 1,080 | 0.2 | 38 | 62 | 1,030 | 0.0 | 22 | 78 |
| Aug. 8, 1982 | 8,350 | 0.0 | 54 | 46 | 2,070 | .1 | 43 | 57 |
| | 7,560 | 0.0 | 49 | 51 | 2,270 | .0 | 38 | 62 |

1 Maximum for storm
2 Mean for storm

## Trap Efficiency of Sediment-Control Ponds

The trap efficiency of the sediment-control pond below the mine as computed by UNITD for four storms ranged from 91.0 to 99.1 percent (table 7). The measured trap efficiency for the same four storms ranged from 98.1 to 99.6 percent. The calculated sediment discharges from the pond were about five times the measured values. One of the reasons for this difference was that the pond water was relatively high in aluminum (1.45 mg/L). A settling factor greater than 100 may have produced closer agreement. The aluminum promotes flocculation of the clay and silt particles and they settle out much more rapidly than indicated by standard particle settling velocities.

Table 7.--Measured and computed trap efficiency of the sediment-control pond

| | Sediment discharge from sediment-control pond, in tons | | Trap-efficiency of sediment-control pond in percent | |
|---|---|---|---|---|
| Date | Measured | Computed | Measured | Computed |
| September 2, 1981 | 0.028 | 0.13 | 98.1 | 91.2 |
| June 16, 1982 (3:00 p.m.) | .0094 | .02 | 99.6 | 99.2 |
| (6:00 p.m.) | .049 | .60 | 99.1 | 89.4 |
| August 8, 1982 | .053 | .24 | 98.2 | 91.9 |

## SUMMARY

This report describes the results of a study to calibrate and verify a modified version of the PRMS UNITD subroutine using hydrologic data collected from a 4.2-acre surface coal mine site in Fayette County, Pennsylvania, that was in the process of being reclaimed. The PRMS UNITD subroutine is a rainfall-runoff model that calculates water discharge and suspended-sediment concentrations for storms. A total of 21 storms were used to calibrate and verify the model.

In the modified version of UNITD, suspended-sediment computations were changed to include eight particle sizes of soil. Suspended-sediment produced by the impact of precipitation on exposed soil and that produced by channel scour are calculated independently. The model also calculates the deposition and transport of the eight sizes. The eight sizes include medium clay and finer particles, coarse clay, four sizes of silt, fine sand, and medium sand.

UNITD was calibrated for the mine site with 11 storms and verified with 10 storms. For the 11 storms used for calibration, the standard error of estimate for water discharge was 0.038 log units or about 9 percent, and the standard error of estimate for sediment load was 0.100 log units or about 23 percent. For the 10 storms used for verification the standard error of estimate of water discharge was 0.133 log units or about 31 percent, and the standard error of estimate of sediment load was 0.229 log units or about 55 percent.

The trap efficiency of the sediment-control pond below the mine site as computed by the model for four storms ranged from 91.0 to 99.1 percent. The measured trap efficiency for the same four storms ranged from 98.1 to 99.6 percent. The particle-size distribution of the suspended sediment discharged from the site was determined on samples collected during a storm used for calibration and during a storm used for verification. The particle-size distribution of suspended-sediment calculated by the model was within about 10 percent of the actual measured values.

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# APPLICATION OF STREAMTUBE COMPUTER MODEL

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## ABSTRACT

A computer model based on the stream tube concept is applied to study the erosion patterns at Stage I cofferdam site of Lock and Dam No. 26 (Replacement) project for the period of January-April 1982. The five mile modeling reach is located between the existing Lock and Dam No. 26, near St. Louis, Missouri and Hartford, Illinois, on the Mississippi River. The computer model was originally developed for the U.S. Bureau of Reclamation to route water and sediment in alluvial channels. The concept of stream tubes is designed to allow the lateral and longitudinal variation of hydraulic conditions as well as sediment activity at cross sections along a study reach. The computer program is a semi-two-dimensional program with the third dimension, depth, being intrinsically incorporated into the computations. Daily stage-discharge values at the gaging station, located at the downstream end of the study reach, were used for the hydraulic computations. Sediment routing computations were performed starting with the existing sediment size distribution. Simulation results at the cofferdam site, presented in the form of topographic maps of the channel bed, indicate close agreement to actual measured data.

## INTRODUCTION

Lock and Dam No. 26 (Replacement) is presently under construction on the Mississippi River near Alton, Illinois at River mile 200.78 (above the confluence of the Ohio River). Construction in the river began in April 1980 by driving a series of 63 foot diameter sheet pile cells in order to cofferdam a section of the river for construction of the first stage dam. The first stage cofferdam built to Elevation 430 NGVD constricted about 48 percent of the river channel at mile 200.78 and was completed in September 1981. To complete the lock and dam, a second stage cofferdam will be used for construction of a 1200-foot lock, and a third cofferdam will tie the remainder of the dam with the Illinois bank (see Figure 1).

Numerous physical model tests were performed at the U.S. Army Engineer Waterways Experiment Station in Vicksburg, MS. The movable bed tests were performed in order to give the district a qualitative assessment of potential river bed scour with the first stage cofferdam in place. The bed for the tests was composed of coal dust which simulates the upper river bed layer. No attempt was made to simulate a graded material as existed in the prototype. Scour in the model (1:120 scale) extended to bedrock in specific areas, mainly adjacent to the upstream deflector arm, where the high velocity currents are directed away from the main river side leg of the cofferdam. Scour in the prototype has extended to about Elevation 335 NGVD which is about 35 feet below the original river bed of 370. In order to provide the best design for the second stage cofferdam a more quantified analysis of scour potential was

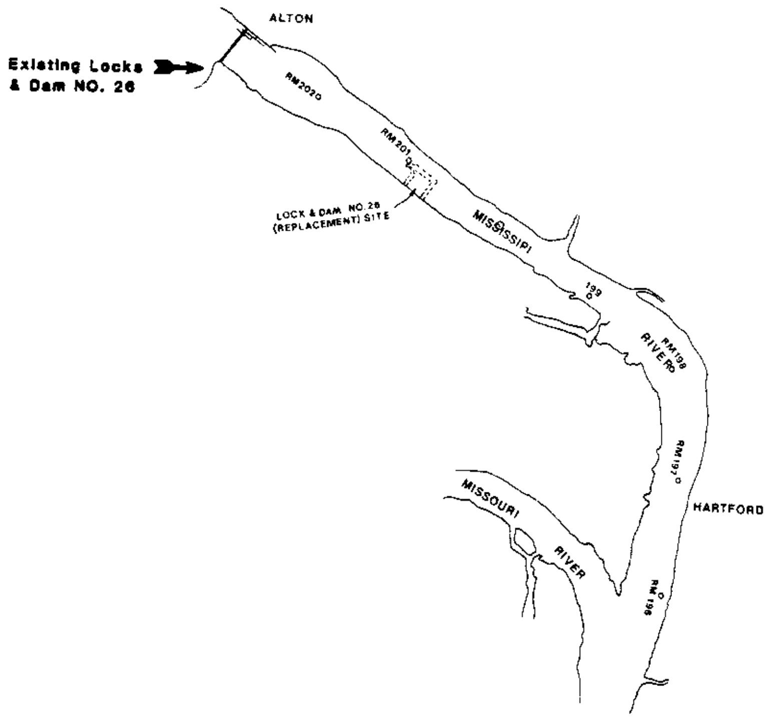


Figure 1. General layout of the Lock and Dam No. 26 (Replacement) site.

needed. To this end the St. Louis District, CE contracted with CSU to apply the Streamtube computer model developed by Molinas (2,3) to Lock and Dam No. 26 (Replacement). Since good prototype data on scour was available for the first stage cofferdam, the computer model could be easily verified. The model then could simulate the scour potential for the second stage cofferdam.

## GENERAL DESCRIPTION OF THE MODEL

The Streamtube Computer Model was developed by Dr. A. Molinas to simulate long-term streambed variations in rivers for which sediment and hydraulic data is limited. The computer model using stream tubes can be applied into a variety of river problems. It can be used as a fixed-bed model to compute water surface profiles for subcritical, supercritical or the combination of both flow conditions involving hydraulic jumps. This option allows the applications involving the computation of water surface profiles in man-made channels with clear water, flow profiles over spillways, or flow profiles in natural river channels where the interaction between the sediment-water mixture and the channel bed is negligible, in other words where the bed elevation changes are negligible. As a movable-bed model, the computer program can be applied to route water and sediment through natural river channels. The use of stream tubes allow the variation of hydraulic conditions and sediment activity not only in the longitudinal, but also in the lateral direction. With the selection of a single stream tube, the model becomes one-dimensional. Average channel response to changes in certain river flow or sediment conditions can be studied. With the selection of multiple stream tubes the model becomes two-dimensional. The changes in the cross section geometries in the lateral direction can be simulated. Since the bed-elevation changes are not averaged over the entire active channel widths as in one-dimensional models, more realistic channel erosion or aggradation can be simulated. This option provides valuable information where certain navigation depths have to be maintained. It can also be used in bank stability problems to identify expected regions of bank instabilities. The armouring process provided in the program allows to study river sedimentation problems for longer periods of time.

The computer program is a semi-two-dimensional program with the third dimension, depth, being intrinsically incorporated into the computations. As such, it has the basic limitations of every two-dimensional program; secondary flows cannot be simulated. The channel is divided into preselected number of tubes. The bed elevation in each stream tube is allowed to move vertically up or down depending on the flow conditions. As a result, while one section of channel is eroding another section might be aggrading. Depending on the number of stream tubes to be used, the channel cross section changes are averaged across different channel sections of different widths. Since the computer time and space is directly related to the number of stream tubes to be used, the user is required to decide on the optimum number of tubes. Bed forms are not simulated due to the lack of a generally accepted methodology for determining them. Even though provisions are made to expand the program to include river confluences, and middle islands, at this point these options are not available. The channel boundaries are fixed in the lateral direction and formation of meander bends cannot be simulated.

The Streamtube Computer Model (2,3) for routing water and sediment is composed of three major components: i) Backwater Computations; ii) Streamtube Computations, iii) Sediment Routing Computations. These computational blocks are linked together as shown in Fig. 2.

At each time step, first, backwater computations are carried out for the entire reach treating the channel as a single tube. Secondly, using the computed water surface elevations, lateral locations of stream tubes at each cross section is determined. Treating each stream tube as independent channels, new water surface profiles and hydraulic variables along them are computed. Thirdly, sediment is routed through each stream tube satisfying the sediment continuity equation. At the end of these computations bed material compositions are revised and channel bed elevations are updated. An armouring procedure is incorporated into the sediment routing computations.

Computations are proceeded in time through defined water and sediment discharge hydrographs.

Backwater Computations

Water discharge hydrographs are approximated by burst of constant discharges. During each constant discharge time block, backwater computations are carried out without interruptions for subcritical, supercritical or a combination of both flow conditions modeling hydraulic jumps. The details of these computations are presented in several publications (Molinas and Yang, 1985, Molinas, 1983). These uninterrupted water surface profile computations are one of the most significant features of the Streamtube Computer Model. It is this unique component that makes the model applicable to water and sediment routing computations through complex flow conditions.

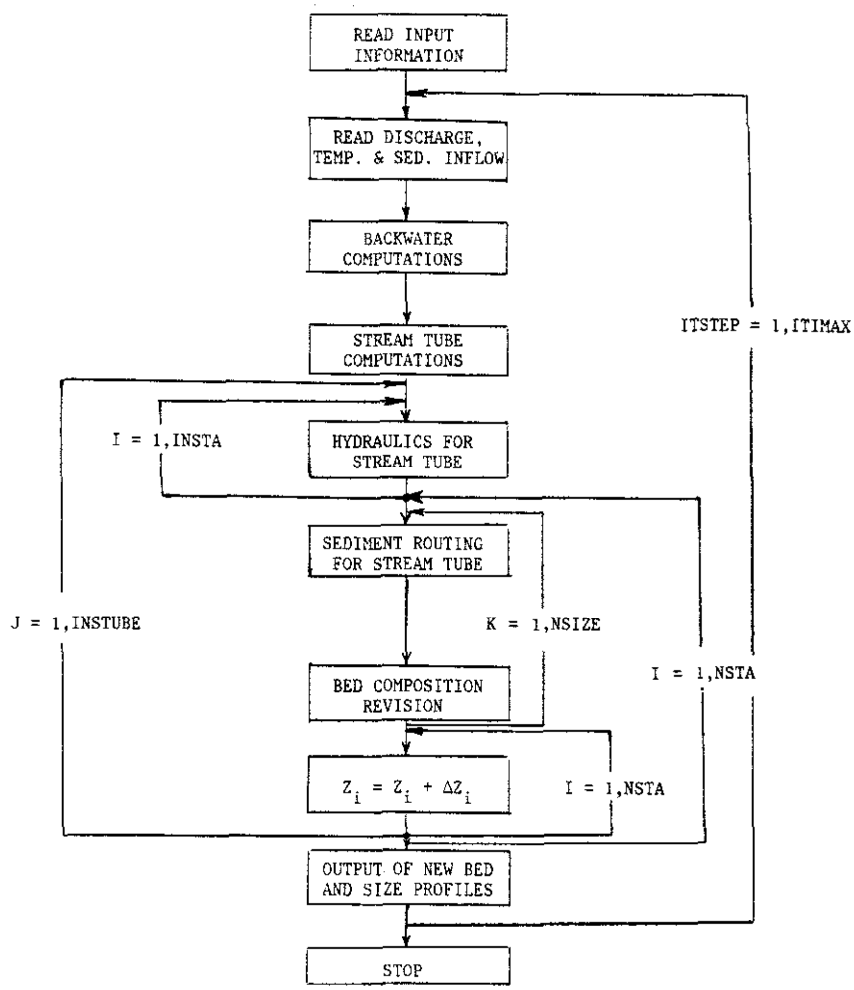


Figure 2. Flow chart for the Streamtube Computer Model for routing water and sediment.

Streamtube Computations

Streamtubes are imaginary tubes bounded by stream lines. Since the discharge between stream lines is constant, each stream tube carries a constant discharge along its length.

For steady, incompressible flows, it is possible to write

$$\frac{P}{\gamma} + \frac{V^2}{2g} + \gamma h = H_t \qquad \text{(Eq. 1)}$$

where $H_t$ is a constant along the stream tube. When applied to real fluids, the Total Head $H_t$ is not a constant. Due to friction and other local losses it is reduced in the direction of flow. Along a river it is possible to determine the variation of this quantity. This is the basic assumption in the Streamtube Computer Model.

The use of streamtubes in routing water and sediment through alluvial channels is another unique feature of the Streamtube Computer Model. In the model the total discharge carried through the channel is distributed equally among the preselected number of streamtubes. Along each streamtube the water discharge remains constant. No lateral inflow into individual streamtubes from neighboring tubes are allowed. Due to the assumptions involved, at a given station water surface elevation across the channel should remain constant. Under these circumstantes the equal discharge locations, and therefore lateral streamtube locations, correspond to equal channel conveyances. Following the initial backwater computations at each station streamtube locations across the channel satisfying equal conveyance requirements are determined. Next, since total energy along each streamtube should remain constant, backwater computations are performed to establish the hydraulic conditions along individual tubes.

Sediment Routing Computations

Sediment routing computations in each stream tube are performed by satisfying the sediment continuity equation, which is given as:

$$\frac{\partial Q_s}{\partial x} + \eta \frac{\partial A_d}{\partial t} = 0 \qquad \text{(Eq. 2)}$$

where $\eta$ is the volume of sediment in a unit bed layer volume or one minus porosity in this program is set equal to a commonly used value of 0.6; $A_d$ is the volume of sediment deposition per unit length; $Q_s$ is the volumetric sediment discharge.

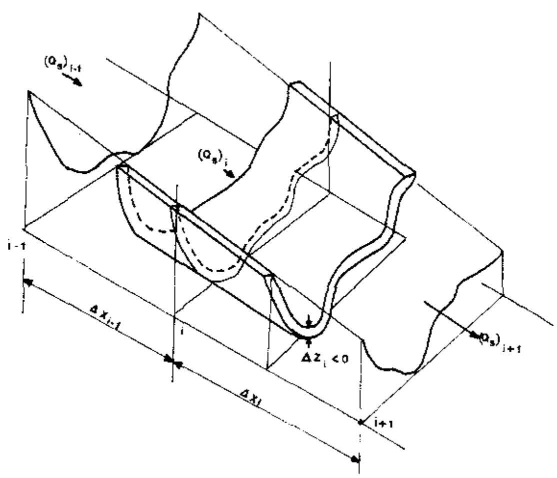


Figure 3. Definition of variables for the sediment routing computations. For simplicity the entire channel is treated as a single streamtube.

The sediment continuity equation is discretized as follows

$$\eta \frac{\partial A_d}{\partial t} = \frac{\eta(2P_i+P_{i+1}+P_{i-1})\Delta Z_i}{4\ \Delta t} \qquad \text{(Eq. 3)}$$

$$\frac{dQ_s}{dx} = \frac{Q_{s_i} - Q_{s_{i-1}}}{\frac{\Delta x_i + \Delta x_{i-1}}{2}} \qquad \text{(Eq. 4)}$$

where $P$ is the wetted perimeter; $Z$ is the bed elevation above a certain datum and $i$ is the cross-sectional index.

The change in bed elevation, $\Delta Z_i$, can be obtained from the sediment continuity equation (Eq. 2) using Eqs. 3 and 4

$$\Delta Z_i = \frac{8\Delta t(Q_{s_{i-1}} - Q_{s_i})}{\eta(2P_i + P_{i-1} + P_{i+1})(\Delta X_i + \Delta X_{i-1})} \qquad \text{(Eq. 5)}$$

The total change in bed elevation at a given station $i$ is computed from

$$\Delta Z_i = \sum_{k=1}^{NSIZE} \Delta Z_{ik} \qquad \text{(Eq. 6)}$$

where NSIZE is the number of size classes.

The sediment transport capacity computations in the present mathematical model can be carried out by the use of:

1. Yang's 1973 and 1984 equations
2. Ackers and White equation
3. Engelund and Hansen's equation

The user is allowed to use any one of these sediment computation methods by defining the variable ISED at the sediment input data. For the study presented in this paper, sediment transport capacity computations were performed by using Yang's equations (5,6).

The armouring process used in the computer model is adapted from Bennett and Nordin (1977). It has been modified to account for bed material compositions across the channel, at those coordinate pairs which are used in defining the channel geometry.

## MODEL APPLICATION

The application of the Streamtube Computer Model to study the hydraulic and sediment conditions at the Lock and Dam No. 26 (Replacement) site, shown in Figure 1 was accomplished in three phases. The first phase was identified as familiarization with the project and development of the basic data for the water and sediment routing. Second phase was the calibration and verification of the computer model utilizing Stage 1 cofferdam hydraulic conditions and river bed measurements. Third phase was the application of the computer model to Stage 2 cofferdam design. This paper will limit itself to the first two phases of the study, to the application of the model to Stage 1 cofferdam scouring simulation.

For the Stage 1 cofferdam construction the Mississippi River was constricted by 48 percent for a 1200 ft section along the river. Since the completion of construction in September 1981 the cofferdam site has experienced up to 35 ft of river channel erosion as of 1984. After reviewing the available channel geometry, hydraulic, hydrologic and sediment data a simulation period of January-April 1982 was selected for model calibration and verification. During this 3-month simulation period in 1982, discharges from 50,000 cfs up to 359,000 cfs were experienced. And, at certain locations along the reach, up to 15 ft of scouring was observed. The channel cross sections had been monitored closely during the simulation period, and therefore provided valuable information for model testing. The 5-mile-long stretch of river selected as the study reach extended between river miles 202.7 (Alton, Illinois) and 196.8 (Harford, Illinois). The Stage 1 cofferdam was located in this reach between river miles 200.92 and 200.63. Model verification runs were carried out by using the daily discharges of the Mississippi River at Alton, Illinois and the corresponding water surface elevations at Harford, Illinois as the downstream boundary conditions.

Starting with January 22, 1982 channel cross sections, the water and sediment routing program was used to compute daily water surface profiles and channel geometries. Figure 4 demonstrates the plan view of the streamtubes at the cofferdam site for different discharges. Also given in this figure are the water velocity vectors.

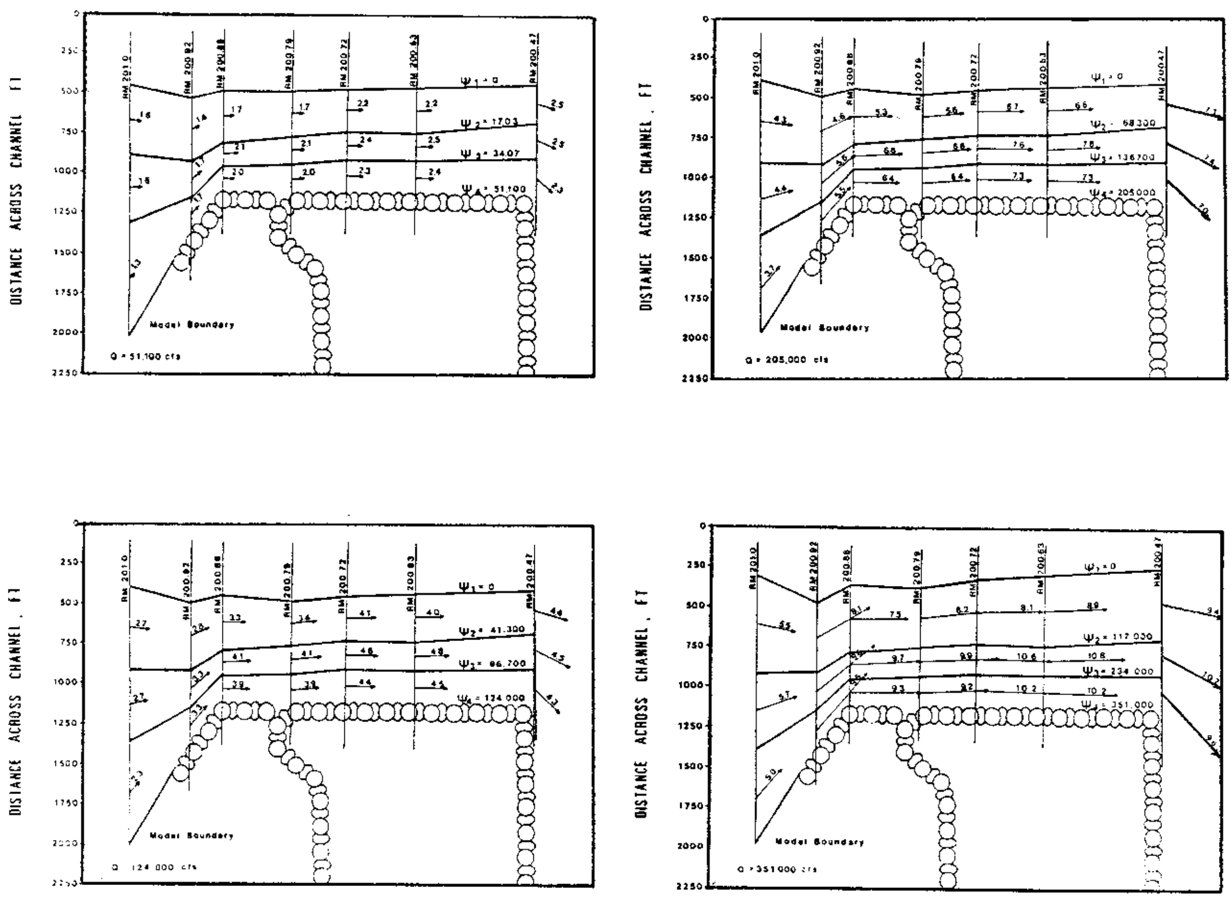


Figure 4. Streamtubes at Stage 1 cofferdam site for various discharges.

Sediment routing computations carried in streamtubes were able to generate bed topographics in close agreement with the measured data (Figure 5).

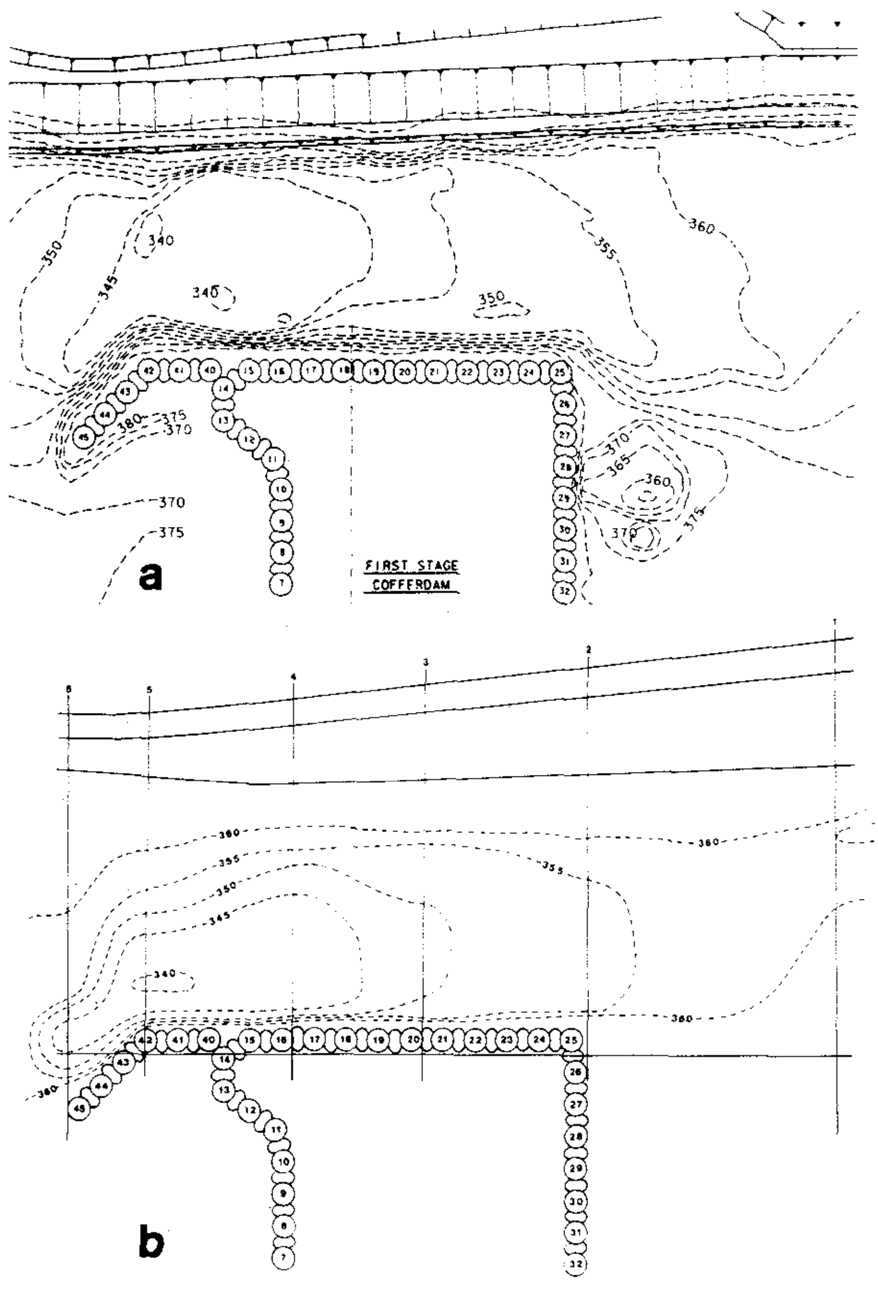


Figure 5. Measured (a) and computed (b) channel bed topographies at Stage 1 cofferdam site (April 1982).

The general view of the river channel at various times are shown in Figure 6. In this figure the cofferdam is located to the left and the view is looking upstream.

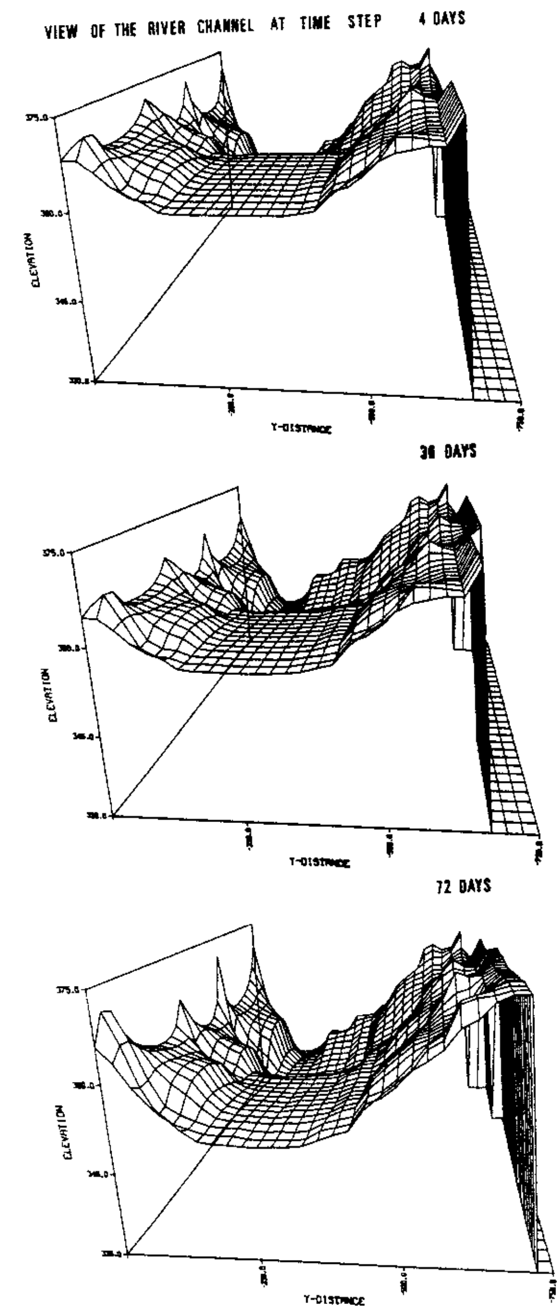


Figure 6. View of the computed river channel at various time steps looking upstream.

## SUMMARY

This paper presented an alternative water and sediment routing method using stream tubes. The computer model was applied to simulate the scouring patterns at the Lock and Dam No. 26 (Replacement) site. The measured and computed scouring patterns around the Stage 1 cofferdam was used for model calibration/verification. The close agreement between observed and computed channel topographies for Stage 1 cofferdam site showed the reliability of the model. Therefore the model was used to predict the scour potential in the next phase of construction.

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# SEDIMENT TRANSPORT AND RIVER SIMULATION MODEL

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## ABSTRACT

The U.S. Bureau of Reclamation STARS (Sediment Transport and River Simulation) model was developed to mathematically simulate the movement of water and sediment through alluvial river channels. The unique feature of this one-dimensional steady-state model is the use of streamtubes to vary the hydraulic and sediment transport characteristics across a cross section. This will allow a more realistic representation of sediment movement. For example, scour can be modeled at the outside of a bend while concurrent deposition occurs on the inside of the same bend.

Data requirements for operation of the STARS model include hydraulic parameters such as cross section geometries, water discharges, stages, and temperatures, as well as the associated sediment parameters such as size gradations of the streambed and sediment supply to the study reach.

This paper describes the development and use of the STARS model. Routines developed to update the cross section geometry, mix transported and streambed sediment, and update time step information are explained. Sediment transport equations programmed into the STARS model are referenced. Results using U.S. Geological Survey data from the East Fork River in Wyoming are presented. Finally, possible applications to aggrading or degrading alluvial channels as well as limitations of the model are offered.

## INTRODUCTION

Rivers in the semi-arid western United States often carry high concentrations of sediment which create problems for designers of bridges, dams, and other hydraulic structures. Aggrading and degrading rivers have been under study by hydraulic engineers, geomorphologists, and others for a number of years. With the advent of present day microcomputers and the ability to store and manipulate large quantities of data, mathematical models have gained wider acceptance and use. The STARS model has been developed by the U.S. Bureau of Reclamation (USBR) based upon concepts of a model formulated at Colorado State University (Molinas, 1983) which simulates the movement and distribution of water and sediment in rivers using streamtubes. The STARS model has been designed to be flexible, efficient, and easy to use. Further testing and refinement of the STARS model will continue as an ongoing process at the USBR. The STARS model holds promise for the short, intermediate, and long-term evaluation of rivers and river response to human intervention.

STARS is a one-dimensional steady-state model which uses streamtubes to help simulate the lateral variation of hydraulic and sediment parameters.
The model may be used to perform either a fixed or moveable bed analysis. When STARS is used as a fixed bed model, no sediment data are required and water surface profiles are computed assuming an unchanging bed.

When a moveable bed analysis is desired, the user must provide a discharge hydrograph (described by a series of discharges and corresponding time steps). A steady-state water surface profile is computed for the initial discharge of this hydrograph. Using these water surface elevations, each cross section is divided into streamtubes of equal discharge and hydraulic properties are determined. Sediment transport rates are then computed for each streamtube and the amount of scour or fill is determined. Finally a new size gradation of the bed is computed and the cross section coordinates are adjusted. Then the model proceeds through the rest of the discharge hydrograph in a similar manner.

## DATA COLLECTION AND MODEL INPUT

Specific field data needed to execute the fixed bed or hydraulic portion of the STARS model are similar to the data required for any of the available water surface profile computer programs. Geometric data to define the channel shape include cross section profiles, channel reach lengths between sections and roughness coefficients. Channel roughness values across a section are segmented with corresponding lateral coordinate endpoints and longitudinal reach lengths. The upstream boundary is specified as a discharge hydrograph, stage-discharge rating curve, or slopes (if normal depth can be assumed). The only additional input is the number of streamtubes which gives the user the ability to further define the channel velocities and associated sediment transport capabilities across the section.

In order to run the moveable bed portion of the model in conjunction with the water routing, additional sediment data are required. Basic input include representative sediment size gradations of the streambed material at each cross section. The user can vary the bed material size gradations across the cross section using lateral station limits and associated gradations. An incoming sediment load hydrograph or sediment-discharge rating curve, corresponding to the water discharge hydrograph, is required along with the water temperature hydrograph to provide the upstream boundary conditions for the sediment transport method or algorithm which best fits the river conditions or available data in the study reach. Limits on the depth of degradation can be supplied by the user for the case where there is a known grade control or bedrock elevation below the streambed.

## WATER SURFACE PROFILES

Water surface elevations are computed assuming steady-state conditions using the standard step method. An upstream boundary discharge hydrograph and a downstream boundary elevation are required by the model. The downstream elevation may be expressed as a stage-discharge rating curve, an elevation hydrograph, or a slope-discharge relationship. Unsteady open channel flow analysis is not used because of prohibitive computational time and cost.

From the initial water surface elevation, calculations proceed upstream satisfying the conditions of conservation of energy. The friction slope is computed using the Manning's equation. A Newton algorithm with special checks for convergence problems is used to solve both the energy and critical depth equations. Convergence is usually obtained in two or three iterations, but in a few cases these equations have discontinuities in slope and a step search is temporarily employed until the trial water surface is past the point of discontinuity. Convergence is always obtained to a minimum tolerance of 0.01 feet.

The energy balance is voided when the computed water surface elevation has an adverse water slope or the flow is supercritical. When an adverse water slope is computed, the upstream water surface is set equal to the downstream water surface elevation. When the computed water surface elevation is supercritical, the model brings the water surface up to the critical depth. This is reasonable because supercritical flow rarely occurs in natural channels as an average condition across the entire section and because the transport equations are calibrated with subcritical discharges.

## STREAMTUBE CONCEPT

The mathematical basis for routing water and sediment in streamtubes begins with two definitions from Chow (1964):

1. "A streamline is an imaginary line within the flow for which the tangent at any point is the time average of the direction of motion at that point," and

2. "A streamtube is a tube of fluid bounded by a group of streamlines which enclose the flow."

The streamtube, in the case of river modeling, is not circular in shape but is an irregular area bounded by the channel geometry, the water surface, and the vertical streamtube divisions. Figure 1 shows a typical cross section divided into five streamtubes. This mathematical approach divides the flow into

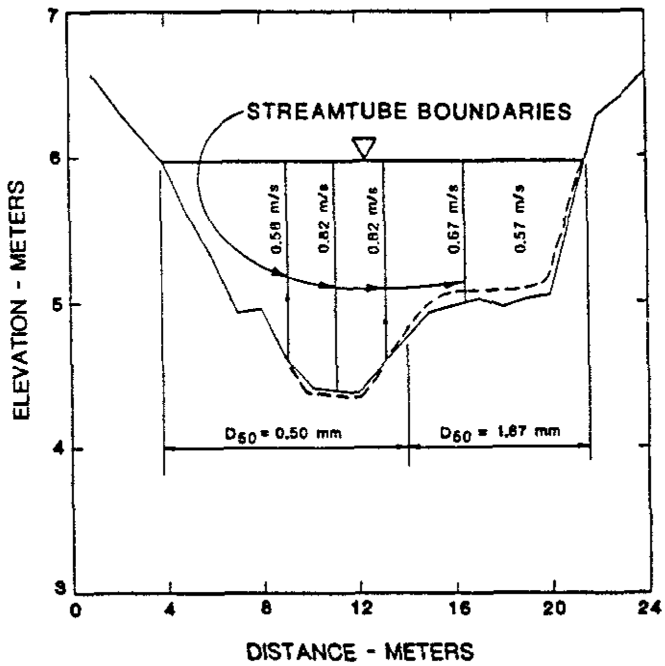


Figure 1. Typical Cross Section With Five Streamtubes

segments of equal conveyance and discharge. By calculating sediment transport in streamtubes, the distribution of the sediment transport across the section can be obtained. In this manner, transport rates calculated in overbank areas are lower than those for the main channel, as would be expected.

Streamtube boundaries are determined after the water surface elevation is computed for a given time step and discharge for the cross section as a whole. The total conveyance, summed between individual coordinate points, is divided by a user supplied number of streamtubes (maximum of 10). The lateral locations of the streamtube boundaries are interpolated between cross section coordinate points. The area, wetted perimeter, and top width can then be calculated for the individual streamtubes. These parameters together with slope, velocity and bed material gradations are essential to computing sediment transport.

## SEDIMENT TRANSPORT EQUATIONS

A number of sediment transport equations have been developed from flume and river data based on bed material ranging from medium gravel to very fine sand. The predictive equations programmed into the STARS model are:

Meyer-Peter and Muller (1948) based on USBR investigations (1960, 1984)

Einstein Bed-Load Function (1950) based on the Velocity - Xi Adjusted Einstein Equations (Pemberton, 1972 and USBR, 1963)

Engelund and Hansen (1967)

Toffaleti (1968, 1969) adaptation of the Einstein Bed-Load Function

Yang (1973) with the updated gravel bed equation from Yang (1984)

Ackers and White (1973)

The best that any sediment transport equation can do (based on a river flow condition) is to predict the sediment transport capability of a given flow for a certain sediment mixture. The sediment transport is considered to be supply limited when there is insufficient material available from upstream and in the bed to supply the computed transport capacity during a given time step. When the transport is supply limited, the model will automatically reduce the computed transport rate to the supply limited rate.

## CROSS SECTION UPDATING ROUTINE

The critical link to making the STARS model accurately simulate a moveable bed is in the ability to apply the predictive sediment transport calculations to the cross section coordinates. Sediment transport calculations proceed in the downstream direction matching the physical movement of the sediment. For each of the streamtubes, sediment transport rates are compared between the upstream and downstream sections and a net sediment flux is computed for the subreach between the two sections. Using the bulk density for sand, a volumetric change can be computed from the net sediment flux. Dividing the volumetric difference by the effective distance between sections gives a new change in cross sectional area to be applied to the coordinate points in the streamtube. After a net change in elevation is computed for each streamtube, the cross section coordinates are adjusted across the entire cross section.

## ACTIVE LAYER AND TIME STEP

The sediment transport process is a gradual sorting and mixing of the incoming sediment load with the existing bed material. A certain thickness of bed material, or active layer, is considered to be in a state of flux at any cross section and time step. The thickness of the active layer must have some relationship to the height of bed-forms in the channel (Bennett and Nordin, 1977).

An active layer, computed at 10 percent of hydraulic depth, is considered to be a first approximation to the height of bed-forms in the channel. While this relationship will underestimate some bed-form heights and overestimate others, it is practical for modeling because too small an active layer would severely reduce computational efficiency and too large an active layer would introduce too much error. Once the active layer is determined, then an appropriate time step is selected.

A time step is the period in which the model allows the river channel to scour and fill before the cross section geometries and bed materials are updated. The model's time steps are limited by either the user-specified time step or the minimum time in which any one streamtube scours or fills to a depth to its active layer. The user provides a hydrograph of water discharges and corresponding time steps (major time steps). When this time step results in a scour or fill depth greater than the active layer, it is automatically divided into smaller (minor) time steps. The minor time step for all cross sections is computed so that the limiting tube and cross section will have a scour or fill depth equal to the active layer.

When fill occurs, an inactive layer is established and maintained in a manner similar to the method used by Bennett and Nordin (1977). The inactive layer is used to keep track of the gradation and thickness of the fill material between the active layer and the original bed. If scour occurs after fill and the inactive layer is removed, the model then uses the gradation of the original bed (see figure 2). This feature of the model may also be used to represent a river with two bed material layers of different gradations. In this case, the surface bed material gradation and its thickness are assigned to the inactive layer while the underlying bed material gradation is assigned to the original bed. Once the surface bed material has been scoured, the model will begin using the underlying bed material gradation (see figure 3).

## APPLICATIONS AND LIMITATIONS

The STARS model has been developed to route water and sediment for alluvial channels. It can be operated as a fixed bed model to compute water surface profiles, or as a moveable bed model for applications to degrading or aggrading rivers as well as those in equilibrium. The existing version will accept any number of cross sections and up to 200 coordinate points for each cross section. A maximum of ten roughness segments can be input at each cross section.

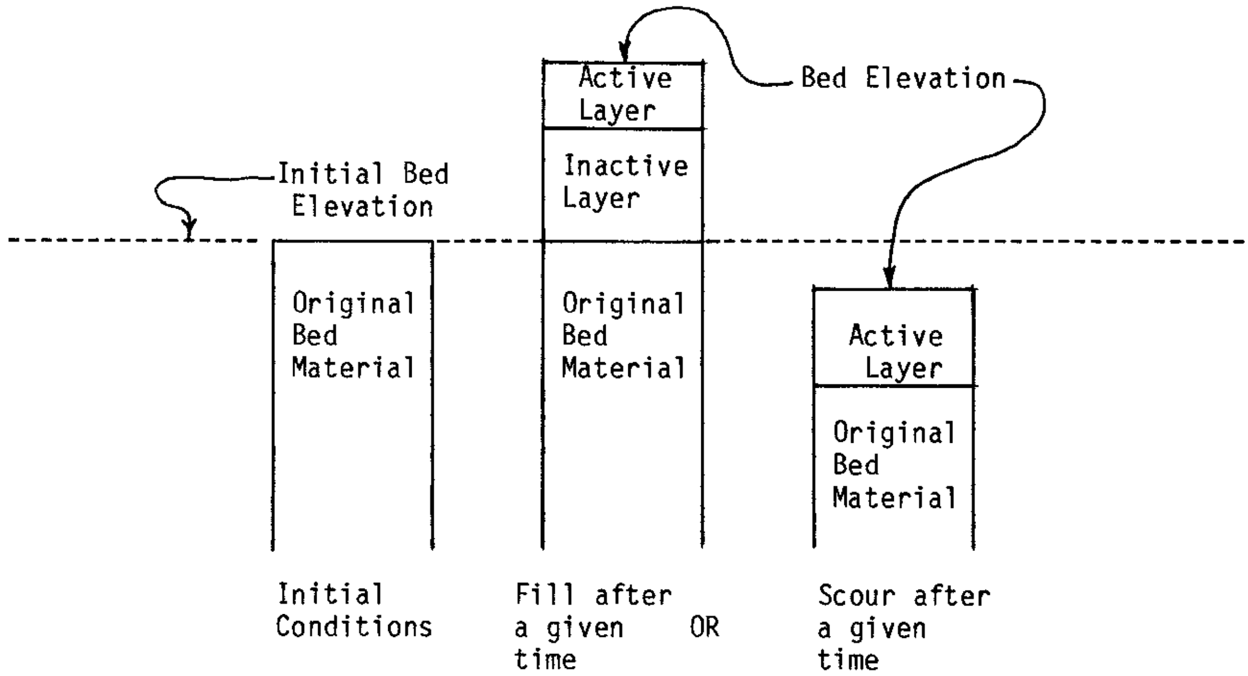


Figure 2. Active and Inactive Layer Definition Sketch
(One Initial Gradation)

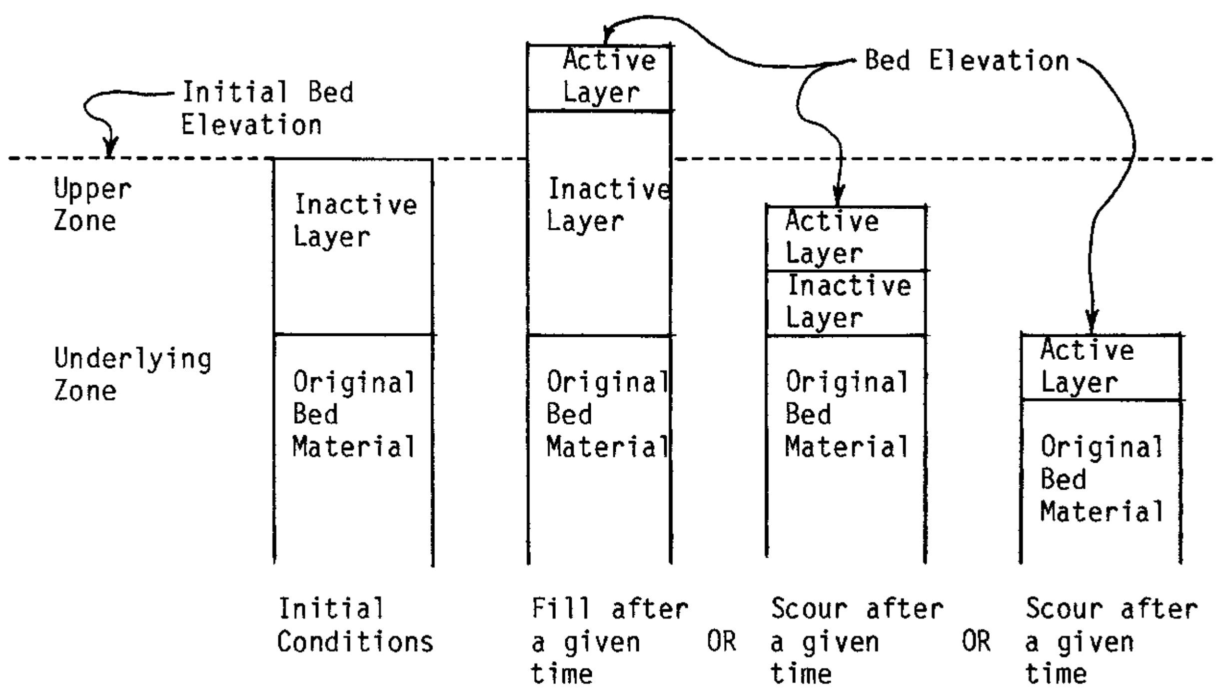


Figure 3. Active and Inactive Layer Definition Sketch
(Two Initial Gradations)

A maximum of 10 streamtubes in the STARS model was found to be a practice limit beyond which coordinate points become too scarce and output becomes too voluminous. Lateral changes in bed material at a cross section are limited to 10. Data collection and laboratory analysis costs will probably limit bed material size gradations to generally fewer than 10 samples at a single section.

The sediment transport equations were not developed for supercritical flow and since hydraulic jumps are seldom encountered across an entire section in natural conditions, a critical velocity constraint was added to the water routing portion of the model. With this constraint, water surface profiles can be easily computed for steep channels. Further testing and verification is needed to determine the applicability of the model to supercritical flow regimes.

## RESULTS USING THE EAST FORK RIVER DATA

A comprehensive data set to describe water and sediment movement was gathered by the U.S. Geological Survey on the East Fork River near Boulder, Wyoming (Emmett, et. al., 1980, and Meade, et. al., 1980). The reach of river simulated with the STARS model is 3213 meters long and has an average slope of 0.0007. At the bankfull stage the river is about 18 meters wide and 1.2 meters deep. Bed material consists of sand and gravel with a median diameter of about 1 mm. A more detailed description of the study reach is given by Leopold and Emmett (1976).

The reach was modeled for a 30-day period from May 22 to June 20, 1979, using 39 surveyed cross sections, corresponding bed material gradations an upstream boundary discharge hydrograph, and a downstream boundary stage hydrograph. The $D_{50}$ of the bed material ranged from 0.39 mm to 14.3 mm. Figure 4 shows successive thalweg profiles plotted with time (into the page). This plot summarizes the entire simulation period and qualitatively shows which cross sections experienced the most fill or scour and at which times. Figure 5 compares the initial thalweg profile with both the measured thalweg profile and the one predicted by the STARS model. The majority of predicted thalweg points followed the trends in scour and fill over the 30-day period.

## CONCLUSIONS

Hydraulic and sediment transport routing schemes have been coupled in the STARS model to reasonably predict changes in the bed profile on the East Fork River. This is partly because the STARS one-dimensional river model is able to represent lateral variations in water and sediment movement through the use of streamtubes. Since the STARS model performed well in the highly variable sand and gravel beds of the East Fork River, it is expected to apply to a variety of other sand and gravel bed rivers.

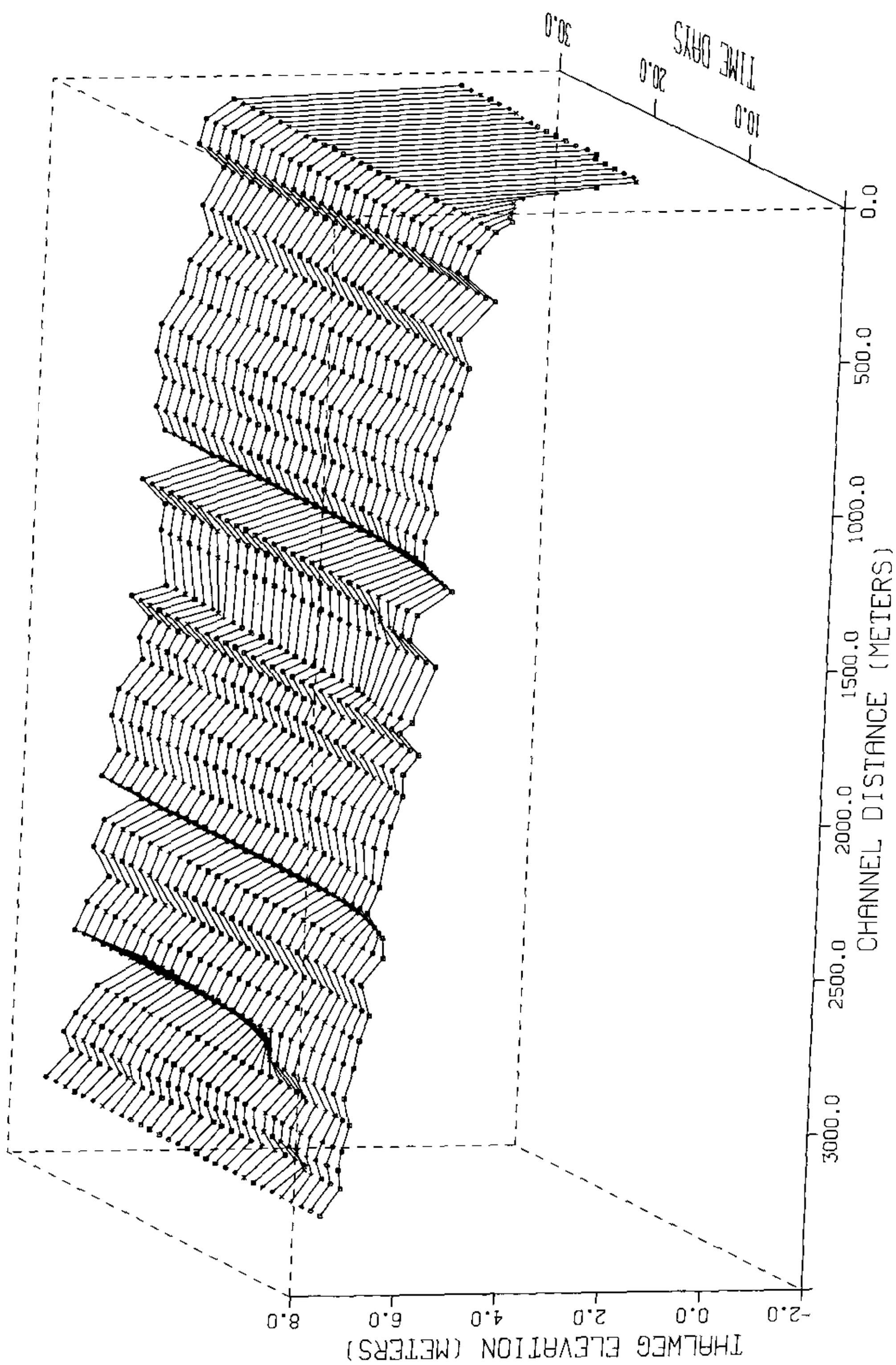


Figure 4. East Fork River Thalweg Profiles Plotted With Time

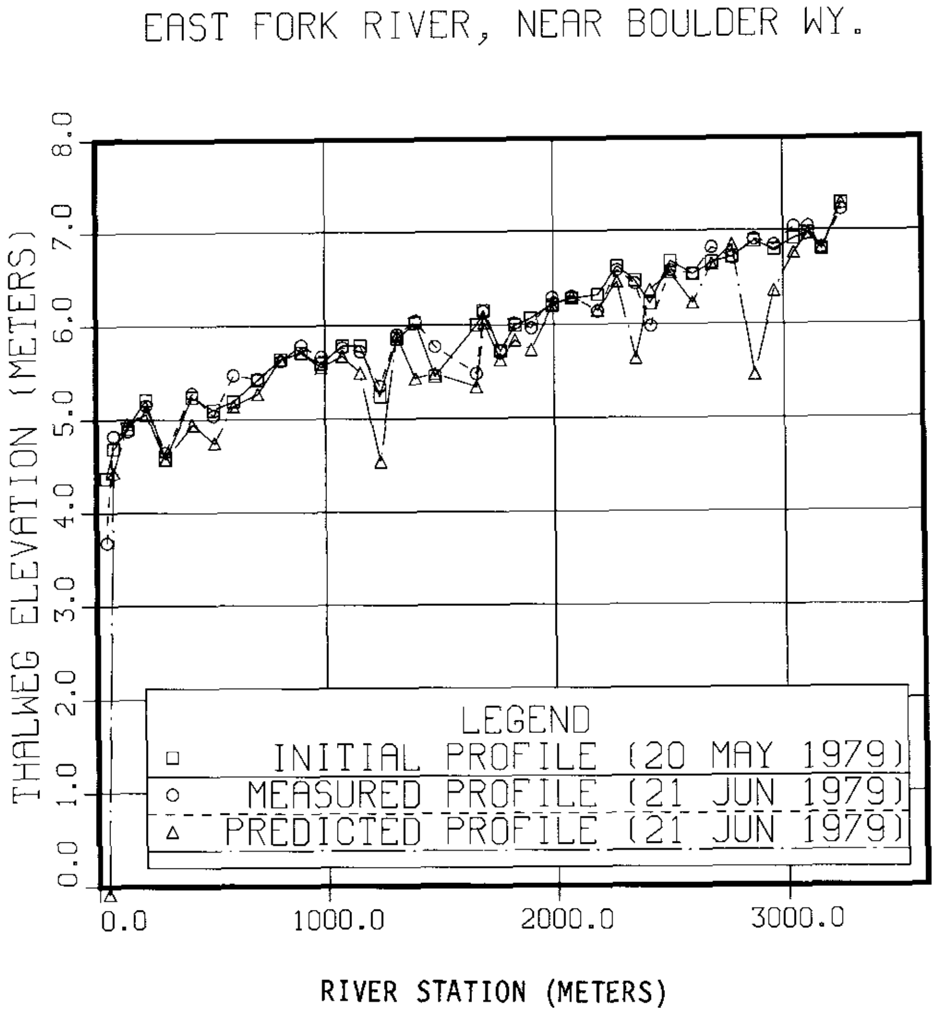


Figure 5. Thalweg Profile Comparisons

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# A RUNOFF-SEDIMENT YIELD MODEL FOR SEMIARID REGIONS

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## ABSTRACT

Watershed or basin-scale models are distributed to account for spatial variations in rainfall, soils, vegetation, and land use, and to accurately represent complex channel networks. A distributed model, ARDBSN (Arid Basin), based on simplified equations approximating infiltration, runoff, erosion, and sediment transport is described. The development and structure of the model is briefly reviewed. Three extensive analyses are described and interpreted: (1) sensitivity analysis for the major model parameters, (2) model validation, and (3) example applications. Theoretical shortcomings are used to illustrate the need for research on specific processes and model components. Strengths of the present model are used to show the necessity of including hydrologic simulation in the development of scientifically defensible range management research. A final manuscript section describes the need for development of expert systems for "front end" or parameter estimation/input file development in support of complex simulation models, and the need for output interpretation to summarize detailed simulation results.

## INTRODUCTION

Land use planners and engineers in semiarid regions are commonly faced with evaluating the impacts of a project on water yield and quality or designing a hydraulic structure on watersheds where insufficient data are available. Empirical methods, such as the Soil Conservation Service (SCS) runoff and peak flow equations (SCS, 1972), and Universal Soil Loss Equation (USLE) (Wischmeier and Smith, 1978) are relatively simple to use, but sacrifice accuracy for simplicity when applied to nonhomogenous or large drainage areas. Computer simulation models, such as CREAMS (Knisel, 1980), incorporate some of the above methods with more sophisticated procedures (e.g., daily soil water accounting) to estimate the impact of agricultural management systems on water, sediment, and plant yields. Extending the concepts developed for these models for application on semiarid uncultivated basin-scale drainage areas requires that the special features of these watersheds be taken into account. Spatial variablity of rainfall and watershed physical characteristics, such as soil types and vegetation density, influence the upland processes of infiltration, runoff, erosion and plant productivity. As drainage area increases, the channel network characteristics become more important in affecting runoff rates and amounts as well as sediment yield. Streamflow will vary in the downstream direction as a result of channel geometry, delivery of water and sediment to the channel network, infiltration into the channel alluvium, and erosion, transport, and deposition in the channel.

In addition to approximating the upland and channel processes, a model should be able to reflect changes in watershed response due to land use changes, be applicable to ungaged areas, and be cost effective, both in data requirements and computer time. SPUR (Simulation of Production and Utilization of Rangeland) (Wight, 1983) is a comprehensive rangeland model, under development by the USDA Agricultural Research Service and cooperating universities, which attempts to approximate the above processes. SPUR is composed of seven major components, including climate, hydrology, plant, livestock, wildlife, insect, and economics. This paper discusses a simplified version of the SPUR hydrology component which will be referred to hereafter as the ARDBSN (short for Arid Basin) model.

The objectives of this paper are to: (1) present and document a distributed runoff/sediment yield model; (2) discuss an analysis of simulation and prototype data agreement from experimental watersheds in southeastern Arizona; (3) demonstrate strengths and weaknesses of the model in relation to potential applications and simplifying assumptions made in the model structure; and (4) offer suggestions for future research and study.

## MODEL DESCRIPTION AND DEVELOPMENT

ARDBSN is a quasi physically based, distributed runoff and sediment yield model based on a continuous simulation of a daily time step water balance, the SCS runoff and transmission loss equations, Modified Universal Soil Loss Equation (MUSLE), and modified DuBoys-Bagnold sediment transport equations.

ARDBSN contains an upland or field component, and a channel component. The field component maintains the daily water balance and calculates surface runoff, field erosion and sediment yield. The channel component assumes surface runoff delivered from the fields is input to the channel network, and routes the water from the watershed upper reaches to the outlet. Flow volume reductions and peak discharge attenuation resulting from losses in alluvial stream beds are accounted for by the model. Sediment discharge is calculated as a function of the channel's sediment transport capacity.

The model is distributed in that a watershed can be subdivided into as many as 27 subbasins consisting of upland (no well defined channels) and lateral or interchannel basins. The channel network can be represented by one to nine channel segments; the 1st order channel segment receives input from one or more upland and two lateral subbasins, while higher order segments receive input from one or two lower order channel segments and lateral subbasins (Fig. 1).

Model input variables include daily rainfall totals, mean monthly temperature and solar radiation, and seasonal leaf area index. Model parameters can be estimated from field data, topographic maps and readily available handbooks so that it is possible to apply the model to ungaged watersheds.

Field Component: The field component of ARDBSN maintains a daily water balance and calculates sediment yield for each of the user chosen subbasins on the watershed. Most of the field component calculations are from the model SWRRB (Williams et al., 1985). Surface runoff is calculated using the SCS Curve Number equation,

$$Q = (P - .2S)^2/(P + .8S) \qquad (1)$$

where Q is surface runoff (in), P is storm rainfall (in), and S is a retention parameter (in). The influence of soil moisture on infiltration rate is approximated by updating the retention parameter, S, as a weighted average unused storage in the soil from 0 to the maximum retention parameter, $S_{max}$, which is calculated by,

$$S_{max} = 1000/CNI - 10 \tag{2}$$

where CNI is the dry antecedent moisture condition Curve Number.

Field erosion and sediment yield, E, is calculated with MUSLE (Williams, 1977) using the equation,

$$E = 95.14(Q*Q_p) \quad KLSCP \tag{3}$$

where Q(ac-ft) and $Q_p$(cfs) are the runoff volume and peak rate respectively, and K, LS, C, and P are the factors in the USLE equation (Wischmeier and Smith, 1978).

Both the Curve Number and MUSLE factors are simplifications of the upland runoff and erosion and sediment yield processes. Because the Curve Number is a storm total rainfall-runoff relationship, it does not account for variablity in rainfall intensity which, in semiarid regions, can be a dominant factor in the runoff amount produced (Osborn and Lane, 1969). MUSLE does not simulate the rill and interrill detatchment and deposition rates as individual processes but in a lumped manner incorporated in the runoff variables and USLE parameters. However, in spite of these drawbacks, both methods have the advantages of being extensively tested and documented (albeit less so for semiarid rangelands), and parameter values are easily estimated from physical watershed characteristics.

Channel Component: The channel component computes runoff rates and volume, sediment transport and yield, and flood flow reductions caused by channel abstraction (transmission losses). Many semiarid watersheds have broad alluvial channels which can abstract large quantities of streamflow. These abstractions, or transmission losses, are important not only because the runoff volume is reduced and flow peak is attenuated as the flood wave travels downstream, but also because they can be an important source of groundwater recharge. The transmission loss calculations used in ARDBSN were developed by Lane et al. (1983), and represent a compromise between simple loss-rate equations (SCS, 1972) and complex kinematic wave models incorporating an infiltration function.

Transmission loss equations developed by Lane use an ordinary differential equation to approximate rate of runoff volume change with distance. Transmission loss amount is a non-linear function of channel length and average width, upstream and lateral inflow, and mean duration and volume runoff. The transmission loss model parameterization was based on analyses of 139 events from 14 channel reaches in Arizona, Texas and Nebraska.

Ephemeral streamflow in semiarid regions usually results from individual thunderstorm rainfall events and the hydrograph is characterized by a rapid rise, a sharp peak, and a slower recession. A double triangle hydrograph approximation, applied to semiarid watersheds by Diskin and Lane (1976), can be estimated if the peak discharge, time to peak, runoff volume and flow duration are known. The peak discharge equation was derived from the SCS method (SCS, 1972), and is calculated as

$$Q_p = C5\ V/D \tag{4}$$

where V is runoff volume (in), D is runoff duration (hr), and C5 is a parameter expressing hydrograph shape. Mean runoff volume ($\overline{V}$) and flow duration ($\overline{D}$) were related to drainage area using data from semiarid watersheds by Murphey et al. (1977), and are calculated from the equations

$$\overline{D} = C1\ A^{C2} \tag{5}$$

$$\overline{V} = C3\ A^{C4} \tag{6}$$

where A is area ($mi^2$) and C1-C4 are parameters estimated as a function of the watershed Curve Number, channel slope and watershed length to width ratio.

Sediment transport is assumed equal to sediment transport capacity and is calculated as a function of flow hydraulics and particle size distribution of the channel sediment. Sediment particles larger than 0.062 mm are assumed to travel as bed load. A modification of the DuBoys-Straub formula is used to calculate bed load transport capacity as

$$g_{sb}(d_i) = \alpha\ f_i\ B_s(d_i)\ \tau\ \left[\tau - \tau_c(d_i)\right] \tag{7}$$

where $g_{sb}(d_i)$ is the transport capacity (lb/s-ft) per unit width for particles of size $d_i$ (mm), $\alpha$ is a weighting factor, $f_i$ and $d_i$ are the proportion and diameter (mm) of particles in size class i, $B_s(d_i)$ is the sediment transport coefficient ($ft^3$/lb-s) and $\tau$ and $\tau_c$ are the effective and critical shear stress ($lb/ft^2$), respectively. Lane (1982) derived estimates for $B_s(d_i)$, $\tau$, and $\tau_c(d_i)$ based on $d_{50}$, or median particle size, and calibrated the model with data from the Niobrara River in Nebraska. The largest particle size in the calibration was 2.0 mm and the largest $d_{50}$ was 1.0 mm.

The suspended load transport equation is a modified Bagnold (1956) equation based on stream power in the form

$$g_{sus} = f_{sc}\ CAS\ \tau\ V^2 \tag{8}$$

where $g_{sus}$ is the suspended transport capacity (lb/s-ft), CAS is the suspended transport coefficient (s/ft), $f_{sc}$ is the proportion of particles smaller than 0.062 mm in the channel bed, and V is the average velocity (ft/s).

The hydraulic variables needed to solve equations 7 and 8 are estimated at 9 time intervals by a piecewise normal flow approximation of the double triangle hydrograph (Lane, 1982). The piecewise normal approximation allows for the discharge rate to vary in time as runoff moves down the channel segment, transmission loss equations account for runoff changes in the downstream direction, and variations in channel geometry and particle size allow for deposition or scour along channel reaches.

In summary, the upland runoff and erosion processes are approximated by the SCS curve number relationship and MUSLE. Stream flow is routed based on surface runoff delivery to the channel system. Runoff volume and flow rate are reduced by transmission losses which are functions of basin characteristic mean runoff volume and duration, and channel characteristics. Sediment transport is assumed to equal transport capacity and calculated as a function of hydrograph characteristics, hydraulic geometry, and channel particle size.

The hydrograph approximates spatially varied unsteady flow and allows the model to account for variability of channel geometry, transmission losses, and lateral inflow and their influence on water and sediment yield.

Applications: ARDBSN is intended to be applied to watersheds from about .01 to 10 $mi^2$ having alluvial channel systems which contain non-cohesive sediments. Stream flow should be ephemeral and occur as a result of individual storm events. The model should not be applied to watersheds where base flow or snowmelt dominate the streamflow. Because average events were used to derive the water and sediment yield equations, simulated individual events may be in error, especially for events associated with abnormally high or low intensity storms or unusual antecedent moisture conditions. Typical applications of ARDBSN include simulating flood frequency, predicting water and sediment yields from semiarid watersheds, and deriving sediment delivery ratios and sediment rating curves. An important model application is evaluating the influence of land use and conservation measures upon water and sediment yield. The model field component contains parameters, such as the Curve Number and MUSLE factors, which are affected by land use changes. Given changes in these parameters, the model can be used to estimate the channel network response to these changes.

## ANALYSIS

The hydrology component has been previously verified with data from 3 small watersheds, ranging from 3.2 - 108 acres, on the Walnut Gulch Experimental Watershed at Tombstone, AZ (Wight, 1983). The model was found to explain 88 - 94% of the variance of observed annual runoff and sediment yield, but tended to overpredict runoff and basin sediment yield during those years in which winter rainfall was greater than normal, indicating that the model will not provide realistic results for meteorologic conditions differing from those used in the development of the runoff and peak flow calculations. However, the model did reproduce the phenomena of decreasing water yield and discharge rate per unit area with increasing drainage area observed in semiarid regions.

Sensitivity Analysis: Those parameters which are important in the water and sediment yield and peak flow calculations were varied from the specified or optimal values to test the model's response to parameter changes and to evaluate the sensitivity in model output due to errors in estimating parameter values. Table 1 is a list of the parameters used in the analysis and their relative significance on the various types of model output for a 108-acre watershed. The Curve Number is the most sensitive parameter, significantly affecting all the outputs. Sediment yield is the most sensitive output, affected by Curve Number, Manning's n, the suspended transport capacity coefficient, channel slope and sediment particle size distribution. Manning's n does not affect peak flow or runoff because of the approximations used in developing the equations used to calculate transmission losses. The C1 and C5 parameters affect peak flow and, because peak flow is used in MUSLE, sediment yield from the fields. Transmission losses are sensitive to changes in channel hydraulic conductivity, but because the amount of losses relative to the amount of runoff is small, runoff is not as sensitive to this parameter. The result of the sensitivity analysis is that care must be taken to subdivide the watershed into homogeneous subbasins as dictated by data availability and application of the model. Errors in estimating the subbasin Curve

Number can cause significant errors in the model output. Channel geometry and particle size significantly affect simulated sediment yield so that the number of channel segments defined by the user should depend on the variability of channel geometry and sediment characteristics along the channel reach.

Table 1. Summary of sensitivity analysis parameters and relative significance to model output.

| Parameter | Runoff | Transmission losses | Peak discharge | Field erosion | Sediment yield |
|---|---|---|---|---|---|
| Curve Number | A[1] | A | A | A | A |
| C1 | C | C | B | B | C |
| C3 | C | C | D | D | C |
| C5 | D | D | B | B | C |
| CAS | D | D | D | D | B |
| Channel Characteristics | | | | | |
| Width | C | C | C | D | C |
| Length | C | C | C | D | C |
| Slope | D | D | D | D | A |
| $d_{50}$ | D | D | D | D | A |
| Manning's n | D | D | D | D | A |
| Hydraulic Conductivity | C | A | C | D | C |

[1]Explanation of Symbols: A = output change greater than parameter change; B = output change in between 1/2 and equal to parameter change; C = output change less than 1/2 parameter change; D = no change in output.

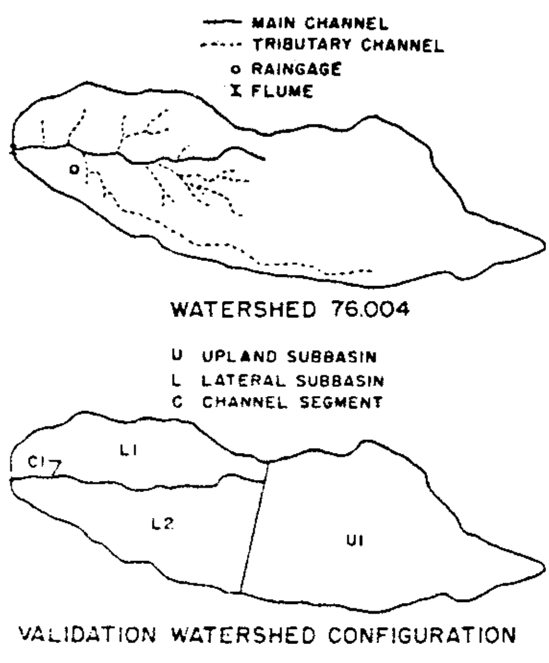


Fig. 1. Santa Rita Watershed 76.004 near Tucson, AZ and equivalent watershed configuration validation simulation.

Validation: Santa Rita Experimental Range watersheds, 76.004 (4.9 acres) and 76.003 (6.8 acres), were chosen to validate the model and test the transferability of parameter estimation techniques to ungaged watersheds. The hydrologic regime of the Santa Rita watersheds is similar to that of Walnut Gulch in that runoff occurs as a result of intense summer thunderstorms. Both watersheds have a gently sloping upland area with up to 30% desert brush and grass cover and an active channel area covering the lower 1/2 of the watershed. Both watersheds were divided into one upland, two lateral subbasins and one channel segment for the simulation runs. Fig. 1 shows watershed 76.004 and the simulation configuration. Parameters were estimated using field data, topographic maps, and procedures outlined in the SPUR User's Manual.

Fig. 2 is a comparison of observed and simulated annual values of runoff volume, maximum peak discharge,

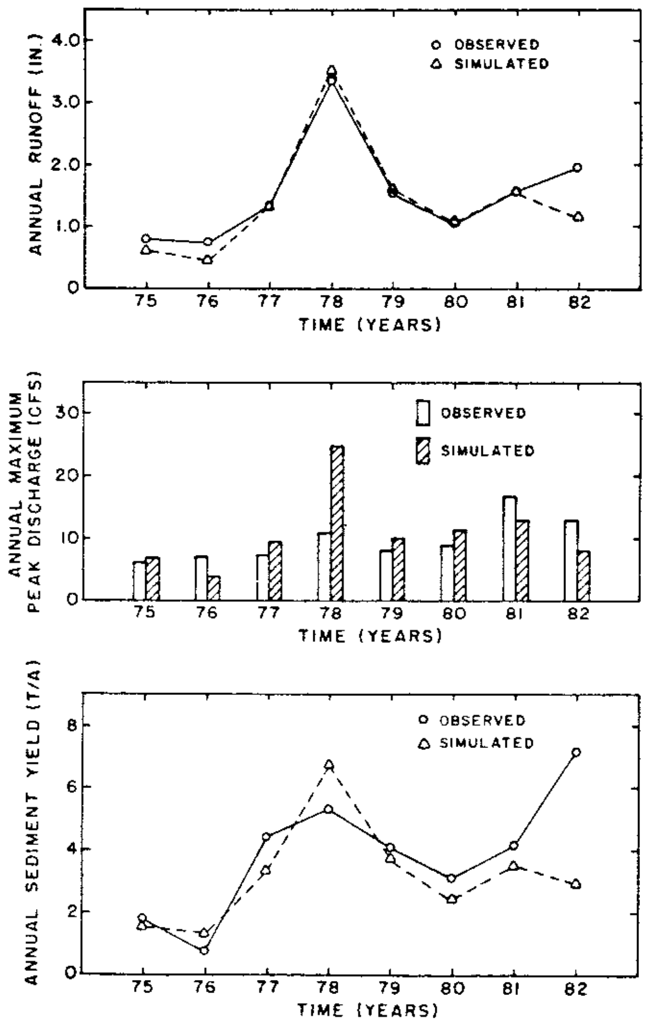


Fig. 2. Observed and simulated annual runoff volume, maximum peak discharge, and sediment yield for the period of simulation, validation, Watershed 76.004.

and sediment yield for watershed 76.004 (watershed 76.003 showed the same trends). Considering that parameters were not optimized, the simulated annual values are in good agreement with the observed data, with the exception of peak discharge. Notice that in 1978, the simulated peak is almost 2.5 times greater than the observed peak. The simulated peak is a result of a 23 hour, 2.2 inch storm which occurred in December. Overestimation of runoff and peak flow results from the use of the single parameter SCS Curve Number model and the peak/volume relationship (eq. 4), which was developed from analysis of short duration thunderstorm events. Thus, large volume, but low intensity, storms which occur in winter will be misinterpreted by the model. However, if used as a first approximation, the model can reproduce annual series of runoff and sediment yield. In addition, the parameter estimation techniques developed from Walnut Gulch data appear to be transferable to areas with similar hydrologic regimes.

Example Applications: Two examples are presented demonstrating use of the model (1) to develop an annual series for maximum peak discharge and water and sediment yield and (2) to evaluate a range management plan.

For the first example, simulated annual peak discharge, runoff volume, and sediment yield were ranked, plotted on log normal probability paper, and the 2, 10, and 100 year return period events were estimated. Fig. 3 is a plot of watershed drainage area and the 2, 10, and 100 year frequency maximum annual peak discharge. The solid lines are least squares regression fits to observed data from 8 watersheds in southeast Arizona (Lane, 1985). The circles are simulated peak flows for the 2, 10, and 100 year return periods for 3 watersheds on the Walnut Gulch Experimental Watershed and 2 watersheds on the Santa Rita Experimental Range. Notice that the points agree with the data based relationship and follow the trend of decreasing peak discharge with increasing area for the return periods.

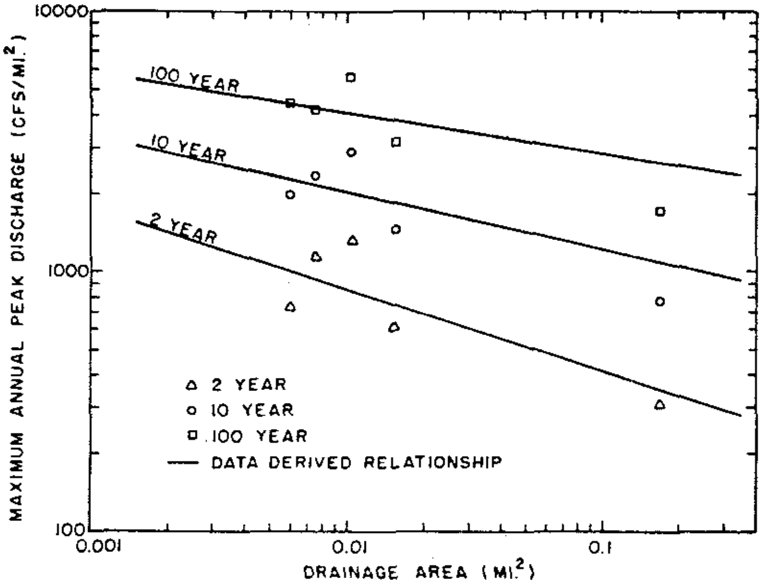


Fig. 3. Comparison of simulated 2, 10, and 100 year return period annual maximum peak discharge (cfs/sq. mi) with data derived relationship.

In the second example, three rangeland conditions were simulated by varying the Curve Number and the C factor of MUSLE. These somewhat representative range conditions are: sparse vegetation or desert brush, poor condition grass cover, and fair condition grass cover. Table 2 is a summary of the mean annual results of a 17 year simulation for the three rangeland conditions for watershed 63.103 on the Walnut Gulch Experimental Watershed; the last row is the magnitude of the output change resulting from changing from sparse vegetation to fair grass. Notice that as the watershed condition is improved, the magnitude of change is between 2 and 3.5 for sediment yield, runoff and peak flow rate, but that field sediment yield changes by a factor of 19. The last column in Table 2 is the percentage of the basin sediment yield contributed by the channel. As field sediment yield decreases, the relative amount that the channel contributes to the total yield increases by a factor of almost 2. There are two important implications of Table 2. One, the results suggest that the channel becomes more

Table 2. Simulated average annual runoff, erosion, sediment yield, and maximum peak discharge for three range conditions for 63.103.

| Range conditions | Runoff | Maximum discharge | Field sediment yield | Sediment yield | Channel contribution |
|---|---|---|---|---|---|
| | (in) | (cfs) | (t/a) | (t/a) | (%) |
| Sparse vegetation (SV) | 1.27 | 13.7 | 1.33 | 2.34 | 43 |
| Poor grass (PG) | .67 | 9.2 | .30 | 1.19 | 75 |
| Fair grass (FG) | .40 | 6.6 | .07 | .69 | 90 |
| Ratio of SV to FG | 3.2 | 2.0 | 19.0 | 3.4 | .5 |

dominant in sediment production as the amount of field sediment delivered to the channel network decreases. It is important to point out that, while a natural channel will adjust its geometry in response to changes in water and sediment delivery to the channel system, ARDBSN assumes constant channel geometry and sediment particle-size distribution for a given simulation period. Therefore, some caution must be used in interpreting sediment yield results, particularly results of long simulations. Two, a significant decrease in field erosion and sediment delivery to the channels may not immediately bring about a correspondingly significant decrease in sediment yield (in this case, a 19 fold versus 3 fold reduction). These results indicate that a program to monitor the effects of a watershed treatment should consider both the upland and channel processes. Evaluating upland improvement treatments by measuring water and sediment yield at the watershed outlet could result in an incorrect decision on the effects of a particular treatment.

## EXPERT SYSTEMS

As hydrologic models grow more complex and incorporate a wide spectrum of the processes active within the hydrologic cycle, it becomes difficult for a user to have the necessary expertise to both estimate input parameters and interpret model output. For example, to evaluate the parameters for all of the components of SPUR requires expertise or knowledge of hydrology, engineering, range management, plant physiology, animal science, and economics. It is incumbent upon the model developer to facilitate use of the model by action agencies if a model is to be more than a research tool. The use of expert systems or knowledge engineering could significantly aid in this process.

An expert system is defined by Bramer (1982) as ". . . a computing system which embodies organized knowledge concerning some specific area of human expertise, sufficient to perform as a skillful and cost effective consultant." Potential applications of expert systems in hydrologic modeling could include (1) determining if the model chosen is appropriate to the given problem, (2) selecting the parameters for model input, (3) summarizing and interpreting model output, and 4) suggesting management alternatives. The first category above would determine from an interactive dialogue with the user or an analysis of input data if the model is applicable to the user supplied conditions, the type of data available, and user needs. The second category would accept input parameters from the user, check their accuracy, given both the watershed physical description and other parameters, and estimate parameter values which the user is unable to evaluate. This is an important category because, as models become more complex, errors in input or parameter estimation become more probable, particulary by users not familiar with a model's stucture. The last two categories are interdependent; interpretation of results will lead to simulating alternative management plans. Given a set of conditions and needs, the results can be interpreted in various ways; the system would decide, based on the user's needs, which type of summary will be presented, and interpret that summary. The fourth category would use the initial watershed condition and the results of the interpretation to suggest management schemes. By simulating the alternative plans, and giving the user both a choice of scenarios and the implications of each on the user's needs, the system could act as a cost effective consultant.

## DISCUSSION AND CONCLUSIONS

The structure of ARDBSN includes simplifications and lumping of parameters in both the field component (Curve Number, MUSLE) and the channel component (average channel geometry, sediment transport equations) which simplify operation of the model, but potentially decrease its accuracy. As shown in the analysis, the model poorly represents extreme events or events caused by low intensity but high depth rainfall. However, considering that parameters were not optimized, the simulated annual series for runoff, peak discharge, and sediment yield are in good agreement with observed data. Parameter estimation techniques appear to be transferable to hydrologically similar areas, but more testing should be done on watersheds composed of a wider range of soil types and vegetation. The model duplicates the phenomena of decreasing yields with increasing areas, suggesting that the processes causing the decrease are being approximated by the model structure. The model results also suggest that as drainage area increases, channel processes become more dominant in determining water and sediment yields, consistent with observations in the field. As a management tool, it indicates that significant changes on the watershed to reduce erosion might have little immediate effect on the channel system. Finally, it underscores the idea that to model semiarid watersheds accurately, the channel processes controlling water and sediment yields must be represented in a manner which allows for spatial variability.

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# APPLICATION OF HEC-6 TO EPHEMERAL STREAMS

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## ABSTRACT

Three ephemeral streams in Arizona were studied to determine the applicability of computer model HEC-6 to quantify sediment scour and deposition in a stream channel. Two topographic maps, of before and after a flood or series of floods, were used for each stream study. The HEC-6 geometric data, consisting of stream bed elevations, etc., was obtained from the pre-flood(s) map; the hydrologic input data comprised discretized hydrograph(s) of the study period. The geometric output data, of stream bed elevations, etc., was compared with data from the post-flood(s) map. Geometric, sediment and hydrologic data development strategies are discussed. Two supplemental computer programs are presented that greatly facilitate the utilization of HEC-6. The results of the study indicate that HEC-6 can be a useful adjunct for the analysis of scour and deposition in streams when the assumptions of the program are rigorously observed.

## INTRODUCTION

HEC-6 is a computer simulation program developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center, to analyze the quantity of scour and deposition in rivers and reservoirs (HEC, 1977). Several other sediment transport computer programs exist and have been used to analyze streams with a wide variation of channel geometry and bed composition (National Research Council, 1983). However, HEC-6 is non-proprietary and, therefore, readily available to the professional engineer. HEC-6 was developed primarily for perennial streams that are characterized by relatively stable banks, well defined main channel, fine sediments and flood flows of fairly long duration. Typically, in the U.S. Southwest, the streams are braided, transport sediments of large size range, and have storm hydrographs of short duration and instantaneous peak discharges.

## THEORETICAL AND NUMERAL BASES OF HEC-6

The program is designed to calculate scour and deposition in rigid-bank channels by simulating steady, gradually-varied water flows and unsteady sediment flow. The principal assumptions in the model are (a) flow is one-dimensional and hydrostatic pressure prevails at all locations in the flow, (b) Manning's n is applicable to gradually-varied flow and can be expressed as a function of water-surface elevation or water discharge, (c) the entire movable-bed part of all cross sections has sediment scoured or deposited at the same uniform rate, and (d) the channel slope is small.

The basic relationships used in the program are the flow-continuity, sediment-continuity and the flow-energy equations; the general structure of HEC-6, consisting of the main program and 29 subroutines, is shown in Figure 1. The program solves the continuity and energy equations, and uses the iterative "standard step method" to calculate the basic hydraulic parameters: velocity, depth, width and slope. These parameters are then employed to calculate "representative hydraulic parameters" using weighting factors. The default values of the weighting factors allow for the "most sensitive" scour and deposition calculations. However, weighting factors can be specified in the program that result in the "most stable" calculations (HEC, 1977).

HEC-6 simulates stream bed armoring by calculating an equilibrium depth (water depth for the condition of no sediment transport) for each grain size and each discharge in the discretized hydrograph. The depth of bed material is then determined that must be removed to attain the equilibrium depth. This depth is very sensitive to the Manning n-values specified for the main channel (DMA, 1983). The depth of sediment between the channel bed surface and the equilibrium depth is the only material subject to scour. The stability of the armor layer is determined by calculating and adjusting the bed surface area exposed to scour.

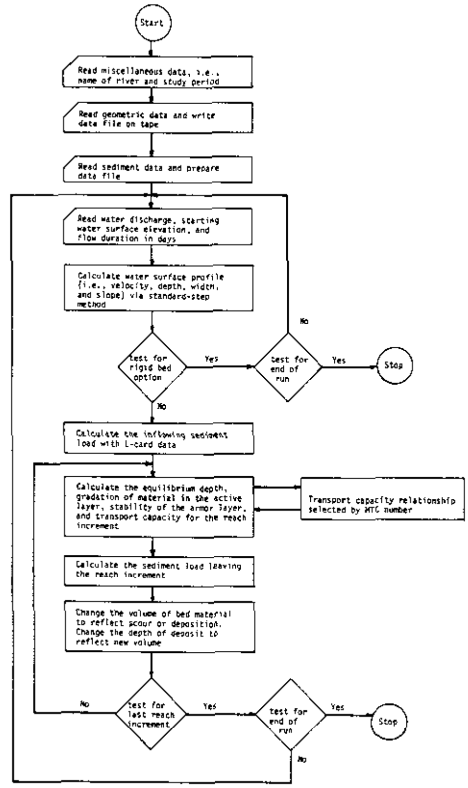


Figure 1. Flow Chart for Computer Program HEC-6

With representative hydraulic parameters specified, HEC-6 calculates the sediment transport capacity, for each grain size, at the beginning of each time step of the discretized hydrograph by using one of the following relationships that are included in the HEC-6 program: Laursen, Toffaleti, Yang, Duboys, or a special function (HEC, 1977; DMA, 1983). The sediment transport relationships of Colby, Ackers and White, Meyer-Peter and Muller, Schoklitsch, Engelund and Hansen, and Shields were also utilized in the study. Sediment loads are calculated by an iterative technique that determines the number of computational time intervals $\Delta t$ in a time step (T) of the discretized hydrograph. The function of the iterative technique is to minimize computational instability and evaluate the changes in the sediment transport load caused by bed armoring and changes in sediment gradation within a time step. The sediment transport capacity is not adjusted for changes in the hydraulic parameters during a time step, that is, the computer model is "uncoupled." The basis for the simulation of bed changes is the use of an explicit finite-difference scheme to solve the sediment-continuity equation. This solution scheme is only "conditionally stable," that is, with unstable conditions, the numerical errors grow unbounded and result in oscillating values of the channel bed elevation. Therefore, stability tests must be performed during the hydrologic input data development.

## DATA DEVELOPMENT STRATEGIES

The input data for HEC-6 are of three basic components - geometric, sediment and hydrologic.

Geometric Data. Cross sections of the stream were chosen at locations that define channel geometry transitions and at crossover points in meandering channels. The selection of reach length or distance between cross sections is an important consideration because this distance does influence the computational stability of the sediment transport computations. The HEC-6 program requires that the cross sections are divided into three subsections -

left overbank, channel and right overbank. These subsections, and their corresponding Manning n-values, are used in the backwater computations of the program.

The program also requires that the movable bed boundaries are specified in the input data. These boundaries designate the part of the cross section allowed to move vertically as a result of calculated scour or deposition. In selecting movable bed boundaries, chronological series of aerial photographs and cross section plots are valuable references. The specified elevation of the movable bed bottom designates the vertical dimension of bed material subject to the sediment transport process. Sensitivity tests indicate that if the specified bed elevation is too deep, the computed movable bed volumes (cubic feet) may become large enough to cause execution termination because of word length limitations of the computer system. Tests also indicate that bed grain gradation can change unrealistically with time if the specified bed elevation is too shallow. Ineffective flow areas, that is, cross sectional areas below the water surface elevation that are not capable of passing flow, are an important consideration in braided stream problems and must be identified.

The Manning n-value is used in equilibrium depth and head loss calculations. Studies by Dust (1983) and Bowers and Ruff (1983) indicate that bed change calculations are very sensitive to even reasonable discrepancies in selected n-values. Methods for the determination of n-values are given by Chow (1959) and Thomas et al. (1981).

Sediment Data. The requisite information is the initial bed material gradation, inflowing sediment rates to the study reach and armoring data. This data essentially provides the initial conditions and boundary conditions for the sediment gradation and movement parameters. The initial bed material gradation, which may be obtained from sediment frequency curves, influences the sediment transport rates, armoring formation and the stability computations. It is important that representative bed sediment samples are used in developing the data.

The inflowing sediment load at the upstream boundary of the study reach is coded in the input data as a table of sediment discharge for each grain size vs water discharge. Field collected sediment data should be used. However, in the absence of field data, the inflowing sediment load can be generated iteratively with HEC-6. One technique is to run the program with zero inflowing sediment; and then use the computed sediment discharge at the downstream-most section or at a section within the study reach, as the inflowing load at the upstream section. This technique should be iterated until the calculated sediment discharges converge to an equilibrium discharge for each grain size. Then, dummy or duplicate cross sections, adjusted to maintain the existing bed slope, should be added upstream of the study reach to reduce the significance of the errors in the generated inflowing sediment load data. Bowers and Ruff (1983) and Dust (1983) found that only one iteration was required for convergence of the inflowing sediment load when studying rivers in Arizona and California.

To simulate initial bed armoring, the percentage of the movable bed protected by armoring is specified in the input data. However, the influence of the specified initial bed armoring on the calculated bed changes is potentially insignificant because the bed armoring formation and stability calculations are usually performed several times within each time step.

Hydrologic Data. The basic components are the discretized discharge hydrograph, water temperature and downstream rating curve. HEC-6 requires that a continuous discharge hydrograph be coded as a sequence of discrete steady flows with duration in days. As previously indicated, the following parameters can influence the numerical stability of the solution scheme used in HEC-6: flow durations or time steps ($\Delta t$), computed or specified computational time interval (T) as determined by iterations of the transport load, distances between cross sections; and the computed sediment transport rates which are a function of: water discharge, specified transport relationship, hydraulic weighting factors, bed gradation, inflowing sediment load and channel geometry. Of these parameters, only the flow durations are essentially arbitrary. The flow duration is, therefore, the parameter adjusted to produce stable HEC-6 calculations. The procedure of estimating the maximum stable computational interval $T_m$ is referenced in Thomas et al. (1981). It is necessary to frequently check the computational stability of HEC-6 calculations because some of the parameters listed above change with time. This check for computational stability is best made with the supplemental computer programs MAXTREND and STAP (Dust, Bowers, Ruff, 1985). The flow durations also govern the significance of the "uncoupled" nature of HEC-6. It is recommended that the time step should be reduced when the calculated bed change at any cross section exceeds one foot or 10% of the water depth within the time step ($T_m$).

The transport capacity relationships, with the possible exception of Laursen, are relatively insensitive to temperature (Vanoni, 1978). This implies, and was verified by sensitivity tests, that HEC-6 computations are not sensitive to the specified water temperatures.

The HEC-6 water surface requirements at the downstream boundary of the study reach can be satisfied by specifying a stage-discharge relationship at the boundary, or assigning a zero water surface elevation at the boundary and allowing HEC-6 to attempt to perform an iterative backwater analysis. If the program is unable to make this analysis, the program defaults to the critical water depth. When the zero water surface elevation is assumed, a rigid bed should be specified for the downstream-most cross section. Several downstream dummy sections should also be incorporated into the data to minimize the influence of the end of the study reach on the computations.

## SUPPLEMENTAL PROGRAMS

The computer programs, MAXTREND and STAP, were developed to analyze HEC-6 executions. The algorithms used in the programs are shown in Figures 2 and 3. MAXTREND was used to determine the maximum stable computational time steps or flow durations; and identify flows and corresponding time steps which resulted in computations that violate the criterion of less than one foot of calculated bed change per time step. After the input data sets were calibrated, MAXTREND was used to evaluate the final trends and extreme values in the calculated bed changes.

The objective of STAP was to evaluate the sediment routing aspects of a HEC-6 simulation. With the geometric data for the initial and actual final conditions specified, STAP calculates the change in volume of bed material (Va) and the change in bed elevation (Ea) for each reach of the channel. The program also calculates for the initial and HEC-6 calculated final conditions, the change in sediment volume (Vc) and bed elevation (Ec). The values of the changes are compared by computing, for each channel reach: volume percent

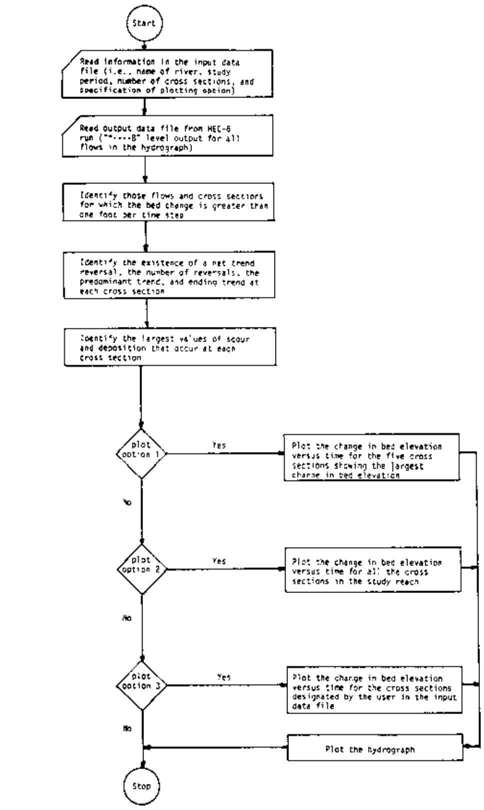


Figure 2. Flow Chart for Computer Program MAXTREND

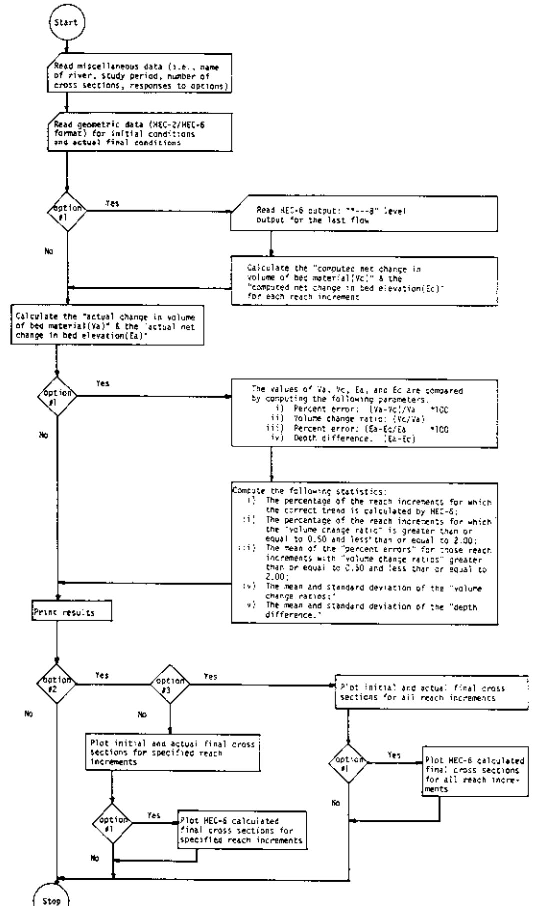


Figure 3. Flow Chart for Computer Program STAP

error, (Va-Vc)/Va*100; volume change ratio, (Vc/Va), depth percent error, (Ea-Ec)/Ea*100; and depth difference (Ea-Ec). The following statistics are then computed: the percent of the channel reachs for which the correct trend is calculated by HEC-6; the percent of the reachs for which the volume change ratio (Vc/Va) is greater than or equal to 0.50 and less than or equal to 2.00; the mean of the percent errors for those reachs with volume change ratios greater than or equal to 0.50 and less than or equal to 2.00; and the mean and standard deviation of the volume change ratio (Vc/Va) and the depth difference (Ea-Ec). As a calibration tool, STAP was used to perform sensitivity tests on various parameters that resulted in the selection of their "appropriate" values. The computed "volume change ratios" and "percent errors" gave a direct quantitative evaluation of the specified sediment transport relationship and, therefore, the "most appropriate" relationship available for the particular HEC-6 study. After the input data set was complete, the computed "volume change ratios" and "percent errors" provided a quantitative evaluation of HEC-6 ability to calculate bed changes.

## CASE STUDY #1: RILLITO CREEK

Rillito Creek has a watershed of 950 square miles. It is a typical desert stream that is dry much of each year but can flow at high rates in response to intense thunderstorms. A peak flow of 29,000 cfs occurred in October, 1983. The stream has a well defined, meandering channel configuration with approximately 85 percent of the banks soil cemented or protected by wire gabions. The stream bed material is composed primarily of gravely sand with ($D_{50}$) = 1.7 mm and $D_{90}$ = 9.5 mm.

The cross sections of the 2.4 mile study reach were developed from two sets of photo-topographic maps (1982 and 1984) with 2 foot contour line intervals. The movable bed elevation was set at 50 ft below the stream bed. An alternate value of 30 ft was tested, but was found to be too shallow. Photo-topographic maps and field observations showed that Rillito Creek has no ineffective flow areas. Initially, the default or "most sensitive" hydraulic weighting factors were specified in the data set. However, sensitivity tests as evaluated by program STAP, determined that the "most stable" factors improved the correlation of the bed change calculations.

Bed sediment samples were collected at three locations. Analyses indicated that the grain size distributions were similar, and a mean distribution was used. Figure 4. Inflowing sediment load data was not available for Rillito Creek. Therefore, the entire study reach was used to generate sediment data for 3 separate discharge hydrographs of 1,000, 10,000, and 30,000 cfs; and for the following relationships: Laursen, Yang, Ackers and White, Engelund and Hansen, and Shields Table 1 shows the computed sediment data using the Yang relationship and three discharges. From field observations, there was no evidence of bed armoring and, therefore, initial armoring conditions were not specified in the HEC-6 input data.

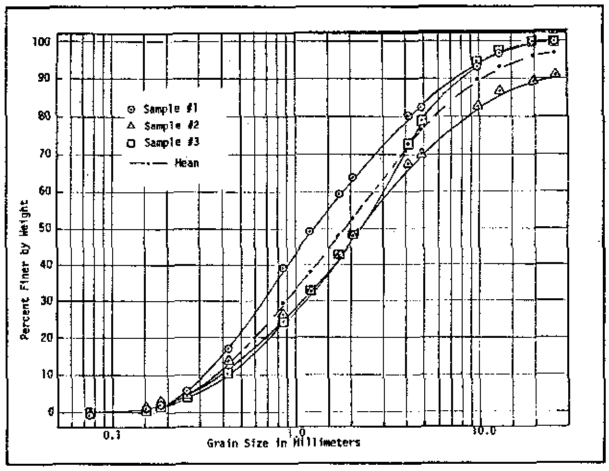


Figure 4. Distribution of Bed Materials

Table 1. Rillito Creek: Inflowing sediment load for Yang relationship (MTC=4): sediment load (tons/day) vs water discharge (cfs).

| Grain Size Class | Discharge (cfs) | | |
|---|---|---|---|
| | 1000 | 10000 | 30000 |
| Very fine sand | 1133 | 29221 | 128859 |
| Fine sand | 491 | 21778 | 121675 |
| Medium sand | 424 | 22130 | 125790 |
| Coarse sand | 370 | 19146 | 105682 |
| Very coarse sand | 306 | 16178 | 85958 |
| Very fine gravel | 272 | 16761 | 85534 |
| Fine gravel | 81 | 13742 | 67330 |
| Medium gravel | | 11332 | 53469 |
| Course gravel | | 7077 | 32780 |

The discharge histogram for the study was the October 1983 flood that had a duration of approximately 4 days and two peak discharges of 23,000 and 29,000 cfs. Figure 5 (a). The instability of the HEC-6 calculated bed changes, as a function of time, is shown in Figure 5(b)-(f). The stable magnitude of the bed changes is reached when the plot becomes horizontal, i.e., independent of time. The computational stability tests are summarized in Table 2 and Figure 6 for three of the sediment transport relationships. The maximum stable time interval is greatly dependent on the transport relationship.

The water temperature during the October 1983 flood was estimated to be 60°F. No stage-discharge data was available at the downstream boundary of this study. Therefore, the HEC-6 default critical depth option was used to generate the downstream boundary discharge rating curve. Four dummy cross sections were added to the study reach to minimize the significance of the critical depth assumption.

The final HEC-6 simulation for Rillito Creek, shown in Table 3, was evaluated with program STAP for each subreach. The subreach or reach increment is the sum of one-half the distance between the specified cross section and each of the adjacent cross sections.

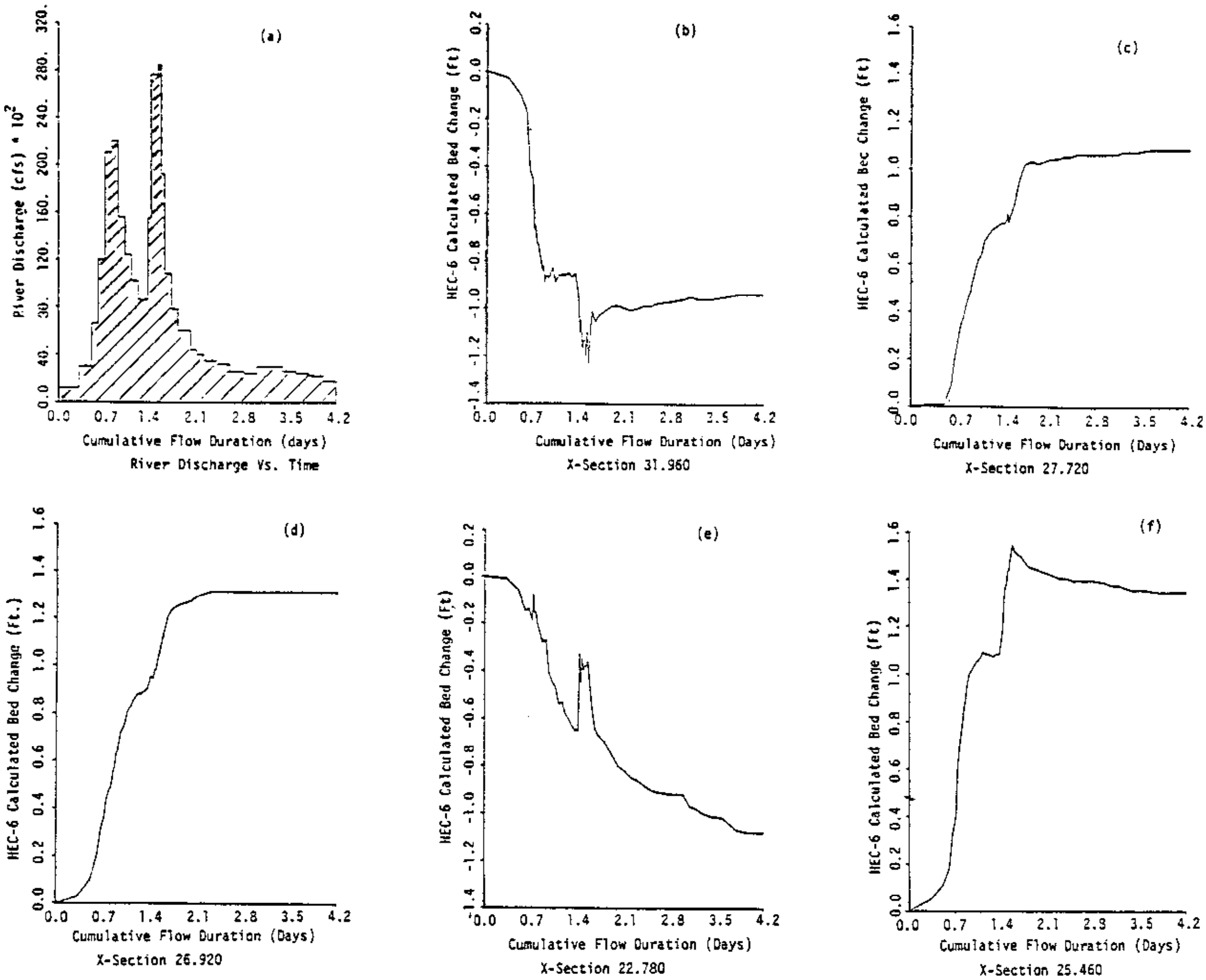


Figure 5. Discharge hydrograph; and flow duration vs calculated bed change for a sequence of cross sections, proceeding downstream (b) to (f), using the Yang relationship. Note that the ordinate scale is not the same for all figures.

## CASE STUDY #2: AGUA FRIA RIVER

Table 2. Rillito Creek: Computational stability tests

| Transport Relationship (MTC) | Discharge (cfs) | Max. Stable Time Interval Tm*(Days) |
|---|---|---|
| Yang | 30,000 | Tm < 0.05 |
| | 10,000 | < 0.10 |
| | 1,000 | < 0.40 |
| Engelund and Hansen | 30,000 | < 0.01 |
| | 10,000 | < 0.05 |
| | 1,000 | < 0.40 |
| Shields | 30,000 | < 0.0008 |
| | 10,000 | < 0.02 |
| | 1,000 | < 0.04 |

*Each time interval, $T_m$, is composed of 5 equal time steps, i.e., $T_m = 5\Delta t^m$

The Agua Fria River is located approximately twenty miles from Phoenix. Flow in the river is controlled by flood gates in Waddell Dam which impounds Lake Pleasant. The drainage area of the irrigation-storage lake is 1460 square miles. The 6.8 mile study reach is braided; and bed armoring with gravels and fine cobbles is evident. Serial photographs show that the banks migrate during flood flows. The surface layer of the bed material is approximately nine inches thick and consists of poorly-graded sands with a maximum of 6% gravel and less than 1% silt and clay. The surface material is underlain by a fairly well-graded sand with 35% gravel and negligible silt and clay. $D_{50}$ = 2.2 mm and $D_{90}$ = 25.0 mm. Three data sets were generated for the study periods:

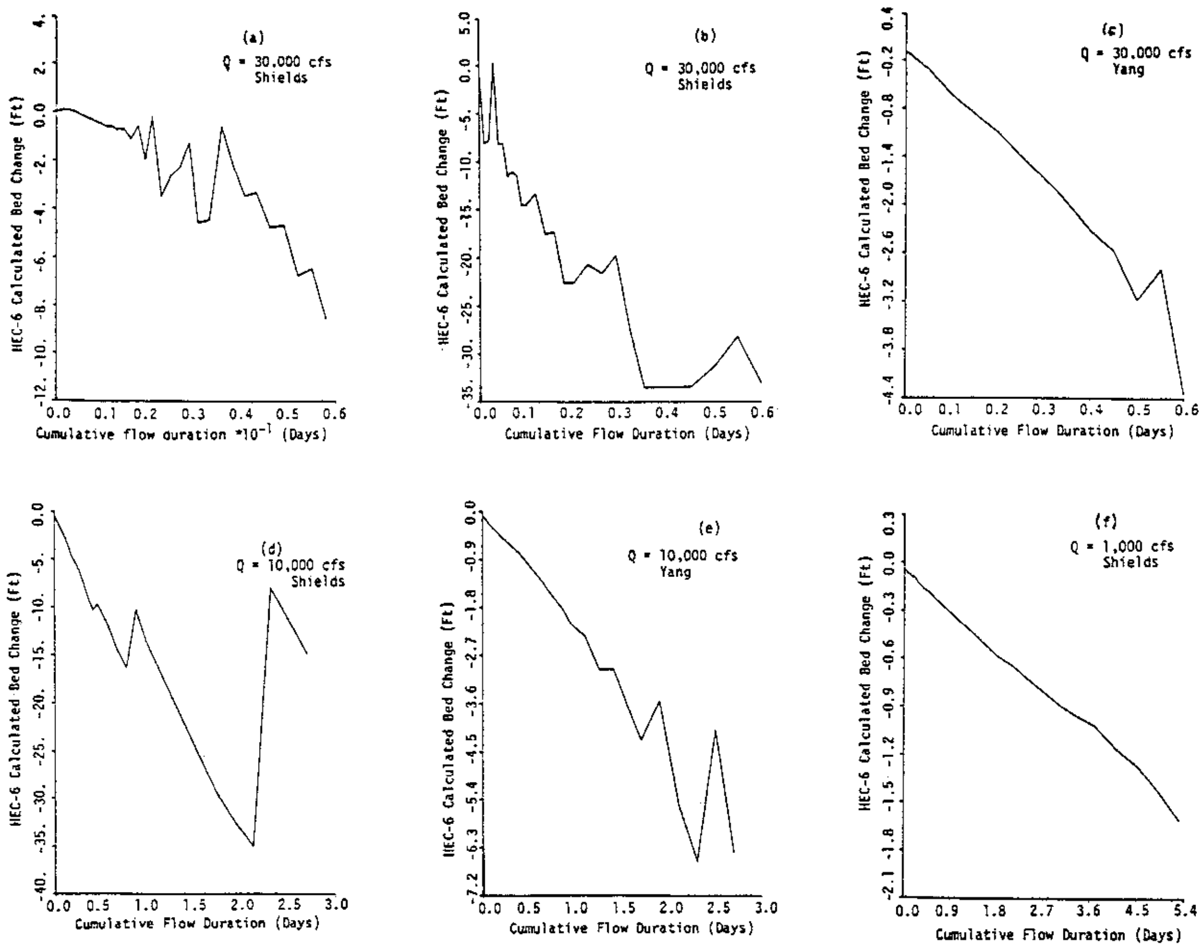


Figure 6. Effect on the HEC-6 calculated bed change stability, for one cross section (22.780) of discharge, sediment transport relation, and the maximum stable time step ($T_m$) = 5Δt. Note that the scales are not the same for all figures.
(a) Δt = 0.0003, 0.0005, 0.0008, 0.001, 0.0015, 0.002, 0.0025, 0.003
(b) - (f) Δt = 0.001, 0.005, 0.010, 0.030, 0.050, 0.100, 0.200

Table 3 (a) Rillito Creek: Actual change in volume and the HEC-6 predicted change in volume for each sub-reach. Yang relationship.

| X Sect. | Actual Volume Change (Cubic Ft) | Computed Volume Change (Cubic Ft) | % Error in Prediction | (Comp/Actual) Volume Change |
|---|---|---|---|---|
| 22.78 | -517376. | -423168. | 18.21 | 0.82 |
| 23.12 | 203520. | 0. | 100.00 | 0.00 |
| 23.60 | 111104. | -25088. | 122.58 | -0.23 |
| 24.36 | 85504. | 94208. | -10.18 | 1.10 |
| 24.90 | 233984. | 105472. | 54.92 | 0.45 |
| 25.46 | 295680. | 300800. | -1.73 | 1.02 |
| 26.00 | 257024. | 211456. | 17.73 | 0.82 |
| 26.92 | 531968. | 480768. | 9.62 | 0.90 |
| 27.72 | 678656. | 349184. | 48.55 | 0.51 |
| 28.66 | -48128. | -20480. | 57.45 | 0.43 |
| 29.24 | 67328. | 0. | 100.00 | 0.00 |
| 30.16 | -156416. | 105472. | 167.43 | -0.67 |
| 30.88 | -98816. | -56320. | 43.01 | 0.57 |
| 31.46 | 19712. | -80384. | 507.79 | -4.08 |
| 31.96 | -185856. | -123392. | 33.61 | 0.66 |
| 32.44 | -128000. | -35840. | 72.00 | 0.28 |
| 32.84 | -14080. | -18432. | -30.91 | 1.31 |
| 33.60 | 41984. | 43008. | -2.44 | 1.02 |

NOTE: negative volume changes indicate scour. The percentage of the reach increments where the "volume change ratio" (VCR) is:
a) VCR > = 0 : 83.33
b) 0.50 < VCR < 2.00: 55.56

Statistics of "Volume Change Ratio": Mean = 0.27353
Standard Dev. = 1.19941

Table 3 (b) Rillito Creek: Actual change and the calculated change in bed elevation. Yang relationship.

| X Sect. | Actual Depth Change (ft) | Calculated Depth Change (ft) | % Error in Calculations | Depth Difference (Actual-Calculated) |
|---|---|---|---|---|
| 22.78 | -2.21 | -1.80 | 18.55 | 0.41 |
| 23.12 | 0.76 | 0.00 | 100.00 | -0.76 |
| 23.60 | 0.47 | -0.10 | 121.28 | -0.57 |
| 24.36 | 0.33 | 0.40 | -21.21 | 0.07 |
| 24.90 | 1.03 | 0.50 | 51.46 | -0.53 |
| 25.46 | 1.28 | 1.30 | -1.56 | 0.02 |
| 26.00 | 0.86 | 0.70 | 18.60 | -0.16 |
| 26.92 | 1.67 | 1.50 | 10.18 | -0.17 |
| 27.72 | 2.76 | 1.40 | 49.28 | -1.36 |
| 28.66 | -0.20 | -0.10 | 50.00 | 0.10 |
| 29.24 | 0.27 | 0.00 | 100.00 | -0.27 |
| 30.16 | -0.59 | 0.40 | 167.80 | 0.99 |
| 30.88 | -0.55 | -0.30 | 45.45 | 0.25 |
| 31.46 | 0.15 | -0.50 | 433.33 | -0.65 |
| 31.96 | -1.37 | -0.90 | 34.31 | 0.47 |
| 32.44 | -0.71 | -0.20 | 71.83 | 0.51 |
| 32.84 | -0.07 | -0.10 | -42.86 | -0.03 |
| 33.60 | 0.29 | 0.30 | -3.45 | 0.01 |

The percentage of the reach increments where Depth-Difference (DF) is:
a) -1.0 < DF < 1.0 : 94.44%
b) -0.5 < DF < 0.5 : 61.11%

Statistics of "Depth-Difference": Mean = -0.09284
Standard Dev. = 0.55031

1964 to 1979: 3 flood flows with a total duration of 10 days; Q = 2,000 to 57,000 cfs.
1979 to 1983: 1 flood flow with a duration of 8 days; Q = 900 to 65,000 cfs.
1964 to 1983: 4 flood flows with a duration of 18 days; Q = 900 to 65,000 cfs.

A partial presentation of the results is shown in Table 4 for the 1979 to 1983 study period. This period has the most accurate geometric data.

Table 4. Agua Fria River: Actual change and the calculated change in bed elevation. Yang relationship.

| X Sect. | Actual Depth Change (Ft) | Calculated Depth Change (Ft) | % Error in Calculations | Depth Difference (Actual-Calculated) |
|---|---|---|---|---|
| 20.08 | 2.23 | -9.80 | 539.46 | -12.03 |
| 20.30 | 1.29 | -9.20 | 813.18 | -10.49 |
| 20.48 | 0.55 | -7.70 | 1500.00 | -8.25 |
| 20.65 | -0.15 | -7.60 | -4966.66 | -7.45 |
| 20.86 | 0.65 | -5.40 | 930.77 | -6.05 |
| 21.09 | 0.94 | -4.10 | 536.17 | -5.04 |
| 21.26 | 1.31 | -2.70 | 306.11 | -4.01 |
| 21.57 | -0.64 | -8.24 | -1187.50 | -7.60 |
| 21.87 | 0.05 | -2.10 | 4300.00 | -2.15 |
| 22.07 | -1.50 | -3.40 | -126.67 | -1.90 |
| 22.32 | -0.24 | -1.70 | -608.33 | -1.46 |
| 22.72 | -3.88 | -3.30 | 14.95 | 0.58 |
| 22.97 | -2.39 | -2.20 | 7.95 | 0.19 |
| 23.33 | -0.31 | -2.70 | -770.97 | -2.39 |
| 23.62 | -0.83 | -1.70 | -104.82 | -0.87 |
| 24.04 | -2.48 | -4.00 | -61.29 | -1.52 |
| 24.32 | -0.81 | -2.90 | -258.02 | -2.09 |
| 24.65 | -1.51 | -3.60 | -138.41 | -2.09 |
| 24.90 | -2.93 | -2.40 | 18.09 | 0.53 |
| 25.10 | -1.46 | -2.00 | -36.99 | -0.54 |
| 25.30 | -2.31 | -2.00 | 13.42 | 0.31 |
| 25.45 | -1.21 | -0.80 | 33.88 | 0.41 |
| 25.65 | -2.99 | -1.90 | 36.45 | 1.09 |
| 25.73 | -2.19 | -1.20 | 45.21 | 0.99 |
| 25.90 | -1.98 | -2.50 | -26.26 | -0.52 |
| 26.10 | -3.08 | -3.50 | -13.64 | -0.42 |
| 26.30 | -4.79 | -5.90 | -23.17 | -1.11 |
| 26.45 | -3.88 | -3.70 | 4.64 | 0.18 |
| 26.60 | -3.84 | -6.60 | -71.88 | -2.76 |

The percentage of the reach increments where Depth-Difference (DF) is:
a) $-1.0 < DF < 1.0$ : 37.93%
b) $-0.5 < DF < 0.5$ : 17.24%

Statistics of "Depth-Difference": Mean = - 2.63560
Standard Dev. = 3.55155

## CASE STUDY #3: SALT RIVER

The Salt River flows through metropolitan Phoenix, where the channel is usually dry. River flows are controlled by a series of dams with a watershed of more than 6,000 square miles. The 2.0 mile study reach channel is slightly braided and with overbanks occupied by industry, agriculture and housing developments. The upper 1.5 to 2.0 feet of the river bed material is composed primarily of sandy gravel and well rounded cobbles with a maximum particle diameter of 3 inches. However, there are locations where the surface material consists of fine to medium sand. A well developed armor layer exists throughout most of the channel with material sizes of 6 to 9 inches. $D_{50}$ = 39.0 mm and $D_{90}$ = 240.0 mm.

The 1977 to 1983 study period experienced 4 major flood flows of 82,000., 100,000., 79,000., and 118,000 cfs; with a total duration of 181 days. Computational stability tests were made for 10,000, 25,000, 75,000, 100,000 and 125,000 cfs.

Table 5. Salt River: Actual change and the calculated change in bed elevation. Shields relationship.

| X Sect. | Actual Depth Change (Ft) | Calculated Depth Change (Ft) | % Error in Calculation | Depth Difference (Actual-Calculated) |
|---|---|---|---|---|
| 9.00 | -4.45 | -1.60 | 64.04 | 2.85 |
| 9.20 | -3.78 | -1.00 | 73.54 | 2.78 |
| 9.40 | -2.62 | -0.20 | 92.37 | 2.42 |
| 9.60 | -2.00 | -0.10 | 95.00 | 1.90 |
| 9.80 | -1.10 | 0.00 | 100.00 | 1.10 |
| 9.99 | -3.44 | -0.80 | 76.74 | 2.64 |
| 10.18 | -2.08 | -0.60 | 71.15 | 1.48 |
| 10.38 | -3.89 | -0.20 | 94.86 | 3.69 |
| 10.57 | -3.28 | -0.30 | 90.85 | 2.98 |
| 10.78 | -1.45 | 0.00 | 100.00 | 1.45 |

The percentage of the reach increments where Depth-Difference (DF) is:
a) $-1.0 < DF < 1.0$ : 0.00%
b) $-0.5 < DF < 0.5$ : 0.00%

Statistics of "Depth-Difference": Mean = 2.32965
Standard Dev. = 0.82089

A partial presentation of the results is shown in Table 5.

## RESULTS

A summary of the analyses to determine the applicability of HEC-6 to quantify sediment transport in ephemeral stream channels is presented in Table 6. The results of the study unquestionably demonstrate that the assumptions, upon which HEC-6 was devised, must be rigorously observed, namely, well defined stream channel with stable banks, maximum bed material diameter less than 64 mm; and the discretized flood hydrographs has segments of fairly long, constant duration. The Rillito Creek study most closely adhered to the HEC-6 assumptions and gave the most satisfactory prediction of sediment transport quantities. The inability of HEC-6 to simulate the movement of bed material in the Salt River, with sediments greater than 64 mm diameter, is primarily due to the numerical techniques used in the equilibrium depth and armor formation/stability part of the program. The inferior results of the Agua Fria River study are, to a large degree, due to the inaccuracies in defining the geometric boundaries of the channel.

Table 6. Summary of study results

| Parameter | Study Reach Agua Fria River study period 1964-83 | 1964-79 | 1979-83 | Salt River Sed Data Set No. 1 | No. 2 | Rillito Creek | Ideal Conditions |
|---|---|---|---|---|---|---|---|
| Depth Dif. (DF) | | | | | | | |
| mean | 0.69 | 0.47 | -2.64 | 2.33 | 2.63 | -.09 | 0.0 |
| standard dev. | 3.05 | 1.58 | 3.55 | 0.82 | 1.06 | 0.55 | 0.0 |
| Volume Change Ratio (VCR) | | | | | | | |
| mean | 0.10 | -0.48 | -0.25 | 0.14 | 0.07 | 0.27 | 1.0 |
| standard dev. | 6.07 | 3.27 | 16.9 | 0.13 | 0.06 | 1.20 | 0.0 |
| The percentage of X-sections for which | | | | | | | |
| a) VCR > = 0.0 | 72 | 62 | 76 | 100 | 100 | 83 | 100 |
| b) .5 < VCR<2.0 | 24 | 24 | 48 | 0 | 0 | 56 | 100 |
| c)-1.0<DF<1.0 | 35 | 51 | 38 | 0 | 10 | 94 | 100 |
| d)-0.5<DF<0.5 | 17 | 24 | 17 | 0 | 0 | 61 | 100 |

PARAMETER DEFINITION:

Depth Difference (DF): Actual ave. bed elevation change minus computed ave. bed elevation change at a x-section.

Volume Change Ratio (VCR): Computed sediment volume change divided by the actual volume change at a x-section.

## ACKNOWLEDGEMENTS

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# DEVELOPMENT AND APPLICATION OF A SEDIMENTATION MODELING SYSTEM

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## ABSTRACT

A study was performed to predict the long-term effects of a channelization project on an alluvial stream. Impacts of the project within, upstream and downstream of, the project were identified. The study methodology incorporated use of the movable boundary model, HEC-6. Application of HEC-6 to this study required development and manipulation of a large amount of hydrologic data. This paper focuses on the suite of computer programs used to accomplish the study objectives.

## INTRODUCTION

The Arkansas River between Pueblo, Colorado and John Martin Dam, a distance of about 125 river miles, is an alluvial, sand bed river. It meanders between bluffs in a flood plain about one mile in width. During geologic time the downstream (eastern) portion of this reach has been migrating southward due to heavy sediment loads from northern tributaries. A local flood control project is being planned for the town of La Junta, which is in the downstream one-third of this reach. A study was undertaken to evaluate the future performance of various flood control alternatives with regard to channel stability, sediment movement, and project maintenance. The alternatives considered were various channel and levee configurations. Evaluations were based on long-term (100-year period) hydrologic scenarios.

The primary tool used in this study was the movable boundary mathematical model HEC-6 "Scour and Deposition in Rivers and Reservoirs." (U.S. Army Corps of Engineers, 1977) The hydrologic and sediment regimes of the study reach are complex due to four tributaries and eleven major irrigation diversions (Figure 1). This paper describes development of representative data sets, operation of the model, calibration and simulation strategies employed, interpretation of model results, and use of those results to improve project design.

A unique aspect of this study was the development and use of automated procedures for generating, storing and manipulating the large amount of flow data necessary to perform 100 years of continuous simulation. The linkages and data flow among various existing programs, yielding a sedimentation modeling system, are described.

## THE MODELING STRATEGY

Simulation of 100 years of sediment movement in an alluvial river requires the assemblage and use of a multitude of disparate data. Geometric (cross-section) information is needed to describe the present river, sediment data are needed to depict the character of the stream, and flow records are necessary to replicate the hydrology. The process of acquiring, sanitizing and merging these data leads to the notion of a modeling system. The system developed for this study is shown in Figure 2.

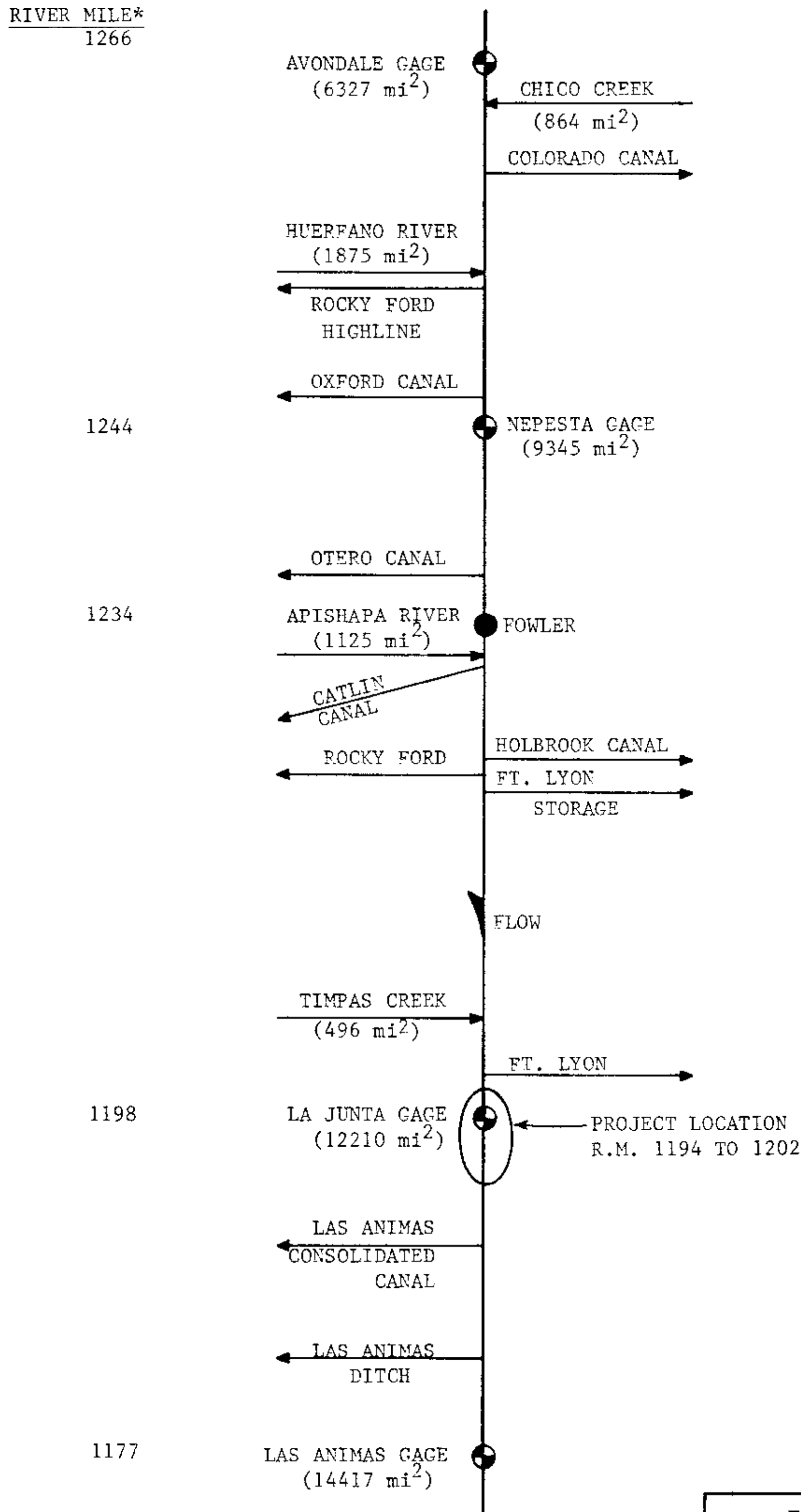


FIGURE 1
SCHEMATIC DIAGRAM
OF STUDY REACH

*Note: 1 mi = 1.6 km

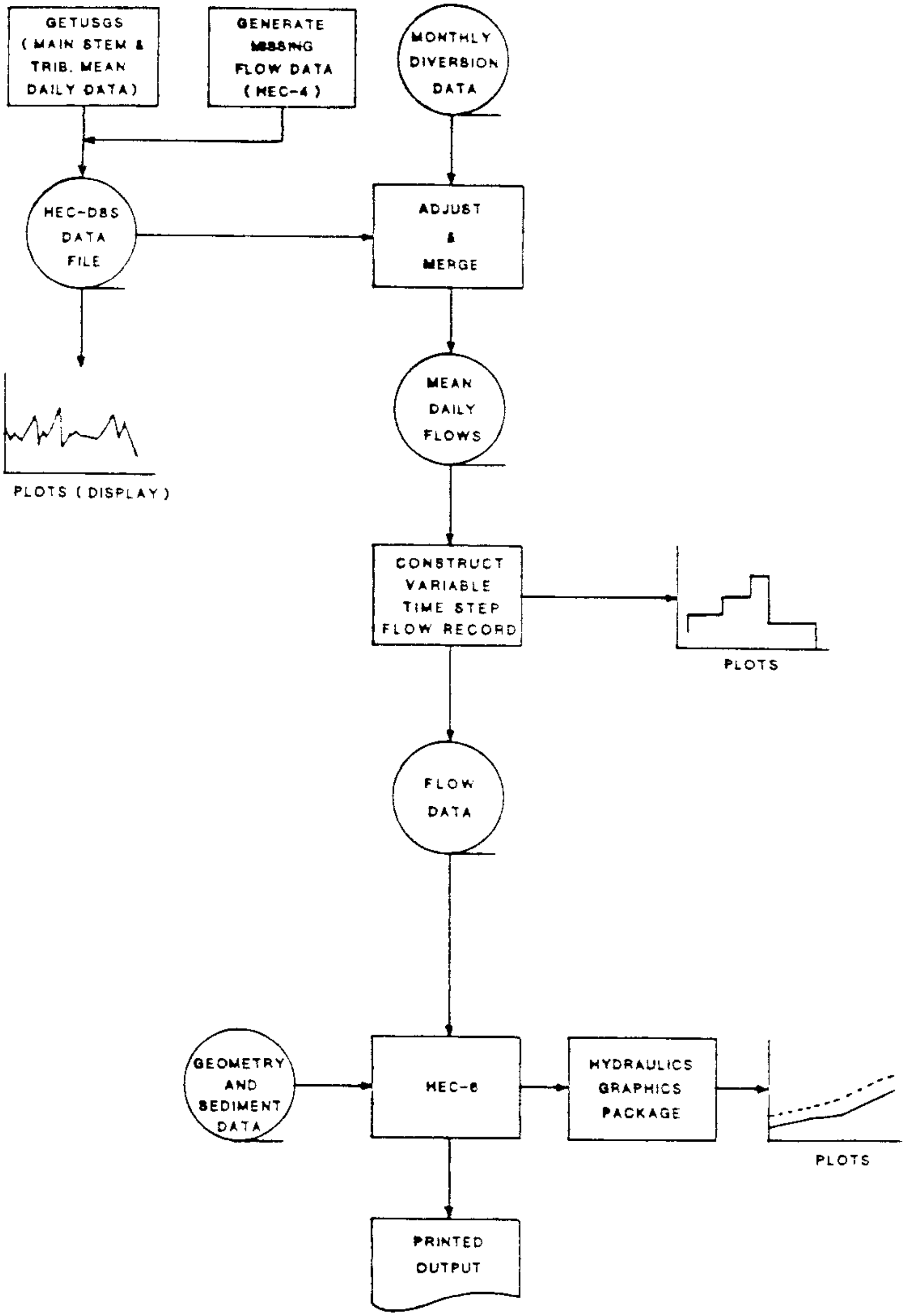


FIGURE 2

HEC-6 DATA FLOW AND PROGRAM LINKAGE

System components:

• GETUSGS

This is a utility routine developed by the Hydrologic Engineering Center (HEC) to access and manipulate WATSTORE (U.S. Geologic Survey, 1975) files which contain mean daily flow data for U.S. Geological Survey (USGS) streamflow gages.

• HEC-4 Monthly Streamflow Simulation (U.S. Army Corps of Engineers, 1971)

This program was used to interpolate streamflow data for those tributaries that did not have a complete gaged record. This was done by patterning the monthly flows (based on drainage area) on a nearby stream with a complete record (Gee, 1983).

• HEC-Data Storage System (HEC-DSS)

The HEC-DSS (U.S. Army Corps of Engineers, 1982) was used to assemble, plot and manage the daily flow records.

• Adjust and Merge

Data for the irrigation diversions were monthly. Furthermore, calculated volumetric flow balances indicated that substantial ungaged irrigation returns existed in the study reach (Gee, 1983). This required that hypothetical tributaries be inserted at each gage to maintain the flow balance. Utilizing the HEC-DSS files, this program assembled the basic daily flow record file for the main stem, four tributaries and the eleven irrigation diversions for the selected base period of 1941-1980.

• THISTOS

This program constructs a variable time step flow record for use by HEC-6. HEC-6 could operate with mean daily flows, however, computational efficiency requires longer time steps. THISTOS is designed to optimize the flow record time steps considering both accuracy and computational efficiency.

• HEC-6 Scour and Deposition in Rivers and Reservoirs (U.S. Army Corps of Engineers, 1977)

Simulates one-dimensional sediment transport, scour, and deposition in a river system which may have reservoirs. Steady-state water surface profiles are computed for each time step; at each cross section, discharge, inflowing sediment load, gradation of bed and transported sediment, and armoring are considered in computing scour or deposition.

• Hydraulics Graphics Package (U.S. Army Corps of Engineers, 1981D)

This software was developed for display and interpretation of results of HEC-2 (U.S. Army Corps of Engineers, 1981C). It has been expanded for use with GEDA (U.S. Army Corps of Engineers, 1981A) and HEC-6. Data that can be displayed include: cross sections; profiles of water surface and bed elevations; time history of bed elevation changes at any cross section; bed material gradations; and sediment transport rates.

## GEOMETRIC DATA

Application of HEC-6 to predict future trends in the behavior of the Arkansas River required development of a geometric description of the river. A study reach of about 90 river miles from Las Animas to Avondale was identified. Good flow records exist for both Las Animas and Avondale as well as at two intermediate locations: La Junta and Nepesta. Flow records were also available for all the irrigation diversions in this reach. The reach contains four significant tributaries and 11 irrigation diversions (Fig.1). The intervening drainage area between Las Animas and Avondale is 8090 $mi^2$ (20,950 $km^2$) of which 3730 $mi^2$ is (9660 $km^2$) ungaged.

The basic geometric information necessary for HEC-6 was developed from a set of river cross sections that cover the reach from Great Bend, Kansas, to Pueblo, Colorado. The sections are dated 1940 and 1945 with additional surveys in 1953 and 1977. The 1945 sections were selected for use because bed material size distribution information was available for 1946. All 1945-surveyed cross sections for the study reach were digitized by hand into HEC-6 input format. Where additional spatial resolution was necessary, supplemental data were obtained from the 1953 survey. In some locations it was necessary to repeat cross sections for additional resolution or to provide unique entry or exit points for all tributaries and diversions. The surveyed sections were biased towards constrictions (bridges, diversion dams); therefore, when necessary, valley sections were used as repeated sections. The resulting data set contained 41 cross sections.

## PROJECT CHANNEL

The channel project being investigated directly affects about 8 miles (13 km) of the river in the vicinity of La Junta. The project condition was simulated by replacing the surveyed cross sections with those representing the proposed project design within this reach. The anticipated shortening of the river was captured through the new channel and overbank lengths. Thus, the interaction of the project channel with both the upstream and downstream reaches of the river was simulated.

## SEDIMENT DATA

Bed material gradations for the study reach were obtained from 1946 surveys performed in conjunction with the construction of John Martin Dam (U. S. Army Corps of Engineers, 1968A & B). It was decided to use a single, average gradation for the entire study reach. This is the initial condition gradation and is updated continuously by the HEC-6 sediment sorting algorithm during the simulation. The average gradation curve was broken into 10 size classifications ranging from silt to coarse gravel.

Measurements of instantaneous transport rates and size distributions were not available at the main stem gages and could not be obtained within the scope of this study. Based on field inspection, it was decided to develop an inflowing load that is in equilibrium with the transport capacity of the upstream end of the reach. This was accomplished by operating HEC-6 for a range of discharges, each with very short duration (on the order of seconds) so that insignificant changes to the bed material size distribution or bed elevations would take place. An inflowing load curve with zero sediment load was then input to HEC-6. The calculated load passing each section (by grain size fraction) is equivalent to the equilibrium transport rate for those conditions. These calculated loads for the upstream-most sections were used to define the load curve.

Transport capacity was calculated using the Toffaleti procedure (Vanoni, 1975). The Toffaleti method was chosen because this reach of the Arkansas is similar to sand bed streams for which the Toffaleti method has worked well in past studies. The model performed satisfactorily with this method; therefore, no others were tried. As the Toffaleti function computes bed material load for sands and coarser grain sizes, wash load must be estimated from suspended load measurements. The wash load component was included in the inflowing load for completeness, but has an insignificant impact on channel behavior for this particular situation.

Four major tributaries occur within the study reach: Timpas Creek, Apishapa River, Huerfano River, and Chico Creek. Although instantaneous sediment transport measurements were made on these tributaries in conjunction with early John Martin sedimentation surveys, the original data are no longer available. Monthly and/or yearly averages of those data were available for use in the study. Those averaged data were used to estimate "instantaneous" relationships between water discharge and sediment load for the tributaries. These load curves, while yielding correct average volumes of sediment, do not reflect the true relationship between instantaneous water and sediment discharges. Grain size distributions for the tributary suspended loads were available (U.S. Army Corps of Engineers, 1953). These distributions were applied directly to the total load curves to yield loads for the individual grain sizes.

The irrigation diversions divert sediment as well as water; although attempts are made to minimize the volume of sediment diverted, particularly of the coarser size fractions. No data were available on the relationship between diverted water and diverted sediment. HEC-6 does not use a load curve to determine quantities of sediment diverted; rather a relation between concentration of sediment in the diverted water and in the main stem is used. For this modeling effort, it was assumed that all the diversions divert silt at a concentration equal to the ambient silt concentration in the main stem. The concentration of all sand fractions diverted was set at 75% of the corresponding main stem concentration based on historical records (U.S. Army Corps of Engineers, 1953). It was assumed that none of the main stem gravels would be diverted. This capability for realistically simulating diversions with HEC-6 was a key factor in establishing credibility of this model application.

## HYDROLOGIC DATA

To simulate the behavior of a stream for a 100-year period with a movable boundary model such as HEC-6, one needs a continuous flow record for a 100-year period. Typically, flow data are obtained as mean daily discharges and then aggregated into longer-period, variable time steps to minimize the computational effort. As continuous flow records rarely exist for 100 years or more, the modeler must assemble, construct, or synthesize the record somehow. Development of appropriate long-term flow sequences for sediment routing is an important research topic yet to be fully addressed. The procedure described below is reasonable and has been used on previous studies. Development of the flow record proved to be the most difficult and time consuming aspect of this study.

Daily stream flow data were obtained for eight gages in the study reach from USGS records. Monthly flow volumes for the eleven diversion structures within the study reach were obtained from the State of Colorado.

A base time period of 1941 to 1980 had the most overlapping and continuous daily data for all the necessary gages. After development of the HEC-6 flow record as described below, that record was repeated 2.5 times to produce a 100-year flow record.

Missing tributary records were interpolated on a monthly basis using HEC-4. The interpolated monthly values were then converted to daily flow values. This was accomplished by taking the interpolated missing monthly volume and dividing it by the observed monthly volume of a nearby tributary stream gage. This produced a ratio which was used to multiply the nearby gages' daily flow values. Tributary flows were adjusted to account for ungaged and non-contributing areas. This was necessary to provide an estimate of sediment being delivered from adjacent ungaged areas (through the tributary load curve).

A daily flow balance at the mainstem gages was performed which indicated significant errors between flow volumes calculated from the tributary and diversion data and the observed volumes at the gages, probably due to ungaged irrigation return flows. It was necessary to remove the error in the water balance; reliable long-term sediment accounting requires accurate water accounting. The procedure used is described in (Gee, 1983).

The final adjusted daily flow data were then processed by the HEC-6 data pre-processor (THISTOS) to generate a sequence of flows of varying time step to optimize computational efficiency and accuracy. The final computational time steps ranged from one day to one month.

## MODEL PERFORMANCE AND CALIBRATION

Evaluation of the performance of HEC-6 consists of a comparison of trends in scour and/or deposition between model simulations and historical stream behavior. It is known that the study reach has been aggrading during recent years. According to gage ratings obtained from the U.S. Geological Survey, the rating curve for the Las Animas gage shifted upwards by about 5 feet (1.5 m) between 1955 and 1966. The model simulations adequately reproduce the aggradation trends. The calculated patterns of deposition also correspond favorably with field observations; deposits appear downstream of tributaries and major irrigation diversions. The observed changes in the stage-discharge relationship at the La Junta gage for the 25-year period, 1951-1979 were used to calibrate HEC-6. This rating shift was due to aggradation rather than roughness changes. Use of historical rating curves is a recommended technique for calibrating HEC-6; rating curves integrate to a certain extent behavior of a stream reach rather than reflecting only local changes (U.S. Army Corps of Engineers, 1981B; Gee, 1984). The calibration results are shown on Figure 3.

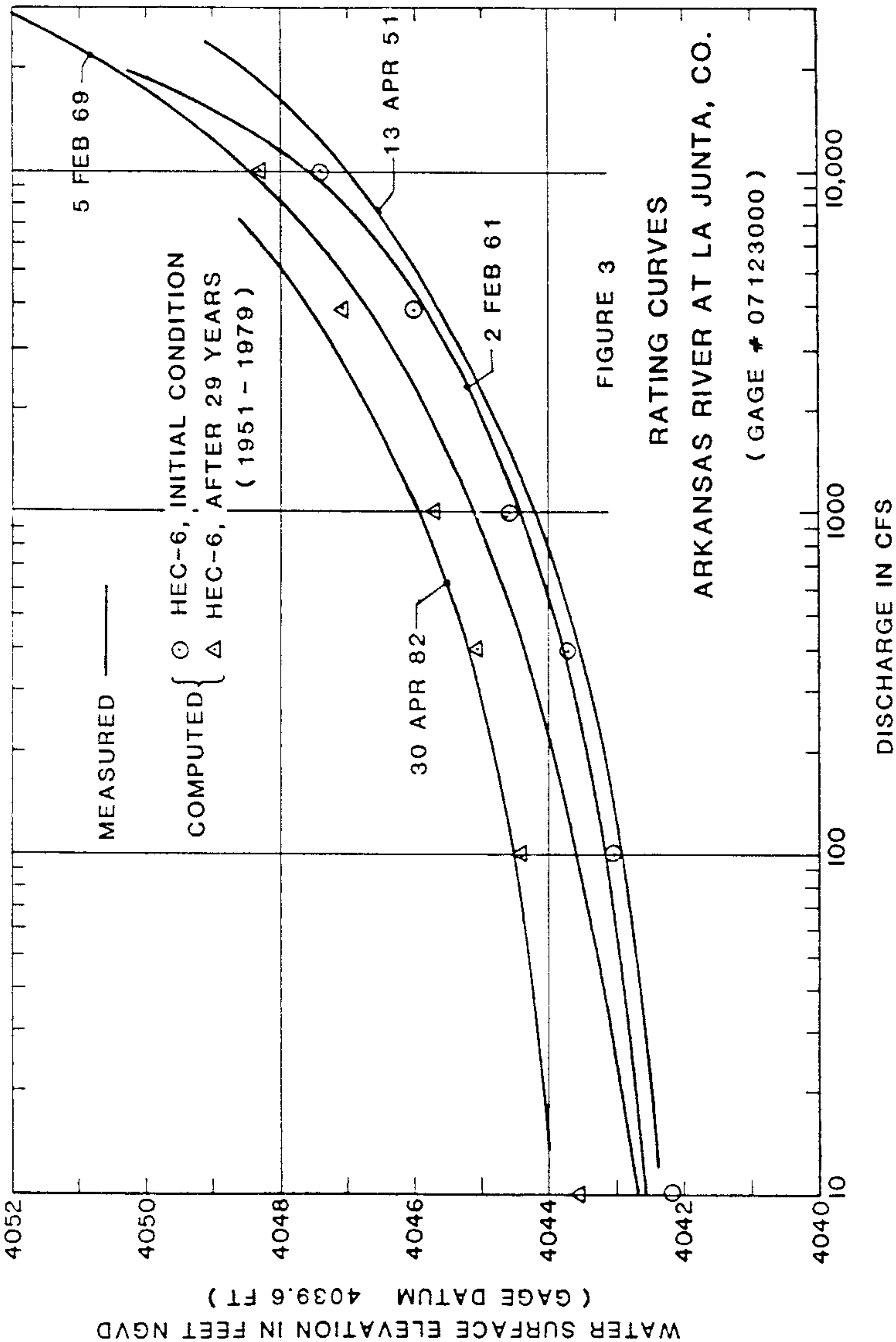


FIGURE 3
RATING CURVES
ARKANSAS RIVER AT LA JUNTA, CO.
( GAGE # 07123000 )

## COMPUTATIONAL ASPECTS

This study required utilization of major computational resources. The operation of HEC-6 to simulate a 100-year long period was the primary element. However, developing, manipulating, and storing the 100 years of daily flow records for four main stem gages, four tributary gages, and eleven diversions was a significant data handling effort in its own right. This study utilized software developed at the HEC for hydrologic data storage and manipulation (U.S. Army Corps of Engineers, 1982), and graphical analysis of data and simulation results (U.S. Army Corps of Engineers, 1981D). The linkage of these various software packages and data files used in this study is shown on Figure 2. This support software has become an integral and necessary component of any major movable boundary modeling effort at the HEC.

The simulations were performed using HEC's Harris 500 minicomputer. The continuous simulations of 100 years of streamflow with time steps ranging from one day to one month (average of one week) took several hours of central processor time. The application of predictive numerical models such as HEC-6 to project impact analyses is a valuable component of professional engineering. Planning of such a study should recognize the following points:

- The support software (pre- and post-processors) is a vital ingredient. Therefore, it must be available, operational on the computer system, and the linkages between programs efficient.

- Data storage, manipulation, and presentation are critical. In the study reported here, the hydrologic data (flow record) was by far the largest and most difficult to assemble data component.

- Execution times/throughput. Many executions of the model are required to sanitize the data, calibrate, and study alternatives; selection of machine and consideration of run scheduling should be addressed in the study plan.

- Graphics. Software and hardware compatibility must be ensured. Both screen (for quick reviewing of results) and hard copy (for results presentation) media must be available.

- Role of Microcomputers. Operation of HEC-6 on microcomputers is not being considered at this time. Use of microcomputers to assemble data, communicate with a host computer, display results, and manage the study effort is viable. Certain sediment transport/channel stability utility routines can be used on microcomputers to check and guide a comprehensive modeling effort (Reese & Ingram, 1985).

## ACKNOWLEDGEMENTS

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# MODELLING SEDIMENT YIELDS INTO STAMFORD LAKE, TEXAS

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## ABSTRACT

The Soil Conservation Service (SCS) completed sediment surveys on Stamford Lake in May 1966 and September 1982. Stamford Lake watershed is a 932 $km^2$ rural watershed in Haskell, Jones, and Stonewall Counties.

Two computer models, HYMO (HYdrologic MOdel) and SWRRB (Simulator for Water Resources in Rural Basins), were utilized to estimate average annual sediment rates and total volumes of deposition from May 1966 to September 1982.

Measured annual sediment yield was 6.1 t/ha, compared to 5.1 t/ha and 5.3 t/ha simulated by HYMO and SWRRB, respectively. An unusually intense storm (greater than the 100-year storm) occurred on the Stamford Lake watershed on August 3-4, 1978. Simulation results show 21-26% of the sediment deposited in the reservoir from 1966 to 1982 occurred during the August 3-4, 1978, storm.

## INTRODUCTION

Stamford Lake is about 23 km northeast of the city of Stamford in Haskell County, Texas. The impoundment is on Paint Creek approximately 21 km upstream from the confluence with the Clear Fork of the Brazos River. The reservoir and dam are owned by the City of Stamford. Water from the reservoir is used for municipal, domestic, and recreational purposes, and generation of electric power. Construction of the dam and its appurtenances was completed in March 1953.

During May 1966, the Soil Conservation Service (SCS) surveyed the lake to estimate volume, deposition rate, locations, and characteristics of sediment accumulated during the previous 12.9 years. The survey was repeated during August and September 1982. Of particular interest is an unusually intense storm that occurred on the Stamford Lake watershed on August 3-4, 1978, producing from 102 to 560 mm of rainfall.

Subsequently, the Agricultural Research Service (ARS) has utilized computer models, Simulator for Water Resources in Rural Basins (SWRRB) and Hydrologic Model (HYMO), to estimate average annual sediment yields for the 16.3 years from 1966 to 1982. The simulated sediment yields were then compared with measured yields from the sedimentation surveys. Both models were also used to estimate sediment yields from the August 3-4, 1978, storm.

## WATERSHED AND RESERVOIR DESCRIPTION

The drainage area of the watershed, including the surface area of Stamford Lake at spillway elevation 431 m m.s.l., is 932 $km^2$. Paint Creek and its tributaries upstream from the reservoir drain 767 $km^2$ in southern Haskell County, 144 $km^2$ in extreme northern Jones County, and 21 $km^2$ in southeastern Stonewall County.

Maximum elevation of 552 m m.s.l. is on the watershed boundary in Stonewall County. The minimum elevation is 415 m m.s.l in the stream channel at the back side of the dam.

The land surface is characterized by relatively level interstream areas and by well-defined stream channels. Flood plain areas are broad and extend well into the western upstream portions of the watershed. The average regional slope of the area is toward the east about 0.0009 to 0.0011 m/m.

Average annual rainfall for the watershed area is 588 mm as determined from a 68-year record. However, annual rainfall for the sedimentation survey period, 1967 to 1982, ranged from 392 mm to 845 mm, averaging 654 mm. During the August 3-4, 1978, storm, 102 to 560 mm of rain fell within 24 hours. Rainfall of 184 mm in 24 hours, adjusted for 932 $km^2$ of drainage area, is a one percent chance storm on the watershed. The larger amounts of precipitation fell on the eastern and northeastern portions of the watershed near the lake.

Soils in the watershed have developed over Permian shale (red-beds) and unconsolidated Quaternary gravel, sand, silt, clay, and caliche. The entire reservoir drainage area is within the Rolling Red Plains Land Resource Area. The principal soil mapping units in the watershed are: Sagerton-Rowena and Tillman-Stamford clay loams and clays on nearly level to gently undulating upland; Vernon clays and clay loams on undulating to erosional upland with gradients up to 15%; and Miller-Norwood silty clay loams in the bottomland areas.

Land use in the watershed is cropland, 559 $km^2$; rangeland, 330 $km^2$; urban, 13 $km^2$; Stamford Lake, 19 $km^2$; and miscellaneous area, 11 $km^2$.

The local soil and water conservation districts are administering programs to control soil erosion and conserve water. Typical erosion suppression and soil and water conservation measures utilized on agricultural land are conservation cropping systems, crop residue use, strip cropping, grassed waterways, and parallel terraces on cropland and brush management, proper grazing use, and deferred grazing on rangeland.

Surface area of the lake at spillway elevation 431 m m.s.l. is 19.0 $km^2$. The lowest water surface elevation of 428 m m.s.l. since the reservoir was filled was recorded on August 3, 1978. As a result of the August 3-4, 1978, storm, the highest water surface elevation of 434 m m.s.l. was recorded August 5, 1978. Length of the main body of the reservoir is 12.9 km. It has two arms of significance on its northwestern side, one being 3.9 km and the other 2.6 km in length.

The original 1953 storage capacity of the reservoir was $7.11 \times 10^7$ $m^3$ at 431 m m.s.l. The 1966 sedimentation survey indicated the capacity had been reduced to $6.66 \times 10^7$ $m^3$, and the 1982 sedimentation survey revealed a capacity of $5.39 \times 10^7$ $m^3$.

## HYMO

When flood routing is performed, a watershed is divided into many small areas according to its hydraulic characteristics. The hydrographs from these areas must be estimated, since streamflow measurements are unavailable. A procedure for computing unit hydrographs was developed by Williams (1968), and modified for use in HYMO (Williams and Hann, 1972). Unit hydrographs are divided into three parts for computation. From the beginning of rise to the inflection point, the hydrograph is computed by the two-parameter gamma distribution. From the inflection point to infinity, the hydrograph is broken into two parts and both are computed with exponential functions. The parameters in the distributions are estimated as functions of the volume of runoff, peak rate, drainage area, relief-length ratio, and length-width ratio.

Storm hydrographs are computed by convolving unit hydrographs with incremental source runoff. To compute incremental source runoff, the mass rainfall curve is broken into equal time increments, and the Green and Ampt infiltration equation is applied (Green and Ampt, 1911).

The variable travel time (VTT) flood routing method is used in HYMO (Williams, 1975). The VTT method was developed to account for the variation in travel time with stage and water-surface slope. The VTT method is suited for HYMO because it requires little computer storage, routes both channel and floodplain flows, perfoms reliably under widely varied conditions, and gives consistent results over a wide range of routing intervals and reach lengths.

Sediment yield is computed for each sub-basin with the Modified Universal Soil Loss Equation (MUSLE) (Williams and Berndt, 1977).

$$Y = 11.8\,(V\,q_p)^{0.56}\,(K)\,(C)\,(PE)\,(LS) \qquad (1)$$

where Y is the sediment yield from the sub-basin in t, V is the surface runoff volume in $m^3$, $q_p$ is the peak flow rate in $m^3/s$, K is the soil erodibility factor, C is the crop management factor, PE is the erosion control practice factor, and LS is the slope length and steepness factor.

HYMO was linked with a sediment routing model (Williams, 1978) to form a comprehensive model capable of flood routing and sediment routing. The sediment routing model is composed of two components for estimating the deposition and degradation processes associated with flow through channels and floodplains.

The deposition equation for application to individual routing reaches is

$$DEP = YI\left(1 - \sum_{i=1}^{M} \omega_i\, e^{-\beta\sqrt{d_i}}\right) \qquad (2)$$

where DEP is the amount of sediment deposited within the reach in t, YI is the inflow sediment yield in t, $\omega_i$ is the portion of particle size $d_i$ (µm) contained in the distribution, β is the routing coefficient, and M is the number of particle sizes used to define the particle-size distribution.

The equation to estimate the sediment routing coefficient is

$$\beta = -\ln \frac{\dfrac{(Vq_p)_O^{0.56} - (Vq_p)_{SB}^{0.56}}{(Vq_p)_I^{0.56}}}{4.47} \qquad (3)$$

where the subscripts O, SB, and I refer to runoff energy factors for outflow, sub-basin, and inflow.

The stream power concept was used by Williams (1978) as the basis of the degradation model. Sediment particles that are deposited may be reentrained by the degradation component if sufficient stream power is available. If the stream power exceeds that required to transport the inflow sediment, degradation will occur. The equation to determine the total amount of sediment reentrained ($DEG_R$) is

$$DEG_R = \alpha \gamma^{1.5} \Delta_x \sum_{x=0}^{x_f} (D_x S_x U_x)^{1.5} \qquad (4)$$

where $\alpha$ is a parameter dependent on maximum stream power, $\gamma$ is the density of the water in $t/m^3$, $x$ is time, $x_f$ is final time, D is the flow depth in m, S is the water surface slope in m/m, and U is the flow velocity in m/s. If $DEG_R$ > DEP, the excessive stream power is used for degradation, predicted with the equation

$$DEG_B = KC\,(DEG_R - DEP) \qquad (5)$$

where $DEG_B$ is the amount of degradation of the bed material, and K and C are MUSLE factors that express soil erodibility and vegetative cover. Finally, total degradation (DEG) is the sum of the reentrainment and bed degradation components.

$$DEG = DEG_R + DEG_B \qquad (6)$$

SWRRB

A model called SWRRB (Simulator for Water Resources in Rural Basins) was developed for simulating hydrologic and related processes in rural basins. The SWRRB model was developed by modifying the CREAMS daily rainfall hydrology model (Knisel, 1980; Williams and Nicks, 1982) for application to large, complex, rural basins. The major changes involved were (a) the model was expanded to allow simultaneous computations on several sub-basins; and (b) components were added to simulate weather, return flow, pond and reservoir storage, crop growth, transmission losses, and sediment movement through ponds, reservoirs, streams, and valleys.

The objective in SWRRB model development was to predict the effect of management decisions on water and sediment yields with reasonable accuracy for ungaged rural basins throughout the U.S. Tests on 11 widely varying watersheds throughout the U.S. have been completed to validate the model (Arnold et al., 1985).

Surface runoff volume is predicted using the SCS curve number (USDA, 1972) as a function of daily soil moisture content. Peak runoff rate predictions are based on a modification of the Rational Formula (Williams et al., 1985). Sediment yield is computed for each sub-basin with the Modified Universal Soil Loss Equation (MUSLE) (Williams and Berndt, 1977). The channel and floodplain sediment routing model is composed of two components operating simultaneously (deposition and degradation). The deposition equation is

$$c_{DP} = c_I \exp(-k_d \, FT \sqrt{D50}) \tag{7}$$

where $c_{DP}$ is the concentration of sediment after deposition between sub-basin outlet and the basin outlet, $c_I$ is sub-basin outflow sediment concentration, $k_d$ is the deposition constant (0.01).

The degradation equation is

$$c_{DG} = (c_u - c_I)(1 - \exp(-k_e \, FT \, q_p / \sqrt{D50})) \tag{8}$$

where $c_{DG}$ is the concentration of sediment after reentrainment of deposited sediment plus channel and floodplain degradation, $c_u$ is the upper limit of sediment concentration in flow ((0.25 t/m$^3$), $k_e$ is the degradation constant ((2.788 X 10$^{-4}$), and $q_p$ is the sub-basin peak flow rate expressed in mm/h. Sediment is also routed through pond and reservoirs.

## PROCEDURE

### HYMO

The watershed was divided in 28 sub-basins for HYMO (Fig. 1). The subdivisions were made on the basis of hydraulic characteristics, reach length, and sub-basin size. The size of the sub-basins range from 5.7 to 79.4 km$^2$ (Table 1). Channel cross section data were obtained and rating curves were developed by HYMO. MUSLE factors were determined from topographic maps, land use and soils data. Table 1 shows the MUSLE factors used in HYMO. Twenty-four hour duration rainfall for 1, 2, 5, 10, 25, 50, and 100-year return period storms was used to drive HYMO (USDC, 1961). Runoff and sediment yields were estimated by HYMO for the seven storms. Average annual runoff and sediment yields were computed by integrating the frequency curves.

Table 1. Range of sub-basin USLE factors for HYMO.

| Sub-basin | Drainage area | K | LS | C | | P |
|---|---|---|---|---|---|---|
| | | | | Ave. ann. | 78 storm | |
| | (km$^2$) | | | | | |
| 1-28 | 5.7-79.4 | .32-.36 | .20-.26 | .12-.36 | .15-.44 | .62-.92 |

SWRRB

The watershed was divided into 4 sub-basins for SWRRB (Fig. 2). The divisions were based on soils, topography, and rain gage locations. Tillman soil series is used for sub-basins 1, 2, and 3, and Vernon soil series is used for sub-basin 4. Daily rainfall (USDC, 1966-1982) was taken from Hamlin for sub-basin 1, Stamford for sub-basins 2 and 4, and Haskell for sub-basin 3. Hamlin is approximately 3.2 km east of the southwest corner of the watershed and Stamford and Haskell are shown in Figures 1 and 2. All soils information was obtained from the SCS Soils-5 data base.

The Vernon soil, predominant in sub-basin 4, is very slowly permeable and has a relatively low water-holding capacity. Slopes up to 15% are found on the Vernon soil, which accounts for the high LS factor. The Tillman soil, predominant in sub-basins 1, 2, and 3, is more permeable and has a higher water-holding capacity than Vernon. Consequently, surface runoff is lower for the Tillman soil.

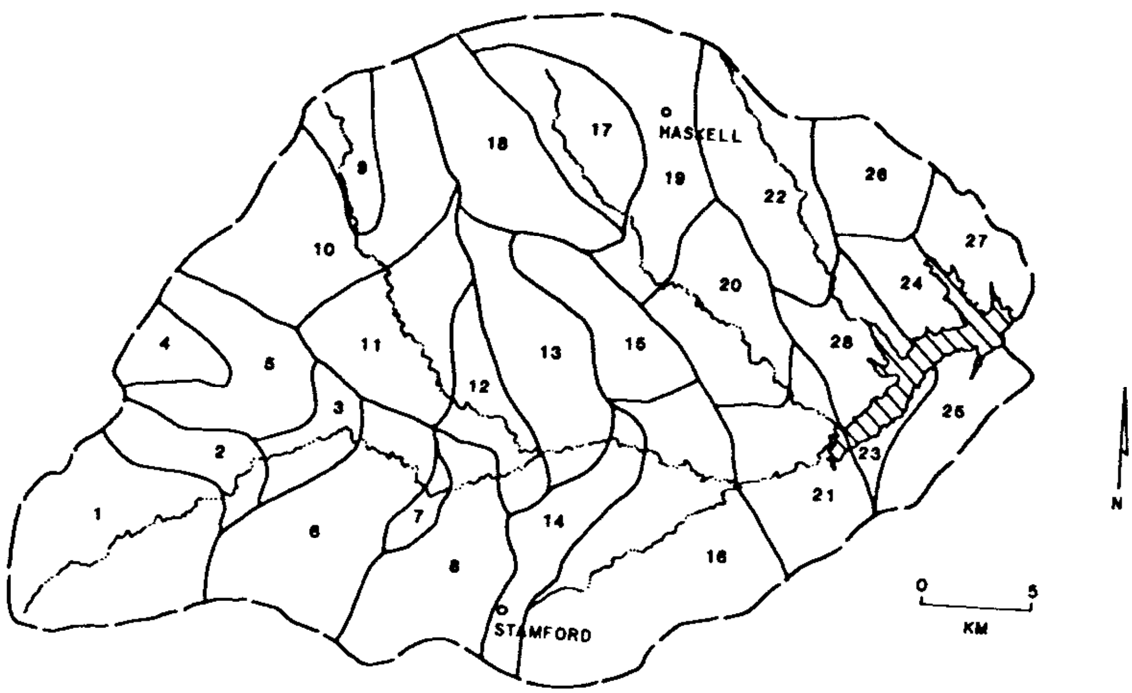


Figure 1. Sub-basin divisions for HYMO.

Table 2. Sub-basin USLE factors for SWRRB.

| Sub-basin | Drainage area | K | LS | C<br>Ave. ann. | P |
|---|---|---|---|---|---|
| | ($km^2$) | | | | |
| 1 | 186 | .32 | .22 | .26 | .82 |
| 2 | 344 | .32 | .22 | .27 | .83 |
| 3 | 262 | .32 | .22 | .27 | .79 |
| 4 | 140 | .32 | .21 | .28 | .82 |
| Basin | 932 | .32 | .22 | .27 | .82 |

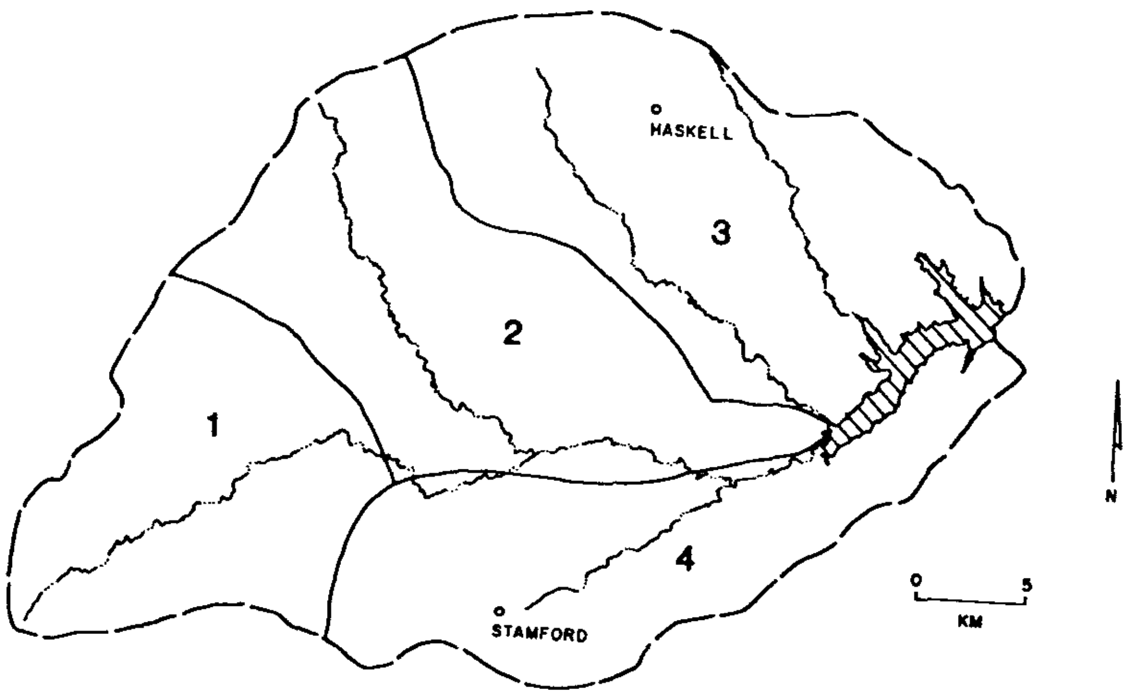


Figure 2. Sub-basin divisions for SWRRB.

## RESULTS

### Average Annual

The SCS completed sedimentation surveys on Stamford Lake in May 1966 and September 1982. The measured average annual erosion sediment yield is 6.1 t/ha. Standard SCS field estimates (USDA, 1983) show that sheet-rill erosion contributes 92.6%; ephemeral gully erosion, 3.9%; perennial gully erosion, 0.8%; steambank erosion, 2.1%; and roadside erosion, 0.6%. Sheet-rill erosion is estimated with the USLE in the standard SCS field estimate. Weighted average annual erosion rates (SCS field estimates) by land use are: cropland, 8.5 t/ha; rangeland, 3.9 t/ha; urban areas, 2.9 t/ha; and miscellaneous areas, 1.2 t/ha (Bircket, 1985).

Return period storms from 1 to 100 years were selected to drive HYMO (USDC, 1961). The rainfall and resulting runoff and sediment yields are shown in Table 3. Average annual surface runoff and sediment yields, computed by integrating the frequency curves, are shown in Table 4. Both the HYMO and SWRRB sediment yields have been adjusted for gully, streambank, and roadside erosion and reservoir deposition. The reservoir trap efficiency was assumed to be 96% (Brune, 1953).

Annual delivery ratios were estimated by dividing the simulated sediment yields by the SCS field estimates. The R value for the Stamford area is 170 (Wischmeier and Smith, 1978). SCS field estimates and delivery ratios are shown in Table 4.

Table 3. TP-40 rainfall and HYMO simulated runoff and sediment yield.

| Return period | TP-40 Rainfall[1/] | Surface runoff | Sediment yield |
|---|---|---|---|
| | -------------(mm)----------- | | ----(t/ha)---- |
| 1 | 66 | 25 | 2.3 |
| 2 | 94 | 46 | 4.4 |
| 5 | 119 | 68 | 6.4 |
| 10 | 137 | 84 | 8.3 |
| 25 | 163 | 107 | 10.3 |
| 50 | 185 | 127 | 13.0 |
| 100 | 211 | 152 | 15.4 |

1/ USDC (1961).

Table 4. Measured and simulated average annual runoff and sediment yield.

| | Ave. annual surface runoff | Ave. annual sediment yield | SCS field estimate | Delivery ratio |
|---|---|---|---|---|
| | (mm) | -------------(t/ha)----------- | | |
| Measured | | 6.1 | | |
| HYMO | 51 | 5.1[1/] | 5.8 | 0.88 |
| SWRRB | 55 | 5.3[1/] | 5.8 | 0.91 |

1/Sediment yields have been adjusted for gully erosion and reservoir deposition.

August 3-4, 1978, Storm

On August 3-4, 1978, rainfall occurred on the Stamford Lake watershed that exceeded the 100-year return period storm. Figure 3 shows an isohyetal map for the storm. Table 5 shows area, rainfall, peak rates, and sediment yields estimated by HYMO for the storm. Values shown are for the sub-basin outlet and include the entire drainage area above the outlet.

Results from SWRRB are shown in Table 6. Rainfall, runoff, and sediment yields are given. Comparison of simulated runoff and sediment yields for the storm are shown in Table 7. In general, the results from the models are in close agreement. HYMO and SWRRB simulations estimate 21% and 26%, respectively, of the total sediment deposited into the lake from 1966 to 1982 occurred during the August 3-4, 1978, storm. Also, SWRRB simulations show that only six events accounted for 44% of the sediment deposited in the lake from 1966-1982.

Table 5. HYMO results from August 3-4, 1978, storm.

| Sub-basin outlet | Area | Rain-fall | Peak rate | Sediment yield |
|---|---|---|---|---|
| | ($km^2$) | (mm) | ($m^3/s$) | (t/ha) |
| 1 | 64 | 127 | 130 | 2.9 |
| 2 | 83 | 139 | 190 | 5.4 |
| 3 | 145 | 164 | 451 | 8.4 |
| 4 | 10 | 178 | 70 | 6.1 |
| 5 | 45 | 197 | 251 | 9.2 |
| 6 | 207 | 176 | 624 | 12.8 |
| 7 | 212 | 178 | 639 | 15.3 |
| 8 | 256 | 191 | 805 | 16.8 |
| 9 | 15 | 254 | 110 | 3.7 |
| 10 | 80 | 254 | 506 | 6.2 |
| 11 | 127 | 254 | 630 | 8.3 |
| 12 | 407 | 216 | 1504 | 18.1 |
| 13 | 451 | 224 | 1658 | 20.4 |
| 14 | 478 | 229 | 1720 | 20.0 |
| 15 | 30 | 356 | 260 | 5.5 |
| 16 | 589 | 253 | 2047 | 21.7 |
| 17 | 33 | 330 | 255 | 4.9 |
| 18 | 43 | 305 | 279 | 4.4 |
| 19 | 130 | 341 | 842 | 11.1 |
| 20 | 171 | 375 | 1102 | 13.9 |
| 21 | 797 | 285 | 2813 | 22.0 |
| 22 | 44 | 483 | 453 | 7.7 |
| 23 | 6 | 381 | 115 | 7.1 |
| 24 | 13 | 533 | 341 | 12.0 |
| 25 | 21 | 381 | 228 | 6.2 |
| 26 | 12 | 533 | 200 | 9.3 |
| 27 | 28 | 533 | 397 | 17.4 |
| 28 | 66 | 491 | 491 | 16.8 |

Table 6. SWRRB results from August 3-4, 1978, storm.

| Sub-basin | Area | Rainfall | Runoff | Sediment Yield |
|---|---|---|---|---|
| | ($km^2$) | -------(mm)------ | | (t/ha) |
| 1 | 186 | 152 | 39 | 5.2 |
| 2 | 344 | 279 | 135 | 18.8 |
| 3 | 262 | 406 | 227 | 31.8 |
| 4 | 140 | 406 | 243 | 36.4 |
| Basin | 932 | 315 | 188 | 22.4 |

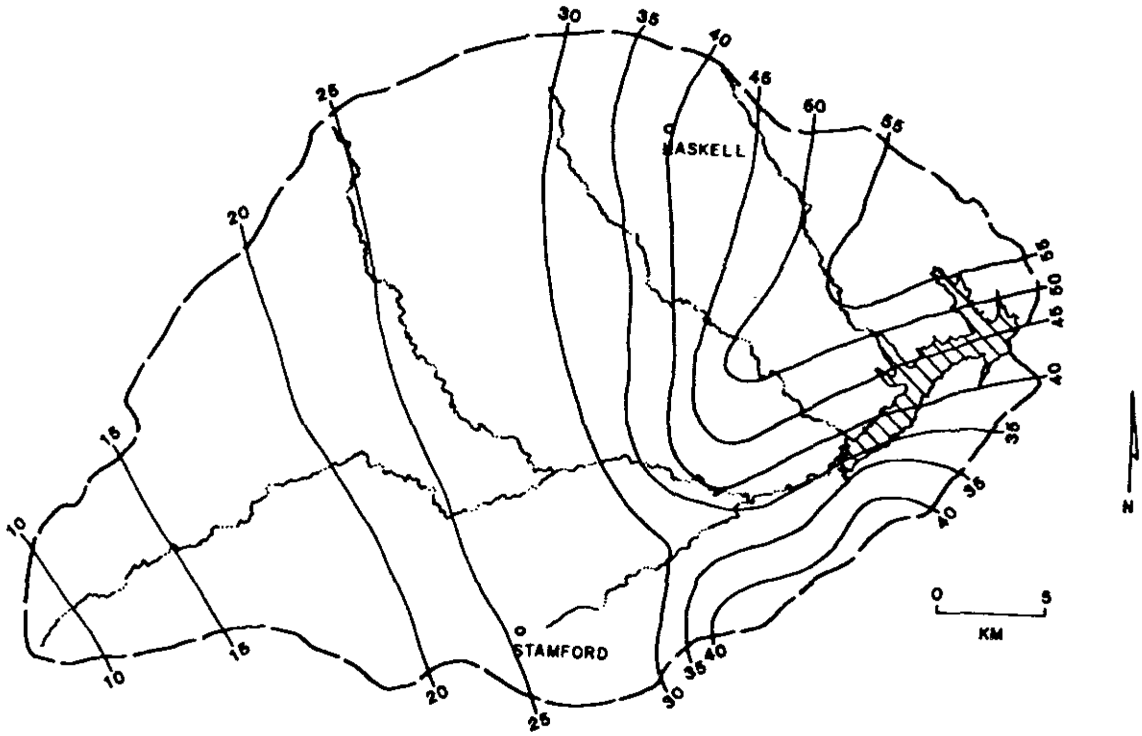


Figure 3. Isohyetal map for the storm of August 3-4, 1978, on Stamford Lake watershed (centimeters).

Table 7. Comparison of HYMO and SWRRB results from August 3-4, 1978, storm.

| | Surface runoff | Peak rate | Sediment yield | Total sediment | Concentration | Percent of total sediment yield |
|---|---|---|---|---|---|---|
| | (mm) | ($m^3/s$) | (t/ha) | (t) | (PPM) | (1966-1982) |
| HYMO | 215 | 2,813 | 17.9 | $1.67 \times 10^6$ | 8,300 | 21 |
| SWRRB | 188 | 3,655 | 22.4 | $2.08 \times 10^6$ | 11,900 | 26 |

## SUMMARY AND CONCLUSIONS

The Soil Conservation Service (SCS) surveyed Stamford Lake in May 1966 and September 1982. The 930 $km^2$ rural watershed is located in Haskell, Jones, and Stonewall Counties.

A continuous simulation model, SWRRB, and an event model, HYMO, were used to estimate sediment yields from the watershed. SWRRB predicts water and sediment yields from large, ungaged rural basins on a daily basis. Major processes included in SWRRB are surface runoff, percolation, return flow, evapotranspiration, pond and reservoir storage, and sedimentation. Sediment yields are predicted for each sub-basin with MUSLE and routed with a sediment routing model. HYMO also uses MUSLE to estimate sub-basin sediment yields while a more sophisticated sediment routing model is used. Rainfall data is taken from 1, 2, 5, 10, 25, 50, and 100-year return period storms to drive HYMO. Average annual sediment yields are computed by integrating the frequency curve.

Measured annual sediment yield was 6.1 t/ha, compared to 5.1 t/ha and 5.3 t/ha simulated by HYMO and SWRRB, respectively. An unusually intense storm (greater than the 100-year storm) occurred on the Stamford Lake watershed on August 3-4, 1978. Simulation results show 21-26% of the sediment deposited in the reservoir from 1966 to 1982 occurred during the August 3-4, 1978, storm.

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# SEDIMENT TRANSPORT SIMULATION IN AN ARMOURED STREAM

By Robert T. Milhous, Hydraulic Engineer, Instream Flow Group, USFWS; Jeffrey B. Bradley, Research Associate, Colorado State University, and Cindy L. Loeffler, Hydraulic Engineer, Instream Flow Group, USFWS, Fort Collins, Colorado.

## ABSTRACT

Improved methods of calculating bed material stability and transport must be developed for a gravel bed stream having an armoured surface in order to use the HEC-6 model to examine channel change. Good possibilities exist for use of a two layer model based on the Schoklitsch and the Einstein-Brown transport equations. In Einstein-Brown the D35 of the armour is used for stability and the D50 of the bed (sub-surface) is used for transport. Data on the armour and sub-surface size distribution needs to be obtained as part of a bed material study in a gravel bed river; a "shovel" sample is not adequate. The Meyer-Peter, Muller equation should not be applied to a gravel bed stream with an armoured surface to estimate the initiation of transport or for calculation of transport at low effective bed shear stress.

## INTRODUCTION

The original title of this paper, "Physical Habitat Simulation and the Moveable Bed," was a report on approaches to link the Physical Habitat Simulation Model (PHABSIM) of the U.S. Fish and Wildlife Service to the HEC-6 moveable bed model of the U.S. Army Corps of Engineers. Unfortunately, HEC-6 does not adequately simulate the transport of bed load in streams having surface material larger than the material just immediately below the surface (i.e., an armoured stream). In contrast, the type of streams most often being studied for physical habitat analysis are gravel bed streams with a coarse surface layer.

Consequently, additional transport functions need to be added to the HEC-6 program before it can be used with any degree of confidence in a gravel bed stream. Possible functions which could be added to HEC-6 to simulate bed load transport in a gravel bed stream are discussed in this paper.

For discussion purposes, streams can be divided into four types from the viewpoint of bed load transport. These are:

1. Sand bed streams,
2. Gravel bed streams,
3. Gravel bed streams with a through-put of sand,
4. Bed rock streams

Sand bed streams have bed material in the sand size range which is generally uniform with depth. Gravel bed streams contain bed material in the gravel size range and often have coarser material on the surface. The third category streams have a gravel bed but also contain sand transported over the top of the gravel at flows too low to move the gravel. Last is the bed rock stream with sufficiently low levels of sediment transport that most sediment is "washed" through the system either within the water column or along the bed. There may be some small bars of either sand or gravel along the stream bed. Only the gravel bed stream will be considered in this paper.

Most gravel bed streams are armoured in that the average grain size on the surface is larger than the average size of the material below the surface layer (sub-surface). An example of an armoured stream is the Williams Fork River in Colorado. The grain size distribution for Williams Fork bed material is given in Figure 1.

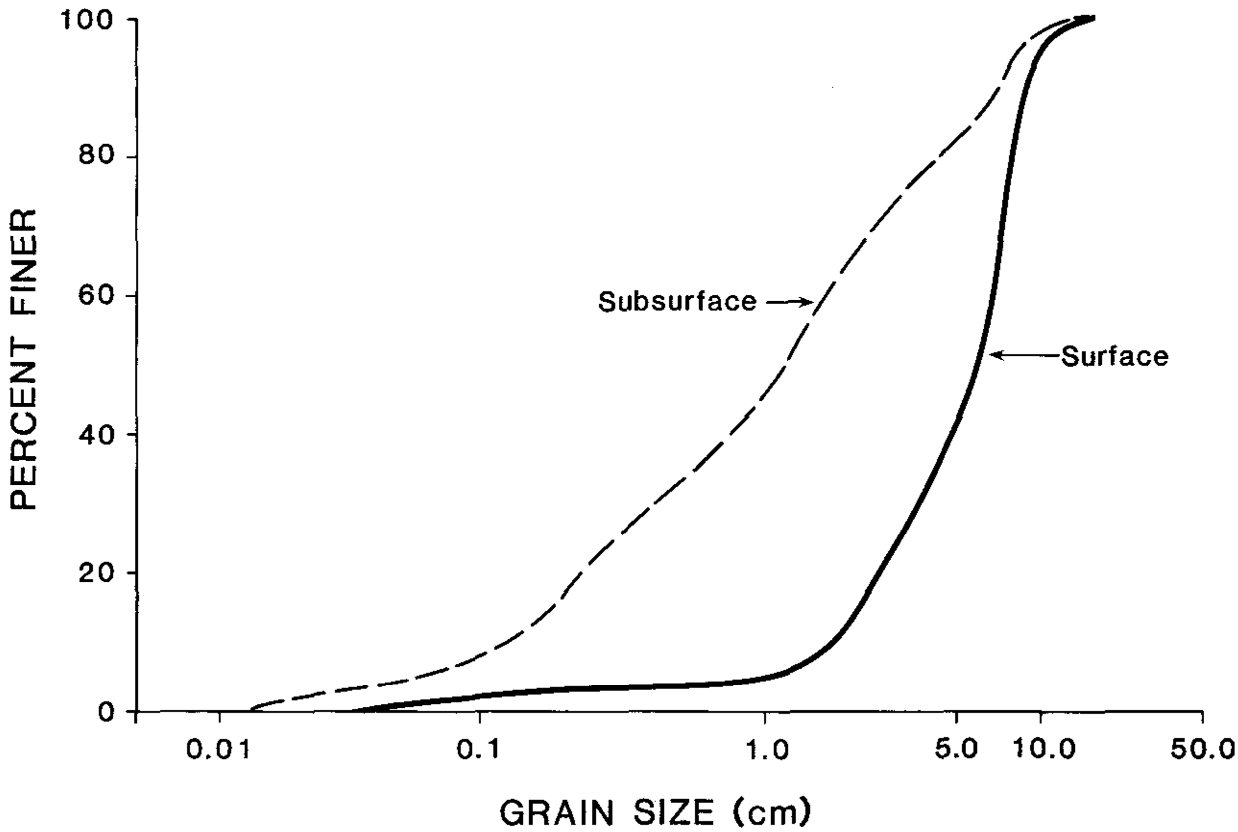


Figure 1. Bed material of the Williams Fork River, Colorado, about one mile below Kinney Creek.

There are two types of armoured streams. The first is represented by Figure 1 where all sizes found in the sub-surface are also found in the armour layer, and the bed is generally mobile in that the armouring material is routinely moved by the stream. The second type is a stationary armour where the surface material is rarely moved by the stream and the armouring sizes are not frequently found in the material below the armour. An example of the second type is the often cited case of the armour of the Missouri River below Fort Randall Dam (Livesey, 1965). Our paper focuses on the stream with a mobile armour.

The usual approach to estimating the bed load is to assume that the bed material does not vary with depth and that the size of the bed material can be represented by a single grain size distribution or a single set of representative sizes. In an armoured gravel bed stream, there are two size distribution curves or two sets of representative sizes--one for the armour and one for the bed material below the armour.

To illustrate the problem, data from Oak Creek in the Oregon Coast Range near Corvallis are used. The properties of the bed material and armour are:

| | |
|---|---|
| median size of armour | 0.21 feet |
| median size of bed material | 0.066 feet |
| specific gravity of bed material | 2.85 |

These data were used with the Shields bed load transport equation to calculate the bed load in Oak Creek.

The Shields equation is:

$$QBL = \frac{10\ Q\ S\ (T - Tc)}{(G_s - 1.0)^2 D50} \tag{1}$$

where $QBL$ = the bed load in tons/day

$Q$ = the discharge in cubic feet per second

$S$ = the energy slope

$T$ = the bed shear stress on the bed in lb/ft$^2$

$Tc$ = the critical shear stress in lb/ft$^2$

$G_s$ = the specific gravity of the bed material

$D50$ = the median size of the bed material or armour in feet

The critical shear stress is calculated using the following equation:

$$Tc = K\ \gamma\ D50(G_s - 1.0) \tag{2}$$

where $\gamma$ = the unit weight of water in lb/ft$^3$

$K$ = the Shields parameters

and the others as given above.

The results of the calculations are given in Table 1. The Shields constant was assumed to be 0.03 for the case presented in Table 1. For the armoured case, the D50 of the armour was used in both the critical shear stress and the bed load equations to calculate the bed load rate; for the sub-surface case, the D50 of the bed material was used in both equations. For the composite case, the D50 of the armour was used for the calculation of the critical shear stress and the D50 of the bed material used in the transport equation. The logic of using a size representative of the armour to calculate stability, and of the bed (sub-surface) to calculate bed load has been given previously in Milhous (1973). The basic logic is that the control on the rate at which bed material is made available for transport is related to the stability of the armour, but when bed material is made available it is the characteristic of the bed material that controls the rate of transport.

Table 1. Bed load in Oak Creek, Oregon. Calculated using Shields Equation and Alternate Representative sizes.

| Discharge (cfs) | Representative size based on Armour | Bed | Composite | Regression equation |
|---|---|---|---|---|
| | (tons per day) | | | |
| 10 | 0 | 69 | 0 | 0.0003 |
| 25 | 0 | 298 | 0 | 0.03 |
| 50 | 76 | 838 | 242 | 0.9 |
| 75 | 192 | 1506 | 612 | 6 |
| 100 | 323 | 2220 | 1027 | 20 |
| 125 | 474 | 2998 | 1508 | 43 |
| 150 | 642 | 3831 | 2042 | 76 |
| 200 | 1022 | 5636 | 3252 | 167 |
| 300 | 1555 | 8424 | 4948 | 456 |

Critical Shields Parameter = 0.030

Regression equations for the bed load as a function of discharge were developed by Milhous (1973) on the basis of measured data; the bed load transport rates calculated using the regression equations are also given in Table 1. Two points are illustrated by Table 1: (1) the assumption made in selecting the representative size can have a major impact on the calculated loads; and (2) the calculated load can be very wrong.

The regression equation developed for flows above the critical discharge is:

$$QBL = 0.0071(Q - Q_{CR})^{2.0} \qquad (3)$$

where $Q_{CR}$ is the critical discharge and the other terms are as described previously.

The regression equation used for streamflow below critical discharge is:

$$QBL = 3.98 \times 10^{-9} \; Q^{4.91} \qquad (4)$$

The two equations intercept each other at about 75 cfs.

The regression results in the tables were obtained using equation (2) for flows at and above 75 cfs, and equation (4) for flows below 75 cfs.

The Shields equation for bed load transport is not often used. Nevertheless, the results presented do give some idea of the nature of the problems faced when working with gravel bed rivers. The next section looks at other potentially applicable equations.

## THE CALCULATION OF BED LOAD

The bed load may be calculated using numerous equations. The results obtained for Oak Creek using six different equations are presented in Table 2. In each case, the representative size for stability of the bed is based on the armour with the representative size used in each equation shown at the bottom of the table. The D35 size is 0.28 feet and the D16 size is 0.12 feet, the other sizes were given previously. Except for the Bathurst and Milhous modifications of the Schoklitsch equations, the equations used are as presented in the ASCE Sedimentation Engineering Manual (Vanoni, 1975).

The Schoklitsch equations as presented by Bathurst et al. (1985) are:

$$Q_{CR} = 0.15\ S^{-1.12}\ \sqrt{g}\ D16^{3/2}\ B \qquad (5)$$

where $Q_{CR}$ = the critical discharge

B = the width of the stream

S = the slope

g = gravity acceleration

D16 = the size at which 16 percent of the armour is smaller

and

$$QBL = 2.5\gamma\ G_s\ \ S^{3/2}(Q - Q_{CR}) \qquad (6)$$

where Q = the water discharge

$\gamma$ = the unit weight of water

$G_s$ = the specific gravity of the bed material

and the other terms are defined above. Note that the bed material size does not appear in the bed load equation but does appear in the equation for critical discharge.

Table 2. Bed load in Oak Creek, Oregon as calculated by various equations using the composite approach.

| Discharge (cfs) | Shields K = 0.03 | Meyer-Peter, Muller K = 0.047 | Meyer-Peter, Muller K = 0.030 | Einstein-Brown |
|---|---|---|---|---|
| | (tons per day) | | | |
| 10 | 0 | 0 | 0 | 0.067 |
| 25 | 0 | 0 | 0 | 0.67 |
| 50 | 242 | 0 | 0 | 6.4 |
| 75 | 612 | 0 | 0 | 27.2 |
| 100 | 1027 | 0 | 0 | 63.7 |
| 125 | 1508 | 0 | 0 | 102 |
| 150 | 2042 | 0 | 0 | 140 |
| 200 | 3252 | 0 | 0 | 235 |
| 300 | 4948 | 0 | 32 | 302 |
| Representative sizes: | | | | |
| Armour | D40 | D40 | D40 | D35 |
| Bed | D50 | D50 | D50 | D50 |

| Discharge (cfs) | ASCE Manual | Schoklitsch Bathurst et al. | Milhous modifications | Regression equations |
|---|---|---|---|---|
| | (tons per day) | | | |
| 10 | 0 | 0 | 0 | 0.0003 |
| 25 | 0 | 0 | 0 | 0.03 |
| 50 | 0 | 0 | 0.9 | 0.9 |
| 75 | 49 | 0 | 6.7 | 6 |
| 100 | 177 | 0 | 17 | 20 |
| 125 | 309 | 165 | 31 | 43 |
| 150 | 442 | 366 | 49 | 76 |
| 200 | 714 | 776 | 96 | 167 |
| 300 | 1195 | 1454 | 192 | 456 |
| Representative sizes: | | | | |
| Armour | D50 | D16 | D50 | - |
| Bed | D50 | - | - | - |

Information in the Bathurst paper was used to derive an alternate set of equations (Milhous, 1985). These are:

$$Q_{CR} = 0.0345\ S^{-1.12}\ \sqrt{g}\ D50^{\ 3/2} B \tag{7}$$

$$QBL = 0.0065 \gamma G_s\ S^{3/2} \left( \frac{Q - Q_{CR}}{B} \right)^{2.0} B \tag{8}$$

where the terms are as presented above.

The assumption is that the bed loads are a function of the square of an effective discharge where the effective discharge is the discharge in excess of some critical discharge based on the bed material size. Again note that the bed material size appears only in the equation for critical discharge.

In reviewing the results presented in Table 2, it is interesting that both the Shields and the Meyer-Peter, Muller equations failed in calculating the sediment load. The relative sediment load calculated using the Shields equation is reasonable but much too high; in contrast, the Meyer-Peter, Muller equation says there is no transport for stream flows in which there is a reasonable amount of measured transport. This is true even when the critical Shields parameter used (0.030) is much below the usual value of 0.047.

The fact that the Meyer-Peter, Muller equation suggests there is no transport may be the reason many individuals working on gravel bed rivers have reported that the gravel bed only moves at flows associated with the 100 year flood or greater. A rough estimate of the mean annual peak discharge in Oak Creek is 250 cfs with a bank full discharge of about 300 cfs. The analysis of the Oak Creek data indicates the Meyer-Peter, Muller equation should not be used to calculate bed load in a gravel bed stream when the bed shear stress is relatively low.

The Einstein-Brown and the ASCE Manual form of the Schoklitsch equation appear to give the best results for flows greater than about 200 cfs, and the Milhous form of Schoklitsch for flows between 50 and 200 cfs. Only the Einstein-Brown equation calculates bed load for streamflows below about 50 cfs even though bed load was measured. Results reported by Milhous (1973) suggest the Einstein equation would also work reasonably well with similar assumptions. Klaasen (1982) has indicated that this approach may not work in all cases and has presented data for the Snake River in Idaho where it clearly did not. This may result from a large quantity of sand moving over the gravel. Further investigation of the sand over gravel case needs to be done.

At this point in our work on estimating the bed load in a gravel bed river our results indicate either the Einstein-Brown, Einstein, ASCE Manual form of the Schoklitsch, or the Milhous form of the Schoklitsch equation could be used. No preference can be made between these four equations at this time.

## A TWO-LAYER CALCULATION PROCEDURE

The basic change from considering sediment transport in a sand bed river to sediment transport in a gravel bed river is a change from a single-layer system to a two-layer system. The equations discussed in the previous section are all single layer equations although they are "tricked" into having some of the characteristics of a two layer system by using a representative size of the armour for stability, and of the sub-surface for transport.

The assumption we now make is that only armour material is available for transport when the shear stress on the bed is low, but for high stress the armour material is relatively unimportant compared to the bed material. In order to examine this hypothesis the Milhous modification of the Schocklitsch equation is used with the following criteria based on the calculated Shields parameter ($f_s$):

| | |
|---|---|
| $f_s < 0.018$ | transport based on armour material only |
| $f_s > 0.060$ | transport based on bed material only |
| $0.018 \leq f_s \leq 0.060$ | linear change from transport based on armour to transport based on bed material |

The equations used are:

$$FB = \frac{f_s - 0.018}{0.42} \tag{9}$$

$$QBL = (QBL_b * FB) + (QBL_a * (1 - FB)) \tag{10}$$

where the subscript b refers to transport based on bed material gradation and the subscript a is based on armour gradation. The results are given in Table 3 and Figure 2 for Oak Creek data. In general the present formulation of the two-layer model tends to over predict the bed load in Oak Creek, except possibly for flows in excess of 150 cfs. The regression equations were based on data for flows less than 150 cfs. We do feel, though further work is required to verify this, that the two-layer approach is more technically sound than those methods presented in the previous section. Maybe Schoklitsch is not the best equation to use but the two layer approach is the most sound.

Table 3. Bed load in Oak Creek based on the two layer analysis using a modified Schoklitsch equation.

| Discharge (cfs) | Bed load: Based on armour | Bed load: Based on sub-surface | Bed load: Based on two layer model | Bed load: Regression equations |
|---|---|---|---|---|
| | (tons per day) | | | |
| 5 | 0 | 0 | 0 | 0.00001 |
| 7.5 | 0 | 0.02 | 0.0004 | 0.0001 |
| 10 | 0 | 0.09 | 0.008 | 0.0003 |
| 15 | 0 | 0.44 | 0.071 | 0.002 |
| 25 | 0 | 1.8 | 0.52 | 0.03 |
| 50 | 0.9 | 9.3 | 5.0 | 0.09 |
| 75 | 6.7 | 22 | 16.4 | 6 |
| 100 | 17 | 39 | 32 | 20 |
| 125 | 31 | 60 | 54 | 43 |
| 150 | 49 | 85 | 80 | 76 |
| 200 | 95 | 146 | 144 | 167 |
| 300 | 195 | 270 | 267 | 456 |

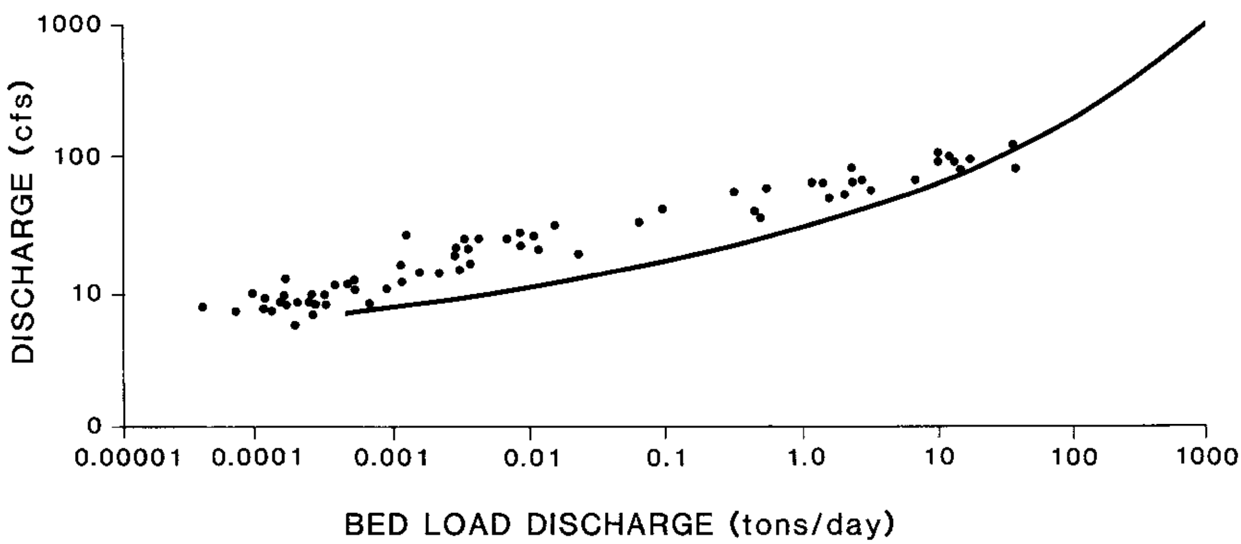


Figure 2. Bed load in Oak Creek as a function of water discharge compared to results from a two layer model.

## TEST OF EQUATIONS ON A RIVER

Oak Creek is about 15 ft wide. Data are not available to adequately test the equations on a larger river at this time. In order to compare the equations when applied to a river about 80 ft wide hydraulic data from an instream flow study on the Williams Fork River in Colorado were used. The results are given in Table 4.

Because bed load data are not available for the Williams Fork, no statement can be made as to which equation is "best." Either the two-layer or Einstein-Brown equations are the most believable. The Meyer-Peter, Muller equation failed the test again for low bed shear stresses.

## A NOTE ON BED MATERIAL SAMPLING

It is worth noting that there is little data available for gravel bed streams in which both armour (surface) and the bed (sub-surface) have been carefully sampled. This is a shortcoming of our present "standard" sampling methods which usually entail taking a shovel and digging a hole, in the process sampling both the armour and the sub-surface material together.

Table 4. Calculated bed load in the Williams Fork River, Colorado.

| Discharge (cfs) | Bed load (tons per day) | | | |
|---|---|---|---|---|
| | Two-layer | Einstein-Brown | Schoklitsch[1] | Meyer-Peter, Muller[2] |
| 100 | 0.0 | 0.0 | 0.0 | 0.0 |
| 200 | 0.16 | 0.0 | 0.0 | 0.0 |
| 250 | 1.19 | 0.26 | 0.0 | 0.0 |
| 300 | 3.00 | 0.60 | 0.0 | 0.0 |
| 400 | 11.4 | 1.16 | 0.0 | 0.0 |
| 500 | 29.8 | 13.5 | 0.0 | 0.0 |
| 700 | 101 | 84.9 | 59.0 | 0.0 |
| 800 | 159 | 171. | 508. | 0.0 |
| 900 | 224 | 241. | 919. | 4.11 |
| 1000 | 306 | 339. | 1380. | 56.9 |
| 1250 | 560 | 665. | 2530. | 274. |
| 1500 | 833 | 806. | 3420. | 520. |

[1]As given in the ASCE Sedimentation Manual.

[2]With critical Shields parameter of 0.03; with 0.047 all values were 0.0.

Because of the lack of bed material data needed to apply either approach used in this paper it is difficult to test the approaches on very many streams. For this reason it is suggested that improvements be made in the bed material sampling of gravel bed rivers. These improvements should result in information on both the surface and the sub-surface. The lack of adequate bed material sampling procedures for gravel bed rivers are significant shortcomings of present water-data collection procedures.

## CONCLUSIONS

At present only two conclusions can be drawn, these are:

1. data is needed on the armour and the bed material below the armour in order for the bed material data collected in a gravel bed stream to be useful in developing methods of calculating bed load in an armoured stream; and

2. the Meyer-Peter, Muller equation should not be used in an armoured stream when the bed shear stress is relatively low because the results are most likely to be misleading.

This paper is a progress report on work underway to develop methods of calculating bed load in an armoured stream. Use of models such as HEC-6 can not be reasonably applied to gravel bed streams until such work is done.

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# ALLUVIAL COMPUTATIONS IN COMPLEX RIVER NETWORKS

By Forrest M. Holly Jr., Iowa Institute of Hydraulic Research, Iowa City, Iowa; Kathleen E. Schneider, Omaha District, Corps of Engineers; and Warren J. Mellema, Missouri River Division, Corps of Engineers, Omaha.

## ABSTRACT

Efforts to predict the future course of channel degradation in the Missouri River between Sioux City and Omaha have led to the development of IALLUVIAL, a computer code for simulation of long-term channel aggradation and degradation in non-uniform alluvium. IALLUVIAL has been successfully used to forecast a progressive slowing down of degradation in the next twenty years. The techniques used in IALLUVIAL have been incorporated in the new BRALLUVIAL code for braided or other multiply-connected channel systems. The basic techniques of IALLUVIAL and BRALLUVIAL, along with some results for the Missouri River, are presented.

## INTRODUCTION

The bed of the Missouri River between Gavins Point Dam and Omaha has undergone extensive degradation since 1950. This scouring has had severe consequences, including potential undermining of structures, loss of wildlife habitat, and lowering of the water table with associated increased pumping costs and dropping of water levels in oxbow lakes used for recreation.

River-bed degradation is a complex process which, before the advent of the computer and associated computational-hydraulics techniques, was not subject to rational, deterministic analysis. However, now that such techniques are available, it has become possible to simulate long term river bed evolution in repsonse to changed river geometry, hydrologic regime, and sediment supply. The IALLUVIAL computer program, developed in 1981-82 at the Iowa Institute of Hydraulic Research under sponsorship of the Omaha District, Corps of Engineers, was first applied in studies to simulate the past twenty years of Missouri River bed degradation.

The success of IALLUVIAL in reproducing past bed degradation laid the groundwork for an additional study, whose goals included modification of IALLUVIAL to treat more realistic river conditions, including tributaries, bank erosion, and vertical nonhomogeneity of bed sediments, and then use of the improved version of IALLUVIAL to simulate bed degradation over the next twenty years, 1980-2000. These prognosis simulations, performed for various hypothetical scenarios of river management, showed the sensitivity of Missouri River bed degradation to geometric, hydrologic, and sediment variables, and furnished indications of the expected pattern of future degradation. The simulations of this study suggested that the worst of the degradation is over. An additional three to four feet might be expected, but the rate of degradation is decreasing, as the river approaches a new equilibrium between sediment supply and transport capacity.

Application of IALLUVIAL to this prototype problem made it possible to identify several areas of the methodology which needed further development and refinement. The simulations also clearly demonstrated the importance of collecting bed-sediment size-distribution data which includes as much detail as possible on the coarser size fractions, as these fractions play a critical role in limiting the rate and ultimate extent of degradation. Continuing Corps-sponsored refinement of IALLUVIAL has been devoted to improvement of computational techniques for armoring and sediment transport by size fraction.

The main purpose of this article is to outline the computational techniques of IALLUVIAL and present limited Missouri River simulation results. An additional purpose is to outline the computational methologies being developed for extension of existing water and sediment-routing techniques to the problem of degradation and aggradation in multiply-connected or braided channel networks conveying non-uniform sediment mixtures.

## REVIEW OF IALLUVIAL COMPUTER PROGRAM

IALLUVIAL is a computer-based flow- and sediment-routing model for simulation of the long-term bed evolution of alluvial streams. It treats the flow as one-dimensional and quasi-steady, solving the following governing equations: equation of motion of water flow; equation of continuity for water flow; friction-factor equation that takes into account the variable roughness of sediment-transporting streams; equation for sediment discharge; and equation of continuity for sediment.

A river reach is divided into subreaches, and the computations for each are performed for successive, discrete time intervals. In each time interval, IALLUVIAL solves the governing equations in two steps: the first is a "backwater" step to obtain water-surface elevation, depth, velocity, and sediment discharge at each computational point; in the second step, the sediment-continuity equation is solved to yield depths of degradation/aggradation, changes in bed-material composition, and changes in armoring of the bed surface. The initial and boundary conditions required for a solution are: known initial bed elevation and sediment size distribution at all computation points; known water and sediment discharge hydrographs at the upstream limit of the model; and a known stage (water-surface elevation) hydrograph, or discharge-stage relationship, at the downstream limit of the model.

Sediment-discharge and friction-factor predictors. The Total-Load Transport Model (TLTM) developed by Karim and Kennedy (1981) is used for the sediment-discharge and friction-factor predictors. The formulation of TLTM takes into account the well-known fact that the friction factors of alluvial streams are heavily dependent on their sediment discharges, and avoids the need to specify a fixed hydraulic roughness, such as Manning's coefficient, a priori. In keeping with this concept, the friction-factor relation includes sediment discharge as one of the independent variables, and an iteration scheme is used to calculate sediment discharge and friction factor from the following pair of simultaneous relations:

Sediment-discharge predictor

$$\mathrm{Log}\left(\frac{q_s}{\sqrt{g(s-1)D_{50}^3}}\right) = -2.2786 + 2.9719\ \mathrm{Log}\ V_1 + 1.0600\ \mathrm{Log}\ V_1 \mathrm{Log}\ V_6 + 0.2989\ \mathrm{Log}\ V_2 \mathrm{Log}\ V_6 \qquad (1)$$

Friction-factor predictor

$$\text{Log}\left(\frac{U}{\sqrt{g(s-1)D_{50}}}\right) = 0.9045 + 0.1665\ \text{Log}\ V_7 + 0.0831\ \text{Log}\ V_4\ \text{Log}\ V_5\ \text{Log}\ V_7$$

$$+ 0.2166\ \text{Log}\ V_4\ \text{Log}\ V_5 - 0.0411\ \text{Log}\ V_2\ \text{Log}\ V_3\ \text{Log}\ V_4 \qquad (2)$$

where $V_1 = \frac{U}{\sqrt{g(s-1)D_{50}}}$, $V_2 = \frac{d}{D_{50}}$, $V_3 = S \cdot 10^3$, $V_4 = \frac{u_*}{w}$

$$V_5 = \frac{wD_{50}}{\nu},\ V_6 = \frac{u_* - u_{*c}}{\sqrt{g(s-1)D_{50}}},\ V_7 = \frac{q_s}{\sqrt{g(s-1)D_{50}^3}}$$

in which $q_s$ = volumetric bed-material discharge/unit width, U = mean flow velocity, d = mean flow depth, $D_{50}$ = median size of bed material, S = energy slope, w = fall velocity of sediment particles, $\nu$ = kinematic viscosity of water, s = specific gravity of sediment particles, $u_*$ = bed shear velocity, and $u_{*c}$ = critical shear velocity obtained from Shields' diagram. the numerical coefficients of Eqs. (1) and (2) were obtained through nonlinear regression analysis of extensive field and laboratory data (Karim and Kennedy, 1981).

Water-surface-profile calculation. Computations for sediment discharge and water surface profile in one time interval proceed simultaneously, because of the interdependence between friction factor and sediment discharge incorporated in Eqs. (1) and (2). Starting from a known or specified water-surface elevation at the downstream end, the calculation scheme simultaneously solves Eqs. (1) and (2), and the steady-state continuity and energy equations of flow, by an iteration scheme analogous to the standard step method for backwater computations. This procedure calculates depth, velocity, energy slope, and sediment discharge at successive upstream sections in a single computational sweep, downstream to upstream.

Change in bed elevation. The depth of degradation or aggradation in a computational subreach of length $\Delta x$ during a time interval $\Delta t$ is calculated by applying the sediment-continuity equation between the two bounding computation points,

$$(1-p)\frac{\partial z}{\partial t} + \frac{\partial q_s}{\partial x} = 0 \qquad (3)$$

where p = porosity, and z = bed elevation. Equation (3) may be discretized to calculate the change in bed elevation for a reach, $\Delta z$, from

$$\Delta z = \frac{\Delta t}{\Delta x(1-p)}\{\theta\,(q_{si}^{n+1} - q_{si+1}^{n+1}) + (1-\theta)(q_{si}^{n} - q_{si+1}^{n})\} \qquad (4)$$

in which $q_{si}$ and $q_{si+1}$ are sediment-transport capacities per unit width at downstream and upstream ends of the subreach respectively, n and n+1 denote successive times, and $\theta$ is a weighting factor between 0 and 1. A positive value of $\Delta z$ indicates degradation when $(q_s)_i > (q_s)_{i+1}$, i.e., a deficit in sediment-transport capacities exists between the downstream and upstream sections of the subreach. When $(q_s)_i < (q_s)_{i+1}$, $\Delta z$ is negative, and its absolute value gives the depth of aggradation in the subreach. In the present version of IALLUVIAL, the entire wetted perimeter is shifted up or down by $\Delta z$.

Changes in bed-material composition. The composition of an alluvial river bed undergoes continuous change in response to degradation or aggradation occurring due to imbalance in sediment transport capacities at the two ends of a reach. The depths (or volumes) of sediments of each size fraction scoured from or deposited in a reach are determined by applying the sediment continuity equation by size fraction. The fraction of sediment discharge in size interval k and reach i, $P_{di,k}$, is calculated from the relation given by Karim and Kennedy (1981):

$$P_{di,k} = \frac{P_{i,k}\left(\frac{D_{50i}}{D_k}\right)^x}{\sum_{k=1}^{m} P_{i,k}\left(\frac{D_{50i}}{D_k}\right)^x} \quad (5); \qquad x = 0.0316\left(\frac{d_i}{D_{50i}}\right)^{0.5} \quad (6)$$

where $P_{i,k}$ = fraction of size interval k in bed material of reach i; $D_{50i}$ = median bed-material size in subreach i; $D_k$ = geometric mean size of fraction k; m = total number of sediment size intervals; and $d_i$ = average flow depth in subreach i.

The size distribution of the bed sediments is updated at the end of each time interval by taking out the calculated depths of degradation from, or adding the deposited volumes to, the mixed layer, and then accounting for the proportionate change in each sediment size interval. The horizon of bed material immediately below the bed surface undergoing continual mixing due to agitation, overturning, bed-form migration, etc. is referred to as the mixed layer, and is assumed to have a thickness equal to the average bed-form (or dune) height and to be homogeneous in size distribution at any given time. The dune height $H_d$, is estimated from the following relation (Allen, 1978):

$$H_d = d\left[b_o + b_1\left(\frac{\phi}{3}\right) + b_2\left(\frac{\phi}{3}\right)^2 + b_3\left(\frac{\phi}{3}\right)^3 + b_4\left(\frac{\phi}{3}\right)^4\right] \quad (7)$$

where $\phi$ = non-dimensional bed-shear stress; $b_o$ = 0.079865, $b_1$ = 2.23897, $b_2$ = -18.1264, $b_3$ = 70.9001, and $b_4$ = -88.3293. This relation accounts for both the growth and decay of bed-form heights with variation in bed-shear stresses.

Bed armoring. In a degrading river, the finer sediment particles are transported preferentially from the bed, resulting in gradual coarsening of the bed surface. If the bed material contains sediments which are sufficiently large that they cannot be transported by the flow, then coarser particles gradually accumulate on the bed surface forming an "armor coat" which protects the underlying finer sediments which would otherwise be transported. The fraction of the bed surface, $A_f$, covered by these immobile sediment particles is expressed by the following relation, developed from volumetric considerations:

$$A_f(t) = C_1(1-p)\, d_s(t) \sum_{k=\ell}^{m} \frac{P_k}{D_k} \quad (8)$$

where p = porosity; $d_s(t)$ = depth of degradation to time t; $P_k$ = fraction of bed material with size $D_k$; $\ell$ = sediment-size interval containing the smallest size which remains immobile on the bed; and $C_1$ = constant determined by the shape of the particles and their array on the bed. $C_1 = 1.90$ for ellipsoidal particles of shape factor 0.70 laying flat in a one-diameter-thick armor layer.

Bed armoring plays an important role in restoring balance between the sediment-transport capacity of the flow and the reduced sediment-supply rate into a reach. Armoring assists the river in seeking a new equilibrium by reducing sediment discharge and also by changing the hydraulic roughness. It is assumed in IALLUVIAL that sediment discharge is reduced in direct proportion to the fraction of the bed surface that is armored ($A_f$). Similarly, the friction factor is taken equal to a weighted average of the fixed-bed roughness for the armored portion ($A_f$) and the movable bed roughness ($1 - A_f$). The thickness of the mixed layer is assumed to decrease linearly with increasing armoring of the bed surface.

Although Eq. (8) is a valid general description of the armoring factor used in IALLUVIAL, additional research has led to use of a more refined procedure, whose details can be seen in the report by Karim, Holly, and Kennedy (1983).

## APPLICATION TO MISSOURI RIVER

Initial Missouri-River modelling efforts were devoted to simulation of 1960-80 historical degradation trends using a somewhat simplified, schematic model data set of a 250-mile reach of the Missouri River between Yankton, SD and Omaha NE. Figure 1 is a summary comparison of observed and computed water surface and bed-elevation changes after 20 years. The simulation reproduced the overall pattern of bed evolution, including the apparent shift to aggradation near Omaha, quite faithfully, although local differences in water surface elevation of as much as 4 feet can be seen in the zone where aggradation begins, between Blair and Omaha (Karim and Kennedy, 1982). In subsequent modelling efforts focussed on a prognosis of future bed degradation, the schematic model data set of the 1960-1980 simulation was replaced by one incorporating all available data on 1980 channel topography, bed-sediment size distribution, tributary and bank erosion rates (treated in the model as sediment inputs to the natural channel above Ponca State Park). The model was extended to below the Iowa-Missouri border, and incorporated water and sediment inflow from nine major tributaries (Holly and Karim, 1983).

The changes in water-surface and thalweg profiles at the end of the base 20-year prognosis run are shown on Figure 2. These results show that apart from an additional two feet of degradation in the immediate vicinity of Gavins Point Dam, (compared to 4.2 feet computed in the earlier 1960-80 study), very little additional degradation is forecast to occur in the uncontrolled reach from the Dam down to Ponca. However in the controlled reach from Ponca to Omaha, as much as four feet of additional degradation is expected to occur, being most severe near Sioux City and Decatur Bend. This is to be compared to the 7.2 feet computed near Sioux City in the 1960-80 simulation. Below Omaha the model predicts continuing aggradation; the large inflow of the Platte River (River mile 595) causes a backwater in the Missouri which provokes deposition of transported sediments, and the Platte itself delivers a sediment load which is coarser than that transported by the Missouri, causing formation of a local delta. A general degradation trend resumes below Plattsmouth.

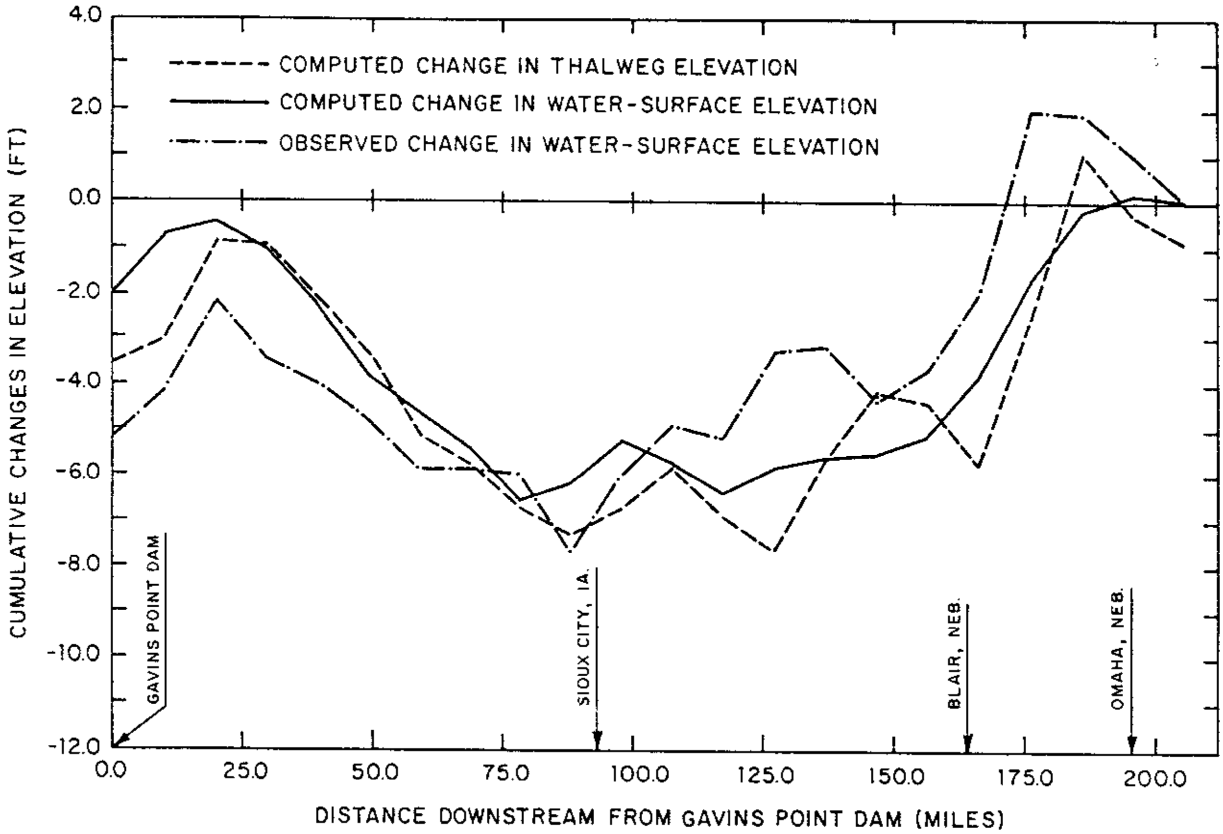


Figure 1. Observed and Computed Water-Surface and Bed Elevation Changes, 1960-1980

**EXTENSION TO MULTIPLY-CONNECTED CHANNELS**

Figure 3 shows a section of a northern braided river as schematized for simulation of water and sediment routing. Generalization of the techniques in IALLUVIAL for multiply-connected networks of this type affects primarily the water routing (backwater) portion of the computation. The flow computation, which proceeds assuming that the bed elevations are momentarily fixed during one iteration, is based on a statement of water continuity at all nodes,

$$\sum_{\ell=1}^{L(m)} Q^n_{m,\ell} + \sum_{\ell=1}^{L(m)} \Delta Q_{m,\ell} = 0 \qquad (9)$$

where m denotes the node number, $\ell=1$, ...., L(m) denotes the links entering or leaving node m, L(m) is the number of links connected to node m, n denotes the known time or iteration level, $Q^n$ is the known water discharge, and $\Delta Q$ is the unknown change in discharge between level n and n+1.

The key to the computation is use of the energy equation within each link to relate $\Delta Q$ (which is constant throughout a link under the quasi-steady assumption) to the nodal water level changes at the extremities of the link. The discrete energy equation, written for two adjacent points i and i+1 of any link $\ell$, can be linearized, yielding:

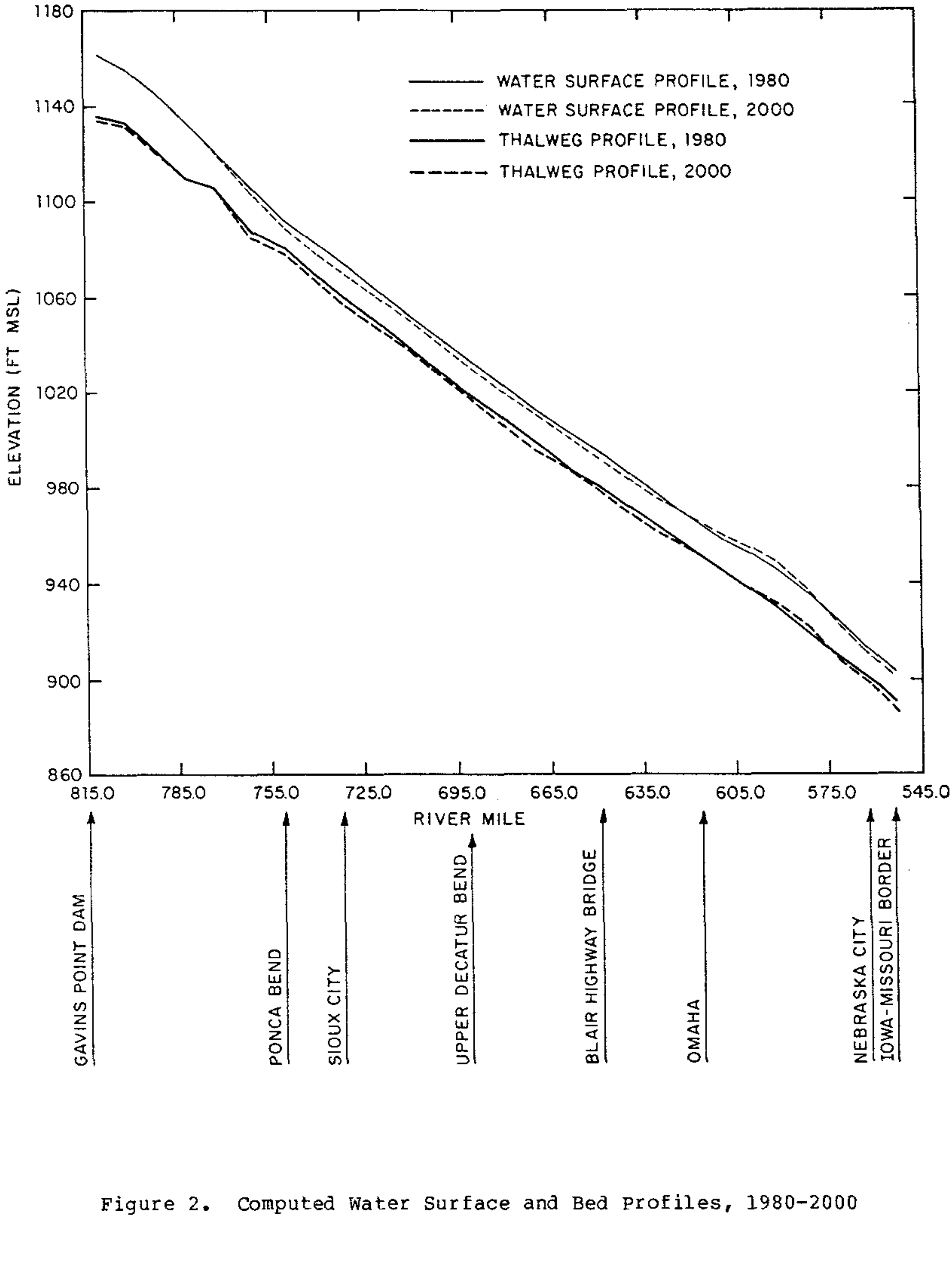


Figure 2. Computed Water Surface and Bed Profiles, 1980-2000

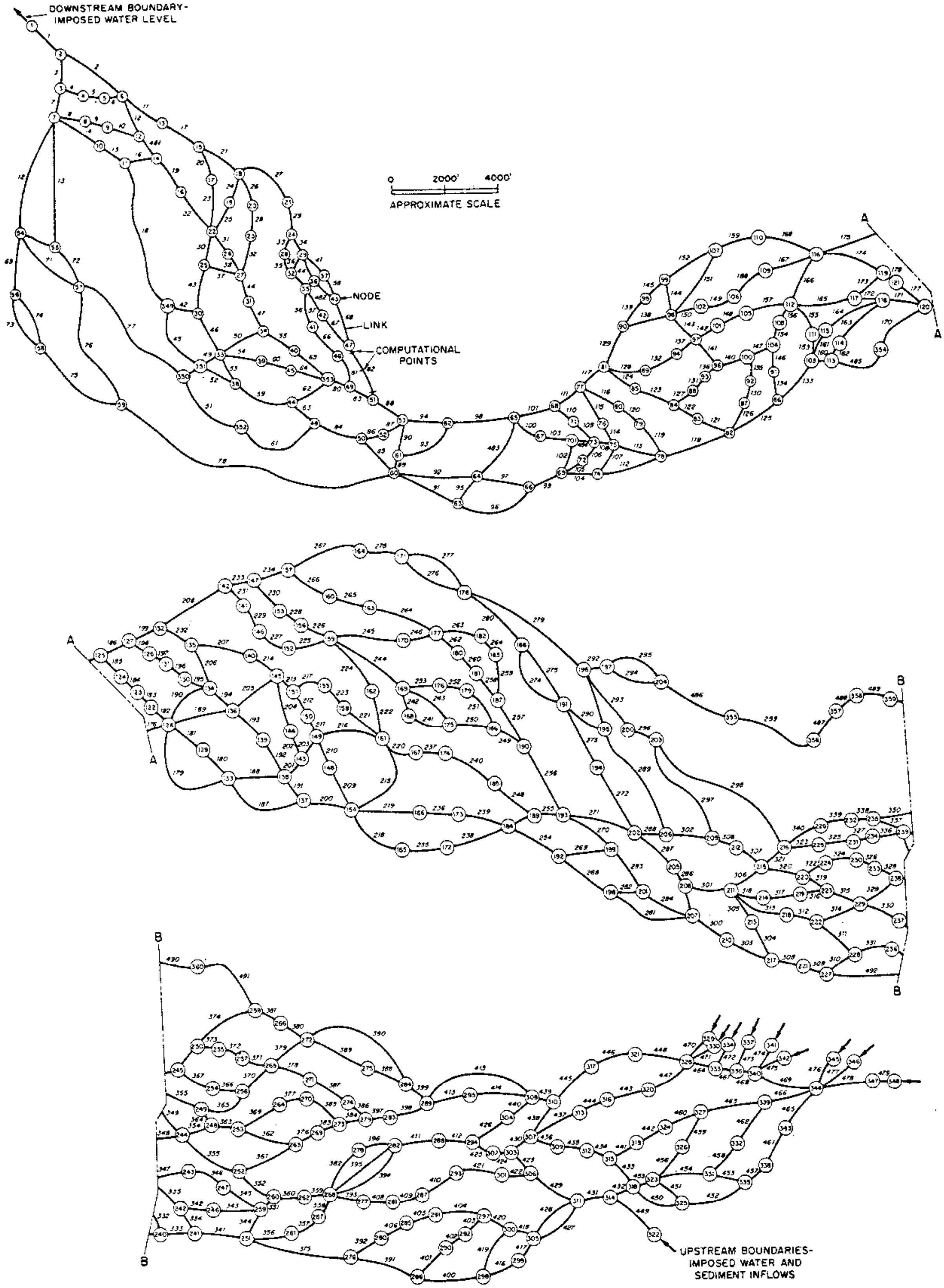


Figure 3. Topological Layout of Braided System for Water and Sediment Routing

$$p_i \Delta y_i + q_i \Delta y_{i+1} + r_i \Delta Q_\ell + s_i = 0 \tag{10}$$

where $p_i$, $q_i$, $r_i$, and $s_i$ incorporate only known quantities from iteration level n. Equation (10) can also be developed for flooded or free flowing weirs.

When Eq. (10) is used to construct a "double sweep" algorithm along a link, in a manner analogous to that used for unsteady flow (Cunge et al., 1980), the following key relation results:

$$\Delta Q_\ell = E_{I-1} \Delta y + F_{I-1} + H_{I-1} \Delta y_I \tag{11}$$

where i = 1 and i = I are the first and last points on link $\ell$. Substitution of Eq. (11) into the node continuity Eq. (9), supplemented by a requirement that the water levels in all links attached to a node are the same at the node, leads to the following system of linear equations:

$$[A] \ \{\Delta y\} = \{B\} \tag{12}$$

where [A] is a known coefficient matrix, B is a known vector, and $\{\Delta y\}$ is the vector of unknown water level changes at the nodes. In essence, each line of Eq. (12) represents the continuity equation for one node; Eq. (12) also incorporates boundary conditions of given inflow at one or more upstream nodes, and imposed water level at a single downstream node. Solution of Eq. (12) yields, in each iteration, the set of updated nodal water levels which simultaneously satisfy node continuity and the energy equation in all links. Finally, a "return sweep" in each link computes the updated link discharges and water levels at each point. In the present application, Eq. (12) is solved through use of a block tridiagonal matrix algorithm, with Gauss-Jordan elimination for inversion of small sub-matrices; use of this procedure renders the matrix inversion cost negligible compared to total simulation costs (Mahmood and Yevjevich, 1975).

Beyond this generalized water-routing computation, the multiply-connected extension of IALLUVIAL requires only one other major addition: treatment of sediment continuity at nodes. The rate of change in bed elevation at a node is simply taken as the net sediment inflow distributed over the bed surface area attributed to the node. The remaining sediment-sorting and bed-armoring computations are identical to those used in IALLUVIAL.

## SUMMARY AND CONCLUSIONS

The techniques employed in IALLUVIAL, taken as a whole, have been validated to some extent by the successful Missouri-River simulations. However, these modelling results do not signal the end, but rather the beginning, of efforts to achieve a better physical understanding and mathematical formulation of constituent physical processes such as armoring, sorting, mixed-layer dynamics, mixed grain-and-form roughness, etc. There is an urgent need for imaginative comprehensive laboratory experiments on non-equilibrium bed evolution in channels having nonuniform bed sediments. Responsible contributions to the alluvial-river modelling capabilities of computational hydraulics will be those devoted, not to the movement towards userfriendliness and distributed computing systems, but to improved mathematical and numerical formulation of some of the most complex processes to be found in nature.


**ACKNOWLEDGEMENTS**

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# OFFSITE SEDIMENT IMPACTS USING BASIN SCALE SIMULATION

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## ABSTRACT

Two basin scale watersheds in the Washita River Basin, in Oklahoma, Winter Creek and the Little Washita River were analyzed to determine the impacts of floodwater retarding structures on the sediment loads delivered to the main stem of the river. The SWRRB model was used to simulate the daily flows, peak rates, and sediment loads with and without the structures installed. The results from the simulation model were compared with measured flow and sediment loads from each of the watersheds. The Little Washita watershed simulation showed a reduction of approximately 29 percent in sediment yield due to structures installed during the period 1973 to 1983. Similarly, a 19% reduction was found on Winter Creek for the period 1966 to 1977. Conventional techniques for analyzing offsite impacts applied to the Little Washita data had shown no significant reduction in sediment yield due to structures. The results are discussed in relation to climatic variability on watershed treatment measures and the value of using a simulation model as a tool to identify offsite impacts.

## INTRODUCTION

The evaluations of offsite impacts of man's activities on the upstream reaches of a river basin and tributaries to the main course of the river and down stream areas has been a major concern for several decades. As early as 1944, Congress authorized works of improvement on 13 river basins across the U.S. The Washita River basin in Oklahoma was one of those authorized for installation of floodwater-retardation and soil-erosion control measures. The Soil Conservation Service started construction on floodwater-retarding structures, drop inlet structures, flood diversions, channel improvement, and floodways in 1946.

Most evaluations of watershed management programs are difficult and costly procedures, particularly so when using conventional methods for measuring before and after effects. In the early 1960's, a program was established to measure the downstream effects of upstream conservation and protection programs such as the treatment of tributary watersheds of a major river basin with flood control structures installed to reduce floodplain damages. A central reach of the Washita River basin in central Oklahoma was instrumented with rain gages and streamflow and sediment transport measuring stations to determine these effects.

Evaluation of the SCS program's impact based on flow measurements before and after construction and on the responses of treated and untreated adjacent watersheds gave inconclusive results. Therefore, other techniques were employed such as continous simulation modeling of the major watershed processes.

The following is an example of the application and testing of the SWRRB (Simulator for Water Resources on Rural Basins) model (Williams et al.,1985) on treated basins in Oklahoma to determine the off-site impacts of flood control projects.

MODEL APPLICATIONS

The Little Washita River and Winter Creek watersheds located in south central Oklahoma near Chickasha were selected for model testing. These watersheds are tributaries to the Washita River. They are part of an experimental watershed study conducted by the Water Quality and Watershed Research Laboratory, USDA-ARS, Durant, OK (figure 1 and 2). The Little Washita watershed was instrumented with recording raingages (1961), runoff (1962) and sediment (1963) measuring devices, with records being taken continuously since these dates. Winter Creek was instrumented with runoff and sediment measuring devices in 1963 and was discontinued in December 1977. The watersheds lay in two Major Land Resource areas (MLRA), the Reddish Prairie and the Cross Timbers. The drainage areas are 538.2 and 86.25 $km^2$.

Daily rainfall data are available from 33 recording rain gages located on the watershed. Runoff data, both storm and base flow are available on a daily basis. Sediment data, consisting of measured suspended load and estimated bed load are also computed on a daily time step.

The Little Washita River watershed was divided into four areas for model testing purposes ( see figure 1). Area 1, the upper region of the watershed is in the Reddish Prairie MLRA, area 2, is in the central Cross Timber MLRA, area 3, the lower section of the watershed, also represents the Reddish Prairie MLRA but with different soils, and area 4 comprises the large alluvial flood plain. These areas represent 29.7, 56.4, 8.5, and 5.4 percent of the total basin drainage area. Winter Creek watershed was divided into 3 areas (see figure 2). Area 1 represents the upland areas above the flood control structures, area 2, the mid slope section, and area 3, the flood plain aluvial zone. These areas represent 54.3, 38.4 and .07 percent of the total watershed area.

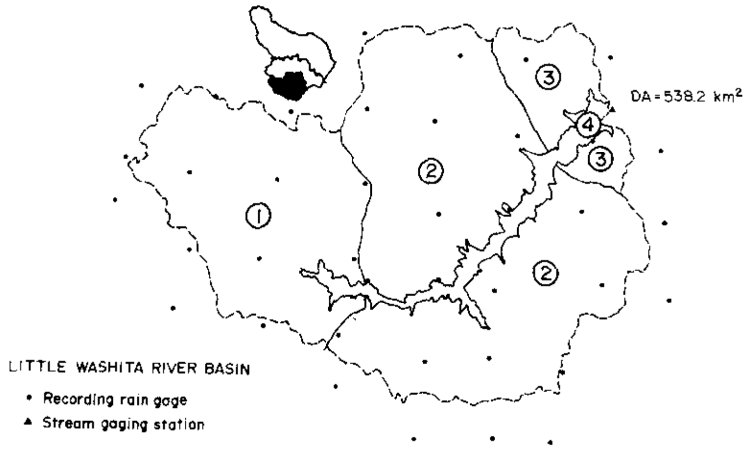


Figure 1. Basin Sub-areas Little Washtia River Watershed

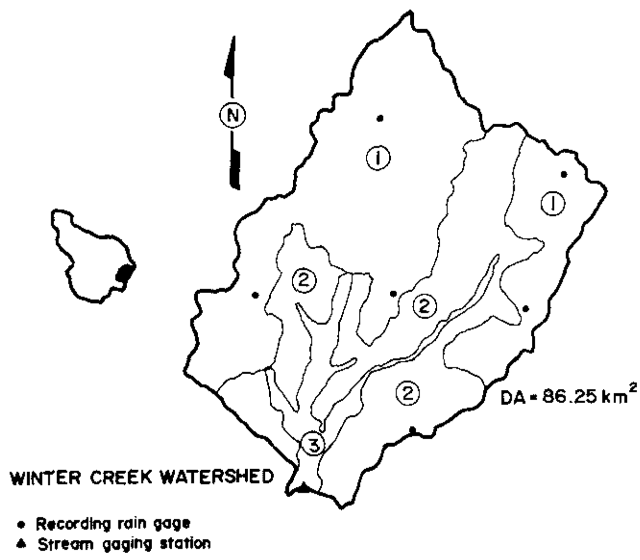


Figure 2. Winter Creek watershed sub-areas.

Soil and land use data were available from Soil Conservation Service surveys (USDA-SCS, 1978; 1978). These data were used with topographic information available from USGS quadrangle maps of the area to develop the parameter set required to run the model. Mean daily rainfall for each of the sub-areas was calculated from the rain gage network data. Climatic data parameters required by the weather generator subroutine were taken from published NWS records (USDC, 1981) for a station near the area.

A survey of farm ponds made from maps and aerial photographs shows that runoff from 19.5% of the drainage area flows into farm ponds on the Little Washita. Also, from late 1969 until 1978 there were 40 SCS flood control structures built on this watershed representing 47% of the drainage area controlled by these structures (figure 3). Land use surveys in 1962 through 1974 showed 18% cropland, 66% rangeland, and 16% miscellaneous or other non-agricultural uses. On Winter creek, 8 structures were built in area 1 (see figure 2) during the period 1965 and 1966. Land use surveys of the watershed in 1962 and 1964 showed land use to be 8.8% cropland and 91.2% rangeland. Runoff from approximately 21 percent of the area was controlled by farm ponds.

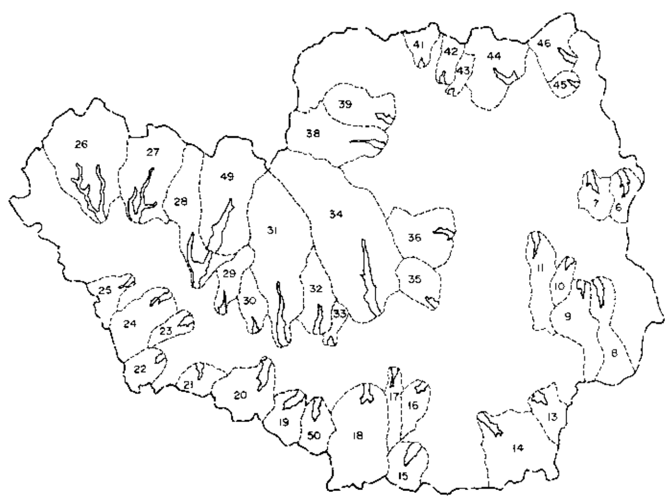


Figure 3. Flood control structures installed 1969-78.

Parameters that were developed from the basic data available for the basins, consist of parameters for sub-basin areas, farm ponds, and reservoir storages from published data (USDA-SCS, 1966). Soil parameters were developed from published soil survey information and soil series sheets. These parameters represent the composited soil data for each basin sub-area. Crop cover factor and Leaf Area Index (LAI) were developed from land use and crop data reports (Wischmier and Smith, 1978; Ritchie, 1972).

Parameter and climate files were developed for the Little Washita watershed, three different model runs were made, and the predicted runoff and sediment yields compared with observed data. The three runs consisted of, 1) a simulated response excluding the present structures for the period 1962 - 1970, 2) a simulation of the watershed's response with the structures installed (1971 - 1981), and 3) a simulated rainfall run with generated multi-gage daily rainfalls. Two model runs were made for the Winter Creek watersheds both for periods during which the structures were installed. However, the first run was made with model parameters selected to reflect the presence of farm ponds, but with no flood control structures in place, and the second, with parameters selected to represent the influence of the structures installed.

## RESULTS and DISCUSSION

Figures 4 and 5 illustrate use of the model for comparing simulated response of the basin to the observed responses of the basin, using installation of flood control structures as a management practice or treatment. The model was run on the Little Washita River basin for the entire period of record assuming no structures had been installed. Next the model was run with the structures in place and functioning from 1971 onward to 1981. Comparing the two runs from 1971 onwards shows the expected cumulative treatment effect of the structures. This differnce in response can be compared with the observed data for 1962 - 1981. The yearly and mean comparisons of these combined runs to observed data are shown in table 1.

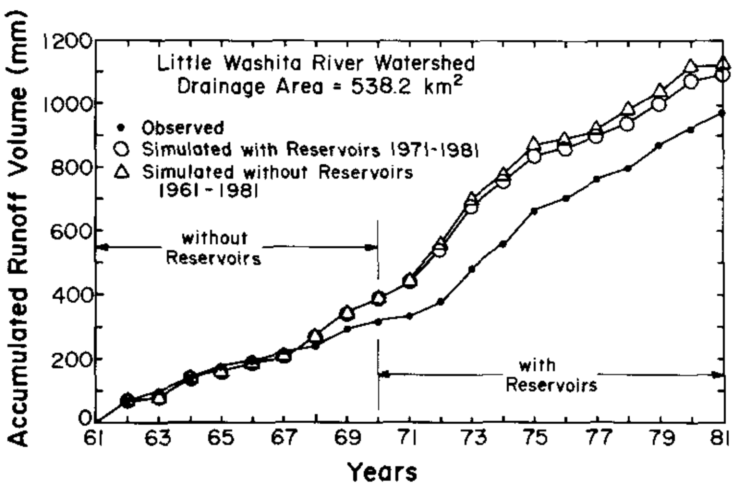


Figure 4. Comparsion of accumulated observed and simulated runoff.

The third test of the model was an assessment of the effectiveness of the model's simulation of basin rainfall as well as runoff and sediment for the Little Washita. Listed in table 2 are the annual amounts of rainfall, runoff, and sediment for 10 eleven-year runs which can be compared to the 11-year observed records with structures installed for the period 1971 - 1981.

While the tests of the model are not exhaustive in scope, they do show the effectiveness of the model in simulating the response of a large complex basin with mixed landuse, numerous water control structures, such as farm ponds and flood control reservoirs, and varied soils. For instance, in figures 4 and 5 the effects of a typical SCS runoff and sediment control program can be seen by comparing the simulated data with observed data from the basin. In figure 4 for the period without structures (1962 - 1971) the simulated and observed flow accumulations are in close agreement. However, runoff is overpredicted for the period 1971 to 1981 when most of the flood control reservoirs were installed. Nevertheless, from 1972 through 1981 the simulated flows follow the observed flows reasonably well. Simulated and observed accumulative sediment yields for the periods before and after installation of the structures follow similar patterns as shown in figure 5.

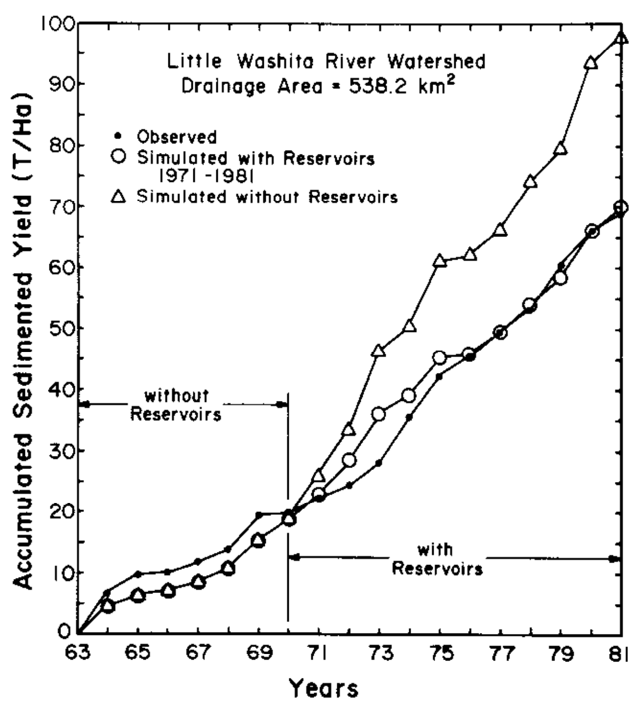


Figure 5. Accumulated, observed and simulated sediment yield.

The importance of these results lies not in the quite small absolute differences between simulated and observed values but in the close correspondence of the simulated response with time to the observed response. Herein lies the value of such models. They provide values on the effectiveness of a range of basin-wide conservation and protection plans. Designers and planners need such information to select an optimum plan, evaluate its effictiveness, and make needed adjustments during the construction phase.

Comparison of observed and model simulations for Winter Creek are listed in table 2. In this application of the model, only a three year period of measurements was available prior to installation of the structures. Therefore, because of the short pretreatment period, these data were not included in the Winter Creek watershed analysis. Winter Creek water yield per unit of area (84.64 mm) is nearly twice as great as the yield for the Little Washita (48.27 mm). But the annual sediment yield is much lower (2.30 t/ha) for Winter Creek than it is for the Little Washita (3.32 t/ha) for the same period (1966-1977). Variabilities of observed and simulated water and sediment yields are high compared to the variability in annual rainfall for the period. The long term yields of water and sediment simulated by the model, with structures in place, agree closely with observed values. These results indicate that the simulated offsite impact of the water control structures on runoff is approximately 6.94 mm/year or an 8% reduction in water yield, and on sediment sediment yield is .65 t/ha/year or a 19% reduction in sediment yield. These relative results would be similar to those expected using the model without calibration on ungaged watersheds.

According to the simulated results one of the benefits of the flood protection program installed on the Little Washita is a reduction in sediment loads of 1.59 tons/ha per year or 29%. Runoff from the basin appears to have been reduced by only 1.87 mm per year or 3%. Without this model such determinations of the flood control plan effectiveness at the basin scale would not have been possible.

The model also performs well when rainfall data are stochastically generated. This feature of the model allows for long term simulation of treatment effects. An example of such an application is shown in table 3. Here, the mean of 10 runs of 11 year length are compared statistically with observed data for the period 1971 - 1981. The mean values of rainfall, runoff, and sediment yields are not significantly different from the observed values at the $t_{.05}$ level.

TABLE 1. Observed and Simulated Water Yield (mm) and Sediment Transport (Tons/ha) Little Washita River Watershed 1962 - 1981 with and without flood control structures.

| YEAR | RAINFALL (mm) | RUNOFF (mm) Obs. | RUNOFF (mm) Sim. with | RUNOFF (mm) Sim. without | SEDIMENT (T/ha) Obs. | SEDIMENT (T/ha) Sim. with | SEDIMENT (T/ha) Sim. without |
|---|---|---|---|---|---|---|---|
| 1962 | 762.64 | 74.72 | 60.84 | 60.84 | | | |
| 1963 | 445.08 | 26.85 | 12.49 | 12.49 | | | |
| 1964 | 776.27 | 42.93 | 68.42 | 68.42 | 6.72 | 4.72 | 4.72 |
| 1965 | 641.77 | 31.67 | 28.37 | 28.37 | 2.91 | 1.50 | 1.50 |
| 1966 | 486.89 | 16.74 | 19.53 | 19.53 | 0.45 | 0.91 | 0.91 |
| 1967 | 659.80 | 17.40 | 26.68 | 26.68 | 1.12 | 1.27 | 1.27 |
| 1968 | 853.78 | 35.20 | 67.21 | 67.21 | 2.69 | 2.55 | 2.55 |
| 1969 | 702.01 | 46.84 | 66.06 | 66.06 | 5.15 | 4.40 | 4.40 |
| 1970 | 513.51 | 17.68 | 27.22 | 27.22 | 0.67 | 3.60 | 3.60 |
| 1971 | 779.53 | 29.57 | 69.80 | 72.65 | 2.46 | 4.35 | 6.89 |
| 1972 | 652.05 | 27.99 | 84.25 | 88.66 | 2.02 | 5.57 | 8.15 |
| 1973 | 1131.22 | 114.45 | 150.84 | 10.99 | 8.96 | 7.77 | 12.03 |
| 1974 | 787.42 | 64.44 | 72.17 | 77.67 | 3.58 | 2.47 | 4.06 |
| 1975 | 928.62 | 112.85 | 93.82 | 97.80 | 7.84 | 7.08 | 11.18 |
| 1976 | 608.19 | 44.78 | 18.77 | 21.75 | 1.12 | 0.52 | 0.82 |
| 1977 | 733.42 | 51.31 | 33.43 | 39.04 | 3.81 | 2.62 | 4.18 |
| 1978 | 669.01 | 49.38 | 43.55 | 44.19 | 4.26 | 5.55 | 7.86 |
| 1979 | 793.47 | 64.77 | 53.89 | 59.27 | 7.17 | 3.46 | 5.53 |
| 1980 | 570.76 | 51.33 | 69.22 | 72.05 | 5.38 | 8.64 | 13.98 |
| 1981 | 911.19 | 44.98 | 50.56 | 54.31 | 2.69 | 2.75 | 4.40 |
| MEAN | 720.33 | 48.29 | 55.86 | 57.76 | 3.83 | 3.87 | 5.45 |
| STD. DEV. | 163.89 | 27.53 | 32.46 | 32.82 | 2.56 | 2.36 | 3.89 |
| COEFF. VAR. | .23 | .57 | .58 | .57 | .66 | .61 | .71 |

TABLE 2. Observed and Simulated Water Yield (mm) and Sediment Transport (tons/ha) Winter Creek Watershed 1966-1977 With and Without Structures.

| YEAR | RAINFALL (mm) | RUNOFF (mm) | | | SEDIMENT (T/ha) | | |
|---|---|---|---|---|---|---|---|
| | | Obs. | Sim. with | Sim. without | Obs. | Sim. with | Sim. without |
| 1966 | 547.09 | 35.31 | 20.09 | 50.54 | .78 | 1.56 | 3.62 |
| 1967 | 680.58 | 38.61 | 33.60 | 44.22 | 1.30 | .26 | .61 |
| 1968 | 858.27 | 62.99 | 70.31 | 74.49 | 2.20 | .86 | 1.07 |
| 1969 | 751.59 | 94.23 | 65.11 | 69.02 | 3.38 | .89 | 1.22 |
| 1970 | 683.86 | 76.45 | 101.82 | 107.12 | 2.78 | 5.50 | 6.36 |
| 1971 | 811.02 | 71.88 | 71.16 | 75.06 | 1.91 | 1.05 | 1.30 |
| 1972 | 674.30 | 76.81 | 113.02 | 118.43 | 1.75 | 5.28 | 5.97 |
| 1973 | 1075.72 | 211.33 | 215.84 | 218.88 | 6.77 | 8.58 | 9.72 |
| 1974 | 141.14 | 97.79 | 80.00 | 84.63 | 1.26 | 2.02 | 2.66 |
| 1975 | 793.23 | 156.21 | 107.88 | 110.86 | 4.28 | 6.42 | 7.10 |
| 1976 | 619.25 | 46.99 | 34.59 | 40.49 | .61 | 1.78 | 2.24 |
| 1977 | 598.40 | 47.24 | 23.84 | 26.29 | .58 | .34 | .52 |
| MEAN | 736.20 | 84.64 | 78.10 | 85.04 | 2.30 | 2.88 | 3.53 |
| STD. DEV. | 140.59 | 51.86 | 54.02 | 51.29 | 1.81 | 2.79 | 3.03 |
| COEFF. VAR. | .19 | .61 | .69 | .60 | .79 | .96 | .86 |

Table 3. Comparison of annual rainfall, runoff volume, and sediment load from 10 simulated runs of 11 year length using the SWRRB model.

| RUN | RAINFALL mm | RUNOFF mm | SEDIMENT T/ha |
|---|---|---|---|
| 1 | 677.7 | 52.27 | 4.547 |
| 2 | 736.8 | 72.88 | 5.485 |
| 3 | 676.2 | 53.93 | 3.301 |
| 4 | 702.4 | 46.76 | 3.469 |
| 5 | 615.1 | 27.92 | 2.093 |
| 6 | 670.7 | 44.84 | 3.707 |
| 7 | 687.9 | 51.33 | 3.349 |
| 8 | 643.8 | 49.42 | 4.163 |
| 9 | 736.9 | 68.48 | 5.964 |
| 10 | 674.7 | 48.90 | 4.148 |
| SIM. MEAN | 682.22 | 51.67 | 4.023 |
| OBS. MEAN | 778.63 | 59.62 | 4.481 |
| DIFF | 96.41 | 7.95 | 0.458 |
| STD MEAN | 324.94 | 44.91 | 3.660 |
| STD DIFF | 102.75 | 14.20 | 1.160 |
| CRIT DIFF | 232.43 | 32.13 | 2.620 |

The simulated average annual rainfall for the period was lower than the observed value for the 11 year period. This resulted from using an average annual rainfall of 682 mm, the observed value for the period from which the generated values were derived, in place of the long-term average annual rainfall for the area, which is 811 mm. Adjustment of the rainfall generating parameters should correct this discrepancy.

A more comprehensive model might give better correlation and better absolute comparison of accumulated data. However, the list of parameters required to reduce such model erros would no doubt exceed the number of parameters used by SWRRB. Comprehensive models commonly require more computer time for simulating system responses and, this of course, would add to the expense of an evaluation program. The added accuracy from such a model may not be warranted when compared to the greater expense and time involved in obtaining a more extensive parametrization of those physical processes that are influenced by man-made changes in land use and climate.

## SUMMARY AND CONCLUSIONS

A basin-scale model (SWRRB) has been developed and tested that will simulate on a daily time step, runoff, peak flow, sediment load, soil water budget, pond and reservoir storage, and sediment budget. The model uses as data input observed daily multiple gage rainfall, or simulated multiple or single-basin gage daily precipitations. Air temperature, and solar radiation inputs are also simulated on a daily basis. SWRRB was developed for a basin or watershed ranging in size from a few hectares to several thousand km**2. The present form of the model allows division of the basin into 5 sub-basin areas and corresponding parameter inputs for each sub-area. It has been tested on basins in Oklahoma, with drainage areas of 538.2 and 88.25 km**2.

These basins were divided into 4 and 3 sub-basins respectively representing different soils, covers, and topographies. Simulated results with and without a flood control project were made which show good correspondence between observed and computed basin responses.

Major conclusions from our results are:

1. This basin-scale model can be used to evaluate the offsite impacts of flood control and conservation programs on large areas.
2. It can be used with readily available data on ungaged basins.

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# PRIORITIZING SURFACE WATER QUALITY PROBLEMS

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## ABSTRACT

A procedure was developed to prioritize potential water quality problems from agricultural watersheds. The procedure simulates flow and transport of sediment, nitrogen, phosphorus, and COD at points of interest within a watershed including the watershed outlet. Single storm events defined in terms of frequency and duration drive the model. The model uses geographic cells of data units at a resolution of 1 to 16 ha to represent upland and channel conditions. Within the framework of geographic cells, runoff characteristics are simulated together with the transport processes of sediment, nutrients, and COD. This framework permits the flow at any point in the watershed to be examined. In this manner, upland sources contributing to a potential problem can be identified and prioritized where remedial measures could be initiated to improve water quality most effectively.

## INTRODUCTION

Runoff from agricultural lands as a potential source of nonpoint pollution has lead to an effort in Minnesota to develop a uniform method of analyzing and prioritizing runoff quality from agricultural watersheds. In the past, inability to analyze pollution problems from different watersheds has caused inconsistencies in directing public funds toward alleviating potential pollution problems. In response to this need, the Minnesota Pollution Control Agency (MPCA), the Minnesota Soil and Water Conservation Board (SWCB), the Soil Conservation Service (SCS), and the Agricultural Research Service (ARS) entered into an agreement to develop models to analyze sediment and nutrient transport within agricultural watersheds. Two models were developed, AGNPS I and AGNPS II, an acronym for <u>ag</u>ricultural <u>n</u>on<u>p</u>oint <u>s</u>ource model. AGNPS I was developed to be used on a mainframe computer for agricultural watersheds ranging in size up to about 12,000 ha. AGNPS II was developed to be used on a programmable calculator for watersheds up to 200 ha.

## MODEL USE

The intended use of the models is to rate the performance of selected agricultural watersheds experiencing the same type of an event. The event could be a design storm occurring at a selected time of the year or it could be any other rainfall event for which the watershed response needed to be analyzed. Analyses could consist of comparisons of runoff, sediment yield, nitrogen and phosphorus yields and concentrations, or chemical oxygen demand yield and concentration.

After a watershed has been identified as needing remedial measures, the models can be used to help assess the effects of alternative management practices. This is accomplished by varying input data consistent with the alternative management practice being investigated and analyzing the resulting watershed responses.

## MODEL STRUCTURE

Both AGNPS models are event-based and are intended to simulate sediment and nutrient transport from agricultural watersheds. The basic model components are hydrology, erosion, sediment and chemical transport. In addition, they consider point sources of sediment from gullies and point sources of nutrients and chemical oxygen demand from animal feedlots. Water impoundments, such as tile-outlet terraces, are also considered as depositional areas of sediment and sediment associated nutrients.

The model operates on a cell basis. Cells are uniform square areas subdividing the watersheds, making analyses possible within the watershed. Potential pollutants are routed through the cells, from the watershed divide to the outlet, in a stepwise manner so that flow at any point may be examined. For watersheds above 800 ha, cell sizes of 16 ha (40 acres) are recommended. Smaller cell sizes are recommended for smaller watersheds. All watershed characteristics and input are expressed at the cell level. To illustrate, Fig. 1 shows a small watershed of 308 ha containing nineteen 16-ha cells. The watershed drainage pattern is shown by the arrows within the cells. The letters within each cell refer to the land use at the time of the storm. The symbols used are C for corn, B for soybeans, SG for small grain, and F for bare soil.

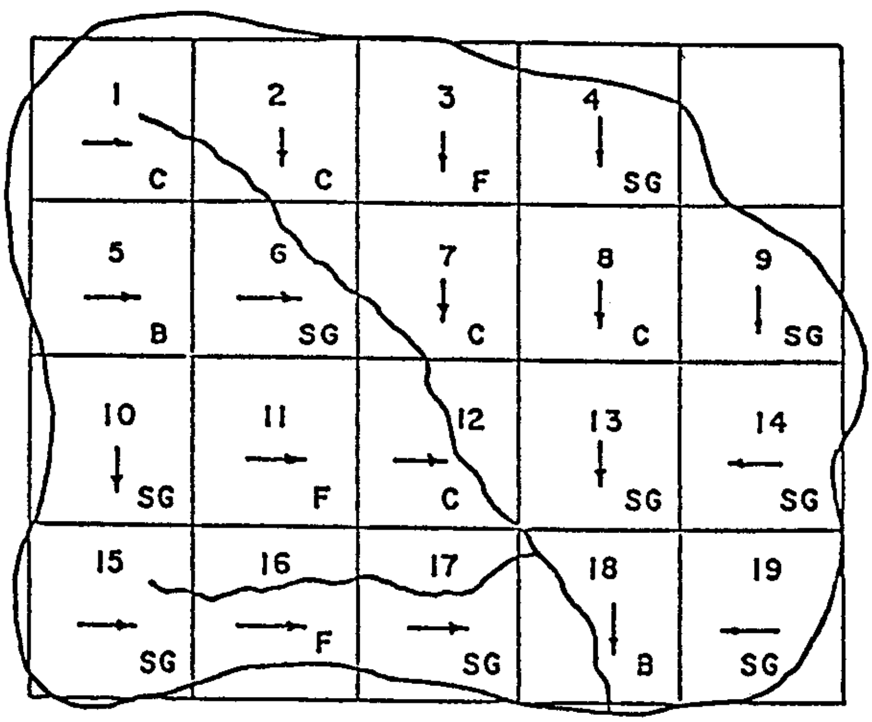


Fig. 1. Sample watershed.

Hydrology

Runoff volume and peak flow rate are calculated in this part of the model. The SCS curve number method (USDA, 1972) is used to estimate runoff volume. The basic equation is

$$Q = \frac{(P-0.2S)^2}{P + 0.8S} \qquad [1]$$

where Q is the runoff volume, P is the rainfall, and S is a retention parameter, all expressed in uniform dimensions of length. The retention parameter is defined in terms of a curve number (CN) by

$$S = \frac{1000}{CN} - 10 \qquad [2]$$

The curve number is a parameter dependent on land use, soil type, and hydrologic soil condition. This method was chosen because of its simplicity and widespread use among the principal user agencies for which the models were developed.

Peak runoff rate for each cell is estimated using an equation from the CREAMS model (Smith and Williams, 1980)

$$Q_p = 3.79A^{0.7}\ CS^{0.16}\ (RO/25.4)^{(0.917\ A^{0.017})}\ LW^{-0.19} \qquad [3]$$

where $Q_p$ is the peak flow rate in $m^3s^{-1}$; A is the drainage area in $km^2$; CS is the channel slope in m/km; RO is the runoff volume in mm; and LW is the watershed length-width ratio which is calculated by $L^2/A$ where L is the watershed length. Equation [3] was tested using data from 20 upper Midwest watersheds. The regression analysis indicated that estimated peak flow was consistently underestimated by slightly more than 1 percent. The coefficient of determination for the relationship was 0.81.

Erosion

A modified form of the univeral soil loss equation (Wischmeier and Smith, 1978) is used to estimate upland erosion for single storms. The equation used is:

$$SL = (EI)\ KLSCP\ (SSF) \qquad [4]$$

where SL is the soil loss; EI is the product of the storm total kinetic energy and maximum 30-min. intensity, K is the soil erodibility factor; LS is the topographic factor; C is the cover and management factor; P is the supporting practice factor; and SSF is a factor to adjust for slope shape within the cell. The erosion equation factors are calculated using procedures found in USDA Handbook No. 537 (Wischmeier and Smith, 1978). Soil loss is calculated for each cell of the watershed. Eroded soil and sediment yield are subdivided into five particle size classes--clay, silt, small aggregates, large aggregates, and sand.

### Sediment Transport

Following detachment, sediment is routed from cell to cell throughout the watershed. The procedure used involves the relations described by Foster, et al. (1981) and Lane (1982). The basic routing equation is derived from the steady state continuity equation.

$$Q_s(x) = Q_s\,(0) + Q_{sl}\,(x/L) - \int_0^x D(x)\,w\,dx \qquad [5]$$

where $Q_s(x)$ is the sediment discharge at the downstream end of the channel reach; $Q_s(0)$ is the sediment discharge into the upstream end of the channel reach; $Q_{sl}$ is the lateral sediment inflow rate; x is the downstream distance; L is the reach length; w is the channel width; and D(x) is the deposition rate. Deposition rate is estimated as:

$$D(x) = V_{ss}/q(x)[q_s(x) - g'_s(x)] \qquad [6]$$

where $V_{ss}$ is the particle fall velocity; q(x) is the discharge per unit width; $q_s(x)$ is the sediment load per unit width; and $g'_s\,(x)$ is the effective transport capacity per unit width. Effective transport capacity is computed using a modification of the Bagnold stream power equation (Bagnold, 1966)

$$g'_s = \eta g_s = \eta k\,\frac{\tau v^2}{V_{ss}} \qquad [7]$$

where $g_s$ is the transport capacity; $\eta$ is an effective transport factor; k is the transport capacity factor; $\tau$ is the shear stress; and v is the average channel flow velocity. Values for the effective transport capacity are described by Young et al. (1986). Sediment load for each of the five particle size classes leaving a cell is calculated by:

$$Q_s\,(x) = \left[\frac{2\,q(x)}{2q(x)+\Delta x V_{ss}}\right] \left[Q_s(o)+Q_{sl}\,\frac{x}{L} - \frac{w\Delta x}{2}\left[\frac{V_{ss}}{q(o)}\,(q_s(o) - g'_s(o)) - \frac{V_{ss}}{q(x)}\,g'_s(x)\right]\right] \qquad [8]$$

Equation [8] is the basic routing equation that drives the sediment transport model. The five particle classes are clay, silt, sand, small aggregates, and large aggregates (Foster, et al., 1981).

### Chemical Transport

The chemical transport part of the model estimates the transport of nitrogen (N), phosphorus (P), and chemical oxygen demand (COD) throughout the watershed. N and P are essential plant nutrients and major contributors to pollution of surface waters. COD is a measure of the amount of oxygen required to oxidize organic and inorganic compounds in water. As such it is used as an indicator of degree of pollution. The relationships used to calculate chemical transport are from the CREAMS model (Frere, et al., 1980) and a feedlot evaluation model (Young, et al., 1982) with some modifications for the effects of soil texture.

Chemical transport calculations are divided into soluble and adsorbed phases. Nutrient yield in the adsorbed phase is calculated using sediment yield from a cell as

$$Nut_{sed} = (Nut_f)\ Q_s\ E_R \qquad [9]$$

where $Nut_{sed}$ is N or P transported by sediment; $Nut_f$ is N or P content in the field soil; and $E_R$ is the enrichment ratio calculated from

$$E_R = 7.4\ Q_s^{-0.2}\ T_f \qquad [10]$$

where $T_f$ is a correction factor for soil texture (Young, et al., 1985) and $Q_s$ is sediment yield.

Estimation of soluble nutrients considers the effects of nutrient levels in rainfall, fertilization, and leaching. Soluble nutrients contained in runoff are estimated by

$$Nut_{sol} = C_{nut}\ Nut_{ext}\ Q \qquad [11]$$

where $Nut_{sol}$ is the concentration of soluble N or P in the runoff; $C_{nut}$ is the mean concentration of soluble N or P at the soil surface during runoff; $Nut_{ext}$ is the extraction coefficient of N and P for movement into runoff; and Q is the total runoff.

COD in the models is assumed soluble. Estimates of COD in runoff are based on runoff volume and average concentrations. Background concentrations of COD available in the literature are used as a basis for predicting COD concentrations from each cell. Soluble COD is assumed to accumulate without any losses.

Point Source Inputs

The models treat nutrient and COD contributions from animal feedlots as point sources and routes them with contributions from diffuse sources. Chemical contributions from feedlots are estimated using the feedlot pollution model developed by Young et al. (1982) as a subroutine. The feedlot model estimates nutrient concentrations and mass at both the feedlot edge and at a receiving body of water.

Streambank and gully erosion are accounted for using estimated values as point sources. Sediment from these sources is added to upland sediment and considered in the transport phase of the models.

Sediment and runoff routing through impoundments is accomplished using relationships described in CREAMS (Laflen, et al., 1978). These relations were developed for impoundment terrace systems having pipe outlets. The fractions of each particle class passing through the impoundment is a function of surface area, depth, diameter of the pipe outlet, and the infiltration rate. In terms of water quality, impoundments decrease peak discharges, sediment yield, and yield of sediment laden chemicals.

## MODEL INPUT AND OUTPUT

Table 1 shows the inputs for AGNPS. The parameters may be obtained from published data or from on site inspection. The model users guide (Young, et al., 1985) contains tables listing standard variables for the required parameters.

Table 1. Input data file.

| No. | Watershed Input |
|---|---|
| 1 | Watershed identification |
| 2 | Cell area |
| 3 | Total number of cells |
| 4 | Precipitation amount |
| 5 | Energy-intensity value |

| No. | Cell Parameter |
|---|---|
| 1 | Cell number |
| 2 | Number of the cell into which it drains |
| 3 | SCS curve number |
| 4 | Average land slope (%) |
| 5 | Slope shape factor (uniform, convex, or concave) |
| 6 | Average field slope length |
| 7 | Average channel slope (%) |
| 8 | Average channel side slope (%) |
| 9 | Mannings roughness coefficient for the channel |
| 10 | Soil erodibility factor (K) from USLE |
| 11 | Cropping factor (C) from USLE |
| 12 | Practice factor (P) from USLE |
| 13 | Surface condition constant (factor based on land use) |
| 14 | Aspect- (one of 8 possible directions indicating the principal drainage direction from the cell) |
| 15 | Soil texture (sand, silt, clay, peat) |
| 16 | Fertilization level (zero, low, medium, high) |
| 17 | Incorporation factor (% fertilizer left in top centimeter of soil) |
| 18 | Point source indicator (indicates existence of a point source input within a cell) |
| 19 | Gully source level (estimate of amount of gulley erosion in a cell) |
| 20 | Chemical oxygen demand factor |
| 21 | Impoundment factor (a factor indicating presence of in impoundment terrace system within the cell) |

Several output options are available. Every option contains values of watershed area, cell size, precipitation amount, erosivity value, runoff volume and peak flow rate, area weighted erosion, calculated sediment delivery ratio, sediment enrichment ratio, mean sediment concentration, and sediment yield. These values can be given for each of the five particle sizes. A nutrient analyses output includes N, P, and COD mass per unit area for both soluble and sediment adsorbed chemicals and the N, P, and COD concentrations in the runoff.

Table 2 shows the output parameters that can be printed for every cell if desired. This has been sufficient for all watershed analyses performed to date. Cell selection for output depends on the problem to be analyzed. For example, if a stream reach needs to be protected for trout habitat, those cells having outlets to the reach need to be examined. If an impoundment at the watershed outlet needs protection then output for only the outlet needs

Table 2. Watershed output at the outlet for any cell.

<u>Hydrology Output</u>

- Runoff volume
- Peak runoff rate
- Fraction of runoff generated within the cell

<u>Sediment Output</u>

- Sediment yield
- Sediment concentration
- Sediment particle size distribution
- Upland erosion
- Amount of deposition
- Sediment generated within the cell
- Enrichment ratios by particle size
- Delivery ratios by particle size

<u>Chemical Output</u>

- Nitrogen
  - Sediment associated mass
  - Concentration of soluble material
  - Mass of soluble material
- Phosphorus
  - Sediment associated mass
  - Concentration of soluble material
  - Mass of soluble material
- Chemical Oxygen Demand
  - Concentration
  - Mass

to be examined. If a problem exists, upland cells need to be selected and examined to identify and prioritize problem areas where management alternatives can be evaluated.

AGNPS has been programmed in Fortran for both the HP-1000[1] and the IBM-PC. It has also been programmed in "reverse Polish" notation for the HP-41CV hand-held calculator. The largest watershed that has been analyzed on the HP-1000 is one containing 3052 cells which required about 20 minutes or about 150 cells/min. Computational efficiency increases as watershed size decreases because bookkeeping requirements decrease for smaller watersheds. The IBM-PC with two floppy disc drives and 256K memory can handle watersheds up to about 700 cells. Computational efficiency is much lower at about 6-8 cells/min, but the costs are also much lower. Although the model can be run on the HP-41CV for watersheds up to 50 cells, it takes a long time. For watershed containing over 20 cells, it is recommended that the model be run on a computer.

## TESTING

Watershed data are being collected in Minnesota for testing the AGNPS model. Parts of the model have been preliminarily tested with data from two experimental watersheds near Treynor, Iowa, and another near Hastings, Nebraska. The data available were runoff and sediment yield. Statistical comparison of the observed and estimated sediment yield at Treynor showed that the model over-predicted by about 2 percent with a coefficient of determination of 0.95. Similar tests using the Hastings data resulted in a coefficient of determination of 0.76.

A sensitivity analysis involving 14 parameters has been done. The most sensitive parameters affecting sediment and chemical yields were land slope, soil erodibility, cover-management factor, and the curve number. Therefore, accurate values for these parameters are needed to obtain satisfactory predictions of upland soil and chemical loss.

## SUMMARY

A procedure called AGNPS has been developed to prioritize water quality from agricultural watersheds in Minnesota. The procedure is based on single storm events defined in terms of frequency and duration and is intended for use on watersheds ranging in size up to 12,000 ha. The model uses geographic cells of data units at a resolution of 1 to 16 ha to simulate flow and transport of sediment, nitrogen, phosphorus, and COD. The framework permits the flow at any point in the watershed to be examined. In this manner, upland sources contributing to a problem can be identified and prioritized where remedial measures might be initiated to improve water quality.

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[1]Trade names and company names are included for the benefit of the reader and do not imply any endorsement or preferential treatment of the product listed by the U. S. Department of Agriculture.

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# SELECTED FUNCTIONS FOR SEDIMENT-TRANSPORT MODELS

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## ABSTRACT

Using data for the Mississippi River, several functions that describe the erosion and deposition processes of suspended sediment in rivers were analyzed. These erosion and deposition functions were programmed as subroutines of a one-dimensional Lagrangian transport model. Flow fields for transport modeling were generated using a one-dimensional flow model. The transport model was calibrated and verified with six sets of suspended-sediment data collected from a 295.6-mile study reach of the lower Mississippi River. Shear-stress functions and a reference equilibrium-concentration function successfully modeled the suspended sediment, but a bed-load layer dependence function was unsuccessful. This analysis will improve the modeler's ability to simulate suspended-sediment transport for many practical applications.

## INTRODUCTION

Several approaches for numerically modeling the movement of suspended sediment are available. Ideally, these methods should determine the rate of change of suspended sediment instead of an equilibrium concentration of suspended sediment to allow for prediction of suspended sediment in nonuniform or unsteady flows in rivers and estuaries. Steady flow and suspended-sediment data from the lower Mississippi River were used in the one-dimensional LTM (Lagrangian transport model) to analyze algorithms that calculate the rate of change of suspended-sediment concentrations.

Three approaches were analyzed for use in a predictive suspended-sediment transport model. The first approach used erosion and deposition functions dependent on shear stress. The second approach used a reference equilibrium-concentration function to determine the rate of change of suspended sediment. The third approach calculated the rate of change of suspended sediment based on the concentration of sediment in the bed-load layer. Each approach that was calibrated at low, medium, and high flow was verified with additional data representing the same three ranges of flow.

A field study was conducted, in cooperation with the Louisiana Department of Transportation and Development, Office of Public Works, to obtain detailed information on the flow, suspended-sediment transport, and water-quality characteristics in the 295.6-mile reach of the lower Mississippi River from Tarbert Landing, Miss. to Venice, La. (Fig. 1). A summary of the data used for suspended-sediment transport modeling is shown in Table 1. Three ranges of flow--low, medium, and high--were used for calibration and verification of the algorithms. Instantaneous sampling at each cross section along the river was not possible because the length of time to complete each sampling trip ranged from 3 to 5 days. Sampling trips were conducted at or as close as possible to steady-state conditions, so a steady state assumption is reasonable. Sampling sites were uniformly distributed along the study reach and numbered from 15 to 50.

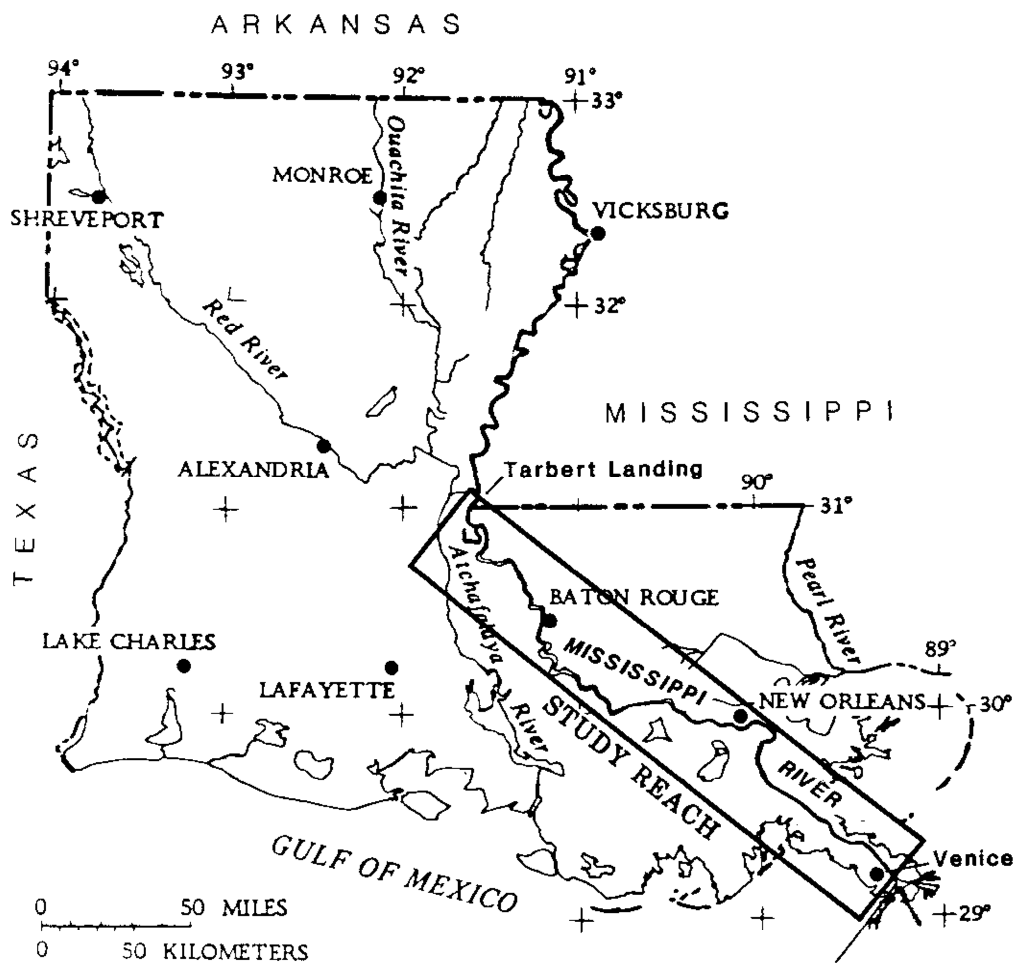


Figure 1.--Location of study reach.

Table 1.--Summary of modeling data

| Trip date: Beginning | Trip date: Ending | Trip length (days) | Discharge ($m^3/s$) | Water temperature (°C) | >63 μm $d_{50}$ (mm) | ≤63 μm $d_{50}$ (mm) | Number of samples |
|---|---|---|---|---|---|---|---|
| | | | Calibration | | | | |
| 11-14-83 | 11-18-83 | 4 | 7,567 | 15.0 | 0.122 | 0.00232 | 50 |
| 11- 5-84 | 11- 9-84 | 4 | 15,070 | 18.5 | .124 | .00678 | 22 |
| 4-30-84 | 5- 4-84 | 5 | 28,070 | 16.0 | .135 | .00194 | 42 |
| | | | Verification | | | | |
| 10-15-84 | 10-19-84 | 4 | 7,349 | 22.0 | 0.137 | 0.00267 | 30 |
| 12- 3-84 | 12- 7-84 | 4 | 19,210 | 9.5 | .140 | .00359 | 15 |
| 1- 7-85 | 1-10-85 | 3 | 25,450 | 9.0 | .118 | .00488 | 15 |

Suspended-sediment samples were collected using a collapsible-bag sampler similar to that described by Stevens and others (1980). The collapsible-bag sampler is designed to collect depth-integrated samples in rivers with flow depths greater than 4.5 meters. The sampler is based on the isokinetic principal. A Price AA current meter is mounted directly above the sampler on a standard hanger bar. A 136-kilogram sounding weight is mounted below the sampler to permit use in high velocities. Placing the sounding weight below the sampler leaves an unsampled zone of 0.46 meter.

Three to five depth-integrated verticals per cross section were taken at equal discharge increments using a vertical transit rate that was no greater than 0.4 of the mean velocity. Constant vertical transit rates were maintained using a hydraulic winch specifically developed for this study (Rodman and Wheat, 1985). The sampler was calibrated at each vertical to assure representative samples were being collected. The sampler nozzle velocity was calculated from the nozzle area, sampling time, and sample volume. Field calibration of the hydraulic efficiency of the nozzle averaged about 0.90 for stream velocities between 1-2 meters per second. Concentration errors for these measured nozzle efficiencies were on the high side and generally less than 5 percent.

Data were collected at several sites to determine concentrations and particle-size distributions of two size fractions. The 63-μm (micrometer) size was used to divide the samples into sand (>63 μm) and fine (≤63 μm) fractions. Raw samples were split in the field by pouring the sample through a U.S. standard No. 230 sieve. The total particle-size distribution was determined at eight sites. Discharge measurements were made at the beginning of each sampling trip and generally at the end. A few discharge measurements also were made at intermediate sites. The moving-boat method of discharge measurement (Rantz and others, 1982) was used. Additional hydraulic information necessary for transport modeling purposes was computed using one-dimensional flow-modeling techniques.

## SEDIMENT-TRANSPORT MODEL

The LTM (Jobson, 1981) was used to simulate suspended-sediment concentrations in the lower Mississippi River. The LTM divided the river into completely mixed parcels of water which moved downstream at the mean cross-sectional velocity. Dispersion coefficients for the lower Mississippi River were determined by calibrating the LTM with data from two previous time-of-travel dye studies (Stewart, 1967; Martens and others, 1974). Each of the various methods for simulating suspended sediment that is presented in this paper was coded within the existing LTM framework.

## EROSION AND DEPOSITION FUNCTIONS

Three different formulations of erosion and deposition functions were tested on both the sand and fine suspended-sediment size fractions of the lower Mississippi River. The first form used erosion and deposition functions dependent on shear stress. The second form calculated the rate of change of suspended-sediment concentration based upon a reference equilibrium concentration. Finally, the third form related the suspended-sediment concentration to the concentration of sediment in the bed-load layer. The supply of bed sediment was assumed to be unlimited for all three approaches.

### Shear-Stress Functions

The shear-stress functions presented herein were used originally to describe the erosion and deposition of cohesive sediments (Krone, 1962; Partheniades, 1962). Although the fine sand is noncohesive and the silt and clay fraction may be only slightly cohesive, the shear-stress functions provide a general expression for the behavior of any suspended sediment. The deposition rate is

$$\left.\frac{\partial C}{\partial t}\right|_{\text{deposition}} = \frac{-P_w C}{h} = -\left(1 - \frac{\tau}{\tau_{cd}}\right)\frac{wC}{h} \qquad (1)$$

where P is the probability of adherence, h is the depth, $\tau$ is the bed shear stress, $\tau_{cd}$ is the critical bed shear stress for deposition, and w is an effective fall velocity determined by calibration. No deposition is assumed to occur for shear stresses above the critical value (Krone, 1962). The rate of erosion is

$$\left.\frac{\partial C}{\partial t}\right|_{\text{erosion}} = \frac{M}{h}\left(\frac{\tau}{\tau_{ce}} - 1\right) \qquad (2)$$

where M is an erosion constant and $\tau_{ce}$ is the critical shear stress for erosion which is greater than or equal to the critical shear stress for deposition (Partheniades, 1962). Hayter's (1983) assumption that deposition can only occur in decelerating flows and erosion can only occur in accelerating flows also was made in this study when applying these shear-stress functions.

Reference Equilibrium-Concentration Function

The basic concept of routing sediments, using an equilibrium concentration concept where suspension is the mode of transport of interest, was developed by Lin and others (1983) and Lin and Shen (1984). The rate of change of suspended-sediment concentration is related to an equilibrium concentration as

$$\frac{\partial C}{\partial t} = \frac{\lambda w}{h}(CEQ-C) \tag{3}$$

in which $\lambda$ is a coefficient to be evaluated in accordance with the conditions prevailing in the river and CEQ is the concentration of sediment corresponding to an equilibrium state of sediment transport. An equilibrium state occurs when the sediment discharge through a cross section is equal to the sediment transport capacity of the flow as dictated by U, w, h, and other factors.

CEQ can be given by empirical or theoretical formula established for the river. The form used successufully by Lin and others (1983) was used in this analysis:

$$CEQ = \frac{kU^2}{h} \tag{4}$$

where k is determined from the field data. The coefficient k is an overall coefficient representing many factors such as size of sediment, flow conditions, turbulence, sinuosity, and many others.

Bed-Load Layer Dependence Function

In practice, the transport of the total sediment load usually is divided into two layers, bed load and suspended load. Bed load is the movement of sediment on the bed or in a layer near the bed below a plane some small distance a' from the bed. Sediment that is transported in the layer above the plane located at a' is defined as suspended load. Exchange between the layers can occur through the lower boundary and is due to gravitational settling and turbulent mass transfer. The sediment flux through the lower boundary can be written:

$$q_s = -wc + K_v \frac{\partial c}{\partial y} \tag{5}$$

where c is the concentration of sediment as a function of distance y, from the bed and $K_v$ is a vertical turbulent diffusion coefficient at a'. Then if we assume that the turbulent mass transfer is proportional to the difference between the depth-integrated concentration, C, in the suspended-load layer, and the concentration in the bed-load layer, termed $C_a$, we can write:

$$\frac{\partial C}{\partial t} = -\frac{wC}{h} + \frac{k'}{h}(C_a-C) \tag{6}$$

where k' would be a mass transfer coefficient, and treated as a calibration coefficient.

Evaluation of the concentration of sediment, $C_a$, in the bed-load layers is necessary. Two methods were considered. The first used an equilibrium bed-load function that was recently developed by van Rijn (1984a, 1984c). A relatively accurate bed-load equation based on a dimensionless particle parameter and a dimensionless transport-stage parameter was determined by van Rijn (1984a). The second method used for calculating $C_a$ was to first calculate the erosion rate, E, from a number of formulas for this purpose (van Rijn, 1984b). Then $C_a$ was estimated assuming equilibrium is reached between the erosion and deposition in the layer near the bed:

$$C_a = \frac{E}{w} \tag{7}$$

## RESULTS OF THE SEDIMENT-TRANSPORT MODEL

The suspended-sediment algorithms were tested with the lower Mississippi River data. The LTM was run with a steady flow and a constant upstream boundary condition. The model was run using a half-hour time step until the first parcel exited the reach. Results were printed at the last time step and compared to the observed data. Data collected in November 1983, November 1984, and May 1984 were used for calibration which represent low, medium, and high flows, respectively. Calibration was performed by visually fitting the LTM results to the observed data. Verification was accomplished with data from October 1984 for low flows, December 1984 for medium flows, and January 1985 for high flows.

### Shear-Stress Functions

Calibration of equations 1 and 2 was accomplished for both size fractions by using the coefficients shown in Table 2. The critical shear stresses for erosion ($\tau_{ce}$) and deposition ($\tau_{cd}$) were assumed to be equal, and equal within each size fraction ($\tau_c$). The critical shear stresses were adjusted until erosion and deposition occurred where expected, and then the other coefficients were adjusted to approximate the actual erosion and deposition rates. The calibrated critical shear stresses were within the range of shear stresses at which suspended sediment has been observed to be transported with negligible net erosion or deposition (Wells, 1980). Hayter's (1983) assumption that erosion occurs only in accelerating flows and deposition occurs only in decelerating flows significantly improved the accuracy of the solution.

The verification results of the shear-stress functions are shown in Figure 2 for fines and Figure 3 for sands. The verification was generally successful. The one poor verification result, for sand in January 1985, was caused by a high value of measured sand at Tarbert Landing, Miss., that is used as the upstream boundary condition in the LTM. The computed profile behaves like the measured profile and even approaches it downstream, but the high sand concentration at the LTM upstream boundary causes the entire computed profile to be too high. This result shows the importance of having an accurate boundary condition for transport modeling.

## Reference Equilibrium-Concentration Function

The equilibrium-concentration approach, using equations 3 and 4, was calibrated successfully using the coefficients listed in Table 2. The k coefficient varied for the different flow conditions. A constant $\lambda$ coefficient was applied for all flow conditions. Neither coefficient was varied in the longitudinal direction. The k coefficient was estimated first for low flow, and then increased in proportion to discharge for other flow conditions. Additional minor adjustments were made to this coefficient to produce erosion and deposition, as expected. The magnitude of the rate of change of erosion and deposition was regulated by the value of the $\lambda$ coefficient. The value of $\lambda$ that produced only gradual changes in concentration was used.

Table 2.--Calibration coefficients for suspended-sediment transport models

| Date | Discharge ($m^3/s$) | Flow condition | $\tau_c$ ($N/m^2$) | | M ($mg \cdot m/L \cdot h$) | | w (m/h) | | k | | $\lambda$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Sand | Fines | Sand | Fines | Sand | Fines | Sand | Fines | Sand | Fines |
| 11-83 | 7,567 | Low | 5.7 | 1.6 | 90 | 9 | 1.40 | 0.50 | 0.10 | 1.0 | 100 | 10,000 |
| 11-84 | 15,070 | Medium | 5.7 | 1.6 | 90 | 9 | 1.40 | .50 | .50 | 2.5 | 100 | 10,000 |
| 5-84 | 28,070 | High | 5.7 | 1.6 | 90 | 9 | 1.40 | .50 | .75 | 4.0 | 100 | 10,000 |

The calibrated coefficients were used for verification simulations with no additional adjustments. The results are shown in Figures 2 and 3. The observed data are matched, with the exception of the fine-concentration simulations for January 1985 and October 1984. Sand-concentration results matched better overall than did fine-concentration results. The model slightly underpredicted deposition at low flow and overpredicted erosion at high flow. One reason the algorithm overpredicted erosion of fine material for high flow is that the assumption of unlimited supply of sediment may be invalid. Fine material will be easily transported at lower flows in the beginning of a rise in stage, thus, limiting the supply available for peak flow and recession falling limb of the hydrograph.

## Bed-Load Layer Dependence Function

van Rijn's (1984c) bed-load layer concentration and equation 6 could not be satisfactorily calibrated with the suspended-sediment data. A successful calibration for one discharge failed to be applicable to other discharges. The calibration apparently failed because neither $C_a$ nor a' were consistently reliable. This unsatisfactory result is not unexpected because bed-load functions generally are accurate only to within a factor of two (van Rijn, 1984a).

The erosion functions used to determine the bed-load layer concentration with equation 7 also could not be satisfactorily calibrated with the suspended-sediment data. All the functions could be calibrated for one set of data, but they could not be calibrated such that they could

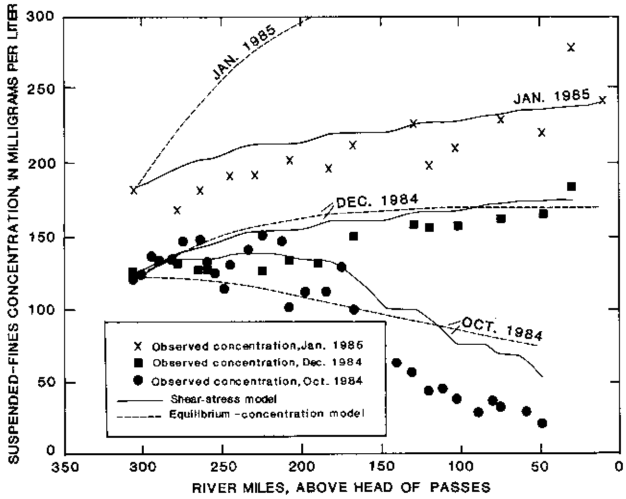


Figure 2.--Results of modeling the fine fraction of suspended sediment of the lower Mississippi River.

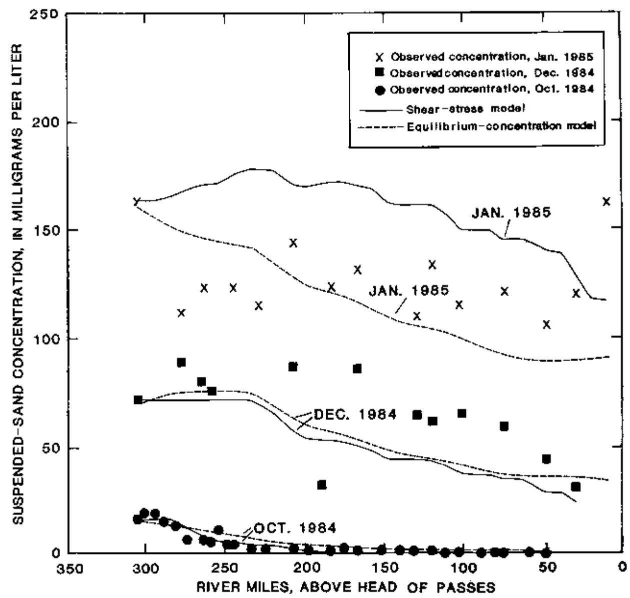


Figure 3.--Results of modeling the sand fraction of suspended sediment of the lower Mississippi River.

reproduce data at all three flow regimes. In all instances, unreliable calculated bed-load layer concentrations apparently were the primary reason that calibration failed. All of the erosion functions were tested by van Rijn (1984b) in a laboratory flume which may not represent the erosion properties of the lower Mississippi River.

The possibility that the calibration failures were due to equation 6, which relates the suspended load to the bed load, was investigated by making two separate modifications. The mass-transfer coefficient, k', was assumed to be constant, and the depth at which the vertically variable concentration was assumed equal to the mean concentration was varied. Neither modification, however, improved the solutions.

## DISCUSSION OF MODELING RESULTS

The shear-stress approach was successful because it accurately simulates the general behavior of any suspended sediment. Although this approach is less physically realistic than the bed-load layer dependence algorithm, it is more useful as a predictive modeling technique because the physical processes at the bed cannot be accurately quantified. The reference equilibrium-concentration approach was almost as successful as the shear-stress approach, but the empirical formulation used for estimating the equilibrium concentration detracts from the utility of the equilibrium-concentration approach for predictive calculations. Additional investigation into the physical interpretation of the coefficients of this approach will make it more attractive for predictive modeling. Alternatively, one could use a different theory to estimate equilibrium concentrations. The existing LTM framework would allow the modeler to readily incorporate any new theory of equilibrium concentration or a new suspended-sediment transport approach.

Modeling suspended sediment as a function of the bed-load layer concentration, although realistic, is not accurate enough for use as a predictive tool on the lower Mississippi River. This problem is due to bed-load formulas that are, at best, accurate only to within a factor of two and generally inapplicable to the fine sediments of the lower Mississippi River. In addition, the use of average cross-sectional flow properties in the formulas may not produce accurate cross-sectionally averaged bed-load layer concentrations, and the algorithm used to relate suspended-sediment concentration to bed-load layer concentration may compound the error. The inaccuracy of the bed-load layer concentration calculation would prevent development of an accurate predictive suspended-sediment transport model even if a perfectly accurate method to relate bed-load layer and suspended-sediment concentrations were developed.

Splitting the existing fine-size fraction into two separate sub-fractions may improve the simulation of suspended-sediment transport in the lower Mississippi River. The existing fine-size fraction contains some wash load, which is generally defined as the conservative portion of the suspended load. Wash load is governed by the upstream supply rate only and not by the composition of the bed material or the river hydraulics. No sharp division between wash load and bed-material load exists. The 63-μm size break is not a satisfactory demarcation for the data of this river because both sands and fines are deposited at low flows and resuspended or eroded at high flows. Splitting the fine-size fraction into silt and clay fractions probably would improve the modeling results. In this way, the effects of not being able to sort the wash load would be reduced.

## CONCLUSIONS

The goal of the present analysis was to determine an approach for modeling suspended sediment in the lower Mississippi River. Both the shear-stress approach and the reference equilibrium-concentration approach were satisfactory, but the bed-load layer dependence approach was not successful in this study. Additional investigation into the physical interpretations of the coefficients of the shear-stress and reference equilibrium-concentration approaches would improve the predictive capability of each approach.

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SMALL WATERSHED MODEL (SWAM)

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## ABSTRACT

The Small Watershed Model (SWAM) is a state-of-the-art model that simulates the movement of water, sediment, and chemicals through a small, mixed-land-use watershed. It was developed to aid planners and others in assessing non-point sources of pollution. To the maximum extent practical, the physical processes are simulated using causal rather than black-box approaches. The watershed is divided into source area, channel, reservoirs, and groundwater segments within which water, sediment and chemicals move, interacting with each other and with the agricultural environment. Agricultural land use and management practices as well as daily changes in vegetative cover and infiltration rates are simulated in source areas, and the state of other model variables are simulated throughout the catchment system.

## INTRODUCTION

Development of SWAM was initiated by the Agricultural Research Service (ARS), USDA in response to U.S. Public Law 92-500, the clean water act. The model is designed to provide action agencies and planners with a means to estimate impact of alternative agricultural management on nonpoint-source pollution from small watersheds.

General objectives of SWAM are (Alonso and DeCoursey, 1985):

1. To produce accurate representation of watershed response when influenced by surface configuration, channels, reservoirs, and groundwater.
2. To provide accurate representation of the physical and chemical processes without curve fitting or subjective distortion due to lumping in space or time.
3. To provide a model that would be a reliable data generator for use in developing simpler application versions and a basin scale model.

The development history of SWAM and related model development efforts (CREAMS, Opus) are described in Seely, et al. (1986). That same paper also gives more complete description of percolation and chemical transport simulation in SWAM than is given in this paper.

## MODEL OVERVIEW

SWAM simulates the simultaneous movement and mutual interaction of water, sediment, nutrients and pesticides through the watershed environment. The model is designed to show the effect of changes in land use or management on the hydrologic, sediment and chemical response of agricultural areas--typically not greater than about 25 $km^2$; it is a continuous simulation system driven by precipitation and other meteorological inputs.

SWAM respresents a watershed by using a network of combinations of four different segments; the output from each segment is the input to another segment. The types of segments include: source areas (the various fields in the watershed), channels, reservoirs, and groundwater segments. Their characteristics are discussed in greater detail below. The process of representing a watershed by a network of segments is described in Alonso and DeCoursey (1985).

Source Area Processes: Source area segments are modeled using a continuous simulation, distributed field-scale model, Opus, which is a second generation, major revision of CREAMS. This submodel simulates the partitioning of precipitation input--including snowmelt--into infiltration and runoff; movement of water in the root zone, overland, and through first order channels; erosion and sediment transport in the field; a variety of water quality processes including pesticide movement, nutrient cycling and transport; soil heat flow; crop growth and residue decay; irrigation additions, and draintile outflow. A wide variety of agricultural management practices are simulated. They include crop rotations, different types of tillages, irrigation, application of fertilizer and animal wastes, grazing, application of pesticides, tile drainage, farm ponds, terracing and buffer strips. More information on the model is presented in Smith and Knisel, (1985) and Smith and Ferriera (1986).

Channel Processes: Longitudinal profiles and flow distributions are computed for each channel segment using a diffusive wave scheme for impounded situations and a kinematic wave scheme for low flow or unobstructed conditions. Input consists of channel geometry and hydrographs from upstream and lateral segments. Average daily water temperature is simulated throughout the channel network. The composition of sediment transported through a channel reach is dependent upon the characteristics of the bed material, composition of sediment from source areas, and flow conditions. Channel aggradation and degradation are simulated as a function of residual transport capacity. Inputs of nitrogen, phosphorous and pesticides are routed with a mass-conserving Lagrangian scheme that simulates longitudinal variations of chemical concentrations due to additions from external sources, transmission losses and sediment deposition. Channel processes in SWAM are discussed more completely in Alonso (1985a & b) and Pionke, et al. (1985).

Reservoir Processes: Reservoirs and small impoundments such as farm ponds probably have more impact on the quality of water leaving a watershed than any other structural or land use conservation practice. Therefore, SWAM incorporates a very detailed treatment of impoundment processes. Given initial temperature, suspended sediments, dissolved solids and chemical distributions and inflow data composition from source areas, channels, and groundwater, the model simulates daily changes that take place in the vertical profiles of temperature, suspended sediment and dissolved solids. The model takes into consideration density currents and changes in thermal stratification that develop. After changes in the profiles are calculated, chemical and biological (plankton and microbial) changes are simulated. Outflow of water and its temperature, suspended sediments and chemical loads are calculated for input to the downstream channel reach.

Groundwater Processes: Groundwater movement into channel and reservoir reaches is based on the streamtube concept. The flow paths are grouped into representative lengths of parallel and convergent streamtubes (Alonso and DeCoursey, 1985). Daily flow from the groundwater segments is a function of the saturated conductivity, porosity, length of the groundwater streamtube, phreatic head at the divide relative to the water surface elevation at the outlet, and thickness of the aquifer at the outlet. Soluble chemicals in the water percolating from the surface are assumed to be uniformly mixed with the groundwater below each contributing source area. These concentrations are mixed with inflowing water from up-gradient and down-gradient source areas to produce realistic chemical flow into the channel. Groundwater returns from channels and reservoirs are not simulated in the current version of SWAM.

## SEDIMENT TRANSPORT MODELING

In SWAM, sediment movement is modeled as the movement of up to 10 different uniform size fractions.

<u>Sediment Response of Fields</u>: Simulation of erosion within a source area (or field) is based on the methodology of KINEROS (Smith, 1981) with additional parameters in equations of rill and interrill erosion rates proposed by Foster (1983). The equation for interrill erosion utilizes the actual rainfall rate which is input to SWAM. The governing equations for sediment transport are those describing sediment mass continuity and transport capacity. Transport capacity in concentrated flow is based on the Yalin relationship. The equations are solved using a finite difference scheme that parallels the solution of surface water flow.

<u>Sediment Movement in Channels</u>: Sediment movement in channels and floodplains is computed using a space-time domain in which the space domain is represented by discrete cross-sections along a stream reach, and the time domain is represented by discrete time steps. The equations governing sediment movement are solved using a finite-difference scheme with appropriate boundary conditions. Sediment transport rates are computed for different size fractions, and transport capacity is estimated using formulae appropriate for the load characteristics. A concept of residual transport capacity is used to estimate channel aggradation/degradation. Rates of sediment deposition or bed material scour are used to establish new load compositions, update bank and bed elevations, and track the material composition of the active bed layer. When this layer is composed of material too large to be transported, it develops into an armor layer.

<u>Sediment in Small Reservoirs</u>: The disposition of sediment in small reservoirs is modeled by accounting for input of each size class, the depth of entering density currents, movement between layers at different depths through settling, and transport with outflow from the reservoir. The approach chosen in the present model is to regard a reservoir as a horizontally well-mixed but vertically stratified water body. Field studies in reservoirs of moderate length have shown that fine suspended sediment is found in fairly uniform concentrations along horizontal layers but is variable in depth. A one-dimensional unsteady numerical model was developed to predict the vertical distribution of suspended sediment concentration and rate of suspended sediment deposition. The model accounts for settling of solid particles and the vertical diffusion of particles by turbulence. The model is derived from the convective-diffusive transport equation and uses a fully implicit finite-difference scheme.

## SUMMARY

The Small Watershed Model (SWAM) is a comprehensive model of the water, sediment, and chemical response of mixed-land-use agricultural watersheds under alternative management practices. It is designed to serve as a vehicle for research in understanding complex watershed processes and to provide a tool for evaluation of a variety of natural resource problems. The model shows good promise to serve as a useful tool in understanding and managing the water, sediment, and chemical response of agricultural watersheds.

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# EVALUATION OF A ROAD SEDIMENT MODEL

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## ABSTRACT

The yield and gradation of sediment that resulted from a 1 inch simulated rainfall on unsurfaced roadways were compared to the yield and gradation of sediment predicted by a physical process model, ROSED. The precipitation was applied in 23-30 minutes on three separate plots. Plot lengths ranged from 50 to 100-foot long with widths of 13-17 feet. Cross slopes ranged from 2 to 4% and longitudinal slopes ranged from 6% to 9%. The unsurfaced roadways were constructed of "border zone" soils and were located in the Boise and Nezperce National Forests.

A modified Colorado State University Rainulator was used to apply the precipitation. The runoff from each plot was measured using a flume with stilling well and flowmeter. Grab samples were obtained at one minute intervals to determine sediment yield and to collect adequate sediment for gradation analysis.

A sensitivity analysis was performed on each of the parameters that affects the yield and gradation of sediment. Important elements for predicting sediment in the ROSED model are raindrop detachment, sediment detachment by overland flow, and sediment transport by overland flow.

Each parameter in a sample data set was systematically varied to determine the effect the parameter has on model output. Known soil properties, geometry and rainfall data for each plot were then used for input and the model was calibrated to obtain output that closely matched the sediment yield and gradation observed in the field.

Sediment input parameters that produced the greatest change in model output for sediment yield and gradation were parameters used for computing bed load transport within the model. Using a set of proposed sediment input parameters based on known soil characteristics and known site data, without calibration, the model predicted sediment yields that were within ±50% of the actual yields. With calibration the model predicted sediment yields within ±1% of the actual yields. The grain size distribution curves and sediment graphs also closely matched measured curves.

## MODEL DESCRIPTION

ROSED (ROad SEDiment) is a numerical physical process model which predicts water and sediment yield from a roadway prism as a function of rainfall, plot topography, surface cover, and soil parameters. ROSED is an event oriented model driven by the runoff. A series of rainfall events may be simulated. For a given rainfall event, the interception and infiltration losses are computed and the excess water is routed down the watershed.

The road prism is divided into three response units, a plane with overland flow which discharges into a ditch which in turn discharges into a culvert. A typical road prism is shown in Figure 1. The process of water and sediment routing is accomplished using a space-time model.

Sediment is generated from the initial loose layer on the surface, raindrop detachment, and overland flow detachment. The Meyer-Peter Muller equation is used for bed materials transport and the Einstein function is used for suspended sediment transport. The yield of sediment may be limited by the supply of sediment or by the transport capacity of the flow. A flow chart of the model is shown in Figure 2. A more detailed description of the model and all experimental data can be found in Reference 1.

The significant functional relationships and the specific parameters that affect the yield and gradation of the material that is removed from a roadway prism were determined to be:

1. The Meyer-Peter Muller equation used to compute bed load transport,

$$q_{bk} = a_1' (\tau_o - \tau_{ck})^{b_1}$$

$q_{bk}$ = bed load discharge for a given sediment size, $ft^2/sec$

$a_1'$ = experimentally derived coefficient

$\tau_o$ = fractive force of flowing water, $lbs/ft^2$

$\tau_{ck}$ = critical tractive force of a given particle size (tractive force at which particle first moves), $lbs/ft^2$

$b_1$ = experimentally derived coefficient

2. The raindrop soil detachment equation:

$$D_r = a_2 \quad i^{b_2} \quad (1 - \frac{Z_w}{Z_m})\ (1 - C_g)\ (1 - C_c)$$

$D_r$ = potential rate of soil detachment by rainfall of intensity i, in/hr

$a_2$ = experimentally derived coefficient

$i$ = rainfall intensity, in/hr

$b_2$ = experimentally derived coefficient

$Z_w$ = depth of surface water plus loose soil, in

$Z_m$ = maximum raindrop penetration depth, in

$C_g$ = ground cover density

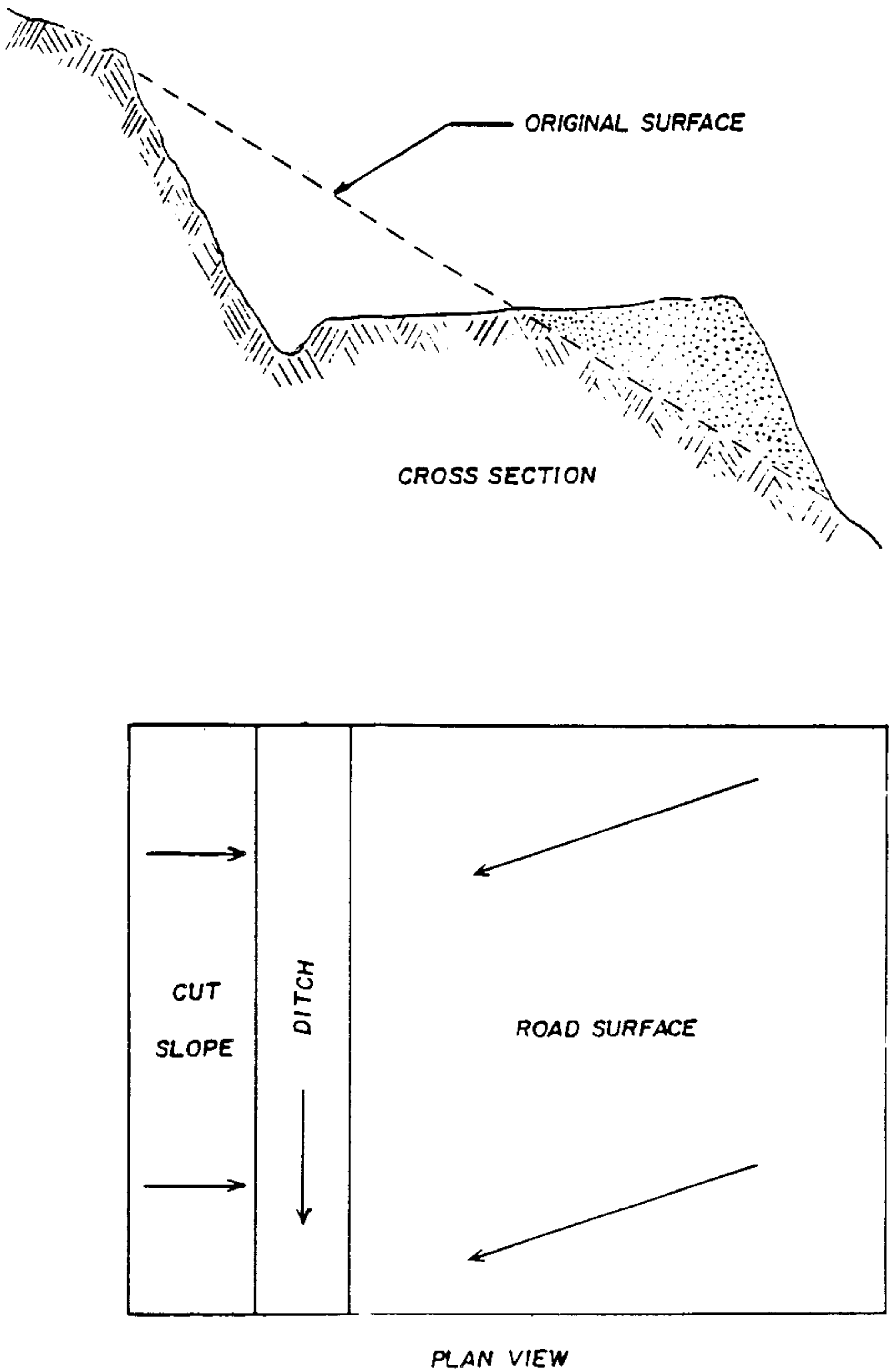



Figure 1. Typical Road Prism

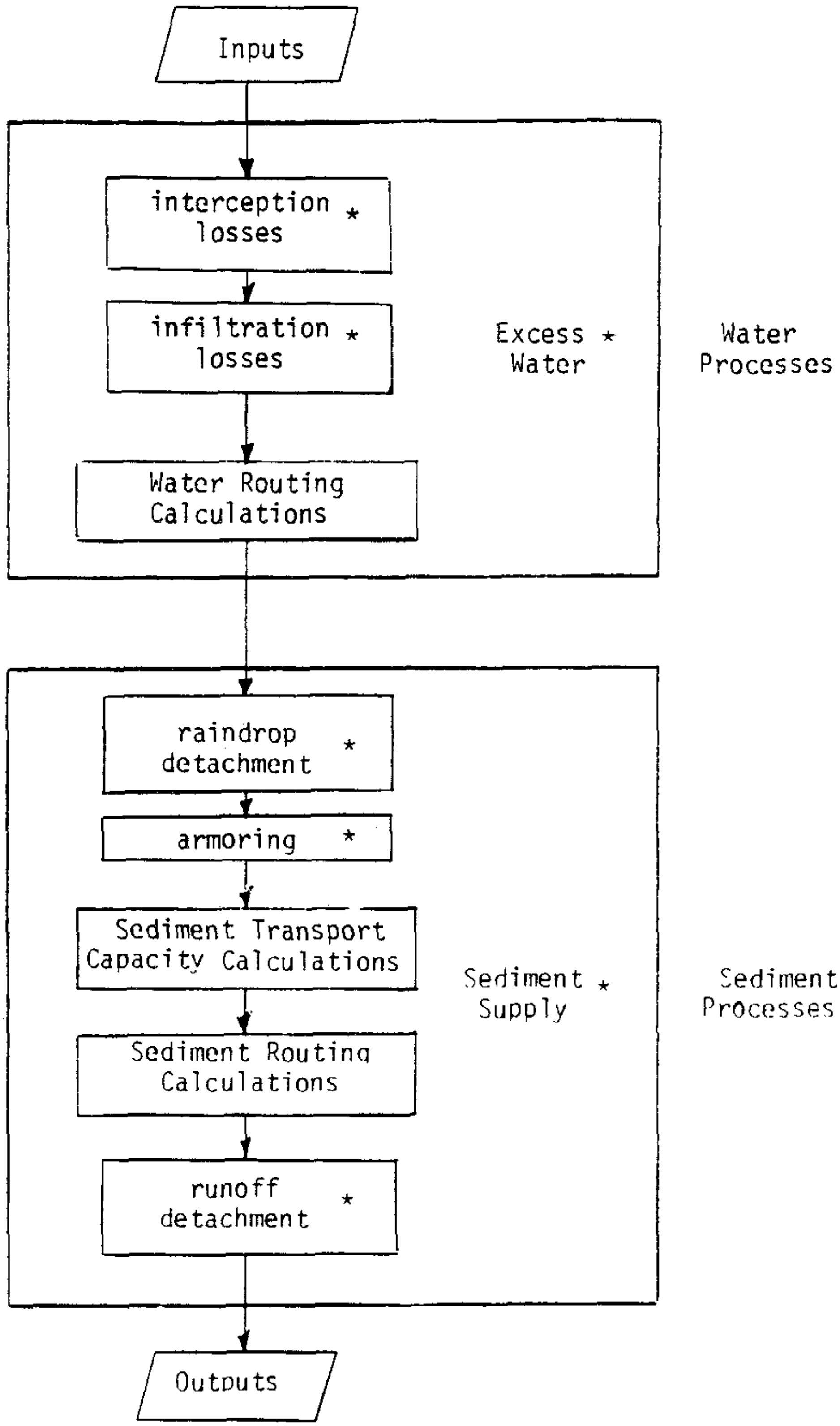



Figure 2. General Flow Chart of ROSED

$C_c$ = canopy cover density

3. The surface runoff detachment coefficient

$$D_F = \frac{KD_s^2}{0.63}$$

$D_F$ = the surface runoff detachment coefficient

$K$ = erosivity index, Universal Soil Loss Equation

$D_s$ = representative grain size, mm

4. The critical tractive force for a particle size,

$$\tau_{ck} = \delta_s(\gamma_s - \gamma)d_{sk}$$

$\tau_{ck}$ = critical tractive force, $lbs/ft^2$

$\delta_s$ = experimentally determined parameter, "Shields" parameter,

$\gamma_s$ = unit wt of soil, $lbs/ft^3$

$\gamma$ = unit wt of water, $lbs/ft^3$

$d_{sk}$ = representative grain size, ft

5. Tractive force of flowing water

$$\tau_o = \frac{1}{8} \rho \frac{K_o}{N_r} V^2$$

$\tau_o$ = Tractive force, $lbs/ft^2$

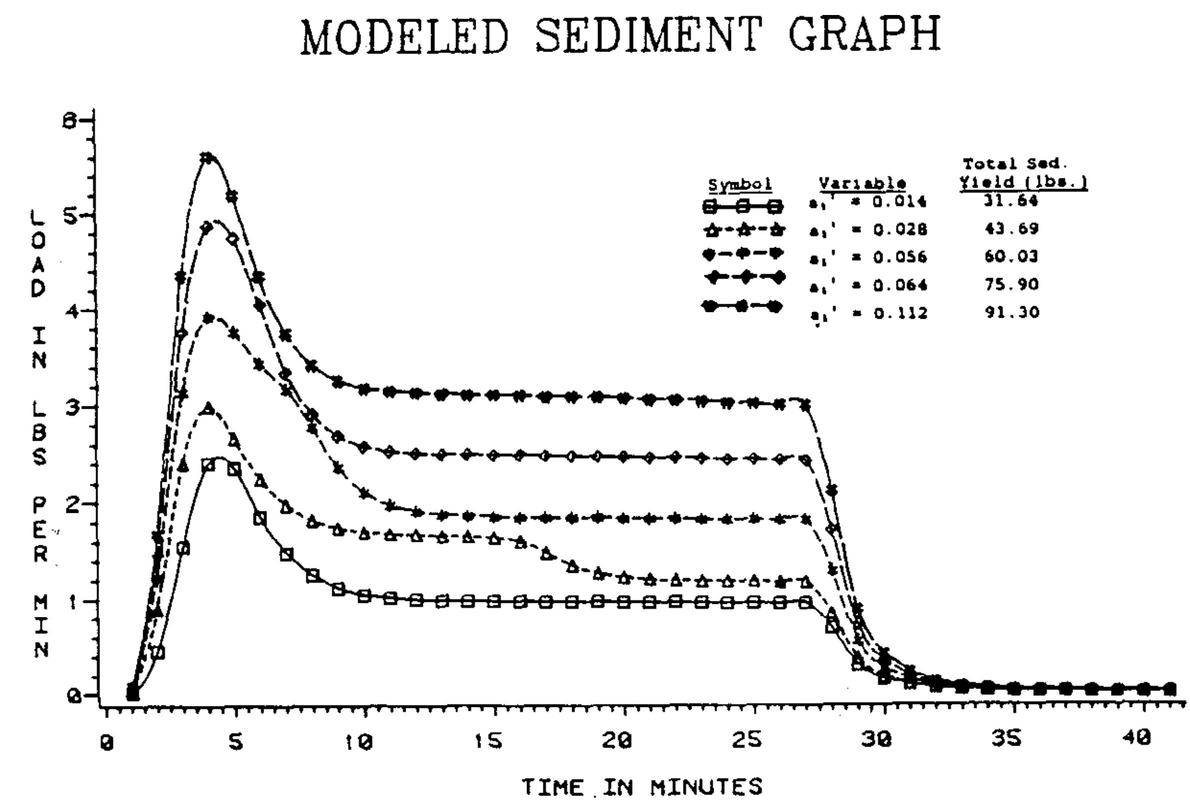
$\rho$ = density of water, $lbs\text{-}sec^2/ft^4$

$K_o$ = experimentally derived grain resistance coefficient

$N_r$ = Reynolds number

$V$ = mean velocity of flow, ft/sec

The parameters that were significant that were systematically varied, the name of the parameter as used in the model and as displayed in the figures that follow, default value of the parameter, and recommended range of the parameter are listed in Table 1 below. Some of the program generated sediment graphs and grain size distribution for a series of runs with the modeled parameter taking on the values shown in the graph with all other parameters at default value are shown in Figures 3 through 6.



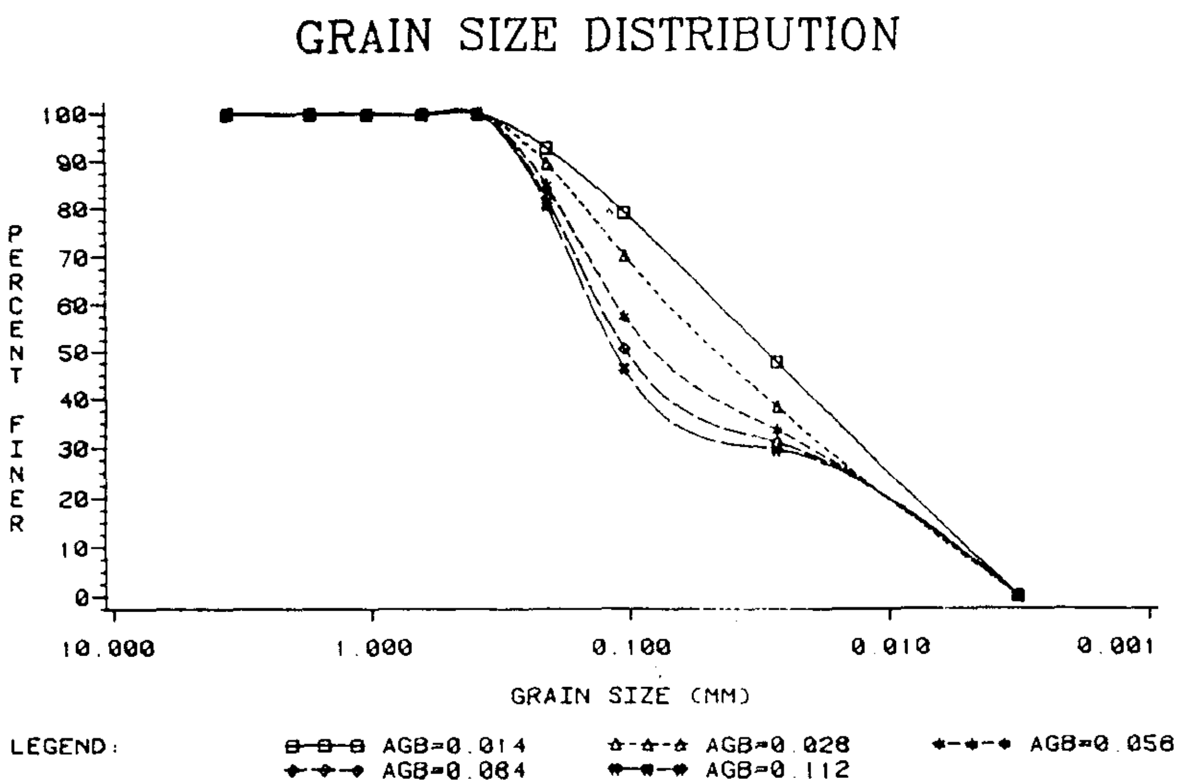




Figure 3. Model Response to Input Variable $a_1$

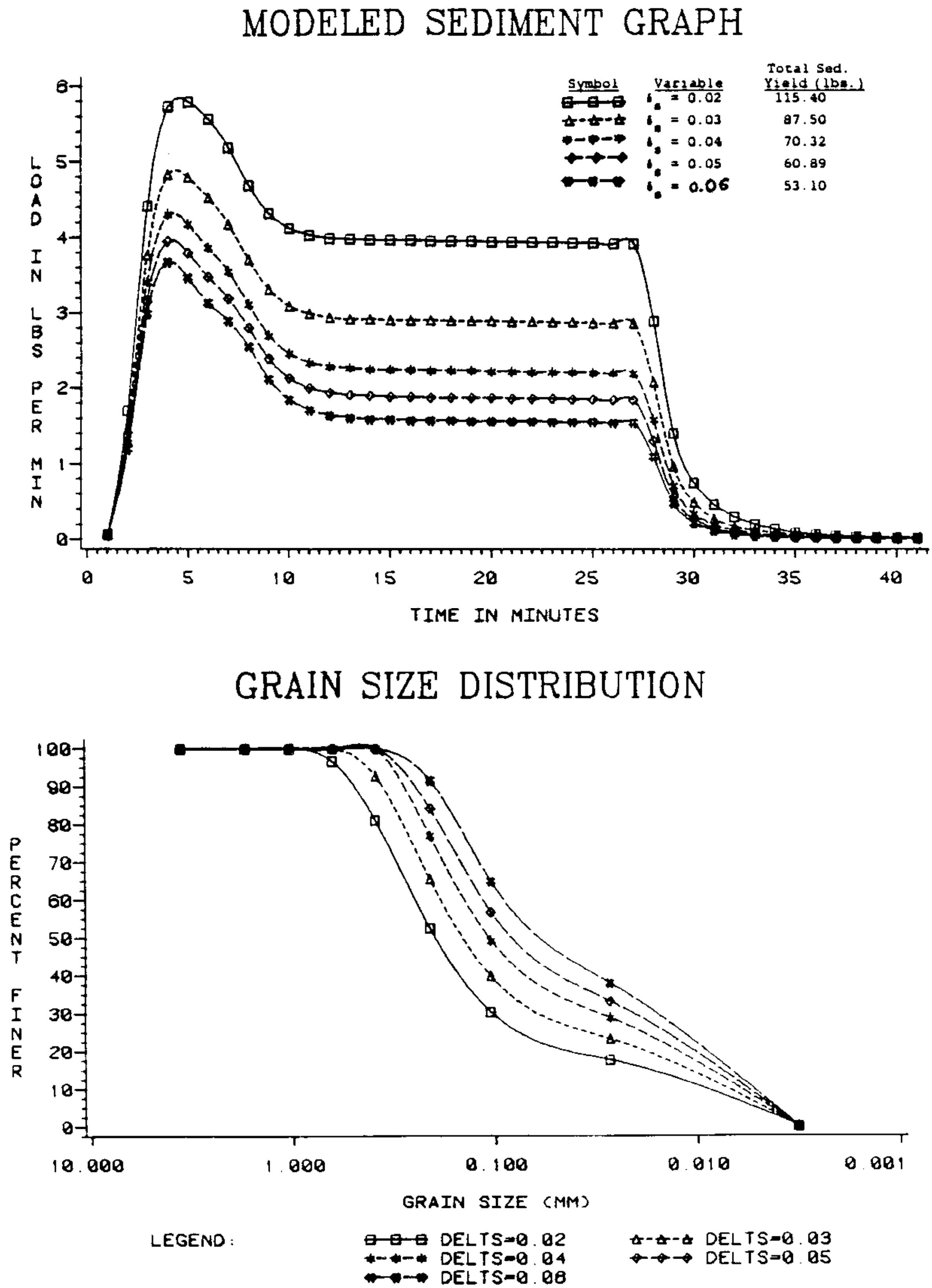
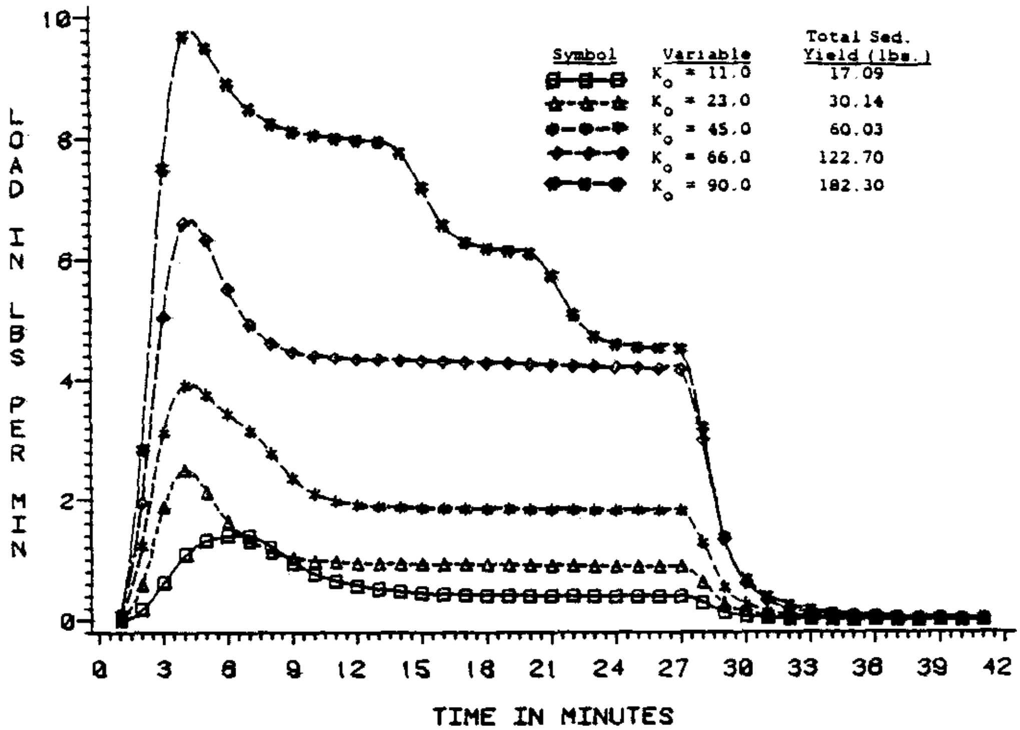



Figure 4. Model Response to Input Variable $\delta_s$

MODELED SEDIMENT GRAPH




GRAIN SIZE DISTRIBUTION

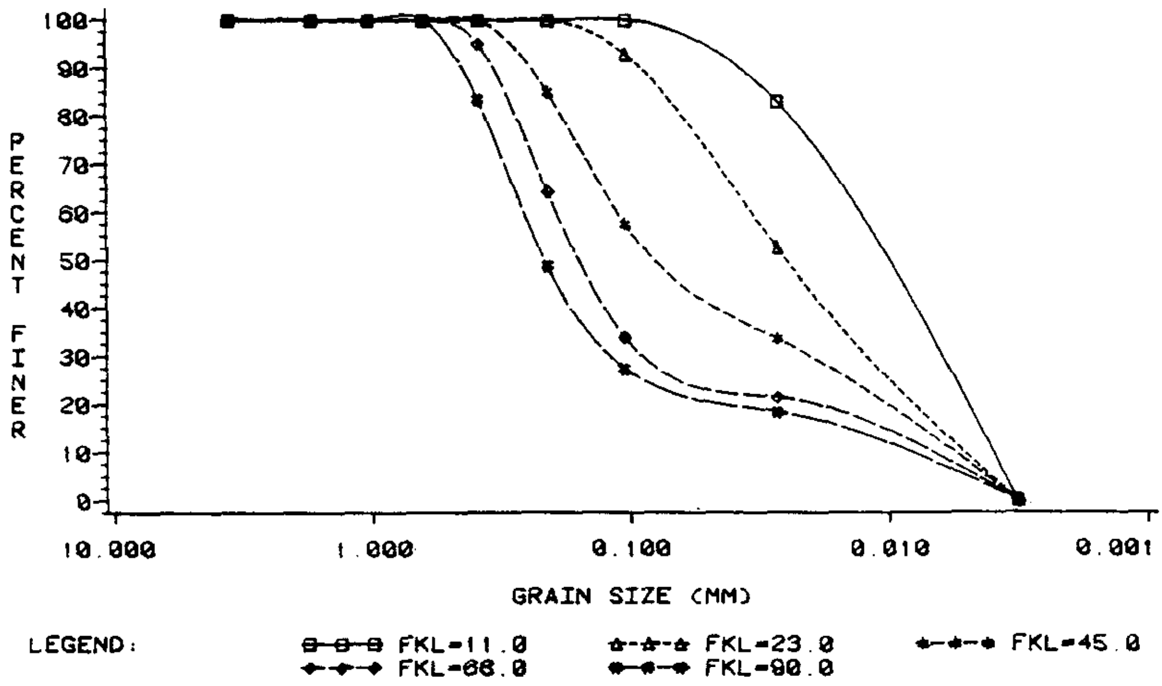
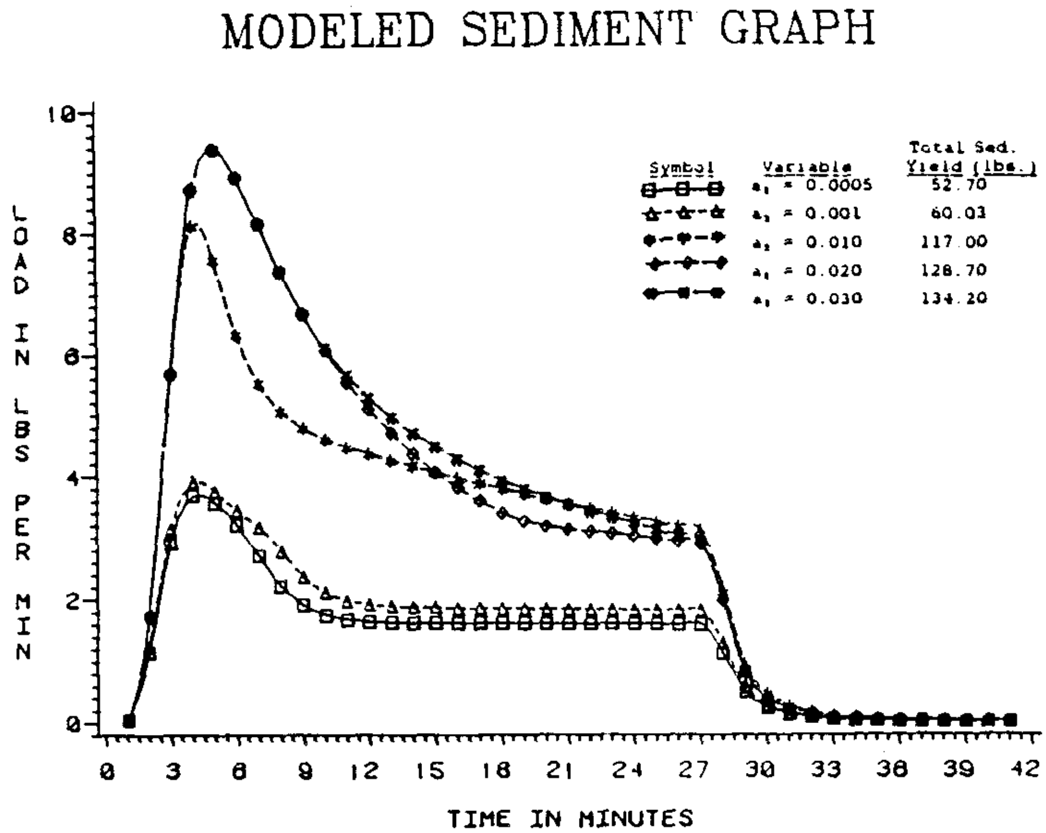



Figure 5. Model Response to Input Variable $K_o$



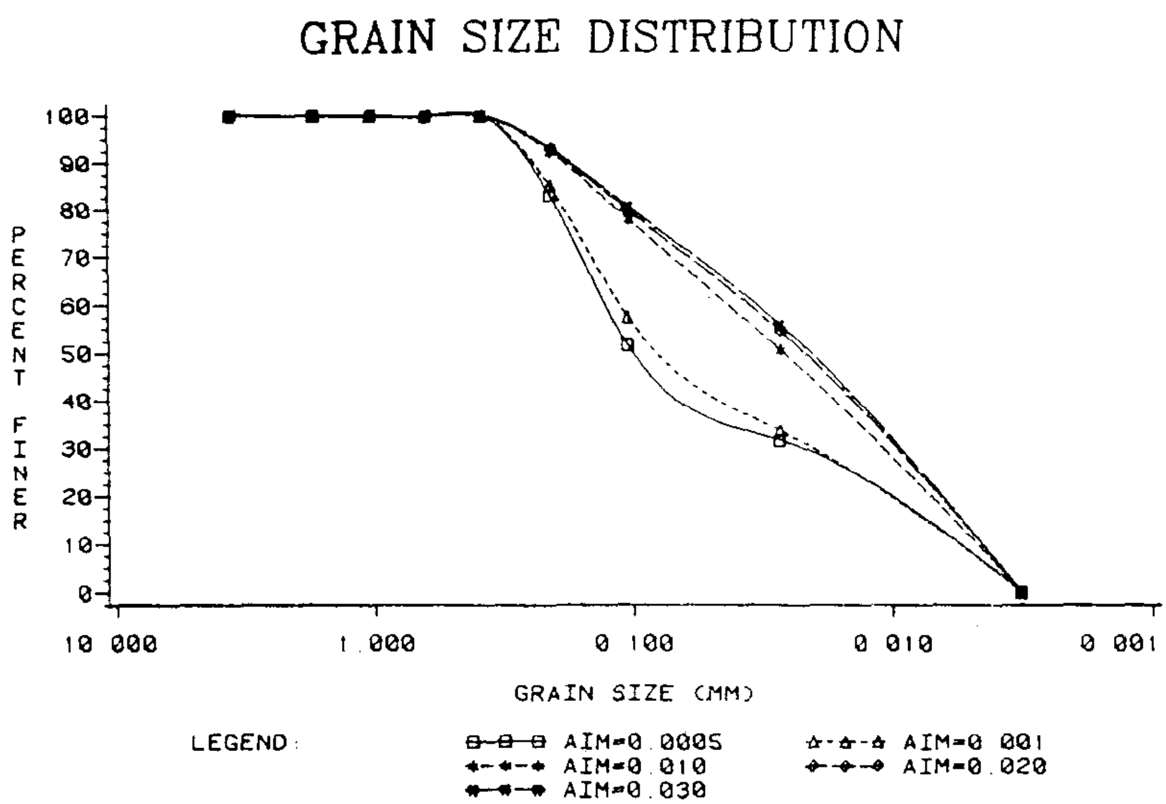




Figure 6. Model Response to Input Variable $a_2$

TABLE 1.

| Variable Name Report | Model | Default Value | Recommended Range |
|---|---|---|---|
| $a_1'$ | AGB | 0.056 | 0.056 |
| $a_2$ | AIM | 0.001 | 0.001-0.030 |
| $b_1$ | BEX | 1.50 | 1.50 |
| $b_2$ | BIM | 2.00 | 2.00 |
| $D_f$ | ADF | 0.25 | 0.0-1.0 |
| $K_o$ | GRF | 45.0 | 10-66 |
| $\delta_s$ | DELTS | 0.051 | 0.010-0.060 |

## RESULTS

A summary of important sediment parameters in the ROSED model default values, and recommended range of values were shown in Table 1. The input parameters $\delta_s$ and $K_o$ had the greatest impact on sediment yield. Based on this study it is recommended that the values shown in Table 1 for $a_1'$, $b_1$, and $b_2$ be used for decomposed granitic soils.

The predicted sediment yield from the plots using the default values of parameters were generally ±50% of the measured yield from the plots. It is stressed that roadway material examined in this study was composed of metamorphosed granitics with gneiss and schist commonly found in the Idaho Batholith. Input parameters may be somewhat different for roadway prisms composed of other material.

## REFERENCE

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