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HYDRAULIC AND SEDIMENT TRANSPORT PROCESSES IN EPHEMERAL STREAM CHANNELS: COMPONENTS OF THE SPUR MODEL

Leonard J. Lane[1]

INTRODUCTION

Stream channels combine in complex patterns to produce channel networks and the interchannel areas. These features, among others, control the routes and rates of movement of water and sediment as runoff occurs in response to precipitation. Because these hydrologic processes are complex and highly variable in time and space, it is impossible to measure them on each watershed where information is needed. Moreover, because hydrologic processes are influenced by climate, geology and geologic materials, soils, vegetation, and land use, it is often impossible to monitor a few watersheds and extend the results over large areas. Therefore, there is a need to develop a predictive capability using mathematical models which simulate hydrologic processes. Such a model for rangeland conditions has been developed to include hydrology, vegetative development, and animal utilization. This paper describes the hydrologic components of that model (see Wight 1983).

As hydrologic processes occur over progressively larger land areas, the relative importance of stream channels increases. Therefore, there is a need to understand hydrologic, hydraulic, and sedimentation processes occurring in stream channels. Streams in natural channels in arid and semiarid regions are often ephemeral, with occasional streamflow following storm periods.

Water is often a limiting factor in arid and semi-arid areas. Thus, streamflow processes, including infiltration or transmission losses in the channel bed and banks, are important components in the hydrologic cycle. Because erosion and sedimentation processes are related to hydrologic processes, there is also a need to understand sediment transport in these stream channels.

The purpose of this short paper is to describe the development and application of rather simplified procedures to simulate hydrologic, hydraulic, and sedimentation processes as used by SPUR (Wight 1983) in semiarid watersheds, with emphasis on processes in ephemeral stream channels. This paper summarizes some observations on channel processes, provides an overview of important features of those processes, and lists selected references for further study. These references provide derivations and more specific information, elaborate on these topics, and provide examples and applications.

TRANSMISSION LOSSES

As water flows in an ephemeral channel system, the of flow varies in the downstream direction as a result of variable subchannel contributions of

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[1]USDA-ARS Southwest Rangeland Watershed Research Center, 2000 E. Allen Rd., Tucson, AZ 85719.

water and sediment, channel hydraulic features, and processes such as infiltration into the channel bed and banks. Infiltration losses and abstractions, or transmission losses, are important, because they reduce the volume of runoff. Although abstractions are called transmission losses, they are an important part of the water balance because they support riparian vegetation and recharge local aquifers and regional groundwater (Renard 1970). In addition to the hydraulic and hydrologic significance of transmission losses discussed above, these losses also influence sediment transport and yield because of their affect on hydraulic processes.

Several procedures have been developed to estimate transmission losses (Babcock and Cushing 1941; Burkham 1970a, 1970b; and others). These procedures range from inflow-loss-rate equations (Burkham 1970a, 1970b; NEH-4 1972) to simple regression equations (Lane et al 1971), to simfplified differential equations for loss rate as a function of distance (Jordan 1977, Lane 1983), to storage-routing as a cascade of leaky reservoirs (Lane 1972, Wu 1972, Peebles 1975), and to kinematic wave models incorporating infiltration (Smith 1972). As a rule, the simplified procedures require less information about physical features of the channel systems, but are less general in their application. The more complex procedures may be more physically based, but they require correspondingly more data and more complex computations.

The transamission loss equations presented here represent an attempt to develop a somewhat simplified procedure for practical applications. As such, the equations represent a compromise between the more physically based deterministic models and the more simplified procedures described earlier. An empirical basis for the transmission loss equations is presented; then, the equations are derived as the solution to a first-order differential equation expressing the rate of change in runoff volume with distance in the channel.

If streamflow, without lateral inflow, occurs in a channel reach with significant amounts of transmission losses, then outflow data at a point downstream can be related to inflow data at an upstream point. If the inflow is lower than a threshold amount, all of the runoff will be lost, and no outflow will occur. Once inflow volumes exceed the threshold, then outflow volumes will increase with increasing volumes of inflow. Based on these observations, observed inflow and outflow data for a channel reach were related by regression analysis (Lane et al. 1971), resulting in an equation of the form

$$V(x,w) = \begin{cases} 0 & V_{up} \leq V_0(x,w) \\ a(x,w) + b(x,w)V_{up} & V_{up} > V_0(x,w) \end{cases} \quad (1)$$

where

$a(x,w)$ = regression intercept (acre-ft or m^3),
$b(x,w)$ = regression slope,
$V_0(x,w)$ = threshold volume (acre-ft or m^3),
V_{up} = inflow volume (acre-ft or m^3),
x = Length of the channel reach (mi or km),

and

w = average width of flow (ft or m).

By setting $V(x,w) = 0.0$ and solving for V_{up}, the threshold volume is

$$V_o(x,w) = -a(x,w)/b(x,w) \qquad (2)$$

Based upon these empirical observations and the work of Jordan (1977), Lane (1983) approximated the rate of change in runoff volume with distance as

$$\frac{dv}{dx} = -wc -wk\, V(x,w) + V_{LAT}/x \qquad (3)$$

where c and k are parameters, and V_{LAT} is the volume of lateral inflow assumed to be uniform along the channel reach. The solution to equation 3 is

$$V(x,w) = \frac{-c}{k}(1-e^{-kxw}) + V_{up}e^{-kxw} + \frac{V_{LAT}}{kxw}(1-e^{-kxw}) \qquad (4)$$

which is in the same form as

$$V(x,w) = a(x,w) + b(x,w)\, V_{up} + f(x,w)\, V_{LAT}/x \qquad (5)$$

if we let

$$a(x,w) = \frac{a}{1-b}(1-e^{-kxw}) = \frac{a}{1-b}\left(1-b((x,w)\right) \qquad (6)$$

$$b(x,w) = e^{-kxw} \qquad (7)$$

$$f(x,w) = \frac{1}{kw}(1-e^{-kxw}) = \frac{1}{kw}\left(1-b(x,w)\right) \qquad (8)$$

and notice that

$$b = e^{-k} \qquad (9)$$

$$a = -\frac{c}{k}(1-b) \qquad (10)$$

Thus, equations 5 through 10 are the transmission loss equations for a single channel reach when the rate of change in runoff volume with distance is described by equation 3. To compute the transmission losses in an entire channel network, these equations are applied to each channel reach or segment in the network.

It is important to note that equation 3 and its solution, given in equation 5, deal with spatial variations in transmission losses but assume steady-state loss rates with time. Of course, transmission losses are highly dynamic, and variations in time are important. Therefore, it should be noted that equations 3 and 5 reflect the steady-state assumption and, as such, emphasize spatial, rather than temporal, variations in flow rate.

OPEN CHANNEL FLOW HYDRAULICS

In anticipation of using hydraulic variables in sediment transport calculations, and for simplicity in the calculations, we make two major assumptions. These are the assumptions of rectangular-channel cross section and of normal flow. Normal flow means that depth, veloctiy, and so forth, are not changing with time at a given cross section, and are not changing with distance bewtween subsequent cross sections. That is, normal flow is both steady

and uniform. Under these conditions, the average velocity in a cross section is given by the Manning equation as

$$V = \frac{a}{n} S^{1/2}R^{2/3} \qquad (11)$$

where

    V = average velocity (ft/s, m/s),
    S = slope of the channel bed,
    R = hydraulic radius (ft, m),
    n = Manning's roughness coefficient ($s/ft^{1/3}$ or $s/m^{1/3}$) and,
    a = a unit's conversion factor, 1.0 in SI units and 1.49 in English units.

The hydraulic radius for a rectangular channel is

$$R = \frac{A}{WP} = WD/(W + 2D) \qquad (12)$$

where A is a cross-sectional area, WP is wetted perimeter, W is channel width, and D is flow depth. The continuity equation is then

$$Q = AV = WDV \qquad (13)$$

where Q is flow rate (ft^3/s or m^3/s). The depth of flow which satisfies equations 11 and 13 is called normal depth. Flow, where depth is normal, is called normal flow.

Hydraulic Roughness

The roughness coefficient, n, in equation 11 has been tabulated for a number of channel types (Barnes 1967) and represents the resistance to flow provided by the channel bed and banks. This resistance, or roughness, is called the total roughness. Values of total roughness coefficients n_T, for various channel types are shown in table 1.

Correction for Wall or Bank Roughness

Since the flow resistance contributed by the channel banks (wall roughness) is not directly involved in transporting sediment near the channel bed, it is possible to separate its influence from the influence of the bed. Following Einstein (1942, 1944, 1950), the total cross-sectional area, A_T, is divided into an area pertaining to the wall, A_w, and an area pertaining to the bed, A_b, as

$$A_T = A_w + A_b \qquad (14)$$

If the energy gradient, S, and the velocity, V, are the same for the wall and bed, and if the area is defined as the product of hydraulic radius and wetted perimeter, $A = RWP$, then equation 14 becomes

$$R_T(W + 2DS) = R_w(2D) + R_b(W) \qquad (15)$$

By the Manning equation, hydraulic radius is

$$R = \left[\frac{Vn}{1.49\, S^{1/2}}\right]^{3/2} \qquad (16)$$

where V is velocity and S is slope. Substituting

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Table 1.--Approximate hydraulic roughness coefficients for open channel flow, presented as total roughness coefficient n_T

Total Manning n		Description of Channel
(.02 - .10)		Excavated or dredged channels[1]
.022	1.	Earth, straight, uniform, and clean.
.027	2.	Same, but with some short grass or weeds.
.025	3.	Earth, winding and sluggish, with no vegetation.
.030	4.	Same, but with some grass or weeds.
.080	5.	Channels not maintained; weeds and some brush.
(.03 - .10)		Natural Streams[1]
.030	1.	Clean and straight; no rifts or deep pools
.040	2.	Clean and winding; some pools and shoals.
.048	3.	Clean and winding; some weeds, stone, and pools.
.070	4.	Sluggish reaches with weeds and deep pools.
(.012 - .040)		Wide Alluvial Channels[2]
.018 - .030	1.	Ripples bed form, sediments finer than 0.6 mm, Froude Nos. < 0.37.
.020 - .040	2.	Dunes bed form, Froude Nos. 0.28 to 0.65.
.014 - .030	3.	Transitional bed form, Froude Nos. 0.55 to 0.92.
.012 - .030	4.	Antidunes bed form, Froude Nos. > 1.0.

[1]Source: Chow (1959).
[2]Source: ASCE (1966) and Simons and Richardson (1971).

equation 16 into equation 15, where V and S are common to all terms, produces

$$n_T^{3/2}(W + 2D) = n_w^{3/2}(2D) + n_b^{3/2}(W) \qquad (17)$$

with solution for the hydraulic roughness of the bed, n_b, as

$$n_b = \left[n_T^{3/2} + \frac{2D}{W} (n_T^{3/2} - n_w^{3/2}) \right]^{3/2} . \qquad (18)$$

Geometric considerations suggest that the least value of R_b is 1/2 R_T, which means that, as a minimum,

$$n_b \geq (1/2)^{2/3} n_T \qquad (19)$$

and that as a maximum,

$$n_w \leq (\frac{W + 4D}{4D})^{2/3} n_T . \qquad (20)$$

Equation 18 is evaluated for n_b subject to equation 19 as a constraint (that is, $n_b \geq (1/2)^{2/3} n_T$), which means that the hydraulic radius of the bed is

$$R_b = \left[\frac{V n_b}{1.49 S^{1/2}} \right]^{3/2} \qquad (21)$$

Table 1 can be used to estimate n_T and n_w subject to the constraints on n_w as

$$n_T \leq n_w \leq (\frac{W + 4D}{4D})^{2/3} n_T . \qquad (22)$$

The procedure is to select a value of n_T from the column in table 1 and to select a value of $n_w \geq n_T$ which represents conditions of the banks.

Correction for Grain Resistance

The grain, or particle resistance coefficient, n_g, is related to a representative grain size to the 1/6 power (Strickler 1923). This can be approximated as

$$n_g = 0.0132(d_{50})^{1/6} \qquad (23)$$

The hydraulic radius for grain resistance can then be estimated as

$$R_g = R_b(n_g/n_b)^{3/2} \qquad (24)$$

where R_b is obtained from equation 21, and n_b is obtained from equation 18, subject to the constraints given by equations 19 and 20.

Effective Shear Stress for Sediment Transportation

The effective shear stress for sediment transportation is given by

$$\tau = \gamma R_g S \qquad (25)$$

where

τ = effective shear stress (lb/ft^2),
γ = specific weight of water (lb/ft^3),
R_g = hydraulic radius for grain resistance (ft), and
S = energy gradient, slope of the channel bed for normal flow.

The effective shear stress, given by equation 25, will be less than the total shear stress averaged over the cross section, $\tau_T = \gamma R_T S$, because some of the total available energy is expanded on the banks due to bank roughness, and because some is expended on the bed due to form roughness.

SEDIMENT TRANSPORT CALCULATIONS

Sediment transport is assumed equal to sediment transport capacity. If sediment load exceeds transport capacity, deposition occurs; and if transport capacity exceeds sediment load, scour or erosion may occur. However, for alluvial channels with noncohesive sediments, it is common to assume sediment transport rate equal to sediment transport capacity. To avoid more elaborate sediment deposition models and channel erosion models, we assume that, as a first approximation, sediment transport rate is equal to sediment transport capacity.

Because sediment transport capacity, hereafter referred to as transport capacity, is strongly related to localized in-channel processes, it is in large part determined by the hydraulic variables described earlier. Inasmuch as the in-channel features, such as channel morphology and sediment properties as well as the hydrologic and hydraulic variables, reflect upland processes, these upland processes are reflected in the transport capacity calculations.

The Bed Load Equation

Following Einstein (1950) and others, a distinction is made between bed load and suspended load. If we assume that sediment transport rate is proportional to the water flow rate, then this distinction is somewhat arbitrary. This is because particles that travel as bed load at one flow rate may be suspended at another. The relationship between mode of transport and flow rate is a dynamically complex one and represents a continuous, rather than distinct, transition.

Nevertheless, it is reasonable to assume that larger particles travel as bed load, and that the smaller particles more easily enter suspension. Moreover, it is computationally convenient to assume a sharp distinction based on particle size. Therefore, we arbitrarily assume that all sediment larger than 0.062 mm in diameter is transported as bed load, and that finer material is transported as suspended load. Separate transport equations are derived for bed load transport and suspended load transport based on this assumption.

Using a modification of the DuBoys-Straub formula (see Graf 1971 for a complete description), transport capacity for bed-load size particles can be computed as

$$g_{sb}(d_i) = \alpha f_i B_s(d_i)\tau \left[\tau - \tau_c(d_i)\right] \qquad (26)$$

where
- $g_{sb}(d_i)$ = transport capacity per unit width for particles of size d_i (lb/s-ft),
- α = a weighting factor to insure that the sum of the individual transport capacities equals the total transport capacity computed using the median particle size,
- f_i = proportion of particles in size class i,
- d_i = diameter of particles in size class i (mm),

- $B_s(d_i)$ = sediment transport coefficient (ft^3/lb-s),
- τ = effective shear stress (lb/ft^2), and
- $\tau_c(d_i)$ = critical shear stress for particles in size class i (lb/ft^2).

Values of B_s and τ_c, in English units, were determined by Straub (1935). The total bed load transport capacity is then found by summing the results from equation 26 over all the size fractions.

However, values of B_s and τ_c, as developed by Straub (1935), were for total shear stress rather than the effective shear stress, corresponding with grain resistance. Parameter estimates, using effective shear stress, are given by

$$B_s(d_i) = 40.0/(d_i)^{1.5} \qquad (27)$$

and

$$\tau_c(d_i) = \begin{array}{ll} 0.0022 + 0.010\, d_i & .062 \leq d_i \leq 1.0 \\ -0.0078 + 0.020\, d_i & 1.0 < d_i \end{array} \qquad (28)$$

where d_i is the representative particle diameter (mm). Equations 27 and 28 were calibrated with observed sediment transport data from the Niobrara River in Nebraska (Colby and Hembree 1955) for particle sizes up to 2.0 mm. Therefore, equations 27 and 28 have not been evaluated for particles larger than 2.0 mm in diameter. Because the weighting factor, α, in equation 26 insures that the sum of the individual transport capacities equals the total transport capacity -- computed using the median particle size, d_{50}, in equations 26 through 28 -- the model has not been evaluated for values of d_{50} in excess of 1.0 mm.

The Suspended Load Equation

Bagnold (1956, 1966) proposed a sediment transport model based on the concept of stream power as

$$i_s = P \frac{e_s\, u_s}{v_s}(1 - e_b) \qquad (29)$$

where

- i_s = suspended sediment transport rate per unit
- $P = \tau V$ = available stream power per unit area of the bed (lb/s-ft),
- e_s = suspended load efficiency factor,
- e_b = bed load efficiency factor,
- u_s = transport velocity of suspended load (ft/s), and
- v_s = settling velocity of the particles (ft/s).

Now, if u_s is assumed equal to the mean velocity of the fluid, V, then equation 29 is of the form

$$g_{sus} = f_{sc} \cdot CAS \cdot \tau V^2 \qquad (30)$$

where

- g_{sus} = suspended sediment transport capacity (lb/s-ft),
- f_{sc} = proportion of particles smaller than 0.062 mm in the channel bed material,
- τ = effective shear stress (lb/ft^2),
- V = average velocity (ft/s), and
- CAS = suspended sediment transport coefficient (s/ft).

The suspended sediment transport coefficient, CAS, incorporates the efficiency parameters and the settling velocity of the suspended particles. Values of CAS have been determined by calibration with observed data. However, because of the scarcity of observed data and the interaction of the efficiency parameters and settling velocity, and their interaction with flow dynamics, values of CAS are not well specified by measurable physical characteristics.

APPLICATIONS AND DISCUSSION

Procedures have been outlined which can be used to estimate reductions in runoff due to transmission losses to estimate hydraulic variables in open channel flow, and to compute sediment transport capacity in alluvial channels. Various aspects of these procedures have been applied to compute runoff and sediment yield on semiarid rangeland watersheds.

Lane (1982a) used the transmission loss equations to develop a simplified routing procedure as part of a basin-scale hydrologic model. The model was used to simulate runoff volume and peak discharge rates for individual storm events. Example applications included estimation of flood frequency curves for semiarid rangeland watersheds. Lane (1982b) used the basin-scale runoff estimation model, together with a hydrograph estimation technique and the sediment transport equations, to estimate sediment yield from semiarid watersheds. More recently, the transmission loss and sediment transport equations were modified to provide the channel component of a hydrologic model used as part of a range resource model (Renard et al. 1983).

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