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Journal of Geographical Research
https://ojs.bilpublishing.com/index.php/jgr

ARTICLE

Sensitivity Analysis Of Geographical River Boundary Layers

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ARTICLE INFO

Article history
Received: 14 March 2019
Accepted: 29 March 2019
Published Online: 12 April 2019

Keywords:
River
Boundary layers
Sensitivity analysis
Regime

ABSTRACT

Bank full discharge is generally considered to be the dominant steady flow which would generate the same regime channel shape and dimensions as the natural sequences of flows would. This is because investigation on the magnitude and frequency of sediment transport have determined that for stable rivers the flow which in the longer term transports most material has the same frequency of occurrence as bankfull flow. For stable gravel-bed rivers, this is considered to be the 1.5-year flood.

1. Introduction

The objective of regime theory is to predict the size, shape, and slope of a stable alluvial channel under given conditions. A channel is characterized by its width, depth, and slope. The regime theory relates these characteristics to the water and sediment discharge transported bye the channel empirically. Empirical measurements are taken on channels and attempts are made to fit empirical equations to the observed data. The channel characteristics are related primarily to the discharge but allowance is also made for variations in other variables, such as sediment size.

For practical purposes, rivers are preserved to be in equilibrium (in regime) or in quasi-equilibrium of this characteristics have not changed over a long period of time. Canals usually maintain constant discharge and regime relations may, therefore, be established using field data. However, field measurements for rivers are not usually suitable for establishing laws for rivers in regime as pointed.

2. Analysis

2.1 Depth, Width and Slope

Natural rivers cover a wide range of discharge and slope, while the range of values for canals is relatively small. Lane (1957) observed that the stream width is a function of the slope and discharge. Therefore Lacey's relation

$$P = 2.67Q^{0.5} \qquad (1)$$

cannot be applied to natural streams, because it contradicts the finding of Lane that steep. slope streams tend to be wider and shallower than streams of the discharge on a

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DOI: https://doi.org/10.30564/jgr.v2i2.632

flat slope.

Depth

From analysis of the Kennedy (1994) data from the upper Bari Doab canal system in which nonsilting velocities had been achieved by aggradation and widening, Lacey (1930) plotted nonsilting velocity V_0 and hydraulic radius R,

$$V_0=1.17R^{1/2} \quad (2)$$

which was comparable to Kennedy's

$$V_0=0.84y^{0.64} \quad (3)$$

where y is the average depth excepting sides.

Width

Lacey plotted wetted perimeter P versus discharge Q, and fitted the equation

$$P=2.67Q^{1/2} \quad (4)$$

The Madras canals were wider and the Bari Doab canals were narrower.

Slope

Lacey employed his own form of the Manning equation:

$$V=\frac{1.3458}{n_a}R^{3/4}S^{1/2} \quad (5)$$

where n_a is the absolute channel roughness. The relations among silt factor, silt size, and channel roughness, he used velocity, hydraulic radius, bed material, and n. The silt factor was calculated from equation (2.8), and R. Kutter's n and Manning's n were known. The value of n_a was defined as Singh [70]

$$n_a=0.0225f^{1/4} \quad (6)$$

2.2 Regime Theory and Geometric Models of Alluvial Channels

An equilibrium state of river behavior is given as the regime concept (Kennedy, 1989, Ackers, 1992a). The study of the dimensions of stable alluvial canals, namely surface widths, cross-mean depths and stream wise slopes are called as regime canals. For predicting the dimensions of a regime fluvial channel from regime theory, a geometric model was also developed in order to design the full cross-sectional shapes of stable alluvial channels.

An alluvial channel can adjust its slope, depth and width, to develop a dynamic stable condition in which it can transport a certain amount of water and sediment (Ackers, 1992a). Empirical regime theory and analytical regime theory are two existing approaches. The field data is used to determine the empirical relationships from data. The two sets of equations are given for analytical regime theory, one for sediment transport and the other for frictional resistance. These equations were derived by relating the width to the discharge based on extensive data collected from the field. The development of rational regime relationships are interrelated by various analytical approaches. One type of approach introduces extremal hypotheses, such as minimum stream power [15,16] or maximum sediment concentration [74,75]. Others types of approach introduce a physical condition, such as bank stability (Ackers, 1980, Stevens [72] and Thorne, 1982), critical bank shear (Singh [69]), or lateral turbulence diffusion (Parker, 1978 a, b).

2.3 Numerical Modeling of Fluvial Processes

The study of the hydraulic geometry properties is very appropriate for natural stable channels. During the last decades it has seen that the transient behavior in fluvial processes are also of interest in river engineering. As a result numerical fluvial models have developed quickly in recent years. These models are used for calculating channel-bed degradation and aggradation (or scour and fill), and changes in bed topography. The theory of a numerical model for any fluvial process is given by response of the river's constant adjustment towards dynamic equilibrium. It has been observed that the dynamic equilibrium may never be actually reached in nature because of the changes in supply. Consequently, as an equilibrium condition the results of a long-term simulation of changes in the river behavior should approach the regime channel.

However, the present river numerical models suffer from the dependency on many empirical coefficients and assumptions concerning the treatment of sediment transport and turbulent structure of the flow field. Therefore these models are more or less limited to specific situations.

2.4. Modeling Width Adjustment of Alluvial Channels

The first numerical model for erodible channels dealing with width adjustment was developed by Chang [12].

Some of these models determine the migrative rates of both river banks using the concept of basal endpoint control (Darby & Thorne, 1994). Darby and Thorne (1994) stated that no model is currently able to simulate the flow within the crucial zone adjacent to the outer bank.

2.5. Aims and Objective of the Hydraulic Geometry Research

DOI: https://doi.org/10.30564/jgr.v2i2.632

The aim is to find a way of combining laws of deterministic fluid mechanics and probability theory in river engineering. The general objectives are therefore to:

(1) find a new way for river engineering research by combining the deterministic laws of fluid mechanics with the probability theory.

(2) gain an improved understanding of the mechanisms involved in regime theory and consequently to find certain similarities between numerous existing regime equations.

(3) Provide a computer-based hydraulic geometric model of stable alluvial channels for use in engineering applications.

2.6. Theoretical Background

Improving regime theories and geometric models of stable alluvial channels as well as fluvial numerical models are very important for river regulation because of their use in solving river engineering problems (Ackers, 1992a; Cao, 1987; Cao et al, 1987 and Yalin [106]). The deficiencies in models were pointed out by the National Research Council of USA (1983) as: (1) unreliable sediment discharge function; (2) inadequate formulation of the friction factor of erodible channels; (3) inadequate understanding and formulation of bed armouring and its effect on sediment discharge and friction factor; and (4) inadequate understanding and formulation of bank erosion mechanics. The accuracy of regime theories, geometric models and fluvial models depends ultimately upon the physical foundation, numerical techniques, and physical relationships for momentum, flow resistance, sediment transport, and bank erosion.

2.7. Boundary Shear Stress Distribution

Bank protection, sediment transport and width adjustment can be explored by the boundary shear stress distribution. There are either numerical methods (Keller & Rodi, [42]; Wormleaton [76]) or analytical methods [58] based on two-dimensional approaches. Three-dimensional turbulence models have also been developed and applied in order to study the pattern of secondary flow cells and the structure of the shear layer region (Kawahara & Tamai [41]; Krishnappan & Lau [59]).

2.8. Basic Approaches Ignoring the Effect of Secondary Currents

The basic approaches to calculate boundary shear stress distributions can be listed as following:

(1) Hydraulic radius method

$$\tau=\gamma RS \quad (7)$$

where τ=boundary shear stress, γ = unit weight of water, R= hydraulic radius; S= slope of water surface. It does however express the overall mean value for all shapes of cross section.

(2) Vertical depth method

$$\tau=\gamma hS \quad (6)$$

where h= vertical water depth. A fair approximation is obtained if the local boundary shear at any point is assumed to correspond to its vertical depth of flow immediately above the boundary.

(3) Normal depth method

$$\tau=\gamma h_n S \quad (9)$$

where h_n=depth along the normal depth of flow. If the inclination of the bed along the transverse direction is appreciable, then it will be found somewhat better to calculate the boundary shear by means of the depth at right angles to the bottom.

(4) Area method

The area method, which is an extension of the normal depth method, gives

$$\tau=\gamma h_n S(1-j/2) \quad (10)$$

here $j= ch_n$; c= curvature of bottom. It is more consistent to let the boundary shear correspond to the area between two normals.

However none of the previous four methods properly considers the transfer of shear in the transverse direction of flow.

(5) Extended area method

Lundgren and Jonsson (1964) extended the Prandtl's turbulence theory to take into account the transfer of momentum across normals to the bottom of the channel. They developed a modified area method to determine the bottom shear stress in shallow, symmetrical channels with a rough bottom and gently varying bottom curvature as

$$\tau=\gamma\bar{h}S \quad (11)$$

where $\bar{h}=h_n a$= effective depth; a= a factor which is a function of the transverse bed slope. However this method can not be used on polygonal shapes, such as a trapezoidal section.

(6) Modified area method

Based upon the original area method by Lundgren and Jonsson (1964), Parker (1978b) and Ikeda et al (1988) developed a modified area method as

$$\tau = \rho g S\frac{dA}{dP}+\frac{d}{dP}\int_0^{h_n}\tau_{nx}dn \quad (12)$$

where ρ=the density of fluid; dA= the area between normals to the bed; dP= the wetted perimeter above dA; n= a spatial co-ordinate along normals to the bed; τ_{nx}= the local downstream-directed shear stress induced by turbulence which acts on the normals.

2.9. Boundary shear stress theory by Knight et al (1994)

Some understanding of the lateral distribution of depth-mean velocity and boundary shear stress in channels of complex shape is given by Knight [47,48,49,50,51, 52,53] Knight et al, [43,45, 46, 38, 48,49, 1985, 57, 52, 67, 55, 50, 87] Shiono and Knight, [53, 55, 68] . High quality experimental data from their laboratory and the SERC-FCF provide a foundation for their theoretical analysis.

Shiono and Knight [68] combined the equation for longitudinal stream

$$\rho\left[\frac{\partial U}{\partial y}+\frac{\partial UW}{\partial z}\right]=\rho g S_o+\frac{\partial}{\partial y}\left(-\rho\overline{uv}\right)+\frac{\partial}{\partial z}\left(-\rho\overline{uw}\right) \qquad (13)$$

where x, y, z are streamwise, lateral and normal directions respectively, U, V, W are temporal mean velocity components corresponding to x, y, z, and u, v, w are turbulent perturbations of velocity with respect to the mean, r is the density of water, g is the gravitational acceleration, S_o is the bed slope gradient (S_o=sin q). Integration gives

$$\frac{\partial H(\rho UV)}{\partial y}=\rho g H S_o+\frac{\partial H\overline{\tau}_{yz}}{\partial y}-\tau_b\left(1+\frac{1}{s^2}\right)^{1/2} \qquad (14)$$

where t_b is the bed shear stress, s is the side slope (1:s, vertical:horizontal). Based on the eddy viscosity approach, the analytical solution to Eq. 8 is derived for a constant-depth domain as

$$U_d=\left[A_1e^{\gamma y}+A_2e^{-\gamma y}+\frac{8gS_oH}{f}(1-\beta)\right]^{1/2} \qquad (15)$$

for a linear-side-slope boundary conditions given as

$$U_d=\left(A_3\xi^{\alpha_1}+A_4\xi^{-\alpha_1-1}+\omega\xi+\eta\right)^{1/2} \qquad (16)$$

where

$$\gamma=\left(\frac{2}{\lambda}\right)^{1/2}\left(\frac{f}{8}\right)^{1/2}\frac{1}{H},\ \beta=\frac{\Gamma}{\rho g S_o H}$$

$$\alpha_1=-\frac{1}{2}+\frac{1}{2}\left(1+\frac{s(1+s^2)^{1/2}}{\lambda}(8f)^{1/2}\right)^{1/2}$$

$$\omega=\frac{gS_o}{\frac{(1+s^2)^{1/2}}{s}\frac{f}{8}-\frac{\lambda}{s^2}\left(\frac{f}{8}\right)^{1/2}}$$

$$\eta=-\frac{\Gamma}{\frac{(1+s^2)^{1/2}}{s}\rho\left(\frac{f}{8}\right)}$$

$$\Gamma=\frac{\partial(H\rho UV)_d}{\partial y}$$

and ξ=H-((y-b)/s), which is the depth function on the side-slope domain for the main-channel side slope.

2.10. Flow resistance

The resistance of flow in a loose boundary channel is composed mainly of bed resistance and and wall resistance. Bed resistance can be further divided into surface drag and bed form drag. The devised bed resistance approach can be commonly expressed by two approaches. One is in terms of the energy slope, developed in Europe by Meyer-Peter and Muller (1948), as

$$S= S'+S'' \qquad (17)$$

Another can be in termed of the hydraulic radius, given by Einstein (1942), as

$$R= R'+R'' \qquad (18)$$

where S' and R' are the energy slope and hydraulic radius resulting from surface grain roughness; S" and R" are the energy slope and hydraulic radius associated with form roughness. Multiplying Eq. 11 by gR gives the division for shear stress as

$$\tau=\tau'+\tau'' \qquad (19)$$

Dividing Eq. 13 by $\rho U^2/8$ gives the division for friction factor as:

$$f=f'+f'' \qquad (20)$$

where τ' and f' are counterparts to S' and τ" and f" are counterparts to S".

2.11 Grain Resistance and Flow Resistance Parameters

The Chezy resistance factor, C, is not dimensionless, it is common to let C=C'√g, and write

$$U= C\sqrt{gRS}= CU_* \qquad (21)$$

The Darcy-Weisbach formula for open channel flow can be presented as

$$U/U_*=\sqrt{8/f} \qquad (22)$$

or

$$\tau_b= (f/8)\ \rho\ U^2 \qquad (23)$$

The three friction factors are related to each other by

$$C/\sqrt{g} =(1/n)R^{1/6}/\sqrt{g}=\sqrt{8/f}=U/U_* \qquad (24)$$

In channels with sand or gravel boundary, the flow resistance in the absence of bed forms can be considered to be mainly caused by grain roughness. The well-known grain roughness formula based on Manning's is given by Strickler (1923) as

DOI: https://doi.org/10.30564/jgr.v2i2.632

$$n=d_{50}^{1/6}/21.1 \qquad (25)$$

Substituting Eq. 19 into Manning’s formula gives the Manning-Strickler formula

$$U/U_*=6.74(R/d_{50})^{1/6} \qquad (26)$$

Meyer-Peter and Muller (1948) developed a similar formula for sand mixtures as

$$N= (d_{90})^{1/6}/ 26 \qquad (27)$$

where d_{90} is the size (in metres) for which 90% of bed material is finer.

Einstein presented a logarithmic resistance equation for plane sand bed:

$$U/U_*= 5.75 \log (12.27\, d/\Delta) \qquad (28)$$

where $U_*=\sqrt{g}RS$ is the shear velocity resulting from grain roughness, D is the apparent roughness which is related to equivalent roughness k_s by

$$\Delta= k_s/ X = d_{65}/X \qquad (29)$$

The parameter X is a correction factor which accounts for the variation in flow regime. The value of X is given as a function of the laminar sublayer thickness δ. In the region of rough wall, X is unity and thus Δ and k_s are identical.

2.12 Form Resistance

The evaluation of the form resistance is more complicated. Different resistance laws are required for different bed forms. The transition between particular bed forms may require special relationships.

(1) Einstein and Barbarossa’s method (1952)

A function was therefore suggested for the lower regime flow:

$$U/U_*''=F(Y') \qquad (30)$$

where Ψ is the intensity of shear on representative particles and is given by

$$\Psi'_{35} = \frac{\rho_s - \rho}{\rho}\frac{d_{35}}{R'S} \qquad (31)$$

The functional relationship for Eq. 24 was based on field data in the form of a diagram for ease of application.

(2) Engelund’s method (1966)

Engelund’s method employs the divided slope approach and assumes that S=S’+S”, where S’ is due to skin friction and S'' is due primarily to expansion losses associated with flow separation downstream of dune crests as

$$S''= \Delta H''/\lambda=(\alpha\, k^2/2\lambda h)F^2 \qquad (32)$$

where ΔH” is the expansion head loss due to bed forms with a wave length of λ, α is the loss coefficient, k is dune height, h is mean depth of water. Substituting Eq. 26 into Eq.11

yields

$$S=S'+(\alpha\, k^2/2\lambda h)F^2 \qquad (33)$$

Multiplying both sides by $\gamma R/(\gamma_s-\gamma)d$ gives

$$\frac{\gamma RS}{(\gamma_s-\gamma)d} = \frac{\gamma RS'}{(\gamma_s-\gamma)d} + \frac{\alpha}{2}\frac{\gamma h^2}{(\gamma_s-\gamma)\lambda d}F^2 \qquad (34)$$

Assuming

$$\Theta = \frac{\gamma RS}{(\gamma_s-\gamma)d}$$
$$\Theta' = \frac{\gamma RS'}{(\gamma_s-\gamma)d}$$
$$\Theta'' = \frac{\alpha}{2}\frac{\gamma h^2}{(\gamma_s-\gamma)\lambda d}F^2 \qquad (35)$$

then Eq. (28) becomes

$$\Theta=\Theta'+\Theta'' \qquad (36)$$

where,

θΘ,Θθ’ and Θθ” are the dimensionless total shear, shear due to grain roughness, and shear due to bed-boundary layer roughness, respectively. Using flume data, Engelund and Hansen (1967) obtained the following relationship for lower regime with a ripple or dune bed (for Θθ’<0.55):

$$\Theta'= 0.06+0.4\,\Theta \qquad (37)$$

For the upper regime flow with 0.55 < Q’< 1, the relationship becomes

$$\Theta'= \Theta \qquad (38)$$

2.13 Discussion

As a basis for concluding the discussion, a sensitivity analysis on the hydraulic geometry of stable channels will now be conducted using the five key parameters and the new geometric model. Based on the results of this sensitivity analysis, the general behaviour of depths, surface widths and streamwise slopes of stable alluvial channels are discussed. Furthermore the new concepts proposed in this chapter are connected together and coupled with other necessary existing relationships to construct a new rational regime theory.

2.14 Sensitivity Analysis of Five Parameters

The parameters to be tested are:

1. median grain diameter of the boundary, d_{50};
2. gradation of the boundary material, $\xi=d_{90}/d_{50}$;
3. the bank stability index, σ;
4. longitudinal slope, S, and
5. discharge, Q.

The resulting curves of calculated values, expressed as number 1 to 7, are shown in Table 1 and 2, respectively. The curves are

1. channel average-depth, H_a, in cm;
2. channel centre depth, H_c, in cm;
3. cross-sectional mean velocity, U, in cms^{-1};
4. surface width, B, in m;
5. bed width, b, in m;
6. bank width (B-b), in m; and
7. aspect ratio, B/H_c.

Table.1 Observed data of the channel, (Andrews [8])

Bankfull Discharge (m^3s^{-1}) 255	**Bankfull depth (m) 1.85**	**Bankfull width (m) 83.8**	**Bankfull velocity (ms^{-1}) 1.64**
Slope $X10^{-3}$ 0.88	d_{50} of river bed (m) 0.034	d_{90} of river bed (m) 0.082	Bank vegetation type : Thick or thin Thin

Table. 2 Sensitivity

Parameters to be tested	**(1) d_{50} (mm)**	**(2) d_{90}/d_{50}**	**(3) σ**	**(4) Sx1000**	**(5) $Q(m^3s^{-1})$**
Values tested	20-40	2-4	1-2	1-2	500-1000
%increased	100	100	100	100	100
H_a (cm) Change by %	145-256 76	249-291 17	228-400 75	206-110 ¯47	240-247 3
H_c (cm) Change by %	150-300 100	287-367 28	255-510 100	224-112 ¯50	255-255 Constant
$U(cms^{-1})$ Change by %	163-222 36	212-222 5	207-244 18	205-186 ¯9	210-212 1
B(m) Change by %	107.8-45.0 ¯58	48- 39 ¯8	54-26 ¯52	61-124 103	99-190 92
b(m) Change by %	97.8-25.0 74	29-15 49	37-9 75	46-117 154	82-173 112
B-b (m) Change by %	10-20 100	19-24 28	17-17 Constant	15-7.5 50	17-17 Constant
B/H_c Change by %	72-15 79	17-11 36	21-5 76	29-113 290	41-77 87

Analysis results of five parameters on:1. Channel average-depth, H_a, in cm; 2. Channel centre depth, H_c, in cm; 3. Cross-sectional mean velocity, U, in cms^{-1}; 4. Surface width, B, in m; 5. Bed width, b, in m; 6. Bank width B-b, in m; and 7. Aspect ratio B/H_c [8].

2.15. Influence of Median Grain Diameter of Boundary Material, d_{50}

Constant input variables: discharge, $Q=255\ m^3s^{-1}$, longitudinal slope, S=0.00088, gradation of boundary material, $\xi=d_{90}/d_{50}=1$, bank stability index, σ=1, β=0.15, μ=0.6, $\rho^*=(\rho_s - \rho)/\rho = 1.65$) are given in sensibility analysis. It is very obvious that d_{50} exerts a strong influence on the hydraulic geometry of a stable alluvial channel. For example, increasing d_{50} by 100% from 20 mm to 40 mm causes the cross-sectional average depth, H_a, to increase by 76% (from 1.454 m to 2.556 m); the centre depth, H_c, to increase by 100% (from 1.5 m to 3.0 m); the cross-sectional mean velocity, U, to increase by 36% (from 1,626 ms^{-1} to 2.216 ms^{-1}); the surface width, B, to decrease by 58% (from 107.8 m to 45.0 m); the width of bank region, B-b, to increase by 100% (from 10 m to 20 m); the aspect ratio, B/H_c, to decrease by 79% (from 71.9 to 15.0); and the width of centre bed region, b, to decrease by 74% (from 97.834 m to 25.046 m). The results show that for given constant input condition the stable Type-A channel will approach a Type-B threshold channel for a median grain diameter of boundary material somewhat larger than 60 mm. In this case, the size of bank material is big enough to keep a large stable centre depth with a enough cross-sectional area to transport the given discharge. As a result, the sediment transport is vanished and the centre bed zone cannot be formed (Ackers and Charlton, 1970a, b, Ackers, 1972, 1992a).

3. Results

The surface width equation is employed as

$$B= 4632\ (H_a^2S/d_{50})^{1.5}$$

The complete set of rational regime equations are given as:

$$S = a_1\ d_{50}^{1.079}/Q^{0.359}$$

$$H_a= a_2Q^{0.359}/\ d_{50}^{0.079}$$

$$B = a_3Q^{0.539}/\ d_{50}^{0.119}$$

$$U = a_4\ Q^{0.102}d_{50}^{0.198}$$

$$Q_s= a_5\ Q^{0.736}d_{50}^{0.778}$$

where Q_s= cross-sectional sediment discharge at bankfull discharge in kgs^{-1}. The coefficients, a_1 to a_5, were the functions of the coefficient, a, and calibrated from observed values of Q, d_{50}, S, H_a, B, Q_s, and U by 53 set data published by Bray (1979), Lane et al (1953) and Ikeda

DOI: https://doi.org/10.30564/jgr.v2i2.632

(1988) as a_1= 0.360, a_2=0.168, a_3= 2.852. a_4= 1.923 and a_5= 2.170. The comparison of the observed data and the calibrated equations are given. The scatter of the observed data around the line given by the equations is acceptable for B, H_a and U. In the case of slope equation the observed data show considerable scatter, perhaps mainly due to the fact that slope is a difficult parameter to measure accurately (Parker, 1979).

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