THE GATE COACH

Revised Study Material

For

GATE – PSU

Civil Engineering

Irrigation

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Chapter 3

Sediment Transport and Design of Irrigation Channels

Whenever water flows in a channel (natural or artificial), it tries to scour its surface. Silt or gravel or even larger boulders are detached from the bed or sides of the channel. These detached particles are swept downstream by the moving water. This phenomenon is known as Sediment Transport.

Mechanics of Sediment Transport

Where R = is the hydraulic mean depth of channel S = channel bed slope

Unit wt. of water P = wetted perimeter Hence Average unit tractive force, also called shear stress

= $\tau_0 = \gamma_\omega RS$ We can hence conclude that the Applied Tractive Stress (average) exerted by the flowing water under uniform flow conditions is given by Eq. (as equal

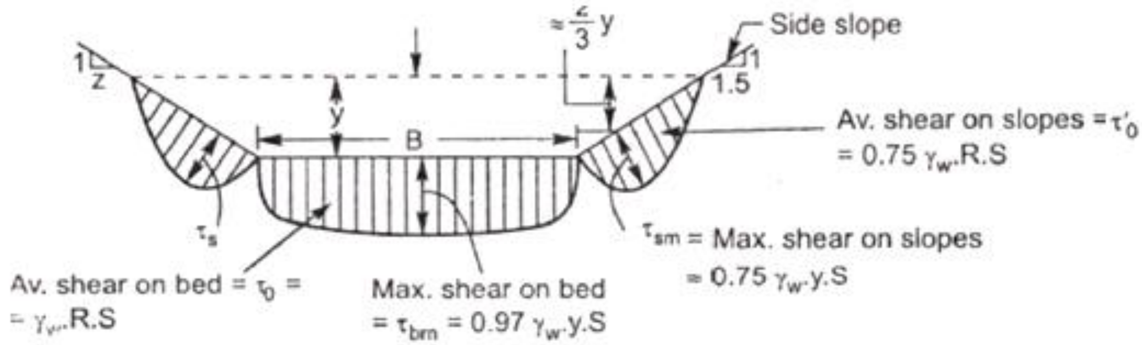
However, the maximum stress induced by any flow occurs at the point of greatest depth or at the centre of any channel with horizontal bed, and is given by:

$$\tau_{max} = \gamma_\omega . yS$$

Where γ_ω = unit wt of water y = water depth in channel S = channel bed slope

It has further been established by lane by lane by using membrane technology that the tractive or shear stress in channels, except for wide open channels, is not uniformly distributed along the wetted perimeter. Variations do occur across the entire cross section of the channel.

A typical shear stress variation along the boundary of a trapezoidal channel is shown in the maximum shear at bed (τ_{bm}) occurs at the centre of the section and reduces gradually and then abroupty to zero at both corners.



Typical shear stress distribution along the boundry of a trapezoidal channel.

The value of τ_{bm} is approximately given as:

$$\tau_{bm} = 0.97 * \tau_{\max} = 0.97 * \gamma_{\omega}.y.S$$

Along the side slopes, the maximum shear (τ_{sm}) occurs at approximately two thirds of the depth of flow with magnitude given as:

$$\tau_{sm} = 0.75 \gamma_{\omega}.S$$

Hence, for designing non scouring channels in coarse alluviums

$$\frac{\tau_c}{\gamma_{\omega} d(S_s - 1)} = 0.056 (for\, d > 6mm)^*$$

Where γ_{ω} = unit wt. of water = 9.18 kN/m^3

Or 1 t/m^3 or 1000 kgf/m^3

The average shear stress caused on the bed of a channel by the flowing water is given by

$\tau_0 = \gamma_{\omega} RS$ Where R = Hydraulic mean radius of the channels, i,e. A/P

Moreover, τ_0 should be $\leq$ τ_c; (for non scouring channels) or $\quad \tau_0 \le \gamma_\omega d(S_s - 1)\,(0.056) \quad$ or $\gamma_\omega RS \le \gamma_\omega d(S_s - 1)\,(0.056)$

or $\quad RS \le d\,(S_s - 1)\,(0.056) \quad$ or $RS \le d\,(2.65 - 1)\,(0.056)$

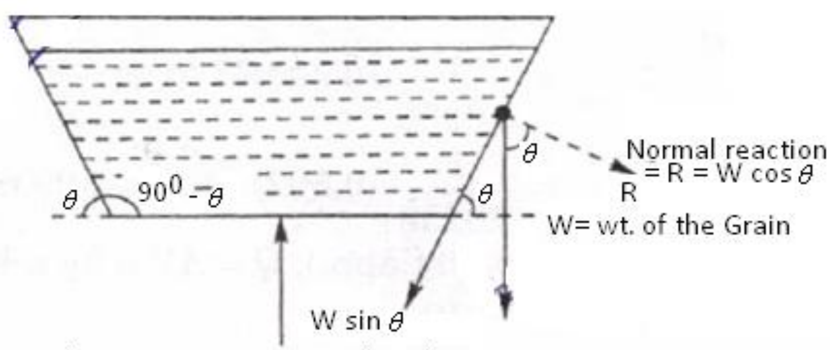
or $\quad RS \le \dfrac{d}{11}$

or $\quad d \ge 11RS$

Equation gives the minimum size of the bed material or lining stone that will remain atrest in a channel of given R and S.

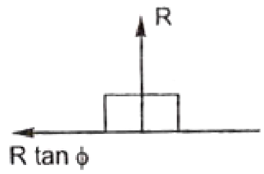
Stability of channel slopes (design of non scouring channels with unprotected side slopes)

Up to now, we have considered the stability of horizontal beds, where the shear stress τ_0 (given by $\tau_0 = \gamma_\omega RS$)was the only disturbing force. But on side slopes of channels, one more disturbing force, i.e., the component of the weight of the partial also comes into picture. We will now consider a grain on the side slope of a channel. Various forces acting on this grain are shown in

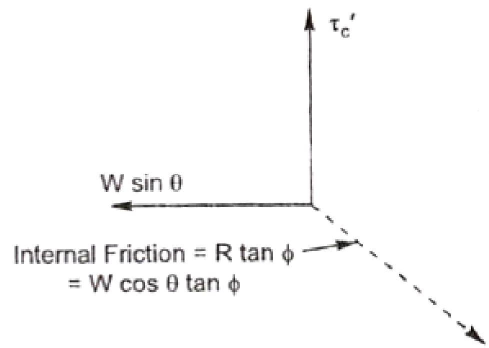


Flow Direction is Perpendicualr to paper
(a) Forces acting on a grain
on the side slope of a channel.

Now, let τ_c’be the shear stress required to move the grain of weight w on the side slope. The free body diagram of various forces acting on the grain is shown



(b) where ϕ = Angle of repose of soil



(c) Free body diagram of forces

Let τ_c represents the critical shear stress or the shear stress required to move a similar grain on a horizontal bed, as shown in

$$\boxed{\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}}$$

The above equation shows that $\tau_c' < \tau_c$: which means that the shear stress required to move a grain on the side slopes is less than the shear stress required to move the grain on canal bed.

Kennedy's Theory

R.G. kennedy, an executive engineer of Punjab P.W.D, carried out extensive investigations on some of the canal reaches in the upper Bari Doab canal system. He selected some straight reaches of the canal section, which had not posed any silting and scouring problems during the previous 30 years or so.

Form the observations, he concluded that the silt supporting power in a channel cross section was mainly dependent upon the generation of the eddies, rising to the surface. These eddies are generated due to the friction of the flowing water with the channel surface, the vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. So if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension, silting will be avoided based upon this concept, he defined he critical velocity (V_0) in a channel as the mean velocity

(across the section) which will just keep the channel free form silting or scouring, and related it to the depth of flow by the equation

The equation for critical velocity was, thus, modified as:

$V_0 = 0.055\ my^{0.64}$

Where V_0 = critical velocity in the channel in m/s

y = water depth in channel in m

m = C.V.R.

Design procedure.

Determine the critical velocity V_0 by the above by assuming a trail depth, and then determine area by dividing discharge by velocity. Then determine channel dimensions. Finally, compute the actual mean velocity (V) that will prevail in the channel of this cross section,, by using Kutter's formula, manning's formula, etc. if the two velocity's V_0 and V work out to be the same, then the assumed depth is all right, otherwise change it and repeat the procedure till V and V_0 become equal

Kutter's formula

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{s}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where V = velocity of flow in meteres/sec

R = Hydraulic mean depth in meters.

S= Bed slope of the channel.

n = Rugosity coefficient

Use of Garret's Diagrams for Applying Kennedy's Theory. A

Lot of mathematical calculations is required in designing irrigation channels by the use of Kennedy's method. To save mathematical calculations, graphical solution of Kennedy's and Kutter's equations, was evolved by Garret. The original diagrams given by him were in F.P.S. system, but here they have been changed into M.K.S./S.I. system. The procedure adopted for design of irrigation channels using Garret's diagrams is explained below:

(i) The discharge, bed slope, rugosity coefficient, value of C.V.R. is given for the channel to be designed.

(ii) Find out the point of intersection of the given slope line and discharge curve. At this point of intersection, draw a vertical line intersecting the various bed width curves.

(iii) For different bed widths (B), the corresponding values of water depth (y) and critical velocity (V_0) can be read on the right hand ordinate. Each such pair of bed width (B) and depth (y) will satisfy Kutter's equation, and is capable of carrying the required discharge at the given slope and
rugosity coefficient. Choose one such pair and determine the actual velocity of flow

(iv) Determine the critical velocity ratio (V/V0) taking V as calculated and
VQ as read.

(v) If the value of C.V.R. is not the same as given in question, repeat the
procedure with other pairs of B and y.

Lacey's Theory

Lacey, an eminent civil engineer of U.P. Irrigation Department, carried out extensive investigations on the design of stable channels in alluviums. On the basis of his research work, he found many drawbacks in Kennedy's Theory (1895) and he put forward his new theory.

Lacey's regime channels.

It was stated by Kennedy that a channel is said to be in a state of 'regime' if there is neither silting nor scouring in the channel. But Lacey came out with the statement that even a channel showing no silting no scouring may actually not be in regime. He, therefore, differentiated between three regime conditions:

(i) True regime ;

(ii) Initial regime ; and

(iii) Final regime.

True regime.

A channel shall be in regime, if there is neither silting nor scouring.

Initial regime

When only the bed slope of a channel varies due to dropping of silt, and its cross-section or wetted perimeter remains unaffected, even then the channel can exhibit 'no silting no scouring' properties, called *Initial regime*.

Final regime.

If there is no resistance form the sides, and all the variables such as perimeter, depth, slope, etc. are equally free to vary and finally get adjusted according to discharge and silt grade, then the channel is said to have achieved permanent stability, called final regime

Design procedure for Lacey's theory

(1) Calculate the velocity from equation

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} m/\sec$$

Where Q is in cumec :

V is in m/s ; and

f is the silt factor given by

f = 1.76. $\sqrt{d_{mm}}$

where d_{mm} = Average particle size in mm, as given in

Table Values of particle size (d_{mm}) for various types of alluvial materials for use in

S. No	**Type of material (soil)**	**Av. Grain size in mm (d_{mm}) (3)**
1	Silt Very fine medium Standard Sand	0.06 to 0.08 0.12 0.16 0.32 (f = 1.0)
2	Medium Coarse Bajri and sand	0.51 0.73

3	Fine Medium Coarse	0.89 1.29 2.42
4	Gravel Medium Heavy	 7.28 26.10
5	Boulders Small Medium Large	 50.10 72.50 188.80

(2) Work out the hydraulic mean depth (R) form the equation

R = $R = \frac{R}{2}\left(\frac{V^2}{f}\right)$

Where V is in m/sec; R is in m.

(3) Compute area of channel section $A = \frac{Q}{V}$

(4) Compute wetted perimeter, P = 4.75 $\sqrt{Q}$

(5) Knowing these values, the channel section is known; and finally the bed slope S is determined by the equation

$$S = \left[\frac{f^{5/3}}{3340 Q^{1/6}} \right]$$

Where f is the silt factor, given by Q is the discharge