4.7.2. Regime Channels. A channel is said to be in a state of 'Regime', if the flow is such that 'silting and scouring' need no special attention. Such a state is not easily possible in rivers, but in artificial channels, such a state can be obtained by properly designing the channel.

The basis for designing such an ideal, non-silting, non-scouring channel is that, whatever silt has entered the channel at its head is kept in suspension, so that it does not settle down and deposit at any point of the channel. Moreover, the velocity of the water should be such that it does not produce local silt by erosion of channel bed and slopes.

4.7.3. Kennedy's Theory (1895). 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.

From 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 the **critical velocity** (V_0) in a channel as the *mean velocity* (across the section) which will just keep the channel free from silting or scouring, and related it to the depth of flow by the equation

$$V_0 = c_1 \cdot y^{c_2}$$

where c_1 and c_2 are constants depending upon silt charge.

c_1 and c_2 were found to be 0.55 and 0.64 (in M.K.S. or S.I. units), respectively.

Therefore, $V_0 = 0.55\, y^{0.64}$ ...(4.18)

Since this formula was worked out especially for the upper Bari Doab canal system, it could not have been applicable in toto to other canals or canal systems due to variation in the type of soil (or silt) at various canal sites. Realising this lacuna, Kennedy later introduced a factor (m) in this equation, to account for the type of soil through which the canal was to pass. This factor, which was dependent upon the silt grade, was named as **critical velocity ratio (C.V.R.)** and denoted by *m*.

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

$$\boxed{V_0 = 0.55\, m y^{0.64}} \quad ...(4.19)$$

where V_0 = Critical velocity in the channel in m/s.

y = water depth in channel in m

$m = C.V.R$

For sands coarser than the standard, the values of *m* were given from 1.0 to 1.2 ; and for sands finer than the standard, *m* was valued between 1.0 to 0.7, as shown in Table 4.2.

Table 4.2. Recommended Values of C.V.R. (m)

S.No.	*Type of silt*	*Value of m*
1.	Silt of River Indus (Pakistan)	0.7
2.	Light sandy silt in North Indian Rivers	1.0
3.	Light sandy silt, a little coarser	1.1
4.	Sandy, loamy silt	1.2
5.	Debris of hard soil	1.3

Design procedure. Determine the critical velocity V_0 by the above Eq. (4.19) by assuming a trial 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 velocities 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

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}} \right] \sqrt{RS} \quad ...(4.20)$$

Manning's Formula

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2} \quad ...(4.21)$$

where V = Velocity of flow in metres/sec.

R = Hydraulic mean depth in metres.

S = Bed slope of the channel.

n = Rugosity coefficient.

The values of n in both these equations depend upon channel condition and also upon discharge. The values of n may be taken as given in Table 4.3.

Table 4.3. Recommended Values of Manning's Coefficient *n* for Unlined Channels

Condition of channel	*Value of n*
Very good	0.0225
Good	0.025
Indifferent	0.0275
Poor	0.030

The Central Board of Irrigation and Power (India) has recommended the following values of n for different discharges.

Table 4.4. Values of Manning's *n* for Different Discharges

Discharge in cumec	*Value of n for Unlined Channels*
14 to 140	0.025
140 to 280	0.0225
280 and above	0.020

Chezy's Formula

$$V = C\sqrt{RS} \quad \text{...(4.22)}$$

where C = a constant depending upon the shape and surface of the channel.

R and S have the same meaning as in eq. (4.21).

The actual mean velocity(V) generated in the channel can be computed by any of these three resistance equations, but generally Kutter's equation is used with Kennedy's theory.

Example 4.6. *Design an irrigation channel to carry 50 cumecs of discharge. The channel is to be laid at a slope of 1 in 4000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.023.*

Solution. $Q = 50$ cumecs, $S = \frac{1}{4000}$

$m = 1.1$, $n = 0.023$

Use equation (4.19), as, $V_0 = 0.55m \cdot y^{0.64}$

Assume a depth equal to 2 m

$$V_0 = 0.55 \times 1.1 \times (2)^{0.64} = 0.605 \times 1.558 = \mathbf{0.942} \text{ m/sec}$$

$$A = \frac{Q}{V_0} = \frac{50}{0.942} = 53.1 \text{ m}^2.$$

Assume side slopes as $\frac{1}{2} : 1 \left(\frac{1}{2} H : 1V\right)$

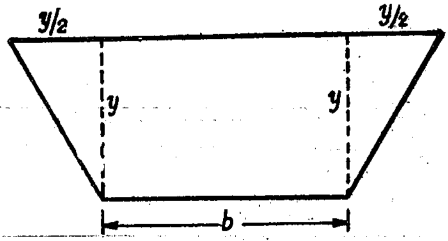

Fig. 4.9.

Now, $\quad A = y\left(b + y \cdot \frac{1}{2}\right)$

$\therefore \quad 53.1 = 2\,(b + 1)$

or $\quad 26.55 = b + 1$

or $\quad b = 25.55$ m

and $\quad P = b + 2\sqrt{\left(1 + \frac{1}{4}\right)} \times y$

or $\quad P = b + 2\frac{\sqrt{5}}{2} y = 25.55 + \sqrt{5} \times 2 = 30.03$

$$R = \frac{A}{P} = \frac{53.1}{30.03} = 1.77 \text{ m.}$$

But, from eqn. (4.20),

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

$$\therefore \quad V = \left[\frac{\frac{1}{0.023} + 23 + \frac{0.00155}{1/4000}}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{1.77}}}\right]\sqrt{1.77 \times \frac{1}{4000}}$$

$$= \left[\frac{43.5 + (23 + 6.2)}{1 + \frac{29.2 \times 0.023}{1.33}}\right]\left[1.33 \times \frac{1}{63.3}\right]$$

$$= \frac{72.7}{1 + 0.505} \times 1.33 \times \frac{1}{63.3} = \frac{72.7}{1.505} \times 1.33 \times \frac{1}{63.3}$$

$$= 1.016 \text{ m/sec} > 0.942 \text{ ; or } V > V_0.$$

In order to *increase* the critical velocity (V_o),, we have to *increase* the depth. So increase the depth.

Use 3 m depth :

$$V_0 = 0.605 \times (3)^{0.64} = 0.605 \times 2.02 = 1.22 \text{ m/sec.}$$

.8 m^2.

3)

or $\quad 13.6 - 1.5 = b = 12.1$ m.

$$P = 12.1 + 2 \times \frac{\sqrt{5}}{2} 3 = 12.1 + 6.72 = 18.82$$

$$R = \frac{A}{P} = \frac{40.8}{18.82} = 2.17 \text{ ; therefore } \sqrt{R} = 1.47.$$

$$V = \frac{43.5 + 29.2}{1 + \frac{29.2 \times 0.023}{1.47}} + \left[1.47 \times \frac{1}{63.3}\right] = \frac{72.7}{1.45} \times 1.47 \times \frac{1}{63.3}$$

$$= 1.16 \text{ m/sec.} < 1.22 \text{ ; or } V < V_0$$

So reduce the depth.

Use 2.5 m depth

$$V_0 = 0.605 \times (2.5)^{0.64} = 0.605 \times 1.797 = 1.087 \text{ m/sec.}$$

$$A = \frac{50}{1.087} = 46$$

$$46 = 2.5\,(b + \tfrac{1}{2} 2.5)$$

$$18.4 - 1.25 = b = 17.15 \text{ m}$$

$$P = 17.15 + \sqrt{5} \times 2.5 = 17.15 + 5.58 = 22.73$$

$$R = \frac{A}{P} = \frac{4}{22.73} = 2.02 \text{ ; therefore } \sqrt{R} = 1.42$$

$$V = \frac{72.7}{1 + \frac{29.2 \times 0.023}{1.42}} (1.42) \left(\frac{1}{63.3}\right) = \frac{72.7}{1.472} \times \frac{1.42}{63.3}$$

$$= 1.1 \text{ m/sec} > 1.087 \text{ ; } V > V_0$$

So increase the depth.

Use 2.7 m depth

$$V_0 = 0.605 \times 1.189 = 1.147$$

$$A = \frac{50}{1.147} = 43.5$$

$$43.5 = 2.8\,(b + \tfrac{1}{2} 2.8)$$

$$15.54 - 1.4 = b = 14.14 \text{ m}$$

$$P = 14.14 + \sqrt{5} \times 2.8 = 14.14 + 6.26 = 20.40$$

$$R = \frac{43.5}{20.4} = 2.13, \text{ therefore, } \sqrt{R} = 1.46$$

$$\therefore \quad V = \left[\frac{72.7}{1 + \frac{29.2 \times 0.023}{1.46}}\right]\left[\frac{1.46}{63.3}\right] = \left[\frac{72.6}{1.46}\right]\left[\frac{1.46}{68.3}\right]$$

$$= 1.148 \text{ m/sec} \approx 1.147 \text{ or } V \approx V_0.$$

Actual velocity V tallies with V_0.

Hence, use the depth equal to 2.7 m and base width 14.14 m. (say 14.2 m) with slopes $\frac{1}{2}$: 1 of trapezoidal section. **Ans.**

Example 4.7. *Design an irrigation channel to carry 40 cumecs of discharge, with B/D, i.e. base width to depth ratio as 2.5. The critical velocity ratio is 1.0. Assume a suitable value of Kutter's rugosity coefficient and use Kennedy's method.*

Solution. $V_0 = 0.55\,(y)^{0.64}$ ($\because\ m = 1$)

Here $y = D$

$\therefore \quad V_0 = 0.55 \cdot D^{0.64}$

$Q = AV$

Using $\frac{1}{2}$: 1 slopes, area (A) of trapezoidal section is given as :

$$A = BD + 2 \cdot \frac{1}{2} \cdot D \frac{D}{2} = D\left[B + \frac{D}{2}\right]$$

$$\therefore \quad 40 = D\left[B + \frac{D}{2}\right] V_0$$

But $\quad B/D = 2.5$; or $B = 2.5D$

$$\therefore \quad 40 = D\,[2.5\,D + 0.5D]\,V_0 = D\,[3D]\,V_0 = 3D^2 \cdot V_0$$

But $\quad V_0 = 0.55 \cdot D^{0.64} \qquad \therefore \quad 40 = 3D^2\,(0.55 \cdot D^{0.64})$

or
$$D^{2.64} = \frac{40}{3 \times 0.55} = 24.2$$

or
$$D = (24.2)^{\frac{1}{2.64}} = (24.2)^{0.379} = \mathbf{3.34\ m}$$

Now $\quad B = 2.5\,D = 2.5 \times 3.34 = \mathbf{8.35\ m}$

Now determine the slope S

$$A = 3D^2 = 3 \times (3.34)^2 = 33.5\ \text{m}^2.$$

$$P = \left[B + 2 \cdot \frac{\sqrt{5}}{2} D\right] = (8.35 + \sqrt{5} \times 3.34) = (8.35 + 7.46) = 15.81\ \text{m}$$

$$R = \frac{33.5}{15.81} = 2.12, \text{ or } \sqrt{R} = 1.456$$

$$V_0 = 0.55\,(3.34)^{0.64} = 0.55 \times 2.163 = \mathbf{1.19}$$

Assume $\quad n = 0.023.$

Using Eq. (4.20), we get

$$V = \left[\frac{\dfrac{1}{0.023} + \left(23 + \dfrac{0.00155}{S}\right)}{1 + \left(23 + \dfrac{0.00155}{S}\right)\dfrac{0.023}{1.456}}\right] 1.456\sqrt{S} \qquad \text{...}(i)$$

Assume $\quad S = \dfrac{1}{4000}.$

Putting this value of S and computing the value of V, we get

$$V = \frac{43.5 + (23 + 6.2)}{1 + 29.2 \times \dfrac{0.023}{1.456}} \times \frac{1.456}{63.3} = 1.114$$

$$1.114 < 1.19 \qquad \text{or} \qquad V < V_0$$

Therefore, to increase the value of V, we must increase/steepen the slope ; hence, use a slope = 1 in 3700 (say)

Putting $\quad S = \dfrac{1}{3700}$ in (i) above, we get

$$V = 1.189 \approx 1.19$$

or $\quad V = V_0$ for value of $S = \dfrac{1}{3700} \qquad$ So use $S = \dfrac{1}{3700}$

Hence, use a trapezoidal channel section as follows :

$$\left.\begin{aligned} \text{Depth} &= 3.34 \text{ m} \\ \text{Base width} &= 8.35 \text{ m} \\ \text{Side slopes} &= \tfrac{1}{2} H : 1 V \\ \text{Bed slope} &= 1 \text{ in } 3700 \end{aligned}\right] \quad \textbf{Ans.}$$

4.7.4. Lacey's Theory (1939). 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. The essential points which he argued, and the design procedure which he suggested, is briefly described here.

4.7.4.1. 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.*

According to him, a channel which is under 'initial' regime, is not a channel in regime (though outwardly it appears to be in regime, as there is no silting or scouring) and hence, regime theory is not applicable to such channels. His theory is therefore applicable only to those channels, which are either in *true regime* or *in final regime.*

4.7.4.2. True regime. A channel shall be in regime, if there is neither silting nor scouring. For this condition to be satisfied, the silt load entering the channel must be carried through, by the channel section. Moreover, there can be only one channel section and one bed slope at which a channel carrying a given discharge and a particular quantum and type of silt, would be in regime. Hence, an artificially constructed channel having a certain fixed section and a certain fixed slope can behave in regime only if the following conditions are satisfied :

(*i*) *Discharge is constant ;*

(*ii*) *Flow is uniform ;*

(*iii*) *Silt charge is constant* ; *i.e.* the amount of silt is constant ;

(iv) *Silt grade is constant* ; *i.e.*, the type and size of silt is always the same, and

(v) *Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil is known as* **incoherent alluvium***), *and is of the same grade as is transported.*

Hence, a designed channel shall be in 'true regime' if the above conditions are satisfied. But in practice, all these conditions can never be satisfied. *And, therefore, artificial channels can never be in 'true regime'; they can either be in initial regime or final regime*, as explained below :

4.7.4.3. Initial regime and Final 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*. Thus, when water flows through an excavated channel with somewhat narrower dimensions and defective slopes, the silt carried by the water may get dropped in the upper reaches, thereby increasing the channel bed slope. Consequently, the velocity is increased, and a non-silting equilibrium is established, called 'Initial regime'. Sides of such channels are subjected to a lateral restraint and could have scoured if the bank soil would have been a true alluvium. But in practice, they may either be grassed or be of clayey soil, and therefore, they may not get eroded at all. Hence, such channels will exhibit 'non-silting, non-scouring' properties, and they will appear to be in regime ; but in fact, they are not. *They have achieved only a working stability due to the rigidity of their banks.* Their slopes and velocities are higher and cross-sections narrower than what would have been if the sides were not rigid. *Such channels are termed as channels in initial regime, and regime theory is not applicable to them, as they are infact, not the channels in alluvium.*

But, if there is no resistance from 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*. Regime theory is applicable to such channels only, and not to all regime channels, as was envisaged by Kennedy.

Such a channel in which all variables are equally free to vary, has a tendency to assume a semi-elliptical section. The coarser the silt, the flatter is the semi-ellipse, *i.e.* greater is the width of the water-surface. The finer the silt, the more nearly the section attains a semi-circle (Fig. 4.10).

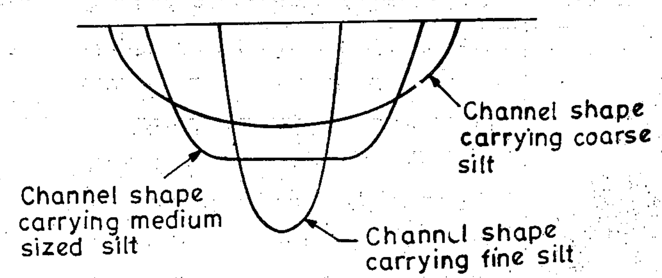

Fig. 4.10

The second point which Lacey argued was that the sediment is kept in suspension not only by the vertical component of the eddies which are generated on the channel

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* **incoherent** soil, as defined in article 4.4, is the soil, which does not possess any cohesion between its soil grains. The alluvial soil is defined in article 3.2, as the soil formed by continuous deposition of silt by the moving water. Such sandy soil deposits which donot possess any cohesion (c = 0) and are formed by the agency of water are called incoherent alluvium.

bed, but also by the eddies generated on the sides of the channel. Kennedy had neglected the eddies that are generated on the sides of the channel, by presuming that such eddies has horizontal movement for greater part, and, therefore, do not have sediment supporting power. Lacey thus, argued that the silt supporting power of a channel is proportional to the wetted perimeter of the channel and not to its width, as was presumed by Kennedy.

Thirdly, Lacey argued that the grain size of the material forming the channel is an important factor, and should need much more rational attention than what was given to it by Kennedy (different values of critical velocity ratio(m) for different types of soils). He, therefore, introduced a term called **silt factor** (*f*) in his equations, and connected it to the average particle size (as per equation (4.24)).

The various equations put forward by Lacey for the design of stable channels are given below :

4.7.4.4. Design procedure for Lacey's theory

(1) Calculate the velocity from equation

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} \text{ m/sec.} \quad ...(4.23)$$

where Q is in cumec :
V is in m/s ; and
f is the **silt factor**, given by

$$f = 1.76 \cdot \sqrt{d_{mm}} \quad ...(4.24)$$

where d_{mm} = Average particle size in mm, as given in Table 4.6.

Table 4.6. Values of Particle size (d_{mm}) for Various types of Alluvial materials for use in eq. (4.24)

S. No. (1)	*Type of material (soil) (2)*	*Av. grain size in mm (d_{mm}) (3)*
1	*Silt,*	
	Very fine	0.05 to 0.08
	Fine	0.12
	Medium	0.16
	Standard	0.32 (f = 1.0)
2	*Sand,*	
	Medium	0.51
	Coarse	0.73
3	*Bajri and Sand,*	
	Fine	0.89
	Medium	1.29
	Coarse	2.42
4	*Gravel,*	
	medium	7.28
	heavy	26.10
5	*Boulders,*	
	small	50.10
	medium	72.50
	large	188.80

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

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) \quad ...(4.25)$$

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

(3) Compute area of channel section $A = \frac{Q}{V}$ ...(4.26)

(4) Compute wetted perimeter, $P = 4.75\sqrt{Q}$ ...(4.27)

where P is in m ; Q is in m^3/sec.

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

$$S = \left[\frac{f^{5/3}}{3340\,Q^{1/6}}\right] \quad ...(4.28)$$

where f is the silt factor, given by Eq. (4.24)
Q is the discharge in cumec.

Lacey's Regime Width and Scour Depth for Alluvial Rivers :

For wide streams or rivers, as we know, wetted perimeter P, approximately equals the river width. Therefore, according to Lacey, for alluvial rivers :

The regime width $= W = 4.75\sqrt{Q}$ ...(4.29)

For such streams, Lacey has also defined the regime scour depth, as

*Lacey's Normal Regime Scour Depth** $= \mathbf{R}'_r = 0.473\left(\frac{Q}{f}\right)^{1/3}$ ...(4.30)

The above scour depth equation will be applicable only when the river width equals the regime width of $4.75\sqrt{Q}$. For any other value of active river width, the normal scour depth is given by the equation :

*Lacey's Normal Scour Depth** ($\mathbf{R}'$)

$$= \mathbf{R}' = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(4.31)$$

where q is the discharge intensity per unit width of stream $= Q/L$, where L is the actual river width at the given site.

Example 4.9. *Design a regime channel for a discharge of 50 cumecs and silt factor 1.1, using Lacey's Theory.*

Solution. $Q = 50$ cumecs, $f = 1.1$

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} = \left[\frac{50 \times (1.1)^2}{140}\right]^{1/6}$$

$$A = \frac{Q}{V} = \frac{50}{0.869} = 56.3 \text{ m}^2$$

* Symbol R is frequently used for representing Lacey's scour depth, but to avoid confusion with the symbol R used for the hydraulic Radius (i.e., hydraulic mean depth = A/P), we are using the symbol $\mathbf{R}'$ here for scour depth. When river width equals the regime width, then the regime scour depth is being represented by $\mathbf{R}'_r$

$$R = \frac{5}{2} \cdot \frac{V^2}{f} = \frac{5}{2} \cdot \frac{(0.869)^2}{1.1} = 1.675 \text{ m.}$$

$$P = 4.75\sqrt{Q} = 4.75 \cdot \sqrt{50} = 33.56 \text{ m}$$

For a trapezoidal channel with $\frac{1}{2}H : 1V$ slopes

$$P = b + \sqrt{5} \cdot y$$

and $$A = \left(b + \frac{y}{2}\right)y$$

$\therefore$ $$33.56 = b + \sqrt{5} \cdot y \quad ...(i)$$

and $$56.3 = by + \frac{y^2}{2} \quad ...(ii)$$

From Eq. (*i*), we get, $b = 33.56 - 2.24y$

Putting this value of b in Eq. (*ii*)

$$56.3 = [33.56 - 2.24y]\, y + \frac{y^2}{2}$$

$$= 33.56y - 2.24y^2 + 0.5y^2 = 33.56y - 1.74y^2$$

or $$1.74y^2 - 33.56y + 56.3 = 0$$

or $$y^2 - 19.3y + 32.4 = 0$$

$\therefore$ $$y = \frac{19.3 \pm \sqrt{372 - 129.6}}{2}$$

$$= \frac{19.3 \pm \sqrt{242.4}}{2} = \frac{19.3 \pm 15.6}{2}$$

Neglecting unfeasible + ve sign, we get

$$y = \frac{19.3 - 15.6}{2} = 1.65 \text{ m}$$

$\therefore$ $y = 1.65$ m. **Ans.**

$b = 33.56 - 2.24 \times 1.65 =$ **29.77 m**

or $b = 29.77$ m. **Ans.**

$$S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{(1.1)^{5/3}}{3340 \cdot (50)^{1/6}} = \frac{1}{5420}$$

Use a bed slope of 1 in 5420. **Ans.**