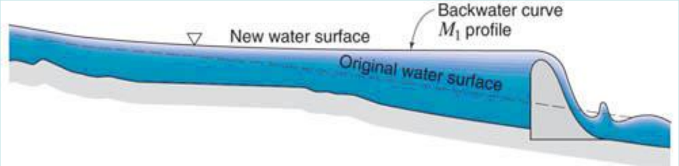
River Mechanics

Nonuniform Flow

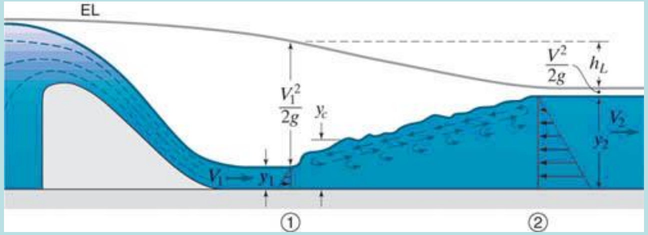
Chapter 4 (Graf, 1998)

Nonuniform Flow

- Two Types of Nonuniform Flow:
  - Gradually Varied Flow – changing conditions extend over a long distance



  - Rapidly Varied Flow – changing flow conditions occur abruptly



Gradually Varied Flow

- Consider prismatic channel with steady, non-uniform flow:

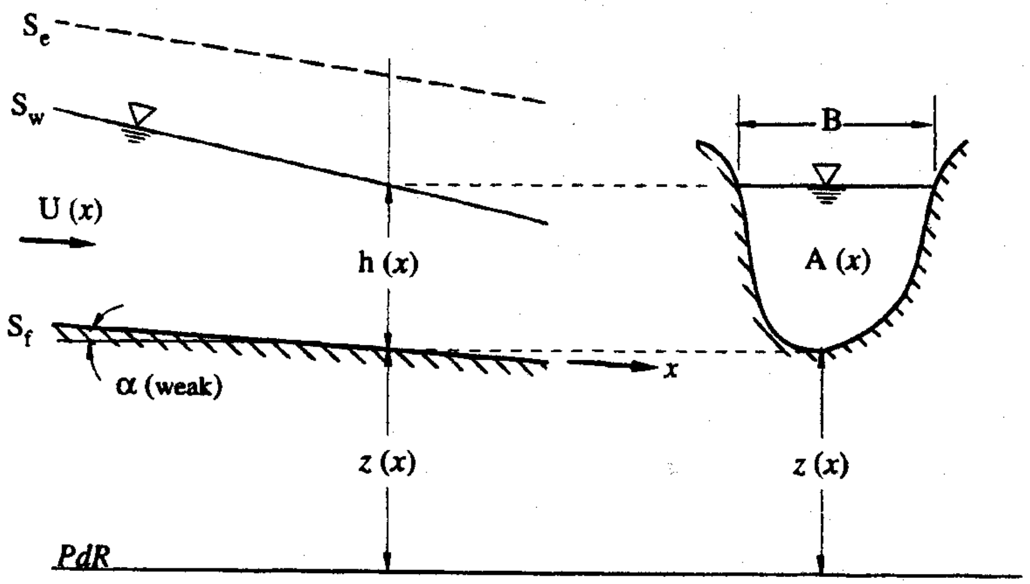


Fig. 4.1 Scheme of steady and non-uniform flow on a fixed bed slope, $z(x)$.

Saint-Venant Equations

- Combined equation of continuity and equation of energy…

$$\frac{\partial Q}{\partial x} + B\frac{\partial h}{\partial t} = 0$$

$$\frac{\partial Q}{\partial x} = A\frac{\partial U}{\partial x} + U\frac{\partial A}{\partial x} = 0$$

$$h\frac{\partial U}{\partial x} + U\frac{\partial h}{\partial x} = 0$$

Saint-Venant Equations

- Combined equation of continuity and equation of energy…

$$H = \frac{U^2}{2g} + h + z$$

$$\frac{d}{dx}\left(\frac{U^2}{2g}\right) + \frac{dh}{dx} + \frac{dz}{dx} = \frac{dH}{dx}$$

$$\frac{dz}{dx} = -S_f \qquad \frac{dH}{dx} = -S_e = f\frac{1}{4R_h}\frac{U^2}{2g} = \frac{8g}{C^2}\frac{1}{4R_h}\frac{U^2}{2g}$$

$$\frac{d}{dx}\left(\frac{(Q/A)^2}{2g}\right) + \frac{dh}{dx} - S_f = -S_e = -\frac{(Q/A)^2}{C^2 R_h}$$

Saint-Venant Equations

- Combined equation of continuity and equation of energy…

$$\frac{d}{dx}\left(\frac{(Q/A)^2}{2g}\right)+\frac{dh}{dx}-S_f=-S_e=-\frac{(Q/A)^2}{C^2R_h}$$

$$Prismatic:$$

$$A=\mathrm{f}(h)$$

$$\frac{d}{dx}\left(\frac{(Q/A)^2}{2g}\right)=\frac{d}{dx}\left(\frac{Q^2}{2g\left[A(h)\right]^2}\right)=\frac{Q^2}{2g}\left(\frac{-2}{A^3}\frac{dA}{dx}\right)=-\frac{Q^2}{gA^3}\left(\frac{dA}{dh}\frac{dh}{dx}\right)$$

Saint-Venant Equations

- Combined equation of continuity and equation of energy…

$$-\frac{Q^2}{gA^3}B\frac{dh}{dx}+\frac{dh}{dx}-S_f=-\frac{(Q/A)^2}{C^2R_h}$$

Prismatic:

$$\frac{dh}{dx}=\frac{S_f-\left(\frac{(Q/A)^2}{C^2R_h}\right)}{1-\frac{BQ^2}{gA^3}}=S_f\frac{1-\left(\frac{(Q/A)^2}{C^2R_hS_f}\right)}{1-\frac{BQ^2}{gA^3}}$$

Wide and Rectangular Channel

- Further simplify for wide and rectangular channel…
  - Normal Depth:

$$Q = UA = C(h_n B)\sqrt{h_n S_f}$$

$$q = \frac{Q}{B}$$

$$q = Ch_n\sqrt{h_n S_f}$$

$$q^2 = C^2 h_n{}^3 S_f \rightarrow h_n{}^3 = \frac{q^2}{C^2 S_f}$$

Wide and Rectangular Channel

- Further simplify for wide and rectangular channel…
  - Critical Depth:

$$Fr = 1 = \frac{U}{\sqrt{gD_h}} = \frac{Q}{A\sqrt{gh_c}} = \frac{Q}{Bh_c\sqrt{gh_c}} = \frac{q}{h_c\sqrt{gh_c}}$$

$$q^2 = h_c^{\ 2} g h_c \rightarrow h_c^{\ 3} = \frac{q^2}{g}$$

Wide and Rectangular Channel

- Using Chezy Equation…

$$\frac{dh}{dx} = S_f \frac{1 - \left(\dfrac{Q^2}{A^2 C^2 h S_f} \right)}{1 - \dfrac{BQ^2}{gA^3}} = S_f \frac{1 - \left(\dfrac{Q^2}{B^2 h^3 C^2 S_f} \right)}{1 - \dfrac{BQ^2}{g(Bh)^3}}$$

$$\frac{dh}{dx} = S_f \frac{1 - \left(\dfrac{q^2}{h^3 C^2 S_f} \right)}{1 - \dfrac{q^2}{gh^3}} = S_f \frac{1 - \left(\dfrac{h_n}{h} \right)^3}{1 - \left(\dfrac{h_c}{h} \right)^3}$$

Wide and Rectangular Channel

- Using Manning's Equation…

$$\frac{dh}{dx} = S_f \frac{1 - \left(\dfrac{q^2}{h^3 C^2 S_f} \right)}{1 - \dfrac{q^2}{g h^3}} = S_f \frac{1 - \left(\dfrac{h_n}{h} \right)^{10/3}}{1 - \left(\dfrac{h_c}{h} \right)^3}$$

Critical Slope

- Bed slope that results in uniform flow at critical depth ($y_n = y_c$) for a given discharge
  - Combine critical flow equation with uniform flow equation:

$$U = C\sqrt{R_h S_f} \leftrightarrow \left(\frac{Q}{A}\right)^2 = \frac{gA}{B}$$

$$C^2 R_h S_c = g\,\frac{A}{B}$$

$$S_c = \frac{gA}{C^2 B R_h}$$

Critical Slope

- If $S_f < S_c$ for a given Q and C, then $h_n > h_c$:
  - Mild Slope
  - Uniform flow corresponding to this normal depth will be subcritical (fluvial)
- If $S_f > S_c$ for a given Q and C, then $h_n < h_c$:
  - Steep Slope
  - Uniform flow corresponding to this normal depth will be supercritical (torrential)

Forms of Water Surface

- Water surface profiles for the possible cases encountered in open-channel flow…
  - First classification based on bed slope, S_f:
    - $S_f = 0$ (Horizontal Slope): H
    - $S_f < 0$ (Adverse Slope): A
    - $S_f > 0$ (Mild Slope, Steep Slope, or Critical Slope): M, S, or C
  - Figure 4.2 in Graf

Conditions Eq. 4.8a	$\frac{h_n}{h}$	Sign num.	$\frac{h_c}{h}$	Sign den.	Sign dh/dx	Change of flow depth	Name
$S_f > 0$							
$S_f < S_c$	< 1	+	< 1	+	+	increase	M1
$h_n > h_c$	< 1	+	> 1	-		not possible	
	> 1	-	< 1	+	-	decrease	M2
	> 1	-	> 1	-	+	increase	M3
$S_f > 0$							
$S_f > S_c$	< 1	+	< 1	+	+	increase	S1
$h_n < h_c$	< 1	+	> 1	-	-	decrease	S2
	> 1	-	> 1	-	+	increase	S3
$S_f > 0$							
$S_f = S_c$	< 1	+	< 1	+	+	increase	C1
$h_n = h_c$	> 1	-	> 1	-	+	increase	C3
$S_f = 0$							
$h_n = \infty$		-	< 1	+	-	decrease	H2
		-	> 1	-	+	increase	H3
$S_f < 0$							
$h_n < 0$	< 1	-	< 1	+	-	decrease	A2
	< 1	-	> 1	-	+	increase	A3

Profiles *vertical scale exaggerated*

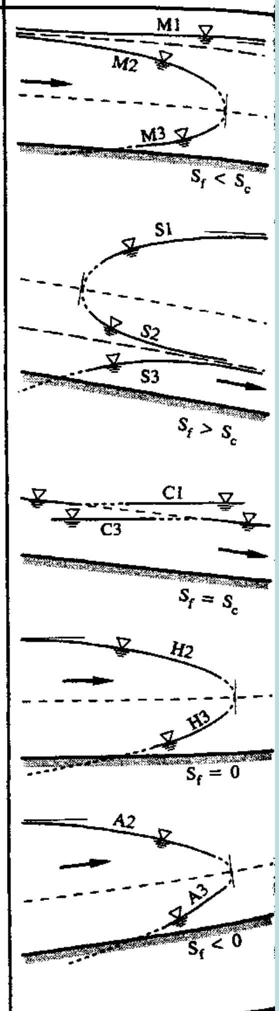


Fig. 4.2 Water-surface profiles for gradually varied flow.

water surface ; normal depth ; critical depth

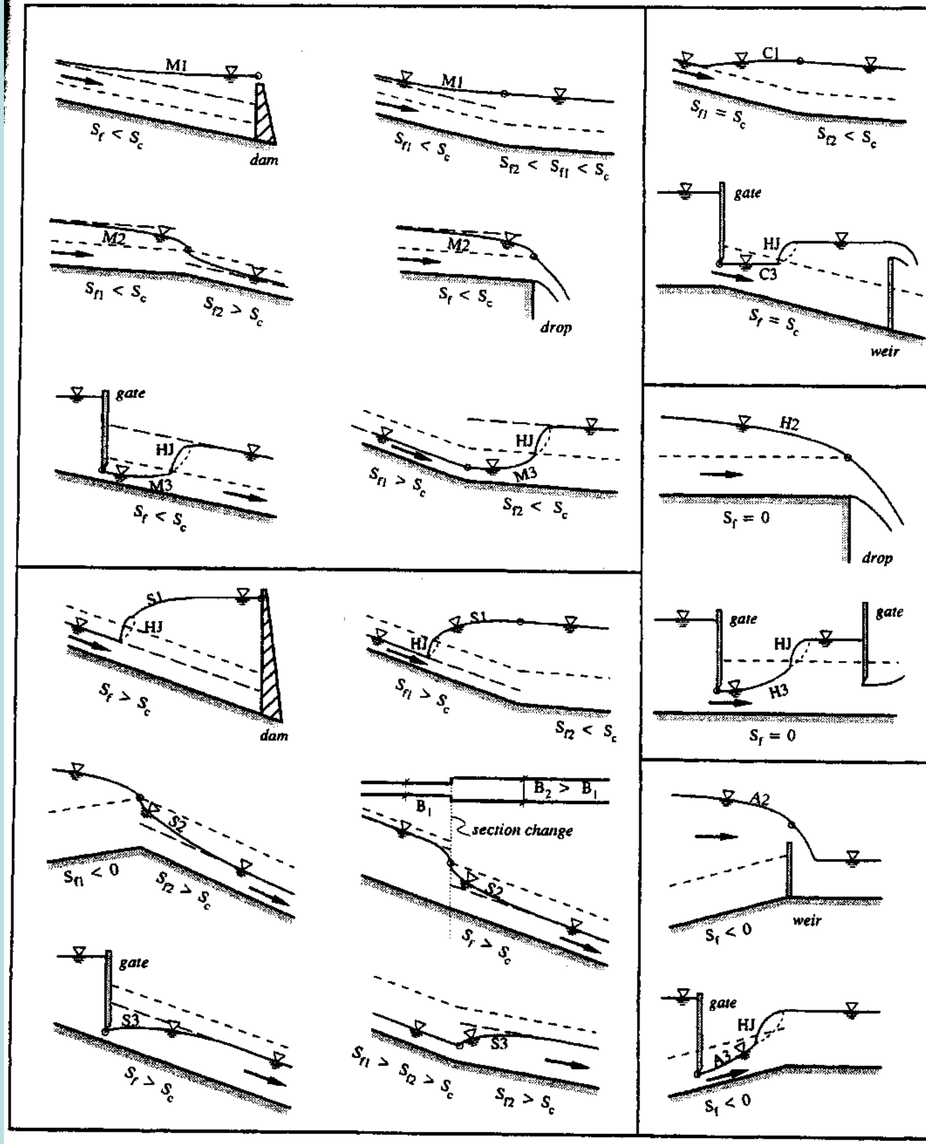
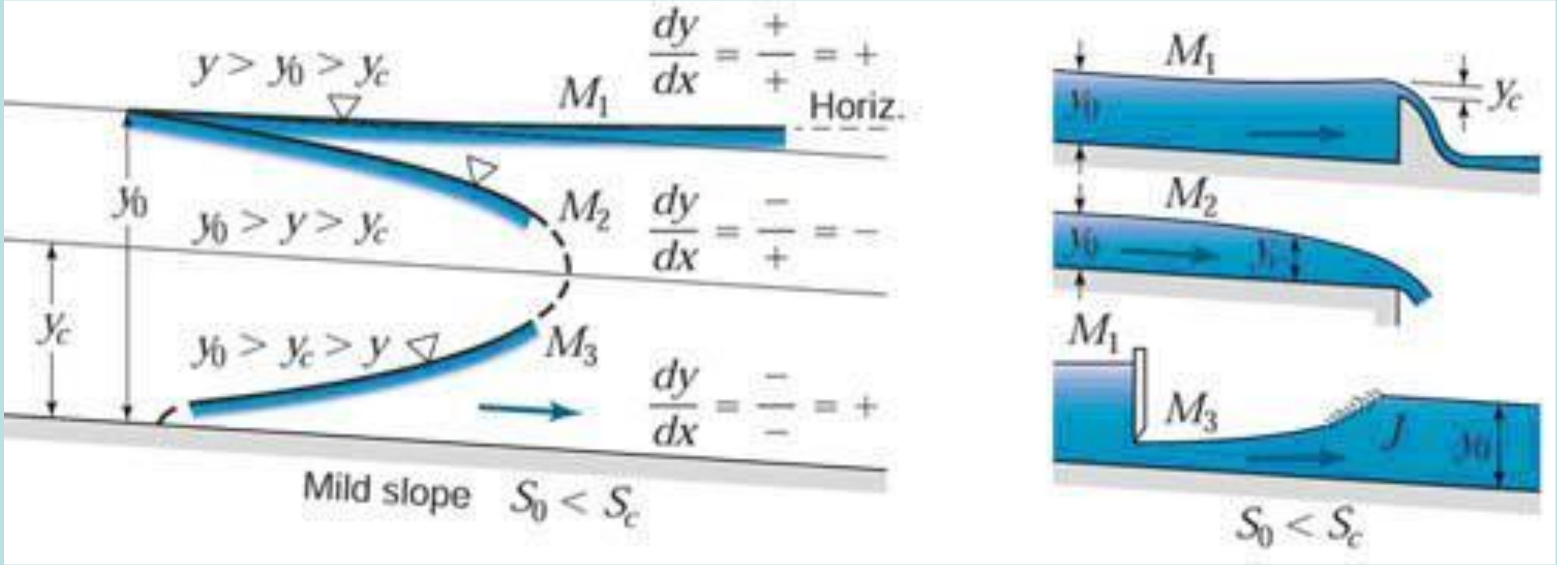


Fig. 4.2a Some examples of flow.

○ control section ; HJ hydraulic jump

Water-Surface Profiles

- M1 –
  - Curves goes downstream towards a horizontal tangent
  - Upstream of a dam or weir, pier, at junctures of certain bed slopes

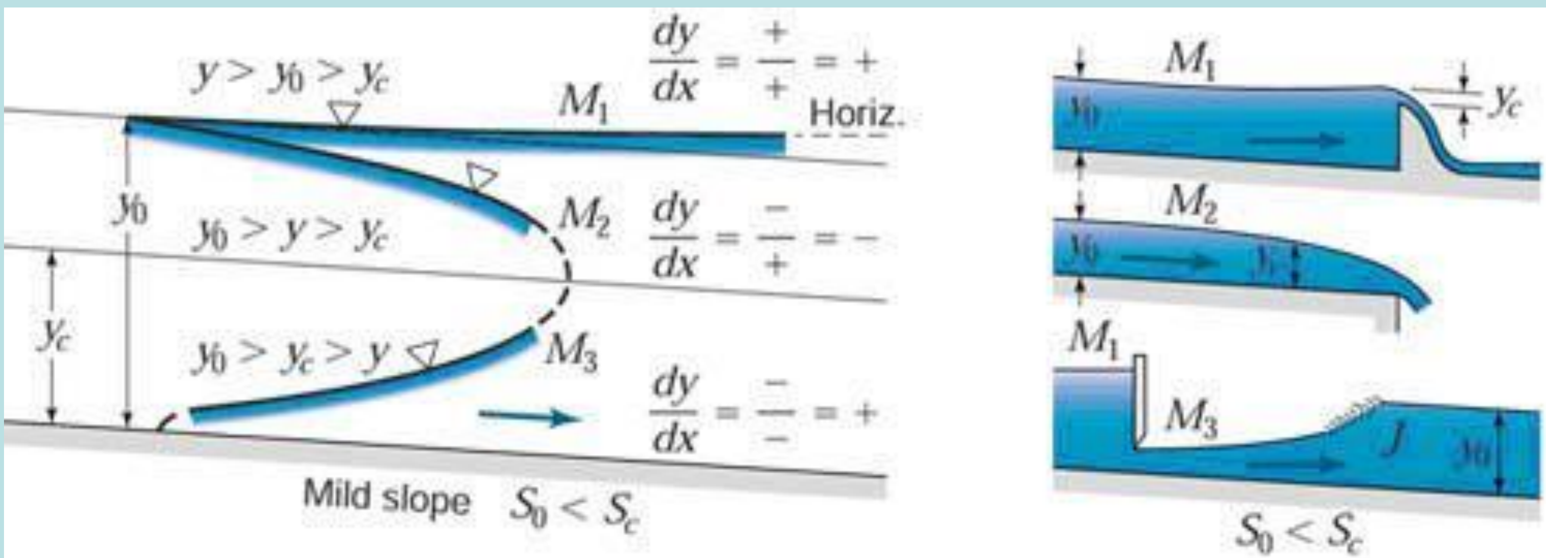


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

- M2 –
  - Curves goes downstream towards the critical depth
  - Upstream of an increase in bed slope and upstream or a hydraulic drop

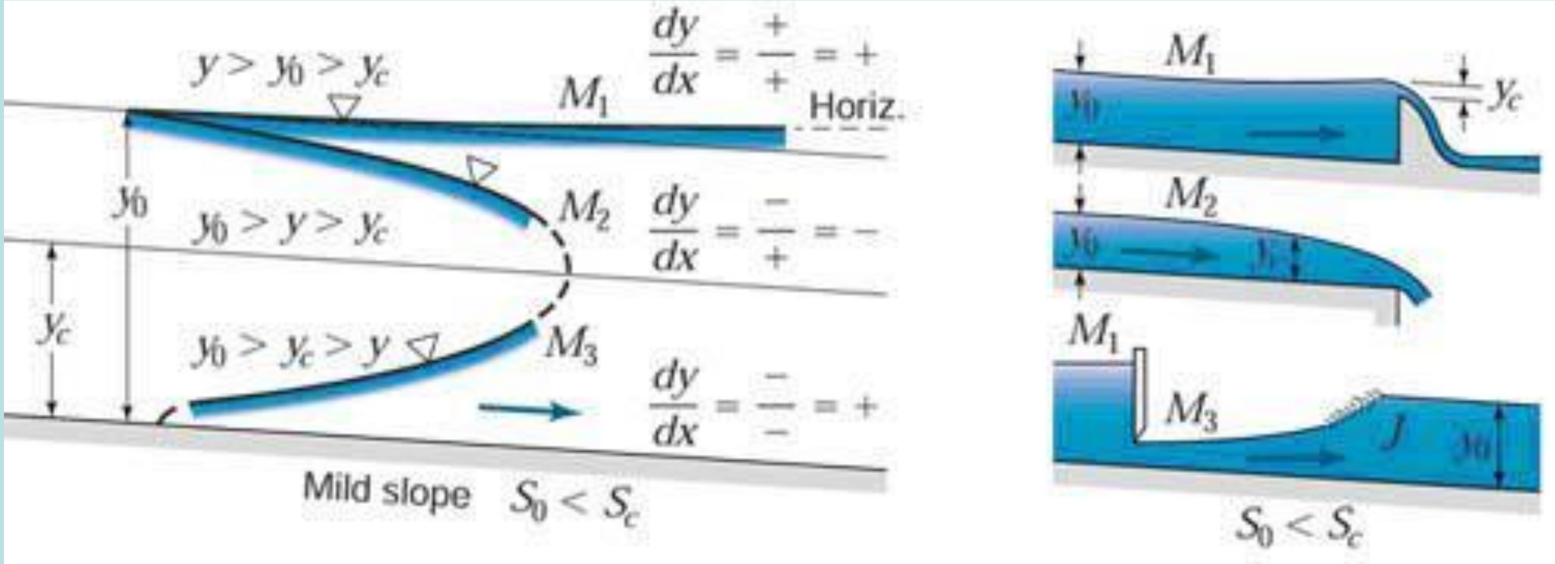


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

- M3 –
  - Curves goes downstream towards the critical depth where it terminates at a hydraulic jump
  - Occurs when supercritical flow enters a mild channel and after a change in bed slope from steep to mild

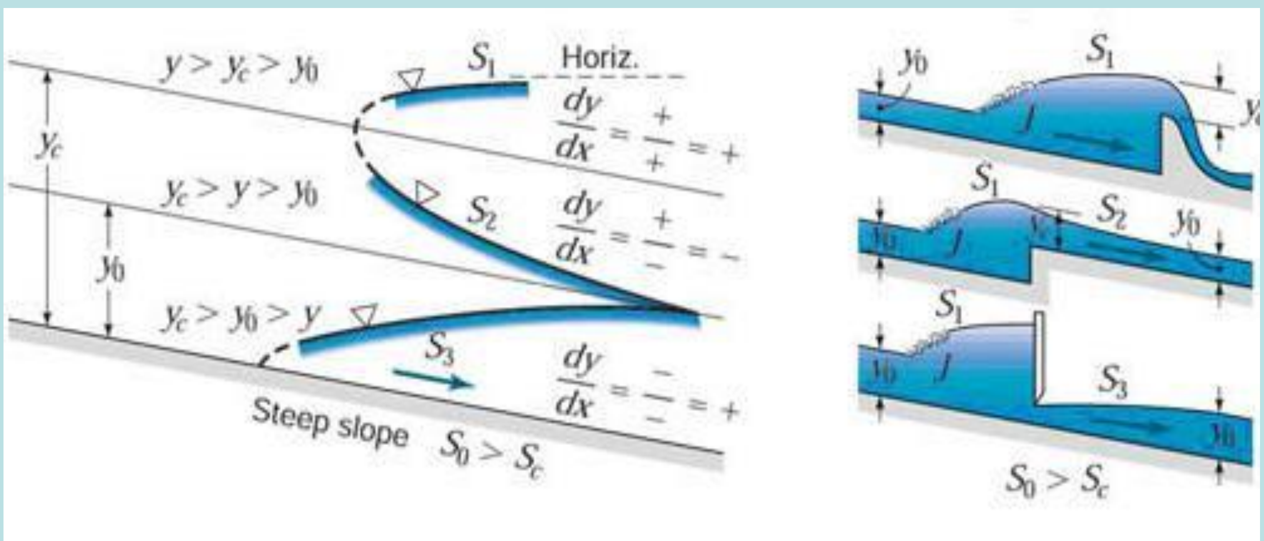


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

- S1 –
  - Curves begins at critical depth of a hydraulic jump and terminates as a tangent to a horizontal line
  - Upstream of a dam or weir and at a juncture of certain bed slopes

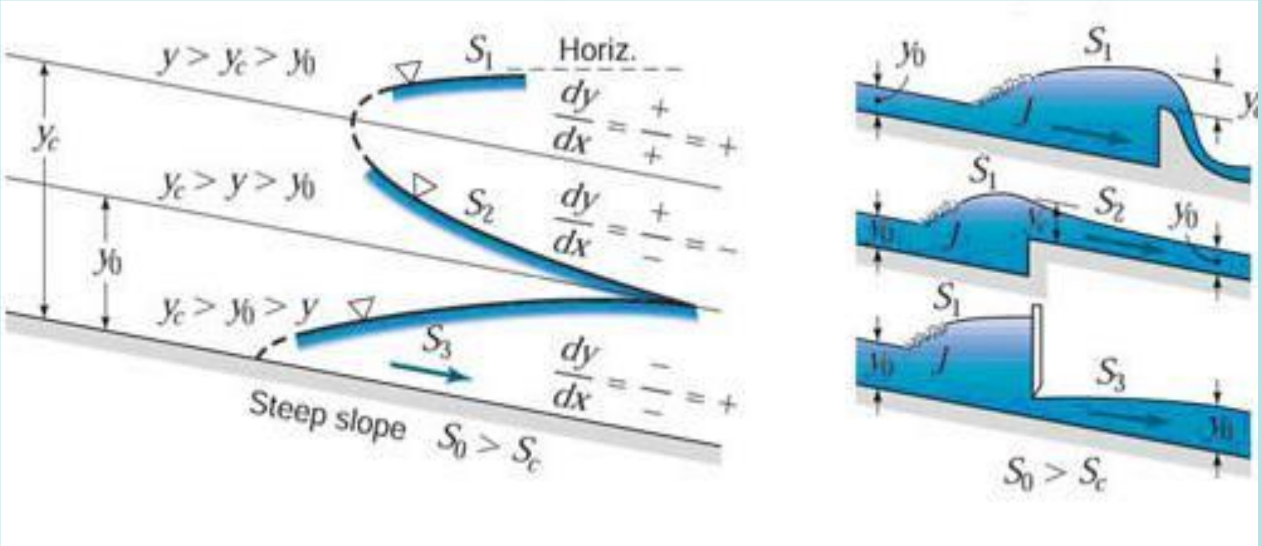


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

- S2 –
  - Takes place in transition between critical depth and uniform flow (usually very short)
  - Occurs downstream of a sudden increase in bed slope and downstream of an enlargement

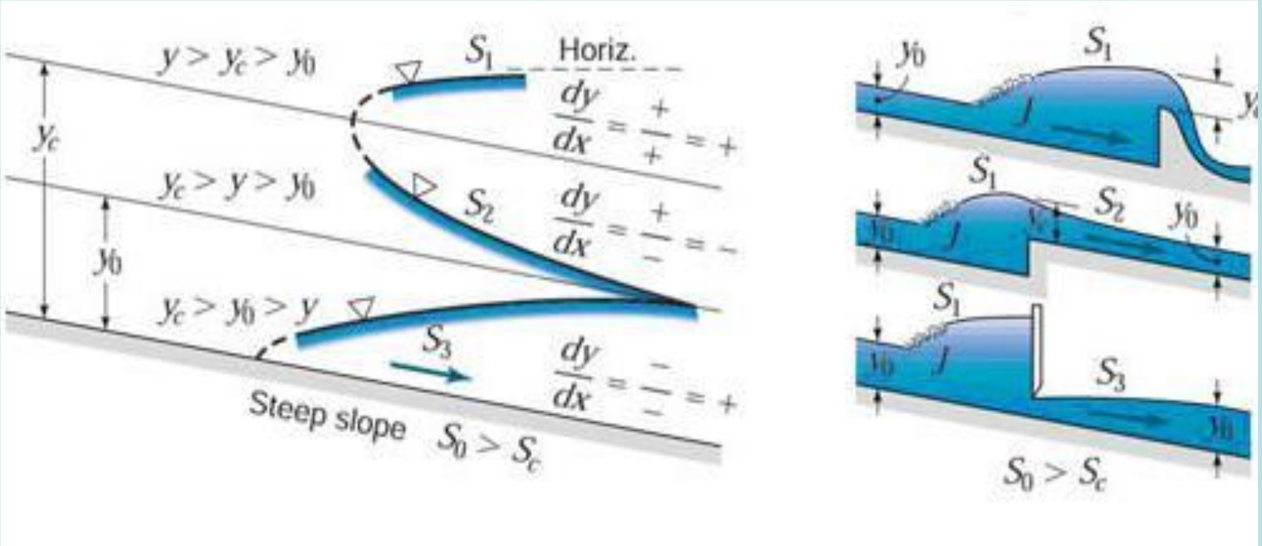


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

- S3 –
  - Takes place in transition between supercritical flow and uniform flow and approaches a tangent
  - Occurs downstream of a gate, when the flow is below the normal depth and when the bed slope is reduced

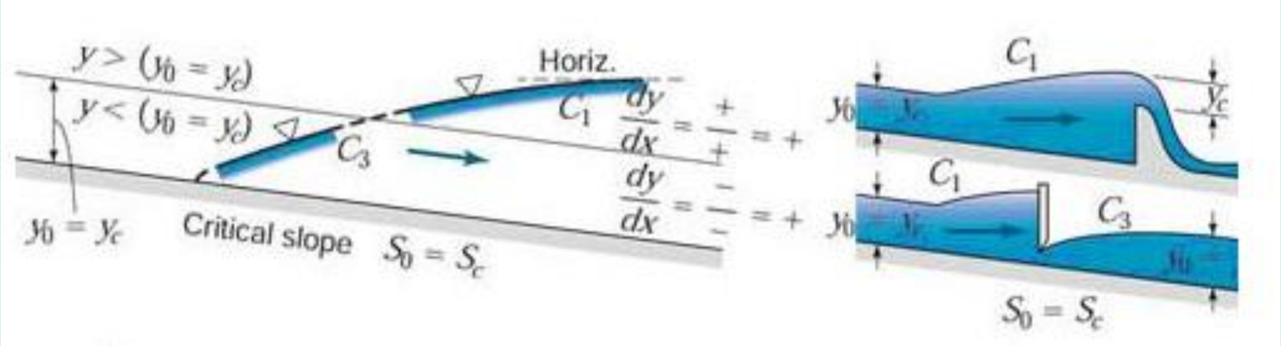


Finnemore and Franzini, Fluid Mechanics

Note: $y = h$, $y_o = h_n$

Water-Surface Profiles

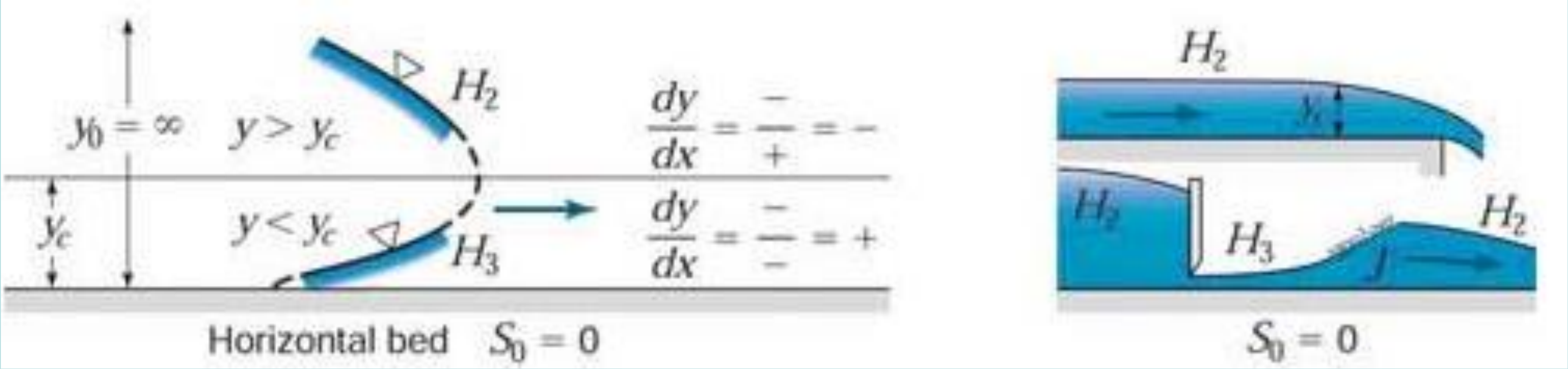
- Critical Slope – C
  - C1 – Curve is horizontal and occurs at a juncture of certain bed slopes and upstream of a dam (weir)
  - C2 - ?????
  - C3 – Curve is horizontal and occurs when the bed slope is reduced to critical slope and downstream of a sluice gate when the flow is below normal depth



$$\frac{dh}{dx} = S_f \frac{1-\left(\frac{q^2}{h^3 C^2 S_f}\right)}{1-\frac{q^2}{gh^3}} = S_f \frac{1-\left(\frac{h_n}{h}\right)^{10/3}}{1-\left(\frac{h_c}{h}\right)^3}$$

Water-Surface Profiles

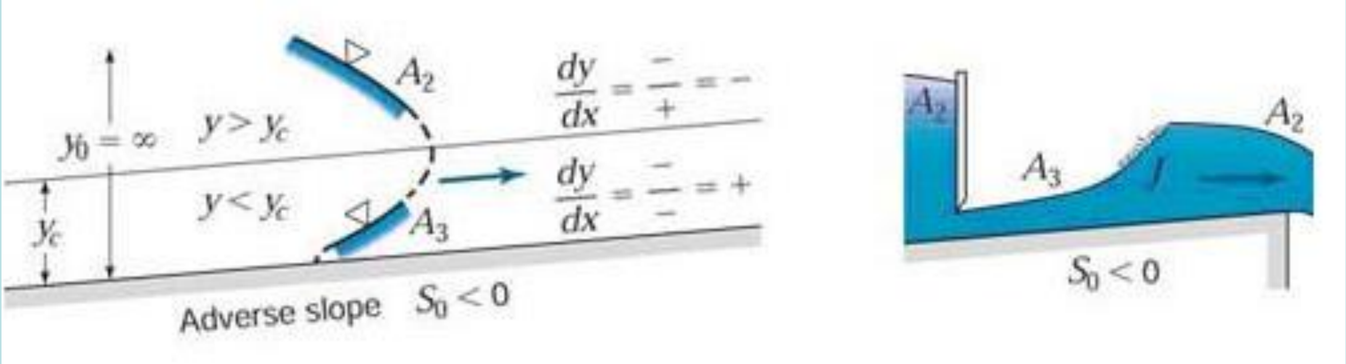
- Horizontal Slope – H (S_f=0, h_n is infinite)
  - H1 – not established because h_n is infinite
  - H2 and H3 correspond to M2 and M3 when the channel bed becomes horizontal
  - H2 is encountered at a hydraulic drop
  - H3 is encountered when supercritical flow enters into a horizontal channel



$$\frac{dh}{dx} = S_f \frac{1 - \left(\frac{q^2}{h^3 C^2 S_f} \right)}{1 - \frac{q^2}{g h^3}} = S_f \frac{1 - \left(\frac{h_n}{h} \right)^{10/3}}{1 - \left(\frac{h_c}{h} \right)^3}$$

Water-Surface Profiles

- Adverse Slope – A (S_f<0, h_n does not exist)
  - A1 – not established because h_n does not exist
  - A2 and A3 correspond to H2 and H3
  - A2 is encountered at a juncture of certain bed slopes
  - H3 is encountered when supercritical flow enters into an adverse channel



$$\frac{dh}{dx} = S_f \frac{1 - \left(\frac{q^2}{h^3 C^2 S_f} \right)}{1 - \frac{q^2}{gh^3}} = S_f \frac{1 - \left(\frac{h_n}{h} \right)^{10/3}}{1 - \left(\frac{h_c}{h} \right)^3}$$

Example Problem 4.21 Flow profile

A wide, rectangular channel is carrying 10 cfs/ft down a 0.5% slope. The channel has a Manning's *n* of 0.025. A 2.5-ft barrier in the channel causes flow to pass over the barrier at critical depth. Compute the flow profile upstream from the barrier to a point where the depth is within 10% of normal depth. Figure 4.25 illustrates the physical situation.

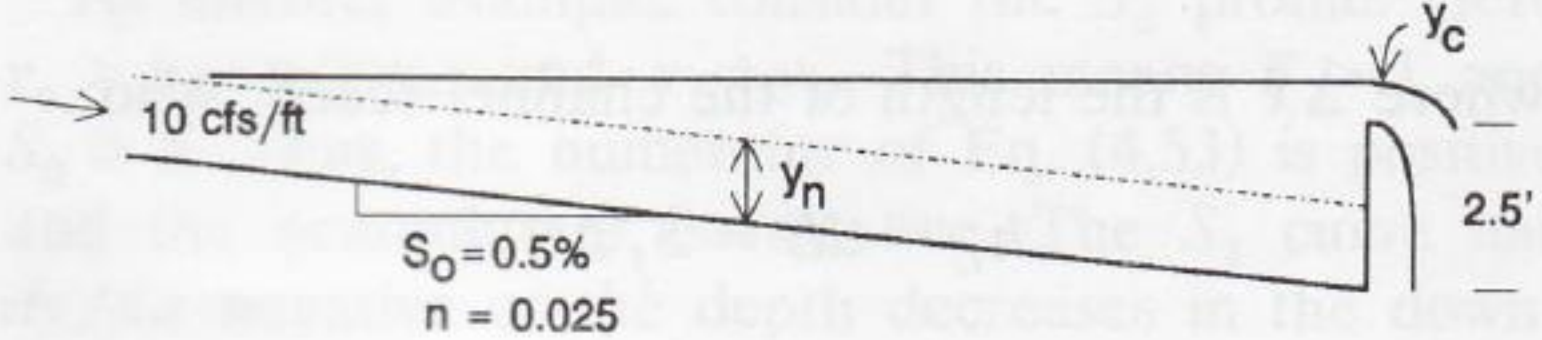


Table 4.15 Profile Calculations for Example Problem 4.21

y (ft)	v[a] (fps)	$V^2/2g$ (ft)	E[b] (ft)	y_m (ft)[c]	S_f[d]	dx (ft)[e]	x[f] (ft)
3.96	2.525	0.099	4.059				0
3.50	2.857	0.127	3.627	3.730	0.00035	–93	–93
3.25	3.077	0.147	3.397	3.375	0.00048	–51	–143
3.00	3.333	0.173	3.173	3.125	0.00063	–51	–195
2.75	3.636	0.205	2.955	2.875	0.00083	–52	–247
2.50	4.000	0.248	2.748	2.625	0.00112	–53	–300
2.25	4.444	0.307	2.557	2.375	0.00157	–56	–356
2.00	5.000	0.388	2.388	2.125	0.00227	–62	–418
1.75	5.714	0.507	2.257	1.875	0.00345	–85	–503
1.70	5.882	0.537	2.237	1.725	0.00456	–45	–548

[a] $v = q/y$.
[b] $E = v^2/2g + y$.
[c] $y_m = (y_1 + y_2)/2$.
[d] $S_f = (qn/1.49 y_m^{1.67})^2$.
[e] $dx = (E_1 - E_2)/(S_f - S_o)$.
[f] $x_2 = x_1 + dx$.

Solution: From Eq. (4.9),

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{100}{32.2}\right)^{1/3} = 1.46 \text{ ft}.$$

From Manning's equation (4.23) using $q = vy$,

$$y_n = \left(\frac{qn}{1.49S^{1/2}}\right)^{3/5} = \left(\frac{10(0.025)}{1.49(0.005)^{1/2}}\right)^{3/5} = 1.68.$$

The depth of flow over the brink in Fig. 4.25 is

$$y = 2.5 + y_c = 3.96 \text{ ft}.$$

The solution is carried out by assuming depths and computing Δx. Table 4.15 shows the computations.

Example Problem 4.22 Channel transition 1

A trapezoidal channel with 2:1 side slopes and a 4-ft bottom width is flowing at a depth of 1 ft. The channel is concrete and on a slope of 0.1%. If the channel bottom is raised smoothly by 0.1 ft over a short distance, what will be the depth of flow at the exit of the transition?

Solution

$$n = 0.015 \qquad \text{for concrete}$$

$$v_1 = \frac{1.5}{n} R^{2/3} S^{1/2}$$

$$A = bd + zd^2 = 4(1) + 2(1)^2 = 6 \text{ ft}^2$$

$$P = b + 2d\sqrt{z^2 + 1} = 4 + 2(1)\sqrt{2^2 + 1}$$

$$= 8.47 \text{ ft}$$

$$R = A/P = 6/8.47 = 0.71 \text{ ft}$$

$$v_1 = \frac{1.5}{0.015}(0.71)^{2/3}(0.001)^{1/2} = 2.52 \text{ fps}$$

$$\mathbf{F} = \frac{v}{\sqrt{g d_h}}$$

$$d_h = \frac{A}{t} = \frac{6}{b + 2zd} = \frac{6}{4 + 2(2)(1)}$$

$$= 0.75 \text{ ft}$$

$$\mathbf{F} = \frac{2.52}{\sqrt{32.3(0.75)}} = 0.51 \qquad \text{subcritical}$$

$$\frac{v_1^2}{2g} + y_1 = \frac{v_2^2}{2g} + y_2 + \Delta z$$

$$\frac{(2.52)^2}{64.4} + 1.0 = \frac{v_2^2}{64.4} + y_2 + 0.1$$

$$0.9986 = \frac{v_2^2}{64.4} + y_2$$

$$v_2 = \frac{Q}{A} = \frac{v_1 A_1}{A_2} = \frac{2.52(6)}{4y_2 + 2y_2^2}$$

$$0.9986 = \frac{3.54}{\left(4y_2 + 2y_2^2\right)^2} + y_2.$$

Solve by trial

y_2	Right-hand side	
0.90	1.03	
0.75	0.958	
0.84	0.996	OK

$$y_2 = 0.84 \quad \text{ft}$$

Check the Froude number:

$$v_2 = \frac{Q}{A_2} = \frac{2.52(6)}{4(0.84) + 2(0.84)^2} = \frac{15.1}{4.77} = 3.16 \text{ ft}$$

$$d_h = \frac{A}{t} = \frac{4.77}{4 + 2(2)(0.84)} = \frac{4.77}{7.36} = 0.65 \text{ ft}$$

$$\mathbf{F} = \frac{v}{\sqrt{g d_h}} = 0.69 \quad \text{still subcritical.}$$

Solution OK.

Example Problem 4.23 Channel transition 2

A rectangular channel 10 ft wide is carrying 75 cfs. The channel smoothly narrows to 8 ft in width. The flow depth in the 10-ft section is 2.5 ft. What is the depth in the 8 ft section assuming no energy losses?

Solution

$$\frac{v_1^2}{2g} + y_1 = \frac{v_2^2}{2g} + y_2$$

$$v_1 = \frac{Q}{A} = \frac{75}{10 \times 2.5} = 3 \text{ fps}$$

$$v_2 = \frac{75}{8y_2}$$

$$\frac{3^2}{64.4} + 2.5 = \frac{(75/8y_2)^2}{64.4} + y_2 = \frac{1.36}{y_2^2} + y_2.$$

The solution may be found by trial to be $y_2 = 2.40$ ft. Thus the depth in the 8-ft section is 2.40 ft.

Example

The flow in a 15-ft wide rectangular channel that has a constant bottom slope is 1400 cfs. A computation using Manning's equation indicates that the normal depth is 6.0 ft. At a certain section the depth of flow in the channel is 2.8 ft. Does the depth increase, decrease, or remain the same as one proceeds downstream from this section?

Example 4.A – Graf

A trapezoidal channel with bottom width of 7.0 m and side slopes of m = 1.5 conveys Q = 28 m^3/s with a bed slope of 0.0010 (n = 0.025). The channel is terminated by a sudden drop of the channel bed. Determine what type of water-surface profile is to be expected.

Computation of Water Surface Profiles

- Integration of dh/dx equation from earlier…

$$\frac{dh}{dx}=\frac{S_f-\left(\frac{(Q/A)^2}{C^2R_h}\right)}{1-\frac{BQ^2}{gA^3}}=S_f\,\frac{1-\left(\frac{(Q/A)^2}{C^2R_hS_f}\right)}{1-\frac{BQ^2}{gA^3}}$$

  - Method of successive approximations
  - Method of direct integration
  - Method of graphical integration

Computation of Water Surface Profiles

- Method of successive approximation:

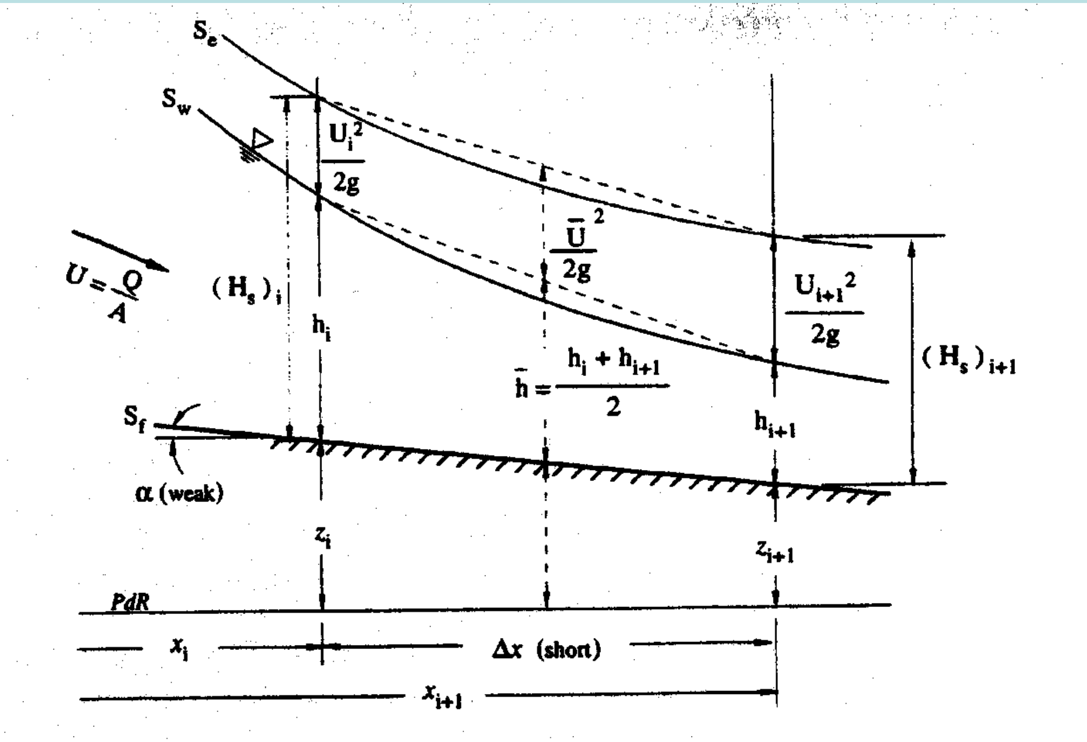


Fig. 4.3 Scheme of a non-uniform flow between two cross sections.

Computation of Water Surface Profiles

- Method of successive approximation:

$$\frac{d}{dx}\left(\frac{(Q/A)^2}{2g}\right)+\frac{dh}{dx}-S_f=-S_e=-\frac{(Q/A)^2}{C^2R_h}$$

$$dh=\left(S_f-\frac{(Q/A)^2}{C^2R_h}\right)dx-\frac{Q^2}{2g}d\left(\frac{1}{A^2}\right)$$

$$h_{i+1}-h_i=\left(S_f-\frac{Q^2}{\overline{C}^2\overline{A}^2\overline{R}_h}\right)\left(x_{i+1}-x_i\right)-\frac{Q^2}{2g}\left(\frac{1}{A_{i+1}^{\;2}}-\frac{1}{A_i^{\;2}}\right)$$

Computation of Water Surface Profiles

- Three forms of this equation:

All Channels:

$$(1)\quad h_{i+1} - h_i = \left(S_f - \frac{Q^2}{\overline{C}^2 \overline{A}^2 \overline{R}_h} \right)(x_{i+1} - x_i) - \frac{Q^2}{2g}\left(\frac{1}{A_{i+1}{}^2} - \frac{1}{A_i{}^2} \right)$$

$$(2)\quad (h+z)_{i+1} - (h+z)_i = \left(- \frac{Q^2}{\overline{C}^2 \overline{A}^2 \overline{R}_h} \right)(x_{i+1} - x_i) - \frac{Q^2}{2g}\left(\frac{1}{A_{i+1}{}^2} - \frac{1}{A_i{}^2} \right)$$

Prismatic Channels:

$$(3a)\quad (H_s)_{i+1} - (H_s)_i = \left(S_f - \frac{U^2}{\overline{C}^2 \overline{R}_h} \right)(x_{i+1} - x_i) \leftarrow Chezy$$

$$(3b)\quad (H_s)_{i+1} - (H_s)_i = \left(S_f - \frac{n^2 U^2}{\overline{R}_h^{4/3}} \right)(x_{i+1} - x_i) \leftarrow Manning's$$

Computation of Water Surface Profiles

- If you arbitrarily select Δx, solve for the variations in the flow depth, Δh –
  - **Standard Step Method** (Method of Reaches)
- If you arbitrarily select Δh, solve for the variations in the distance, Δx –
  - **Direct Step Method** (Method of Depth Variation)

- Before you use either method:

1. Establish control points (known relationship between flow depth and discharge)
2. Computations proceed upstream for subcritical flow, Fr<1 and downstream for supercritical flow, Fr>1
3. When you are closer to the critical depth (curvature of water surface more pronounced), you must use smaller steps

Direct Step Method
(Explicit Method)

- Use known flow depth, h_i, at x_i
- Select h_{i+1} (should be very close to h_i)
- Calculate x_{i+1} using finite-difference equations

Standard Step Method
(Implicit Method)

- Use known flow depth, h_i, at x_i
- Select x_{i+1}
- Guess value of h_{i+1}
  - Calculate average C, A, R_h, and U with corresponds to the average flow depth
  - Use difference equations given above to calculate h_{i+1}
  - Use calculated h_{i+1} as new guess
  - Continue with successive approximations until the calculated h_{i+1} matches previously calculated value

Method of Direct Integration

- Note that integration is a direct solution method:
  - You can proceed from one section to another whatever the distance between the sections
  - In Methods of Successive Approximations, you must use small distances to avoid computational inaccuracy

Method of Direct Integration

- Chow (1959):

$$\frac{dh}{dx} = S_f \frac{1-\left(\frac{h_n}{h}\right)^N}{1-\left(\frac{h_c}{h}\right)^M}$$

  - N = hydraulic exponent for the conveyance (function of cross-section and type of friction coefficient)
  - M = second hydraulic exponent

Method of Direct Integration

- Chow (1959):

$$N(h) = \frac{2h}{3A}\left(5B - 2R_h \frac{dP}{dh}\right) \rightarrow 2.0 < N < 5.3$$

$$M(h) = \frac{h}{A}\left(3B - \frac{A}{B}\frac{dB}{dh}\right) \rightarrow 3 < M < 4.8$$

$$\frac{h}{h_n} = \eta \rightarrow dh = h_n d\eta$$

$$dx = \frac{1}{S_f}\left[1 - \left(\frac{1}{1-\eta^N}\right) + \left(\frac{h_c}{h}\right)^M \left(\frac{\eta^{N-M}}{1-\eta^N}\right)\right] h_n d\eta$$

Method of Direct Integration

- Back to N and M:
  - Assume trapezoidal channel…

$$P = b + 2h\sqrt{1+m^2} \rightarrow \frac{dP}{dh} = 2\sqrt{1+m^2}$$

$$B = b + 2mh \rightarrow \frac{dB}{dh} = 2m$$

  - See equations 4.26a on page 196 of Graf

$$N(h) = [\frac{10}{3}\frac{(1+2mh/b)}{(1+mh/b)} - \frac{8}{3}\frac{(\frac{h}{b}\sqrt{1+m^2})}{(1+2\frac{h}{b}\sqrt{1+m^2})}] \rightarrow 2.0 < N < 5.3$$

$$M(h) = \frac{[3(1+2mh/b)^2 - 2mh/b(1+mh/b)]}{[(1+2mh/b)(1+mh/b)]} \rightarrow 3 < M < 4.8$$

Method of Direct Integration

- Chow (1959):

$$\int dx = \int \frac{1}{S_f}\left[1-\left(\frac{1}{1-\eta^N}\right)+\left(\frac{h_c}{h}\right)^M\left(\frac{\eta^{N-M}}{1-\eta^N}\right)\right]h_n d\eta$$

$$x_i - x_{i+1} = \frac{h_n}{S_f}\left\{(\eta_i - \eta_{i+1}) - \int_0^\eta \left(\frac{1}{1-\eta^N}\right)d\eta + \left(\frac{h_c}{h}\right)^M \int_0^\eta \left(\frac{\eta^{N-M}}{1-\eta^N}\right)d\eta\right\}$$

$$First\ Integral \rightarrow \int\left(\frac{d\eta}{1-\eta^N}\right) = -\int\left(\frac{d\eta}{\eta^N - 1}\right) = \Phi(\eta, N) \rightarrow Table\ 4.1$$

$$Second\ Integral \rightarrow \int_0^\eta \left(\frac{\eta^{N-M}d\eta}{1-\eta^N}\right) = \frac{J}{N}\int_0^\xi \left(\frac{d\xi}{1-\xi^J}\right) = \frac{J}{N}\Phi(\xi, J) \rightarrow Table\ 4.1$$

$$\xi = \eta^{N/J} \qquad J = \frac{N}{N-M+1}$$

Method of Direct Integration

- Chow (1959):

$$x_i' = \frac{h_n}{S_f}\left\{(\eta_i) - [\Phi(\eta_i, N_i)] + \left(\frac{h_c}{h_n}\right)^{M_i} \frac{J_i}{N_i}[\Phi(\xi_i, J_i)]\right\}$$

$$x_{i+1}' = \frac{h_n}{S_f}\left\{(\eta_{i+1}) - [\Phi(\eta_{i+1}, N_{i+1})] + \left(\frac{h_c}{h_n}\right)^{M_{i+1}} \frac{J_{i+1}}{N_{i+1}}[\Phi(\xi_{i+1}, J_{i+1})]\right\}$$

$$\Delta x = x_i' - x_{i+1}'$$

BAE 6333 – Fluvial Hydraulics
Evaluation of Chow (1959) Integral for Direct-Integration Method
Non-Uniform, Gradually Varied Flow

Integral given by Graf (1998) for Chow (1959) integral is given by…

$$-\int\left(\frac{d\eta}{\eta^N-1}\right)=\Phi(\eta,N)\rightarrow Table\ 4.1 \qquad (1)$$

Maple solves this integral to be:

$$\Phi(\eta,N)=-\int\left(\frac{d\eta}{\eta^N-1}\right)=\frac{\eta}{N}LerchiPhi\left(\eta^N,1,\frac{1}{N}\right) \qquad (2)$$

LerchPhi(z,a,v) is given by the following as long as $|z|$**<1** (Erdelyi, 1953), where the limit of the ratio q/K_s is unity:

$$LerchPhi(z,a,v)=\sum_{n=0}^{\infty}\frac{z^n}{(v+n)^a} \qquad (3)$$

This gives you a solution for the left-side of Table 4.1 in Graf (1998).

NOTE: I have actually used this function before for solving for unsaturated flow beneath a stream due to alluvial well pumping (Fox and Gordji, 2007) to solve the following differential equation:

$$s_w-M-H_w=\int_0^{h_{cl}}\frac{\partial h_c}{1-\frac{q}{K_s}\left(\frac{h_c}{h_e}\right)^{\eta}}$$

The integral in equation (12) can be expressed as a Lerch Phi function for ease of incorporation into MODFLOW (Erdelyi, 1953):

$$s_w-M-H_w=\frac{h_{cl}}{\eta}LerchPhi\left(\frac{q}{K_s}\left(\frac{h_{cl}}{h_e}\right)^{\eta},1,\frac{1}{\eta}\right)$$

Fox, G.A. and L. Gordji. 2007. Consideration for unsaturated flow beneath a streambed during alluvial well depletion. *Journal of Hydrologic Engineering – ASCE* 12(2): 139-145.

Table 4.1 Functions for gradually varied flow.

$$\Phi(\eta,N) = -\int_0^{\eta} \frac{d\eta}{\eta^N - 1}$$

The constant of integration is adjusted for $\Phi(0,N) = 0$ and $\Phi(\infty,N) = 0$

η \ N	2.8	3.0	3.2	3.6	4.0	5.0
0.10	0.100	0.100	0.100	0.100	0.100	0.100
0.20	0.201	0.200	0.200	0.200	0.200	0.200
0.30	0.303	0.302	0.302	0.301	0.300	0.300
0.40	0.408	0.407	0.405	0.403	0.402	0.401
0.44	0.452	0.450	0.448	0.445	0.443	0.441
0.48	0.497	0.494	0.492	0.488	0.485	0.482
0.52	0.544	0.540	0.536	0.531	0.528	0.523
0.56	0.593	0.587	0.583	0.576	0.572	0.565
0.58	0.618	0.612	0.607	0.599	0.594	0.587
0.60	0.644	0.637	0.631	0.623	0.617	0.608
0.61	0.657	0.650	0.644	0.635	0.628	0.619
0.62	0.671	0.663	0.657	0.647	0.640	0.630
0.63	0.684	0.676	0.669	0.659	0.652	0.641
0.64	0.698	0.690	0.683	0.672	0.664	0.652
0.65	0.712	0.703	0.696	0.684	0.676	0.663
0.66	0.727	0.717	0.709	0.697	0.688	0.675
0.67	0.742	0.731	0.723	0.710	0.701	0.686
0.68	0.757	0.746	0.737	0.723	0.713	0.698
0.69	0.772	0.761	0.751	0.737	0.726	0.710
0.70	0.787	0.776	0.766	0.750	0.739	0.722
0.71	0.804	0.791	0.781	0.764	0.752	0.734
0.72	0.820	0.807	0.796	0.779	0.766	0.746
0.73	0.837	0.823	0.811	0.793	0.780	0.759
0.74	0.854	0.840	0.827	0.808	0.794	0.771
0.75	0.872	0.857	0.844	0.823	0.808	0.784
0.76	0.890	0.874	0.861	0.839	0.823	0.798
0.77	0.909	0.892	0.878	0.855	0.838	0.811
0.78	0.929	0.911	0.896	0.872	0.854	0.825
0.79	0.949	0.930	0.914	0.889	0.870	0.839
0.80	0.970	0.950	0.934	0.907	0.887	0.854
0.81	0.992	0.971	0.954	0.925	0.904	0.869
0.82	1.015	0.993	0.974	0.945	0.922	0.885
0.83	1.039	1.016	0.996	0.965	0.940	0.901
0.84	1.064	1.040	1.019	0.985	0.960	0.918
0.85	1.091	1.065	1.043	1.007	0.980	0.935
0.86	1.119	1.092	1.068	1.031	1.002	0.954
0.87	1.149	1.120	1.095	1.055	1.025	0.973
0.88	1.181	1.151	1.124	1.081	1.049	0.994
0.89	1.216	1.183	1.155	1.110	1.075	1.015
0.90	1.253	1.218	1.189	1.140	1.103	1.039
0.91	1.294	1.257	1.225	1.173	1.133	1.064
0.92	1.340	1.300	1.266	1.210	1.166	1.092
0.93	1.391	1.348	1.311	1.251	1.204	1.123
0.94	1.449	1.403	1.363	1.297	1.246	1.158
0.95	1.518	1.467	1.423	1.352	1.296	1.199
0.96	1.601	1.545	1.497	1.417	1.355	1.248
0.97	1.707	1.644	1.590	1.501	1.431	1.310
0.98	1.855	1.783	1.720	1.617	1.536	1.395
0.99	2.106	2.017	1.940	1.814	1.714	1.537
0.995	2.355	2.250	2.159	2.008	1.889	1.678

η \ N	2.8	3.0	3.2	3.6	4.0	5.0
1.005	1.818	1.649	1.506	1.279	1.107	0.817
1.01	1.572	1.419	1.291	1.089	0.936	0.681
1.02	1.327	1.191	1.078	0.900	0.766	0.546
1.03	1.186	1.060	0.955	0.790	0.668	0.469
1.04	1.086	0.967	0.868	0.714	0.600	0.415
1.05	1.010	0.896	0.802	0.656	0.548	0.374
1.06	0.948	0.838	0.748	0.608	0.506	0.342
1.07	0.896	0.790	0.703	0.569	0.471	0.315
1.08	0.851	0.749	0.665	0.535	0.441	0.292
1.09	0.812	0.713	0.631	0.506	0.415	0.272
1.10	0.777	0.681	0.601	0.480	0.392	0.254
1.11	0.746	0.652	0.575	0.457	0.372	0.239
1.12	0.718	0.626	0.551	0.436	0.354	0.225
1.13	0.692	0.602	0.529	0.417	0.337	0.212
1.14	0.669	0.581	0.509	0.400	0.322	0.201
1.15	0.647	0.561	0.490	0.384	0.308	0.191
1.16	0.627	0.542	0.473	0.369	0.295	0.181
1.17	0.608	0.525	0.458	0.356	0.283	0.173
1.18	0.591	0.509	0.443	0.343	0.272	0.165
1.19	0.574	0.494	0.429	0.331	0.262	0.157
1.20	0.559	0.480	0.416	0.320	0.252	0.150
1.22	0.531	0.454	0.392	0.299	0.235	0.138
1.24	0.505	0.431	0.371	0.281	0.219	0.127
1.26	0.482	0.410	0.351	0.265	0.205	0.117
1.28	0.461	0.391	0.334	0.250	0.193	0.108
1.30	0.442	0.373	0.318	0.237	0.181	0.100
1.32	0.424	0.357	0.304	0.225	0.171	0.093
1.34	0.408	0.342	0.290	0.214	0.162	0.087
1.36	0.393	0.329	0.278	0.204	0.153	0.081
1.38	0.378	0.316	0.266	0.194	0.145	0.076
1.40	0.365	0.304	0.256	0.185	0.138	0.071
1.42	0.353	0.293	0.246	0.177	0.131	0.067
1.44	0.341	0.282	0.236	0.169	0.125	0.063
1.46	0.330	0.273	0.227	0.162	0.119	0.059
1.48	0.320	0.263	0.219	0.156	0.113	0.056
1.50	0.310	0.255	0.211	0.149	0.108	0.053
1.60	0.269	0.218	0.179	0.123	0.087	0.040
1.70	0.236	0.189	0.153	0.103	0.072	0.031
1.80	0.209	0.166	0.133	0.088	0.060	0.024
1.90	0.188	0.147	0.117	0.076	0.050	0.020
2.00	0.169	0.132	0.104	0.066	0.043	0.016
2.20	0.141	0.107	0.083	0.051	0.032	0.011
2.40	0.119	0.089	0.068	0.040	0.024	0.008
2.60	0.102	0.076	0.057	0.033	0.019	0.005
2.80	0.089	0.065	0.048	0.027	0.015	0.004
3.00	0.078	0.056	0.041	0.022	0.012	0.003
3.50	0.059	0.041	0.029	0.015	0.008	0.002
4.00	0.046	0.031	0.022	0.010	0.005	0.001
5.00	0.031	0.020	0.013	0.006	0.003	0.000
10.00	0.009	0.005	0.003	0.001	0.000	0.000

Example 4.A – Graf

A trapezoidal channel with bottom width of 7.0 m and side slopes of m = 1.5 conveys Q = 28 m^3/s with a bed slope of 0.0010 (n = 0.025). The channel is terminated by a sudden drop of the channel bed. Calculate and plot the profile upstream from the drop using:

(i) method of direct integration (Chow)

(ii) direct step method

(iii) standard step method

Method of Direct Integration - Chow (1959)

b	7 m
m	1.5
n	0.025
S_f	0.001
Q	28 m^3/s
h_n	1.866 m
h_c	1.085 m

	h	h/b	M	N	J	η	ζ	Φ(η,N)	Φ(ζ,J)	x'	x
For Control Point:	1.085	0.155	3.249	3.484	2.820	0.581	0.512	0.60	0.53	98.35	0.00
Select new h	1.200	0.171	3.274	3.506	2.846	0.643	0.581	0.68	0.62	93.22	5.13
Select new h	1.800	0.257	3.400	3.620	2.967	0.965	0.957	1.46	1.54	-551.55	649.90
Select new h	1.864	0.266	3.412	3.632	2.978	0.999	0.999	2.45	2.93	-2002.84	**2101.19**

What happens if I go to 1.866 m (i.e., h = h $_c$)?

	h	h/b	M	N	J	η	ζ	Φ(η,N)	Φ(ζ,J)	x'	x
Select new h	1.866	0.267	3.413	3.632	2.978	1.000	1.000	Infinity	Infinity	N/A	N/A

Direct Step Method

b	7 m
m	1.5
n	0.025
S_f	0.001
Q	28 m^3/s
h_n	1.866 m
h_c	1.085 m

	h	B	P	A	Rh	U	$U^2/2g$	Hs	ΔHs	Se	Avg. Se	Avg. Se-Sf	Δx	x
For Control Point:	1.085	10.255	10.912	9.361	0.858	2.991	0.456	1.541	----	0.007	----	----		
Select new h	1.100	10.300	10.966	9.515	0.868	2.943	0.441	1.541	0.000	0.007	0.007	0.006	0.060	0.060
Select new h	1.150	10.450	11.146	10.034	0.900	2.791	0.397	1.547	0.006	0.006	0.006	0.005	1.093	1.153
Select new h	1.200	10.600	11.327	10.560	0.932	2.652	0.358	1.558	0.011	0.005	0.005	0.004	2.713	3.866
Select new h	1.300	10.900	11.687	11.635	0.996	2.407	0.295	1.595	0.037	0.004	0.004	0.003	11.396	15.262
Select new h	1.400	11.200	12.048	12.740	1.057	2.198	0.246	1.646	0.051	0.003	0.003	0.002	22.962	38.224
Select new h	1.500	11.500	12.408	13.875	1.118	2.018	0.208	1.708	0.061	0.002	0.002	0.001	40.978	79.202
Select new h	1.600	11.800	12.769	15.040	1.178	1.862	0.177	1.777	0.069	0.002	0.002	0.001	71.432	150.634
Select new h	1.700	12.100	13.129	16.235	1.237	1.725	0.152	1.852	0.075	0.001	0.002	0.001	131.246	281.880
Select new h	1.800	12.400	13.490	17.460	1.294	1.604	0.131	1.931	0.079	0.001	0.001	0.000	294.203	576.083
Select new h	1.866	12.598	13.728	18.285	1.332	1.531	0.120	1.986	0.054	0.001	0.001	0.000	779.917	**1356.000**

What happens if I don't take small steps?

	h	B	P	A	Rh	U	$U^2/2g$	Hs	ΔHs	Se	Avg. Se	Avg. Se-Sf	Δx	x
For Control Point:	1.085	10.255	10.912	9.361	0.858	2.991	0.456	1.541	----	0.007	----	----		
Select new h	1.200	10.600	11.327	10.560	0.932	2.652	0.358	1.558	0.017	0.005	0.006	0.005	3.575	3.575
Select new h	1.866	12.598	13.728	18.285	1.332	1.531	0.120	1.986	0.427	0.001	0.003	0.002	223.389	**226.964**

SOLUTION :

1° Before computing the water-surface profile, the normal depth, h_n , and the critical depth, h_c , of the channel should be determined :

The geometrical characteristics, A, P, R_h, B and D_h, for a trapezoidal channel are given in Table 1.1.

a) *Computation of the normal depth, using the Manning-Strickler formula :*

Expressing the average flow velocity with Manning-Strickler formula, eq. 3.16, the discharge in the channel is :

$$Q = U\,A = \frac{1}{n}\;R_h^{\;2/3}\;S_f^{\;1/2}\;A$$

On a spreadsheet, the normal depth can readily be obtained from the above equation by trial-and-error . The computation sheet is presented below :

Computation of *normal* depth by trial-and-error						
$b = 7.0$ [m] ; $m = 1.5$ [-] ; $n = 0.025$ [$m^{-1/3}s$] ; $S_f = 0.001$ [-] ; $Q = 28.0$ [m^3/s]						
trial	h_n [m]	A [m^2]	P [m]	R_h [m]	$Q_{calc.}$ [m^3/s]	Remarks
1	1.500	13.875	12.408	1.118	18.908	no ! $18.908 < 28.000$ try : $h_n > 1.5$ [m]
2	1.900	18.715	13.851	1.351	28.933	no ! $28.933 > 28.000$ try : $1.50 < h_n < 1.9$ [m]
3	1.866	18.285	13.728	1.332	27.999	yes ! $27.999 \cong 28.000$ $h_n = 1.866$ [m]

The *normal depth* is therefore : $h_n = 1.866$ [m]

b) *Computation of the critical depth :*

When the flow is critical, one can write (see eq. 2.22) : $\frac{U_c}{\sqrt{g D_{h_c}}} = 1$

where $U_c = Q / A_c$ (see eq. 1.3). By substituting the relationships for $\mathbf{U_c}$ and $D_{h_c} = A_c / B$ into eq. 2.22, one obtains : $Q = \sqrt{g\ A_c^3 / B}$.

On a spreadsheet, the critical depth can readily be obtained from the above equation by trial-and-error . This is presented below :

Computation of *critical* depth by trial-and-error						
$b = 7.0$ [m] ; $m = 1.5$ [-] ; $n = 0.025$ [$m^{-1/3}s$] ; $S_f = 0.001$ [-] ; $Q = 28.0$ [m^3/s]						
trial	h_c [m]	A_c [m^2]	B [m]	D_{h_c} [m]	$Q_{calc.}$ [m^3/s]	Remarks
1	1.200	10.560	10.600	0.996	33.012	no ! 33.012 > 28.000 try : $h_c < 1.2$ [m]
2	1.000	8.500	10.000	0.850	24.545	no ! 24.545 < 28.000 try : $1.00 < h_c < 1.2$ [m]
3	1.085	9.358	10.254	0.913	27.999	yes ! 27.999 = 28.000 $h_c = 1.085$ [m]

The *critical depth* is therefore : $h_c = 1.085$ [m]

c) *Determination of the type of the water-surface profile :*

Since $h_n > h_c$, the flow is taking place on a mild slope. The water-surface profile will thus be a *type M curve* .

The same conclusion can also be reached by comparing the bed slope with the critical slope. The critical slope is the one for which $h_n = h_c = 1.085$ [m] (see sect. 4.1.3).

If $\quad h_c = h_n = 1.085 \text{ [m]} \quad \Rightarrow \quad A = 9.358 \text{ [m}^2\text{]} \quad$ and $\quad R_h = 0.858$ [m] .

These values are introduced in the Manning-Strickler formula, eq. 3.16 :

$$Q = \frac{1}{0.025}\ 0.858^{2/3}\ S_c^{1/2}\ 9.358 = 28.0 \text{ [m}^3\text{/s]}$$

in order to obtain the critical slope : $\qquad S_c = 0.00686$ [-]

Given that : $\quad S_f$ (= 0.001) > 0 and also S_f (= 0.001) < S_c (= 0.00686), it can be concluded that the flow is taking place on a mild slope and the water-surface profile is a *type M curve* (see Fig. 4.2).

2° To determine in which zone the water-surface profile lies (see sect. 4.2.1), a control section has to be identified.

At the upstream extremity of the channel, the flow is subcritical and uniform with a normal depth of $h_n = 1.866$ [m]. The water-surface profile will therefore be controlled by a singularity at the downstream side. As a matter of fact, the channel terminates by a drop (see sect. 4.4.2) where, it can be assumed that the flow depth passes through the critical depth, $h_c = 1.085$ [m]. This section will be the *control section* for the water-surface profile computation.

The water-surface profile falls between the normal and critical depths. By proceeding towards upstream the flow depth increases from the critical depth, $h_c = 1.085$ [m], to the normal depth, $h_n = 1.866$ [m]. *The water-surface profile is a M2 type curve* (see Figs. 4.2 and 4.2a).

3° *Computation of water-surface profile, using the method of direct integration* :

The *method of Chow* (see point 4.3.2; 5 °) will now be used since it is free from the simplifying assumptions used in other methods. According to the method of Chow, the abscissa, x_i', (with respect to an arbitrary origin) of a section (i), where the flow depth is equal to h, can be written as follows (see eq. 4.29) :

$$x_i' = \frac{h_n}{S_f} \left\{ \eta_i - \Phi(\eta_i, N) + \left(\frac{h_c}{h_n}\right)^M \frac{J}{N} \left[\Phi(\zeta_i, J) \right] \right\}$$

This equation can be implemented in a spreadsheet program for calculating the water-surface profile in a tabular form.

In order to create a fully automatic computation sheet, the analytical expre[illegible] have to be established for the hydraulic exponents, N and M, which are given by :

$$N(h) = \frac{2h}{3A} \left(5B - 2R_h \frac{dP}{dh} \right) \quad , \quad M(h) = \frac{h}{A} \left(3B - \frac{A}{B} \frac{dB}{dh} \right) \qquad (4.2[illegible])$$

The terms, dP/dh and dB/dh , will be evaluated using the geometrical charac[illegible] of the section. For a trapezoidal section, one has (see Table 1.1) :

$$P = b + 2h\sqrt{1 + m^2} \quad \Rightarrow \quad dP / dh = 2\sqrt{1 + m^2}$$

$$B = b + 2mh \quad \Rightarrow \quad dB / dh = 2m$$

Substituting these relationships in eqs. 4.26, and simplifying, the fo[illegible] expressions can be obtained (see *Chow* 1959, p. 131 and p. 66):

$$N(h) = \frac{10}{3} \frac{1 + 2m\,(h/b)}{1 + m\,(h/b)} - \frac{8}{3} \frac{\sqrt{1 + m^2}\,(h/b)}{1 + 2\sqrt{1 + m^2}\,(h/b)} \qquad (4.2[illegible])$$

$$M(h) = \frac{3\,[1 + 2m\,(h/b)]^2 - 2m\,(h/b)\,[1 + m\,(h/b)]}{[1 + 2m\,(h/b)]\,[1 + m\,(h/b)]}$$

Method of direct integration : Chow's method										
$b = 7.0$ [m] ; $m = 1.5$ [-] ; $n = 0.025$ [$m^{-1/3}s$] ; $S_f = 0.001$ [-] ; $Q = 28.0$ [m^3/s]										
$h_n = 1.866$ [m]				$h_c = 1.085$ [m]			$h_c/h_n = 0.581$ [-]			
[illegible]	2	3	4	5	6	7	8	9	10	11
h [m]	h/b [-]	M [-]	N [-]	J [-]	η [-]	ζ [-]	Φ(η,N) [-]	Φ(ζ, J) [-]	x' [m]	x [m]
[illegible]	0.155	3.25	3.48	2.82	0.581	0.512	0.603	0.534	98.243	0.000
[illegible]	0.157	3.25	3.49	2.82	0.589	0.521	0.613	0.544	97.798	0.445
[illegible]	0.164	3.26	3.50	2.83	0.616	0.550	0.645	0.580	96.742	1.501
[illegible]	0.171	3.27	3.51	2.85	0.643	0.581	0.678	0.617	93.192	5.051
[illegible]	0.186	3.30	3.52	2.87	0.697	0.641	0.748	0.697	81.116	17.127
[illegible]	0.200	3.32	3.54	2.89	0.750	0.703	0.826	0.787	56.379	41.865
[illegible]	0.214	3.34	3.56	2.91	0.804	0.765	0.916	0.891	12.509	85.734
[illegible]	0.229	3.36	3.58	2.93	0.857	0.829	1.026	1.021	-63.247	161.490
[illegible]	0.243	3.38	3.60	2.95	0.911	0.892	1.177	1.200	-202.640	300.883
[illegible]	0.257	3.40	3.62	2.97	0.965	0.957	1.457	1.531	-548.543	646.786
[illegible]	0.266	3.41	3.63	2.98	0.999	0.999	2.454	2.931	2010.449	*2108.693*

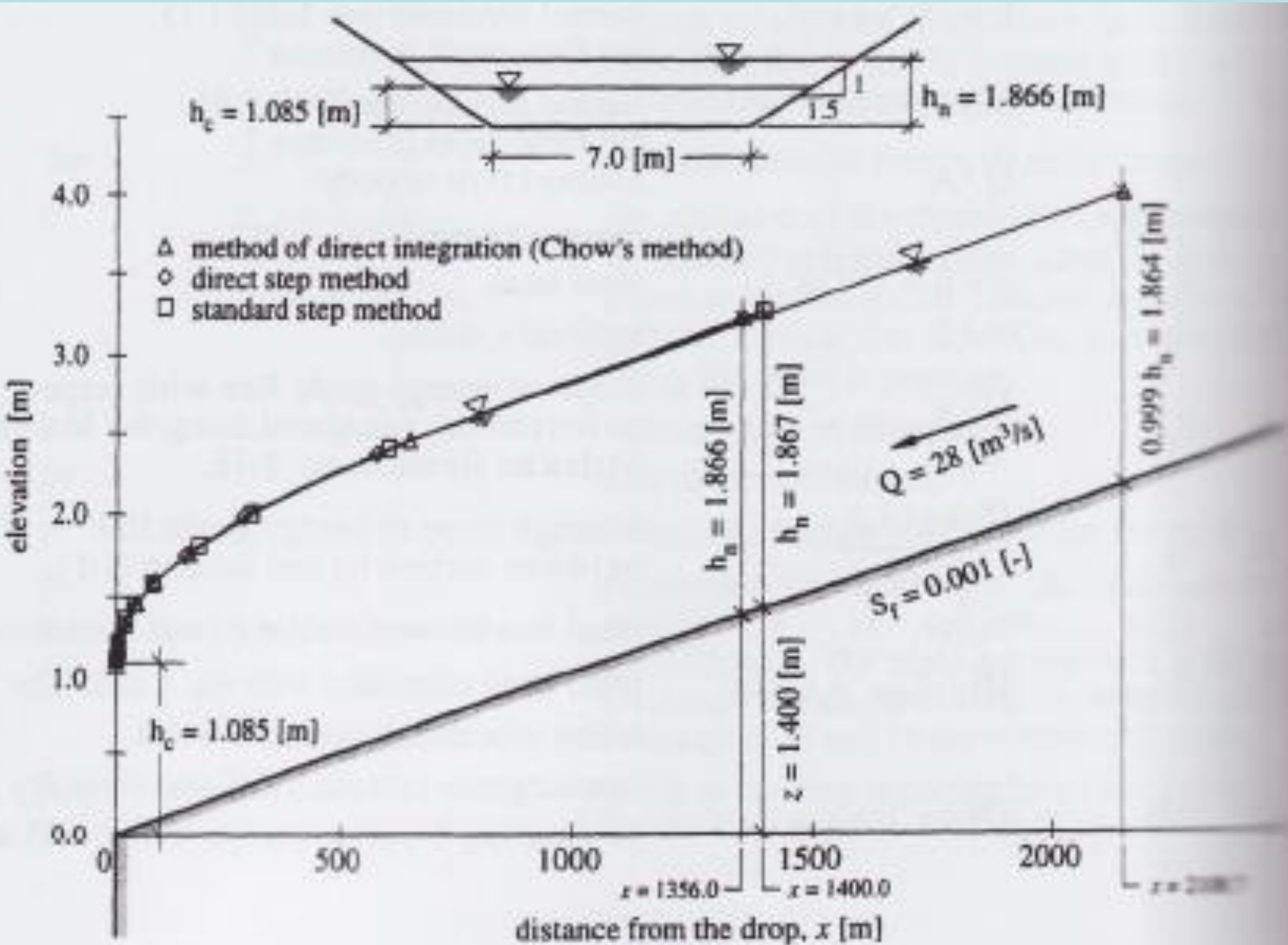
col.	symbol	explanations
1	h	flow depth. The water-surface profile increases from the critical depth, $h_c = 1.085$ [m], to the normal depth, $0.999h_n = 1.864$ [m]. This difference in flow depth is arbitrarily divided into several intervals.
2	h/b	relative flow depth (with respect to bottom width).
3-4	M , N	hydraulic exponents, calculated according to eqs. 4.26a.
5	J	$N / (N-M+1)$
6	η	h / h_n , relative flow depth.
7	ζ	$\eta^{N/J}$, dimensionless flow depth.
8	$\Phi(\eta, N)$	function defined by eq. 4.24a. The values are read from Table 4.1, being functions of η and N .
9	$\Phi(\zeta, J)$	function defined by eq. 4.28a. The values are read from Table 4.1, being functions of ζ and J .
10	x'	distance from an *arbitrary* origin, calculated using the expression : $x_i' = \frac{h_n}{J_f} \left\{ \eta_i - \Phi(\eta_i, N) + \left(\frac{h_c}{h_n} \right)^M \frac{J}{N} \left[\Phi(\zeta_i, J) \right] \right\}$ (4.29
11	x	distance from the drop ($x = 0$ [m]): $x_i = x'_{(h = 1.085)} - x_i'$.

Step method :						
b = 7.0 [m] ; m = 1.5 [-] ; n = 0.025 [$m^{-1/3}s$] S_f = 1.0E-03 [-] ; Q = 28.0 [m^3/s]						
1	2	3	4	5	6	7
h [m]	B [m]	P [m]	A [m^2]	R_h [m]	U [m/s]	$U^2/2g$ [m]
1.085	10.255	10.912	9.361	0.858	2.991	0.456
1.100	10.300	10.966	9.515	0.868	2.943	0.441
1.150	10.450	11.146	10.034	0.900	2.791	0.397
1.200	10.600	11.327	10.560	0.932	2.652	0.358
1.300	10.900	11.687	11.635	0.996	2.407	0.295
1.400	11.200	12.048	12.740	1.057	2.198	0.246
1.500	11.500	12.408	13.875	1.118	2.018	0.208
1.600	11.800	12.769	15.040	1.178	1.862	0.177
1.700	12.100	13.129	16.235	1.237	1.725	0.152
1.800	12.400	13.490	17.460	1.294	1.604	0.131
1.866	12.598	13.728	18.285	1.332	1.531	0.120

col.	symbol	expression	explanations
1	h		flow depth. The water-surface profile increases from the critical depth, $h_c = 1.085$ [m] , to the normal depth, $h_n = 1.866$ [m]. This difference in flow depth is arbitrarily divided into several *small* intervals.
2	B	$b + 2mh$	water-surface width (see Table 1.1) for the flow depth in column 1.
3	P	$b + 2h\sqrt{1+m^2}$	wetted perimeter (see Table 1.1) for the flow depth in column 1.
4	A	$h\,(b + m\,h)$	wetted surface (see Table 1.1) for the flow depth in column 1.
5	R_h	A / P	hydraulic radius for the flow depth in column 1.
6	U	Q / A	average flow velocity.
7	$U^2/2g$		dynamic head.

Direct step method (Δh is fixed)						
h_n = 1.866 [m] ; h_c = 1.085 [m]						
convention : C = 1, towards upstream (Fr < 1) ; C = -1, towards downstream (Fr > 1)						
8	9	10	11	12	13	14
H_s [m]	ΔH_s [m]	S_e [-]	$\overline{S_e}$ [-]	$\overline{S_e} - S_f$ [-]	Δx [m]	x [m]
1.541	—	6.86E-03	—	—	—	0.000
1.541	0.000	6.54E-03	6.70E-03	5.70E-03	0.060	0.060
1.547	0.006	5.60E-03	6.07E-03	5.07E-03	1.093	1.153
1.558	0.011	4.82E-03	5.21E-03	4.21E-03	2.713	3.866
1.595	0.037	3.64E-03	4.23E-03	3.23E-03	11.396	15.262
1.646	0.051	2.80E-03	3.22E-03	2.22E-03	22.962	38.224
1.708	0.061	2.19E-03	2.50E-03	1.50E-03	40.978	79.202
1.777	0.069	1.74E-03	1.97E-03	9.67E-04	71.432	150.634
1.852	0.075	1.40E-03	1.57E-03	5.71E-04	131.246	281.880
1.931	0.079	1.14E-03	1.27E-03	2.70E-04	294.203	576.083
1.986	0.054	1.00E-03	1.07E-03	6.98E-05	779.917	*1356.000*

col.	symbol	expression	explanations
8	H_s	$h + U^2/2g$	*specific* head, eq. 2.14.
9	ΔH_s	$(H_s)_i - (H_s)_{i-1}$	specific head difference between section (i) and previous section (i-1).
10	S_e	$\frac{U^2 n^2}{R_h^{4/3}}$	slope of energy-grade line with respect to horizontal, calculated using the Manning-Strickler formula, eq. 3.16.
11	$\overline{S_e}$	$\frac{(S_e)_i + (S_e)_{i-1}}{2}$	average slope of energy-grade line between section (i) and section (i-1).
12	$\overline{S_e} - S_f$		average slope of energy-grade line with respect to channel bed.
13	Δx	$C \frac{\Delta H_s}{(\overline{S_e} - S_f)}$	distance between section (i) section (i-1), eq. 4.14. The flow being subcritical, one has $C = +1$.
14	x	$\sum_{1}^{i} \Delta x$	accumulated distance from the control section to section (i).



h_c = 1.085 [m]
1
1.5
h_n = 1.866 [m]
7.0 [m]
4.0
3.0
2.0
1.0
0.0
elevation [m]
method of direct integration (Chow's method)
direct step method
standard step method
h_n = 1.866 [m]
h_n = 1.867 [m]
0.999 h_n = 1.864 [m]
Q = 28 [m³/s]
S_f = 0.001 [-]
z = 1.400 [m]
h_c = 1.085 [m]
0
500
1000
1500
2000
x = 1356.0
x = 1400.0
distance from the drop, x [m]