HP
Technical Assistance
HYDROLOGY PROJECT

VOLUME 4
HYDROMETRY

REFERENCE MANUAL

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1   INTRODUCTION TO HYDRAULICS

1.1   GENERAL

In this chapter some more background is presented of some basic elements of hydraulics, dealt with in Chapter 2 of the Design Manual on Hydrometry. The following topics are dealt with:

- Classification of flows
- Properties of water
- Flow velocity profiles
- Unsteady flow equations, and
- Gradually varied flow.

1.2   CLASSIFICATION OF FLOWS

Main forces acting on a mass of fluid:

-	gravity	$F_g = M.g = \rho L^3 g$
-	pressure	$F_p = p.A = pL^2$
-	viscosity	$F_v = \tau.A = \rho v u L$
-	surface tension	$F_\sigma = \sigma.L = \sigma L$
-	elasticity	$F_e = \kappa A = \kappa L^2$
-	inertia	$F_I = Ma = \rho u^2 L^2$

Generally, one of these forces predominates. The inertia force is always present. To characterise the physical phenomena, the forces are compared to the inertial force F_i. Of importance are here:

- Reynolds number
  (viscosity)

$$\frac{Fi}{F_v} = \frac{\rho u^2 L^2}{\rho v u L} = \frac{uL}{v} \equiv Re$$

- Froude number
  (gravity)

$$\frac{Fi}{F_g} = \frac{\rho u^2 L^2}{\rho L^3 g} = \frac{u^2}{gL} \equiv F_r^2$$

For open channel flow the flow-depth h_w is taken as characteristic length. Then L in the above formulae is replaced by h_w. Following numbers are of importance.

- The Reynolds number, to distinguish between laminar and turbulent flow:

$$Re \equiv \frac{uh_w}{v}$$

Re < 600	laminar flow
$600 \leq Re < 2000$	transitional flow
Re > 2000	turbulent flow

- The Froude number, which distinguishes between sub-critical and super critical flow:

$$F_r = \frac{u}{\sqrt{gh_w}}$$

$F_r < 1$	subcritical flow
$F_r = 1$	critical flow
$F_r > 1$	supercrirtical flow

Open channel flow can also be classified based on temporal and spatial changes in mean velocity u and mean depth h_w: $u = f_1(x,t)$ and $h_w = f_2(x,t)$

- **Steady flow**   : $\dfrac{\partial u}{\partial t} = 0$ and $\dfrac{\partial h_w}{\partial t} = 0$

  - <u>Uniform flow</u>   : $\dfrac{\partial u}{\partial x} = 0$, $\dfrac{\partial h_w}{\partial x} = 0$

  - <u>Varied flow</u>   : $\left|\dfrac{\partial u}{\partial x}\right| > 0$, $\left|\dfrac{\partial h_w}{\partial t}\right| > 0$

    - <u>Gradually varied flow</u>   : $\dfrac{\partial u}{\partial x}$, $\dfrac{\partial h_w}{\partial x}$ relatively small

    - <u>Rapidly varied flow</u>   : $\dfrac{\partial u}{\partial x}$, $\dfrac{\partial h_w}{\partial x}$ relatively large

- **Unsteady flow**   : $\left|\dfrac{\partial u}{\partial t}\right| > 0$ and $\left|\dfrac{\partial h_w}{\partial t}\right| > 0$

    - celerity kinematic waves $c = \dfrac{5}{3}u$ (Manning) or $c = \dfrac{3}{2}u$ (Chezy)

    - celerity dynamic waves   : $c = u \pm \sqrt{gh_w}$

1.3   PROPERTIES OF WATER

Some relevant properties of water are:

Property	Symbol	Dimension	Remarks
Density	ρ	$kg.m^{-3}$	
Relatively density of sediment under water	Δ	-	$\Delta = \dfrac{\rho_s - \rho}{\rho}$
Dynamic viscosity	η	$kg.m^{-1}.s^{-1}$	$\tau = \eta\, \partial u / \partial z$
Kinematic viscosity	ν	$m^2.s^{-1}$	$\nu = \eta / \rho$
Surface tension	σ	$N.m^{-1}$	$\sigma_{water/air}$ = 0.074 Nm^{-1} at atmospheric pressure

Table 1.1:   Some properties of water

Density of water varies with   - temperature (little variation)
                                                        - salinity : fresh water ρ = 1000 $kg.m^{-3}$
                                                                            salt water ρ = 1026 $kg.m^{-3}$

Viscosity of water varies with temperature, see Table 1.2:

$$\text{for}\quad T > 10^\circ C:\quad \nu \approx \frac{4 \times 10^{-5}}{20 + T}\quad [m^2.s^{-1}]$$

T (°C)	0	5	10	15	20	25	30	35	40
$\nu(10^{-6}\ m^2/s)$	1.79	1.52	1.31	1.14	1.01	0.90	0.80	0.72	0.65

Table 1.2:   Kinematic viscosity as function of temperature

1.4   VELOCITY PROFILES

1.4.1   INTRODUCTION

To develop expressions for the variation of the flow velocity with depth, consider steady uniform flow conditions, which includes: - streamlines // bed

- bed slope S_0 = water surface slope S_w = energy gradient S_e

- hydrostatic pressure distribution.

The velocity profile is obtained from:

(1) balance of forces in flow direction

(2) relation between shear stress and velocity
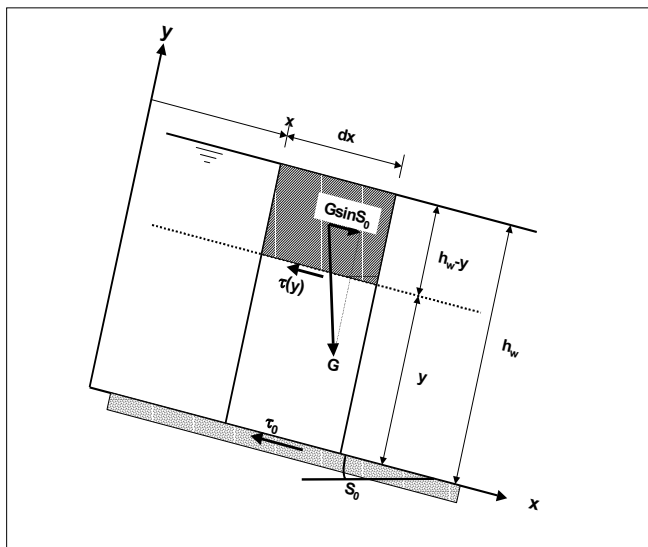
Figure 1.1:
Definition sketch

Balance of forces in x – direction, see Figure 1.1, with G = $\rho g.(h_w$-y).dx.1:

$\rho g \sin S_0.(h_w - y)dx = \tau(y).dx$

or: $\tau(y) = \rho g(h_w - y)\sin S_0$

Since S_0 is small $\rightarrow \sin S_0 \approx S_0$, so:

$\tau(y) = \rho g(h_w - y)S_0$ (1.1)

The relation between $\tau(y)$ and $u(y)$ is generally given by an equation of the form:

$$\tau(y) = \rho\nu\frac{du}{dy} \tag{1.2}$$

- for **laminar** flow (ν) is constant with y

- for **turbulent** flow (ν) varies with y $(\nu$ has a different meaning)

1.4.2   LAMINAR FLOW VELOCITY PROFILE

With (1.1) = (1.2) and elimination $\tau(y)$ gives:

$$du = g\frac{S_0}{\nu}(h_w - y)\,dy \tag{1.3}$$

Integration of (1.3) leads to:

$$u(y) = \frac{gS_0}{\nu}(h_w\,y - \frac{1}{2}y^2) + C$$

No slip condition at y = 0 $\rightarrow$ u(0) = 0 $\rightarrow$ C = 0, so:

$$\left. \begin{array}{l} u(y) = \dfrac{gS_0}{\nu}\left(h_w\,y - \dfrac{1}{2}y^2\right) \\[3em] u(h) = u_{max} = \dfrac{gS_0}{2\nu}h_w^2 \end{array} \right\} \qquad \dfrac{u(y)}{u_{max}} = \dfrac{2y}{h_w}\left(1 - \dfrac{y}{2h_w}\right) \tag{1.4}$$
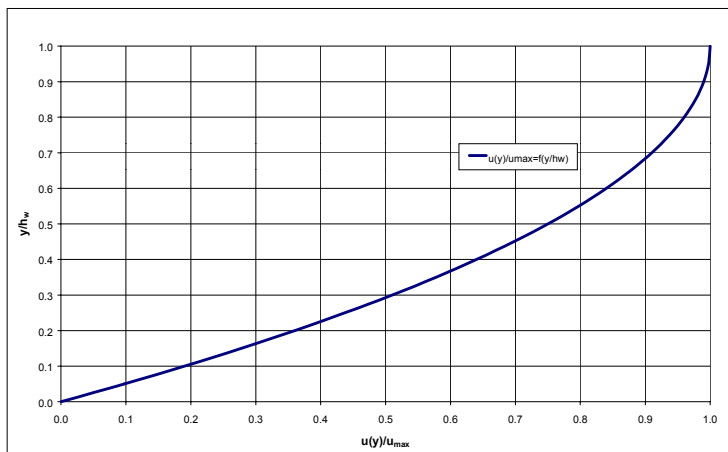
Figure 1.2:
Velocity profile for laminar flow

The function (1.4) is shown in Figure 1.2. The velocity profile for laminar flow is seen to be **parabolic**.

The depth-averaged flow velocity becomes:

$$\bar{u} = \frac{1}{h_w}\int_0^{h_w} u(y)\,dy = \frac{gS_0}{3\nu}.h_w^2 \tag{1.5}$$

Note:

$\bar{u}$ = :: S_0, which is characteristic for laminar flow

$\bar{u} = \dfrac{2}{3}u_{max}$

1.4.3   TURBULENT FLOW VELOCITY PROFILE

Shear stress

The shear stress τ now consists of a **laminar** part τ_L and a **turbulent** part τ_T.

$$\tau(y) = \tau_L + \tau_T = \rho\nu\frac{du}{dy} - \rho\overline{u'v'} \tag{1.6}$$

$$\tau_T = -\rho\overline{u'v'} \quad : \qquad\qquad \text{Reynolds stress}$$

u', v' = turbulent fluctuations about **time averaged** horizontal and vertical velocity components

For small y (very close to the bed) (1.1) becomes:

$$\tau \approx \tau_0 = \rho g h_w S_0 \tag{1.7}$$

Define **shear velocity**: $u_* = \sqrt{\dfrac{\tau_0}{\rho}}$ so : $\tau_0 = \rho u_*^2$ (1.8)

$\Rightarrow$ for **steady uniform flow** : $u_* = \sqrt{g h_w S_0}$

For equation (1.1) one can express $\tau(y)$ with (1.7) as a function of the bed shear stress τ_0:

$$\tau(y) = \tau_0 (1 - y/h_w) \tag{1.9}$$

Velocity profile

To get expressions for the velocity profile 3 areas in the vertical are distinguished:

- very close to the bed, where $y \approx 0$
- at larger distance from the bed where $y > 0$ but $y \ll h_w$
- at large distance from the bed, where $y \ll h_w$ no longer holds

1.   Very close to the bed: $y \approx 0$ and v' = 0

$\Rightarrow$ From (1.8) and (1.9): $\tau \approx \tau_0 = \rho u_*^2$

$\Rightarrow$ From (1.6): $v' = 0 : \tau \approx \tau_L = \rho\nu\dfrac{du}{dy}$ $du = \dfrac{u_*^2}{\nu}dy$

Integration with u(0) = 0 gives:

$$u(y) = \frac{u_*^2}{\nu} \cdot y \tag{1.10}$$

2.   At larger distance from the bed, where $|v'| > 0$ but $y \ll h_w$

$\Rightarrow$ From (1.8) and (1.9): $\tau = \tau_0 (1 - y/h_w) \approx \tau_0 = \rho u_*^2$ (1.11)

$\Rightarrow$ In (1.6): $\tau_T \gg \tau_L : \tau = -\rho\overline{u'v'}$

Using the **mixing–length hypothesis of Prandtl**: $u' = \lambda \left| \dfrac{du}{dy} \right|$ and assuming u' and v' are of the same order of magnitude:

$$\tau = -\rho\overline{u'v'} = \rho\lambda^2 \left| \frac{du}{dy} \right| \frac{du}{dy} \tag{1.12}$$
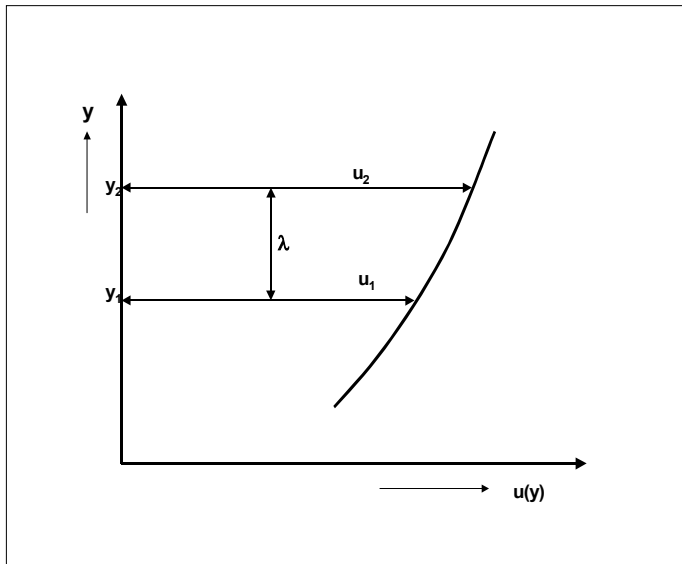
Figure 1.3:
Mixing-length hypothesis of Prandtl

The mixing-length hypothesis of Prandtl is illustrated in Figure 1.3. At y_1 and y_2 the time-averaged velocities are respectively u_1 and u_2. Due to turbulence water particles move from one layer to the other thereby exchanging momentum. This exchange creates a shear stress between the layers, since a particle arriving at y_2 which originates from y_1 has a velocity defect and therefore slows down the upper layer. Inversely, particles originating from y_2 and arriving at y_1 drag the lower layer to a higher velocity. As a result in turbulent flow the gradients with depth in the time-averaged flow velocities will be less than in laminar flow. From the definition sketch in Figure 1.3 the time-averages velocities u_2 and u_1 are related as follows: $u_2 = u_1 + \lambda du/dy$. Prandtl assumed that a particle leaving level y_1 over a distance λ retains its identity and that the order of magnitude of the turbulent fluctuation u' equals the difference in average velocity over a distance λ, so: u' = $\lambda \left| du/dy \right|$ leading to the result presented in (1.12)

Writing (1.12) in the form of (1.2) with $\nu = \nu_T$, i.e. the turbulent viscosity coefficient:

$$\rho\nu_T \frac{du}{dy} = \rho\lambda^2 \left| \frac{du}{dy} \right| \frac{du}{dy} \qquad \Rightarrow \qquad \nu_T = \lambda^2 \left| \frac{du}{dy} \right| \tag{1.13}$$

The problem now is to find an expression for the mixing length λ. Prandtl assumed:

$$\lambda = \kappa y \tag{1.14}$$

with:　　κ = the Von Karman constant, with $\kappa \approx 0.4$.

Substitution of (1.14) in (1.12) and elimination of τ with (1.11) gives:

$$\rho u_*^2 = \rho(\kappa y)^2 \left| \frac{du}{dy} \right| \frac{du}{dy} \qquad \Rightarrow \qquad du = \frac{u_*}{\kappa y} . dy \tag{1.15}$$

$$\Rightarrow \qquad \int du = \frac{u*}{\kappa} \int \frac{dy}{y}$$

$$\Rightarrow \quad u(y) = \frac{u_*}{\kappa} \ell n(y) + C$$

The constant C is determined by putting $u(y_0) = 0$, hence:

$$u(y) = \frac{u_*}{\kappa} \ell n \left(\frac{y}{y_0} \right) \tag{1.16}$$

The quantity y_0 is a theoretical value, i.e. the depth at which u = 0 according to the logarithmic profile.

3.   Large distance from the bed; here the assumption $y \ll h_w$ is **no** longer true. Then it follows:

$$\Rightarrow \qquad \tau(y) = \tau_0 \left(1 - y / h_w \right) = \rho u_*^2 \left(1 - y / h_w \right) \tag{1.17}$$

Experiments show that the logarithmic profile also applies at a larger distance from the bed. The problem is closed by assuming for λ (see also Figure 1.4):

$$\lambda = \kappa y \sqrt{1 - y / h_w} \tag{1.18}$$

From (1.18) it follows that λ is maximum at $y = 2/3 \, h_w$ with $\lambda = 0.15 \, h_w$. Substitution of (1.18) in (1.12) and elimination of τ with (1.17) gives:

$$\rho u_*^2 \left(1 - y / h_w \right) = \rho (\kappa y)^2 \left(1 - y / h_w \right) \left| \frac{du}{dy} \right| \frac{du}{dy}$$

which leads to (1.15) and (1.16) as before.
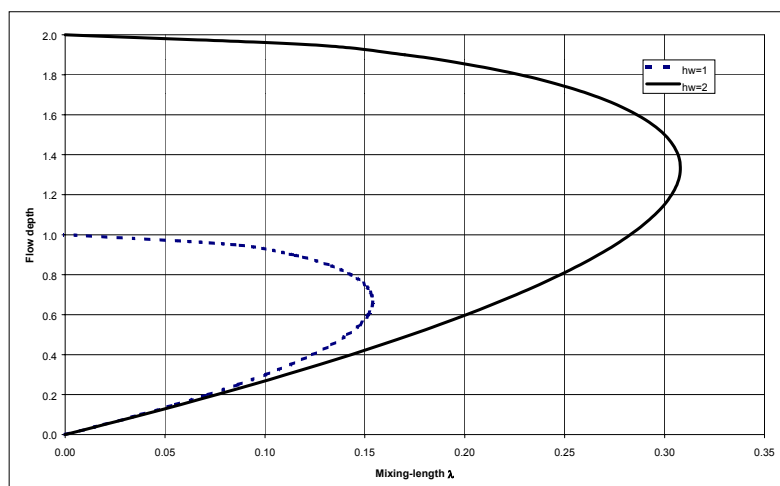
Figure 1.4:
Mixing-length for water depths of 1 and 2 m

Summing up for turbulent flow

In summing up of the above three cases one obtains:

(a)   close to the bed (laminar sublayer):

$$u(y) = \frac{u_*^2}{\nu} y$$

(b)   above the sublayer a logarithmic profile (fully developed turbulent flow):

$$u(y) = \frac{u*}{\kappa} \ell n \left(\frac{y}{y_0} \right)$$

The profiles intersect at $y = \delta$, where δ is the thickness of laminar sublayer, see Figure 1.5.

$$\delta = 11.6 \frac{\nu}{u_*} \tag{1.19}$$

In fact there is a transition zone between a and b extending from (5 to 30) ν/u_*. The laminar sublayer or transition zone are very thin layers, generally less than 1 mm.

Average velocity

$$\bar{u} = \frac{1}{h_w} \int_{y_0}^{h_w} u(y)dy = \frac{u_*}{\kappa h_w} \int_{y_0}^{h_w} \ell n \left(\frac{y}{y_0} \right) dy$$

Since $\int \ell n\, x\, dx = x\, \ell n\, x - x + C$ it follows:

$$\bar{u} = \frac{u_*}{\kappa} \left\{ \ell n \left(\frac{h_w}{y_0} \right) - 1 + \frac{y_0}{h_w} \right\} \quad \text{qr with } y_0 \ll h_w$$

$$\bar{u} = \frac{u_*}{\kappa} \ell n \left(\frac{h_w}{e.y_o} \right) \tag{1.20}$$

Comparison of (1.16) with (1.20) gives for $y = y_1$

$$y_1 = \frac{h_w}{e} = 0.368\, h_w \quad \Rightarrow \quad u(y_1) = \bar{u}$$

Note: from (1.20) it is observed that for **turbulent flow** $\bar{u} :: u_*$, so: $\bar{u} :: \sqrt{S_0}$
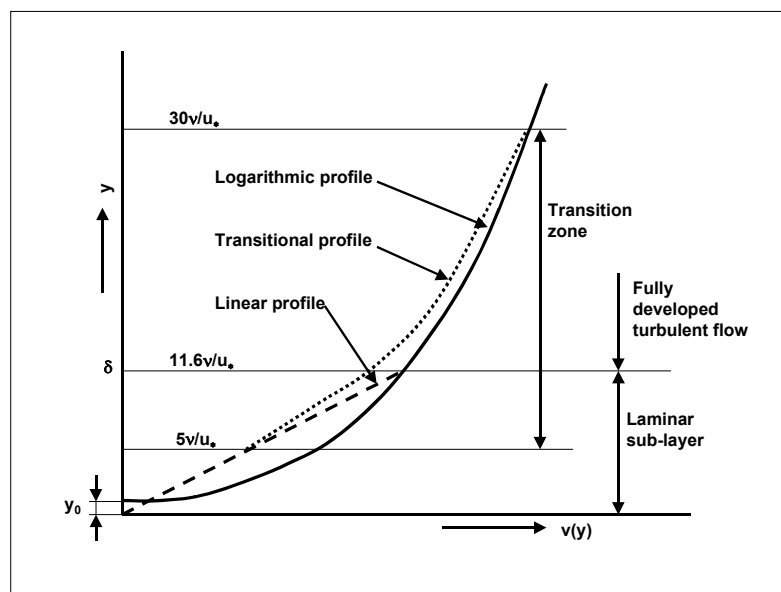
Figure 1.5:
Velocity profiles in the boundary layer

Hydraulically smooth and rough boundaries

In equations (1.16) and (1.20) y_0 has still to be determined. Its value depends on the roughness of the bottom, characterized by the **equivalent sand roughness** k_s. A bed with roughness k_s produces the same resistance as a flat bed covered with fixed, uniform, closely packed sand grains of diameter k_s (according to Nikuradse).

$\Rightarrow$ Now if $k_s << \delta$ the laminar or viscous sublayer is present; then the roughness is not 'felt' by the turbulent flow and y_0 is determined by δ only: the boundary is hydraulically **smooth**.

$\Rightarrow$ If $k_s >> \delta$ then the laminar sublayer is not present and y_0 is related to k_s only: the boundaries is hydraulically **rough**.

$\Rightarrow$ In between these two conditions there is a transition where y_0 depends on δ as well as k_s.

In formula's (from experiments)

1.   hydraulically smooth   : $k_s < 0.3\delta$ and $y_0 \approx \dfrac{\delta}{117}$ $\hspace{4cm}$ (1.21a)

2.   hydraulically rough   : $k_s > 6\delta$ and $y_0 \approx \dfrac{k_s}{32}$ $\hspace{4cm}$ (1.21b)

The velocity profiles then become:

- **smooth boundary**

$$u(y) \ = \ \frac{u_*}{\kappa}\, \ell n\!\left(\frac{y}{y_0}\right) \ = \ \frac{u_*}{\kappa}\, \ell n\!\left(\frac{117y}{\delta}\right) \hspace{4cm} (1.22)$$

$$\overline{u} \ = \frac{u_*}{\kappa}\, \ell n\!\left(\frac{117 h_w}{e\delta}\right) = \frac{u_*}{\kappa}\, \ell n\!\left(\frac{12 h_w}{\delta/3.5}\right) \hspace{3cm} (1.23)$$

- **rough boundary**

$$u(y) = \frac{u_*}{\kappa}\, \ell n\!\left(\frac{y}{y_0}\right) = \frac{u_*}{\kappa}\, \ell n\!\left(\frac{32y}{k_s}\right) \hspace{4cm} (1.24)$$

$$\overline{u} = \frac{u_*}{\kappa}\, \ell n\!\left(\frac{32 h_w}{e k_s}\right) \approx \frac{u_*}{\kappa}\, \ell n\!\left(\frac{12 h_w}{k_s}\right) \hspace{3cm} (1.25)$$

- **transition** between smooth and rough it was found:

$$\overline{u} \ = \ \frac{u_*}{\kappa}\, \ell n\!\left(\frac{12 h_w}{k_s + \delta/3.5}\right) \hspace{4cm} (1.26)$$

The above formulas are valid for wide channels. For other cross-sections h_w is replaced by the hydraulic radius R.

A comparison of velocity profiles for various values of h/y_0 is presented in Figure 1.6. In this figure for comparison reasons also the laminar velocity profile is shown. It is observed that the variation in flow velocity with depth under turbulent flow conditions (apart from the layer close to the bed) is less than under laminar flow conditions. For turbulent flow the variation with depth is less the larger h_w/y_0 i.e. the smaller y_0 or k_s is. Hence, a smooth riverbed gives the least variation in the vertical velocity profile.

From Figure 1.6 it is also observed that when the flow velocity is measured at one point in the vertical one should measure it at approximately $0.4h_w$. If one measures at $0.5h_w$ a correction factor has to be applied which is dependent on the roughness of the bed and may vary between 0.9 and 0.95.

It is noted that above velocity profiles are theoretical ones. In practice the profiles are more disturbed due to variation of depth in the cross-section and wind effects.
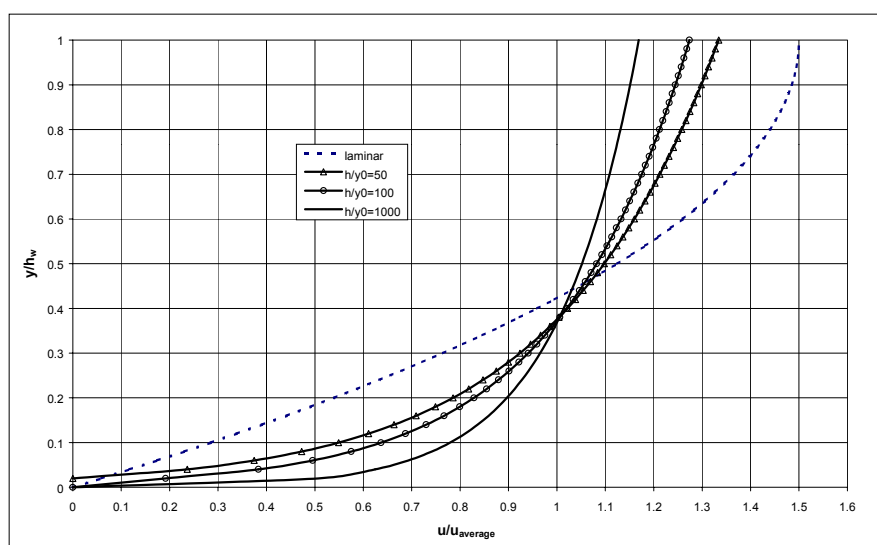
Figure 1.6:
Relative velocity profiles
expressed as function of
relative depth

From the velocity profiles the accuracy of measuring the velocity at 1 or 2 points in the vertical can easily be determined as far as depth is concerned. In Table 1.3 the 0.6, 0.5 and the 0.2-0.8 methods have been compared. For the one-point methods it has also been indicated what errors are made in the mean velocity if the point velocity is taken when a $\pm 4\%$ error is made in the total depth measurement. In computing the deviation for the 0.5-method a multiplier of 0.95 has been assumed to estimate the depth-average velocity.

Method	y/h_w	h_w/y_0=100		h_w/y_0=1000	
		$u/u_{average}$	deviation %	$u/u_{average}$	deviation %
0.6	0.36	0.99	-1	1.00	0
	0.40	1.02	2	1.01	1
	0.44	1.05	5	1.03	3
0.5	0.46	1.06	1	1.04	-1
	0.50	1.08	3	1.05	0
	0.54	1.10	5	1.06	1
0.2-0.8	0.20	0.829	2	0.897	1
	0.80	1.212		1.131	

Table 1.3:   Accuracy of 1 and 2-point methods for estimating depth average flow velocities

From the table it is observed that if the theoretical velocity profiles apply the errors in the depth average flow velocity estimated by 1 or 2-point methods are approximately the same. If the riverbed is smooth (note that $y_0 \approx k_s/32$) it is observed that the 0.5-method is slightly less sensitive for errors in the depth than the 0.6-method. Note also that when the result deviates from the depth-average mean the methods will, in general, give a positive bias (in the table in all cases but two).

1.5   HYDRODYNAMIC EQUATIONS

1.5.1   INTRODUCTION

Consider the river cross-section as shown in Figure 1.7.
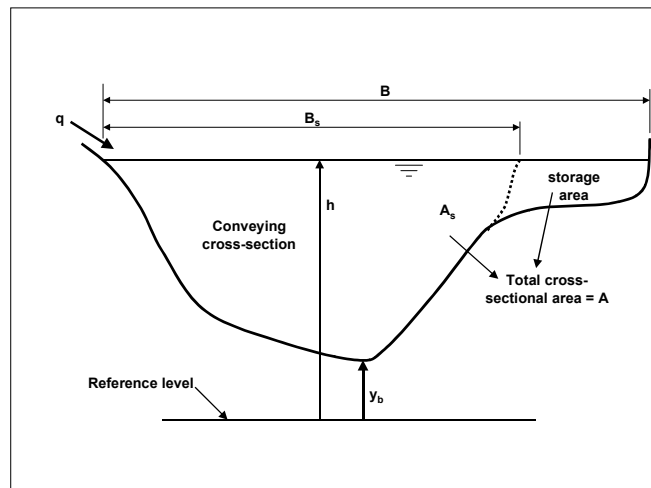
Figure 1.7:
Definition sketch river cross-section

The cross section is divided into a flow conveying area A_s and a storage area. Total area is A.

B   =   width at the water surface of A

B_s =   width at the water surface of A_s

h   =   water level relative to reference level

h_w =   water depth

y_b =   bottom level relative to reference level

q   =   lateral inflow (+)

In the derivation of the hydrodynamic equations the **Eulerian approach** is followed, i.e. a **fixed control volume** is considered with boundaries ⊥ to the flow. First the co-ordinate system is taken parallel to the main flow direction: s = distance along the bed.

The equations are:

- the continuity equation: conservation of mass.

- the momentum equation: the sum of the forces acting on the mass in the control volume = the change of momentum per unit of time Newton's law : $\frac{Q^2}{A_s^2}\left(\frac{1}{A_s^2}\sum_{i=1}^{n}C_i A_{si}\sqrt{R_i}\right)^2 = \varepsilon$

1.5.2   CONTINUITY EQUATION

Mass balance for **control volume** (see Figure 1.8):

$$\left\{ \begin{matrix} \text{Net mass inflow} \\ \text{into control volume in} \\ \text{time dt} \end{matrix} \right\} \quad = \quad \left\{ \begin{matrix} \text{increase of mass} \\ \text{in control volume} \\ \text{in time dt} \end{matrix} \right\} \qquad\qquad (1.27)$$
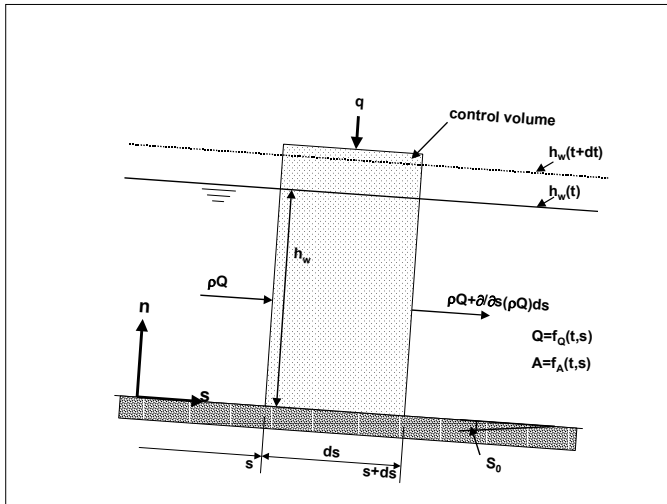
Figure 1.8:
Definition sketch for continuity equation

ρ = density of water
Q = discharge
S_0 = bed slope

Effective mass inflow   :   left plane(s)   : $\rho Q.dt$

right plane (s+ds): $-\left\{ \rho Q + \dfrac{\partial}{\partial s}(\rho Q)ds \right\}dt$

lateral inflow   : $\rho q\, ds\, dt$

$$\text{sum} \quad = \quad -\frac{\partial}{\partial s}(\rho Q)ds\, dt + \rho q\, ds\, dt$$

Increase of mass   :   mass at time t   : $\rho A\, ds$

mass at time $t + dt$: $\left(\rho A + \dfrac{\partial}{\partial t}(\rho A)\, dt \right)ds$

increase in dt   : $\dfrac{\partial}{\partial t}(\rho A)dt\, ds$

$\Rightarrow$ Total Balance: $-\dfrac{\partial}{\partial s}(\rho Q)ds\, dt + \rho q\, ds\, dt = \dfrac{\partial}{\partial t}(\rho A)dt\, ds$

Density is constant with time and distance equation reads: $\rightarrow \dfrac{\partial \rho}{\partial s} = \dfrac{\partial \rho}{\partial t} = 0$. After division by $\rho.ds.dt$ the continuity

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial s} - q = 0 \tag{1.28}$$

1.5.3   MOMENTUM EQUATION

Newton's second law reads;   (note the vector quantities $\vec{F}$, $\vec{u}$)

In words:

$$\sum \vec{F} + \left\{ \begin{array}{l} \text{inflow of momentum} \\ \text{into the control volume} \\ \text{per unit of time} \end{array} \right\} = \left\{ \begin{array}{l} \text{increase of momentum} \\ \text{inside the control volume} \\ \text{per unit of time} \end{array} \right\} \tag{1.29}$$

1.   Forces acting on mass of water in control volume

$\sum \vec{F}$ comprises: - pressure force
                                   - gravity force
                                   - bottom shear force
                                   - wind shear force
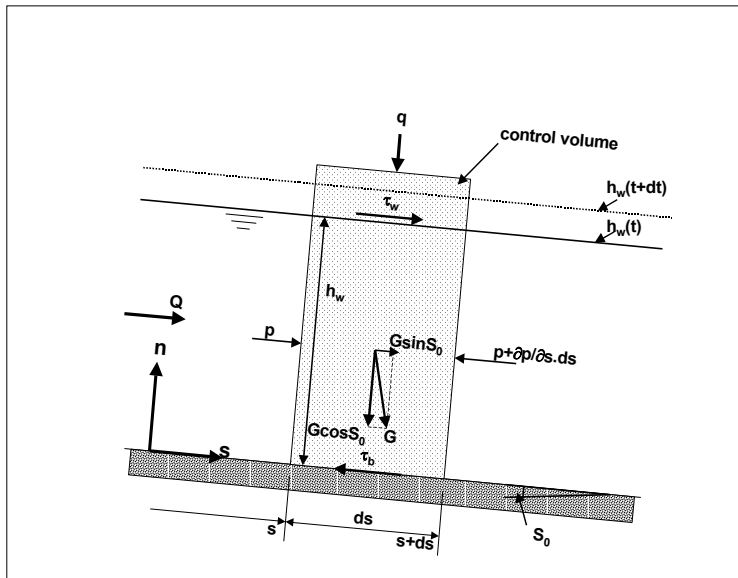
The control volume covers the cross-sectional area A_s $\Rightarrow$ momentum equation is set up for the area A_s

Figure 1.9:
Definition sketch for momentum equation

p = pressure
τ_w = wind shear stress
τ_b = bottom shear stress
G = weight of mass in control volume: $G = \rho g A_s ds$

- **pressure force** :   left plane : $\int_{A_s} p \, dA$

     right plane :$- \int_{A_s}\left(p + \dfrac{\partial p}{\partial s} ds \right) dA$

$\Rightarrow$ resultant pressure force is : $- \int_{A_s} \dfrac{\partial p}{\partial s} \, ds \, dA$

For long waves (no accelerations $\perp$ s-directions) $\partial p/\partial s$ is constant across the cross-section, so:

$$- \int_{A_s}\frac{\partial p}{\partial s} ds \, dA \;=\; - A_s \, ds \frac{\partial p}{\partial s}$$

- **gravity force**   : $\rho g A_s \, ds \sin S_0$

- **bottom/wall shear force** : $-\tau_b . P ds$, where P is wetted perimeter of cross-section A_s

- **wind shear force**   : $\tau_w B_s \, ds$, where B_s is the surface width of cross-section A_s

Summing up:

$$\sum \vec{F} = - A_s \;\; ds \;\frac{\partial p}{\partial s} + \rho g A_s \; ds.\sin S_0 \; - \; \tau_b P ds \; + \; \tau_w B_s \; ds \tag{1.30}$$

2.   Inflow of momentum in time interval dt and per unit of time

left plane   : $\rho Q \overline{u}\, dt$

right plane   : $-\left\{\rho Q\overline{u} + \dfrac{\partial}{\partial s}\left(\rho Q\overline{u}\right)ds\right\}dt$

lateral   : $\rho q\, ds\, u_\ell\, dt$ u_ℓ = velocity component of q in s − direction

_____

sum   : $-\dfrac{\partial}{\partial s}\left(\rho Q\overline{u}\right)ds\,dt + \rho q\,ds\,u_\ell\,dt$

$\Rightarrow$ per unit of time : $\left\{-\dfrac{\partial}{\partial s}\left(\rho Q\overline{u}\right) + \rho q u_\ell\right\}ds$ (1.31)

3.   Increase of momentum in control volume in dt and per unit of time

momentum at time t   : $\rho A_s\, ds.\overline{u}$

momentum at t + dt   : $\rho A_s\, ds.\overline{u} + \dfrac{\partial}{\partial t}\left(\rho A_s\, ds\,\overline{u}\right)dt$

_____

increase in dt   : $ds.\dfrac{\partial}{\partial t}\left(\rho A_s\,\overline{u}\right)dt$

$\Rightarrow$ per unit of time   : $ds.\dfrac{\partial}{\partial t}\left(\rho A_s\,\overline{u}\right)$ (1.32)

Substitution of (1.30), (1.31) and (1.32) in (1.29) and after division by ds gives:

$$\underbrace{-A_s\frac{\partial p}{\partial s} + \rho g A_s \sin S_0 - P.\tau_b + B_s\,\tau_w}_{\sum \overline{F}}\underbrace{-\frac{\partial}{\partial s}\left(\rho Q\overline{u}\right) + \rho q u_\ell}_{\text{inflow of momentum}} = \underbrace{\frac{\partial}{\partial t}\left(\rho A_s\,\overline{u}\right)}_{\text{increase of momentum}}$$

(a)   (b)   (c)   (d)   (e)   (f)   (g)

or after rearrangements and with ρ = constant and $A_s\,\overline{u} = Q$

(g)   (e)   (f)   (a)   (b)   (c)   (d)

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}\left(\frac{Q^2}{A_s}\right) - q u_\ell + \frac{A_s}{\rho}\frac{\partial p}{\partial s} - g A_s \sin S_0 + P.\frac{\tau_b}{\rho} - B_s\frac{\tau_w}{\rho} = 0$$

(1)   (2)   (3)   (4)   (5)   (6)   (7)

Equation (1.33) is further elaborated in the following.

Term (1) : **inertia**

Term (2) : **convection**

This term is non-linear. In its derivation use was made of $\bar{u}$ and the inflow of momentum via A_s was set equal to: $\rho \dfrac{Q^2}{A_s} = \rho . Q \cdot \bar{u} = \rho A_s (\bar{u})^2$

In fact this is not fully correct as the velocity varies across A_s and the inflow of momentum via A_s actually reads : $\rho \displaystyle\int_{A_s} u^2 \, dA$

So, the momentum correction coefficient α' becomes:

$$\alpha' = \frac{\rho \displaystyle\int_{A_s} u^2 \, dA}{\rho Q^2 / A_s} = \frac{A_s}{Q^2} \int_{A_s} u^2 \, dA \tag{1.34}$$

and the corrected convection term becomes:

$$\frac{\partial}{\partial s}\left(\alpha' \frac{Q^2}{A_s} \right) \tag{1.35}$$

Term (3): **lateral inflow of momentum**

Very often the lateral inflow is directed perpendicular to the main flow direction of $\bar{u}$; in these cases $u_l = 0$ and hence term (3) can be omitted: $qu_l = 0$ (1.36)

Term (4) : **pressure gradient**

Since straight flow lines are considered, the pressure distribution will be hydrostatic, so the component of the gravity force $\perp$ to the flowlines balances pressure, hence:

$P = \rho g(h_w - n)\cos S_0$ (n is normal to s)

and $\dfrac{\partial p}{\partial s} = \rho g \dfrac{\partial h_w}{\partial s} \cos S_0$,   hence : $\dfrac{A_s}{\rho}\dfrac{\partial p}{\partial s} = g A_s \dfrac{\partial h_w}{\partial s}.\cos S_0$ (1.37)

Term (5) : **gravity component paralled to flow**

Term (6) : **bottom shear stress**

Assuming that the condition for uniform flow can also be applied here locally so:

$\tau_b = \rho g R S_0$ and $\bar{u} = C \sqrt{R S_0} \Rightarrow \tau_b = \rho g \dfrac{\bar{u}^2}{C^2}$

where: C = Chezy coefficient for hydraulic roughness.

With $R = A_s / P$ Term (6) becomes:

$$P \frac{\tau_b}{\rho} = \rho g \frac{\bar{u}^2}{C^2}.\frac{P}{\rho} = g \frac{A_s \bar{u}^2}{C^2 R} = g A_s \frac{\bar{u}\,|\bar{u}|}{C^2 R} \tag{1.38}$$

Term (7) : **wind shear stress**

$$\tau_w = \rho_a \, \gamma \, C_r \, v_w^2 \, \cos \psi \tag{1.39}$$

where : v_w =   wind velocity

ρ_a =   air density $\approx$ 1.25 kg.m^{-3}

C_r =   friction coefficient ($C_r \approx$ 0.002 (normal) to 0.003 (wide channel))

γ =   shielding factor $0 < \gamma < 1$

Ψ =   angle between wind and channel

With (1.34) to (1.39) equation (1.33) becomes:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}\left(\alpha' \frac{Q^2}{A_s}\right) + g \, A_s \, \frac{\partial h_w}{\partial s}.\cos S_0 - g \, A_s \, \sin S_0 + g \, A_s \, \frac{\overline{u}|\overline{u}|}{C^2 R} - B_s \, \frac{\tau_w}{\rho} = 0 \tag{1.40}$$

Generally S_0 is very small, so $\cos S_0 \approx 1$. By replacing the (s,n) coordinates by the horizontal and vertical x – and y-axis and because S_0 is very small $\rightarrow \partial/\partial s \approx \partial/\partial x$ and $\sin S_0 \approx S_0 = -\partial y_b/\partial x$ where y_b is the bottom level relative to the reference level. The gradient terms become:

$$g A_s \, \frac{\partial h_w}{\partial x}.\cos S_0 - g A_s \, \sin S_0 = g A_s \left\{\frac{\partial h_w}{\partial x} + \frac{\partial y_b}{\partial x}\right\} = g A_s \, \frac{\partial h}{\partial x} \tag{1.41}$$

because : $h_w + y_b \approx h$ (h = level of water surface relative to reference level)

Furthermore in the momentum equation an **extra resistance term** is introduced to take into account local resistances, which are not covered by the terms discussed before. If the energy head loss per unit length is denoted by η_* then the extra resistance term is of the form:

$$g A_s \eta_* \tag{1.42}$$

With (1.41) and (1.42) substituted in (1.40) then the momentum equation reads:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha' \frac{Q^2}{A_s}\right) + g A_s \frac{\partial h}{\partial x} + g \frac{Q|Q|}{C^2 R A_s} - B_s \frac{\tau_w}{\rho} + g A_s \, \eta_* = 0 \tag{1.43a}$$

or with the hydraulic roughness expressed in Manning's n, which is related to C by: $C = R^{1/6}/n$, hence $C^2 R = R^{4/3}/n^2$:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha' \frac{Q^2}{A_s}\right) + g A_s \frac{\partial h}{\partial x} + g \frac{n^2 Q|Q|}{R^{4/3} A_s} - B_s \frac{\tau_w}{\rho} + g A_s \eta_* = 0 \tag{1.43b}$$

By putting $\alpha' = 1$, omitting windshear stress τ_w and the additional resistance term and replacing $\partial h/\partial x$ by $\partial h_w/\partial x - S_0$ then (1.43b) reduces to the momentum equation presented in the Design Manual, equation (2.34).

The continuity equation in the x-y co-ordinate system reads:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q = 0$$
(1.44)

1.5.4   NON UNIFORM FLOW IN CROSS-SECTION

Assume that the cross section is divided into (i = 1, 2,..n) sub sections with usually n = 2 or 3 (river with flood plain on one or both sides). Due to differences in depth and hydraulic roughness the flow will be non-uniform across the cross-section. A simple way to incorporate such conditions in the momentum equation is to adapt the term $C^2 R$ and α' as follows.

Continuity requires:

$$Q = \sum_{i=1}^{n} Q_i \Rightarrow Q^2 = \left(\sum_{i=1}^{n} Q_i \right)^2$$
(1.45)

Suppose the flow is steady and convection is small, then:

$$Q = C A_s \sqrt{R \frac{\partial h}{\partial x}}$$ slope of water level is constant   (1.46)

$$Q_i = C_i A_{si} \sqrt{R_i \frac{\partial h}{\partial x}}$$ across the cros ssection   (1.47)

Substitution of (1.46) and (1.47) in (1.45) gives:

$$C^2 A_s^2 R \frac{\partial h}{\partial x} = \left(\sum_{\tau=1}^{n} C_i A_{si} \sqrt{R_i} \right)^2 \frac{\partial h}{\partial x}$$

So the term $C^2 R$ can be replaced by:

$$C^2 R = \frac{1}{A_s^2} \left(\sum_{\tau=1}^{n} C_i A_{si} \sqrt{R_i} \right)^2 = \varepsilon$$
(1.48)

The momentum correction coefficient α' becomes:

$$\alpha' = \frac{A_s}{Q^2} \int \int_{A_s} \overline{u}^2 \, dA_s = \frac{A_s}{Q} \left(\sum_{i=1}^{n} \overline{u}_i^2 A_{si} \right)$$

So: $$\alpha' = \frac{A_s}{Q^2} \sum_{i=1}^{n} C_i^2 A_{si} R_i \frac{\partial h}{\partial x} = A_s \cdot \frac{\partial h/\partial x}{Q^2} \cdot \sum_{i=1}^{n} C_i^2 A_{si} R_i$$

From (1.46) it follows that: $$\frac{1}{Q^2} \frac{\partial h}{\partial x} = \frac{1}{C^2 R A_s^2} = \frac{1}{\varepsilon A_s^2}$$

So $$\alpha' = \frac{1}{\varepsilon A_s} \sum_{i=1}^{n} C_i^2 R_i A_{si}$$
(1.49)

1.5.5   MOMENTUM EQUATION EXPRESSED IN $\overline{U}$

The momentum equation can also be expressed in $\overline{u}$ with $\alpha' = 1$, $A_s = A$, $\tau_w = 0$ and $\eta_x = 0$:

Consider the acceleration terms $$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\frac{Q^2}{A_s} \right)$$

From (1.44) $$\frac{\partial Q}{\partial t} = \frac{\partial}{\partial t} \left(A_s \overline{u} \right) = A_s \frac{\partial \overline{u}}{\partial t} + \overline{u} \frac{\partial A_s}{\partial t}$$

$$\frac{\partial A_s}{\partial t} = \frac{\partial A}{\partial t} = -\frac{\partial Q}{\partial x} + q$$

So for the local and convective acceleration terms one gets:

$$\frac{\partial Q}{\partial t} = A_s \frac{\partial \bar{u}}{\partial t} + q\bar{u} - \bar{u} \frac{\partial Q}{\partial x}$$

$$+ \frac{\dfrac{\partial}{\partial x}\left(\dfrac{Q^2}{A_s}\right) = \dfrac{\partial}{\partial x}\left(Q\bar{u}\right) = Q \dfrac{\partial \bar{u}}{\partial x} + \bar{u}\dfrac{\partial Q}{\partial x}}{\dfrac{\partial Q}{\partial t} + \dfrac{\partial}{\partial x}\left(\dfrac{Q^2}{A_s}\right) = A_s \dfrac{\partial \bar{u}}{\partial t} + A_s \bar{u}\dfrac{\partial \bar{u}}{\partial x} + q\bar{u}}$$

Filling in (1.43a) gives

$$A_s \frac{\partial \bar{u}}{\partial t} + A_s \bar{u}\frac{\partial \bar{u}}{\partial x} + q\bar{u} + gA_s \frac{\partial h}{\partial x} + gA_s \frac{\bar{u}\left|\bar{u}\right|}{C^2 R} = 0 \qquad (1.50)$$

Omitting lateral inflow (q = 0) and division by gA_s gives:

$$\frac{1}{g}\frac{\partial \bar{u}}{\partial t} + \frac{\partial}{\partial x}\frac{\bar{u}^2}{2g} + \frac{\partial h}{\partial x} + \frac{\bar{u}\left|\bar{u}\right|}{C^2 R} = 0$$

For steady flow $\partial \bar{u}/\partial t$, then:

$$\frac{d}{dx}\left(\frac{\bar{u}^2}{2g} + h\right) = -\frac{\bar{u}\left|\bar{u}\right|}{C^2 R} \qquad (1.51)$$

Inside the brackets is given the energy height $H_e = \dfrac{\bar{u}^2}{2g} + h$

$$\frac{dH_e}{dx} = -\frac{\bar{u}\left|\bar{u}\right|}{C^2 R} \qquad (1.52)$$

Due to variation of the flow velocity in the cross-section the velocity head $\bar{u}^2/2g$ has to be multiplied by a coefficient α, which is energy correction factor, defined by:

$$\alpha = \frac{1}{A_s \bar{u}^3}\int_{A_s} u^3 dA \qquad (1.53)$$

The energy correction factor $\alpha > \alpha'$. Both coefficients are >1, but generally close to 1.

1.5.6   CRITICAL DEPTH

Consider the Bernoulli equation (with correction factor α):

$$H_e = h + \frac{\alpha \bar{u}^2}{2g} \qquad (1.54)$$

Since h = h_w + y_b

$$H_e = y_b + h_w + \frac{\alpha \bar{u}^2}{2g} \qquad (1.55)$$

and putting the reference level at the bottom $\rightarrow y_b = 0$, then with q = Q/m (discharge per unit of width) q = $\overline{u}h_w$, $\overline{u}$ can be eliminated from (1.55):

$$H_e = h_w + \frac{\alpha q^2}{2gh_w^2} \tag{1.56}$$

The minimum value for H_e for a fixed q follows from :

$$\frac{dH_e}{dh_w} = 1 - \frac{2\alpha q^2}{2gh_w^3} = 1 - \frac{\alpha q^2}{gh_w^3} = 0 \tag{1.57}$$

The depth for which H_e is minimum is called the critical depth h_c. From (1.57):

$$h_c = \sqrt[3]{\frac{\alpha q^2}{g}} \qquad \text{and also :} \quad \overline{u}_c = \sqrt{\frac{gh_c}{\alpha}} \tag{1.58}$$

Substitution of h_c in (1.56) gives:

$$H_{min} = \frac{3}{2}h_c \qquad \text{or} \qquad h_c = \frac{2}{3}H_{min} \tag{1.59}$$

The Froude number reads:

$$F_r = \overline{u}/\sqrt{gh_w}$$

At the critical depth for the Froude number it follows with (1.57):

$$F_r = \frac{\overline{u}_c}{\sqrt{gh_c}} = 1$$

The possibilities of H > H_{min} include:

$h_w > h_c \rightarrow$ subcritical flow $F_r < 1$

$h_w < h_c \rightarrow$ supercritical flow $F_r > 1$

1.6   BACKWATER CURVES

1.6.1   BÉLANGER EQUATION

From equation (1.51) we have:

$$\frac{d}{dx}\left(\frac{\overline{u}^2}{2g} + h\right) = -\frac{\overline{u}|\overline{u}|}{C^2 R}$$

Now backwater curves are developed for a channel with a constant bed slope S_0.

$$h = h_w + y_b \qquad \text{and} \qquad \frac{dy_b}{dx} = -S_0$$

So (1.51) can also be written as:

$$\overline{u}\frac{d\overline{u}}{dx} + g\frac{dh_w}{dx} = gS_0 - g\frac{\overline{u}^2}{C^2 h_w} \tag{1.60}$$

where R is replaced by h_w (hence a wide channel is assumed).

From the continuity equation one gets:

$$\frac{dq}{dx} = 0 = \frac{d}{dx}\left(\overline{u}\,h_w\right) = h_w\,\frac{d\overline{u}}{dx} + \overline{u}\,\frac{dh_w}{dx}$$

$$\Rightarrow \frac{d\overline{u}}{dx} = -\frac{\overline{u}}{h_w}\,\frac{dh_w}{dx}$$

Substitution of this expression in (1.60) results into:

$$\left(-\frac{\overline{u}^2}{h_w} + g\right)\frac{dh_w}{dx} = gS_0 - g\frac{\overline{u}^2}{C^2\,h_w}$$

$$\Rightarrow g\left(1 - \frac{\overline{u}^2}{gh_w}\right)\frac{dh_w}{dx} = gS_0\left(1 - \frac{\overline{u}^2}{C^2\,h_w\,S_0}\right)$$

$$\Rightarrow \frac{dh_w}{dx} = S_0\,\frac{1 - \dfrac{q^2}{C^2\,h_w^3\,S_0}}{1 - \dfrac{q^2}{gh_w^3}}$$

Multiplying nominator and denominator by h_w^3 it follows:

$$\frac{dh_w}{dx} = S_0\,\cdot\,\frac{h_w^3 - \dfrac{q^2}{C^2\,S_0}}{h_w^3 - \dfrac{q^2}{g}}$$

For $h_w^3 = q^2/C^2 S_0$, $dh_w/dx = 0$ then the flow is uniform and $h_w = h_n = $ normal depth

So:

$$\frac{dh_w}{dx} = S_0\,\cdot\,\frac{h_w^3 - h_n^3}{h_w^3 - h_c^3} \tag{1.61}$$

which is **the Bélanger equation.** The effect of non-uniform flow in the cross-section is accounted for by using for h_c the definition according to (1.58).

For given q, S_0 and k_s, h_n and h_c are known quantities. So (1.61) is an ordinary differential equation in h_w.

1.6.2   APPROXIMATE BACKWATER EQUATION

Consider the case of a wide prismatic channel with, $h_w \approx h_n$ so: $h_w = h_n + \Delta h_w$ with $\Delta h_w \ll h_n$ so: First order approach with Taylor series for (1.61) gives:

Simplified backwater equation:

$$\frac{dh_w}{dx} = \frac{d}{dx}(\Delta h_w) = \Delta h_w \cdot S_0 \cdot \frac{3 h_n^2}{h_n^3 - h_c^3} + 0(\Delta h_w^2) \qquad \text{where} \quad O(..) = \text{order of magnitude}$$

For slight positive slopes where $\Delta h_w > 0$ the positive x – axis is chosen in upstream direction. Then:

$$\frac{d(\Delta h_w)}{\Delta h_w} = -S_0 \frac{3 h_n^2}{h_n^3 - h_c^3} dx = \frac{-3 S_0}{h_n} dx \left(\frac{1}{1 - F_r^2}\right) \quad ; F_r^2 = \left(\frac{h_c}{h_n}\right)^3 \qquad (1.62)$$

- For $F_r^2 \ll 1$ then:

$$\ell n(\Delta h_w) = -3 \frac{S_0 x}{h_n} + \text{ Constant}$$

If the increase in water level relative to $h_n = \Delta h_{w0}$ at x = 0, it follows:

$$\Delta h_w = \Delta h_{w0} \exp\left(-3 \frac{S_0 x}{h_n}\right) = \Delta h_{w0} \exp\left(-\frac{x}{\lambda_b}\right) \quad \text{where } \lambda_b = \frac{1}{3}\frac{h_n}{S_0} \qquad (1.63)$$

Note at that at distance $L = 3\lambda_b$ from x = 0 only approximately 5% of Δh_{w0} is left as backwater.

- If F_r^2 is not very small relative to 1 then (1.62) gives:

$$\Delta h_w = \Delta h_{w0} \cdot \exp\left(\frac{-3 S_0 x}{h_n - h_c^3/h_n^2}\right) = \Delta h_{w0} \cdot \exp\left(\frac{-3 S_0 x}{h_n \left(1 - F_r^2\right)}\right) \qquad (1.64)$$

Reduction of the backwater with ½ Δh_{w0} can be computed from the previous equations by inserting exp (y) = 0.5 for y = -0.70. Hence according to (1.63):

$$-3 \frac{S_0 L_{0.5}}{h_n} = -0.7 \qquad \rightarrow \quad L_{0.5} = +0.23 \frac{h_n}{S_0} \qquad (1.65)$$

For according to (1.64)

$$\frac{-3 S_0 L_{0.5}}{h_n - h_c^3/h_n^2} = -0.7 \qquad \rightarrow \quad L_{0.5} = +0.23 \frac{h_n\left(1 - Fr^2\right)}{S_0} \qquad (1.65)$$

An analytical approach to solve the backwater equation is by using the Bresse functions, see Design Manual.

2   BUREAU OF INDIAN STANDARDS (BIS) CRITERIA FOR SELECTION OF VELOCITY-AREA METHOD MEASUREMENT SITE

The ideal requirements for a good gauging site are enunciated in IS 1192-1981 "Velocity - Area methods for measurement of flow in open channels" and are summarised below. The more general criteria apply both to current meter gauging and float measurements:

The accuracy of measurement is increased if the site is selected considering these aspects:

1. the channel shall be straight and its cross section well-defined ;

2. the conditions of flow do not change within the period of measurement ;

3. the observation verticals at all points are parallel to one another and at right angles to the measuring cross section ;

4. the measuring contours (iso-vels) are regular in the vertical and horizontal planes on which they are measured.

5. The site selected should comply, closely with the following essential requirements:

6. The reach of the open channel at the gauging site shall be straight and of uniform cross section and slope as far as possible, in order to avoid irregularities in velocity distribution. The length of the reach need not be more than 1600 m and should not be less than 400 m. When the length of the straight channel is restricted, it is recommended for current meter measurements that the straight length upstream of the measuring cross section should be twice that on the downstream.

7. Note: - In the case of artificial channels, the minimum length of straight reach should preferably be such as to give a drop of 0.06 m. or the minimum length should be equal to four times the width of the channel, whichever is larger.

8. The depth of water in the selected reach shall be sufficient to provide for the effective immersion of either the current meters or floats, whichever are to be used.

9. When near a confluence, the site, if located on a tributary shall be sufficiently upstream preferably beyond the backwater effect: and if located on the main stream, upstream or downstream of the confluence, it shall be beyond the disturbances due to the tributary.

10. The site should be easily accessible at all times of the year.

In addition to the requirements specified above, the following points shall be taken into consideration as desirable attributes in the selection of the gauging site:

1. The flow should be confined in a single channel and there should be no overflow as far as possible. Where this is not possible, the site in which the minimum number of channels exist and the floodplain has minimum width should be preferred.

2. Where these requirements cannot be set (for instance, when in all alluvial rivers the river bed is changing during the period of measurement, or when, under flood conditions, the river is not confined to a single channel in embankments), a gauging-site shall be chosen such that the bed change and or overflow is minimum. Floodplain, if can not be avoided, shall be minimum width, as smooth as possible, with a distinct channel, and clear of bushes and trees. The flow in the over bank or floodplain section(s) shall be measured separately and added, treating the whole as a composite section.

3. The site shall be remote from any bend or natural or artificial obstruction if disturbance of the flow is likely to be caused thereby.

4. The orientation of the reach should be such that the direction of flow is as closely as possible normal to that of the prevailing wind.

5. Sites at which there is tendency for the formation of a vortex or development of return flow shall be avoided.

6. The site should, as far as possible, be free from trees and obstruction, which may interfere with flow and clear vision during observation.

7. The site shall be free from aquatic growth, which is likely to interfere with the measurement of depth and the current meter reading.

8. The site shall be away from the backwater zone caused by any structure on the river

9. The site should be sufficiently away from the disturbance caused by rapids and falls, etc.

3   SHAFT ENCODERS

3.1   INTRODUCTION

The shaft encoder is a device which can convert the movement of the pulley wheel into a water level i.e. instead of a mechanical signal transmitted by gears from the pulley to the chart recorder pen arm, the rotation of the pulley wheel is monitored by the shaft encoder. The linear movements of a conventional float, pulley and counterweight sensor which responds to variations in the water surface level are converted into the angular movements of a shaft and are monitored by a "shaft angle transducer". The term is a general one since there are a number of devices, which can output a variable signal response to the rotation of a shaft. The output from shaft encoders can be recorded on electronic state data loggers and shown on a LCD display.

3.2   TYPES OF SHAFT ENCODERS

There are basically two general types of shaft encoder:

a) The absolute encoder, and
b) The incremental encoder.

The **absolute encoder** is able to signal exactly where its reference zero is located relative to the 360^0 field of rotation of its shaft, to a required level of resolution, which is defined in "Bits". An 8-bit encoder can resolve angular position to one part in 2^8 (256 decimal); a 10-bit encoder can resolve to one part in 2^{10} (1024 decimal). The shaft-mounted disk is of glass (or similar light-transmitting material), patterned into "tracks" that, according to a binary pattern that alternatively obscures and transmits light from one side of the disk to the other.

The **incremental encoder** is simpler and cheaper and is most commonly used in hydrometric applications, The incremental encoder signals the fact that shaft rotation is taking place as a train of logic pulses. Predominantly the incremental encoder is a revolution counter but which when linked to an appropriate counting mechanism can be readily converted into a rotational positioning device. Apart from cost the incremental encoder has the advantage over the absolute encoder that by its associated up/down counter it can track changing level multiple rotations of its shaft. This is particularly attractive for hydrometric applications where a high resolution is often required over a large level range.

There are also three fundamental construction technologies used in Shaft Encoder manufacture - of which one predominates in hydrometric applications, but with the other variants still being of relevance.

- The **Mechanical Encoder** rotates a disk on which conducting tracks are laid, and sectioned into Conducting/Non-Conducting patterns of binary significance. A series of conducting wipers makes intermittent electrical contact with the surface of this disk, one wiper per track, one track per Bit resolution.

    - The device is mechanically active but electrically inert until powered by an external electronic device, which can immediately "see" the contemporary binary pattern set up by the wipers and tracks, and "read" the binary number that indicates how the shaft is currently oriented with respect to its zero datum.

    - Such devices suffer the predictable limitations of electro-mechanical operation, potentially exacerbated in hydrometric applications by the operational characteristic of predominantly very slow shaft movement with long spells of no movement at all.

- Even though this type of encoder has the welcome characteristic of low energy requirements, the Mechanical Shaft Encoder did not lend itself to typical hydrometric circumstance and has not prevailed as a commonly-used device.

- The **Optical Encoder**, on the other hand, is essentially a non- contacting, and largely frictionless device. In this variant, the shaft-mounted disk is of glass (or similar light-transmitting material), patterned into "tracks" that, according to a binary pattern in the case of Absolute variants but, otherwise simply as a graticule of lines that alternately obscure and transmit light from one side of the disk to the other.

  - A light source located to one side of the disk is juxtaposed with light-detecting sensors on the other and, as the shaft and disk rotate, a train of logic pulses is generated that can be interpreted as "Logical Zeros" and "Logical Ones" by appropriately-designed sensing circuitry, as light is alternately revealed to and obscured from the detectors by the movement of the disk and its "transparent-alternating-with-opaque engraved graticule. Figure 3.1 illustrates the general principle.

Figure 3.1:
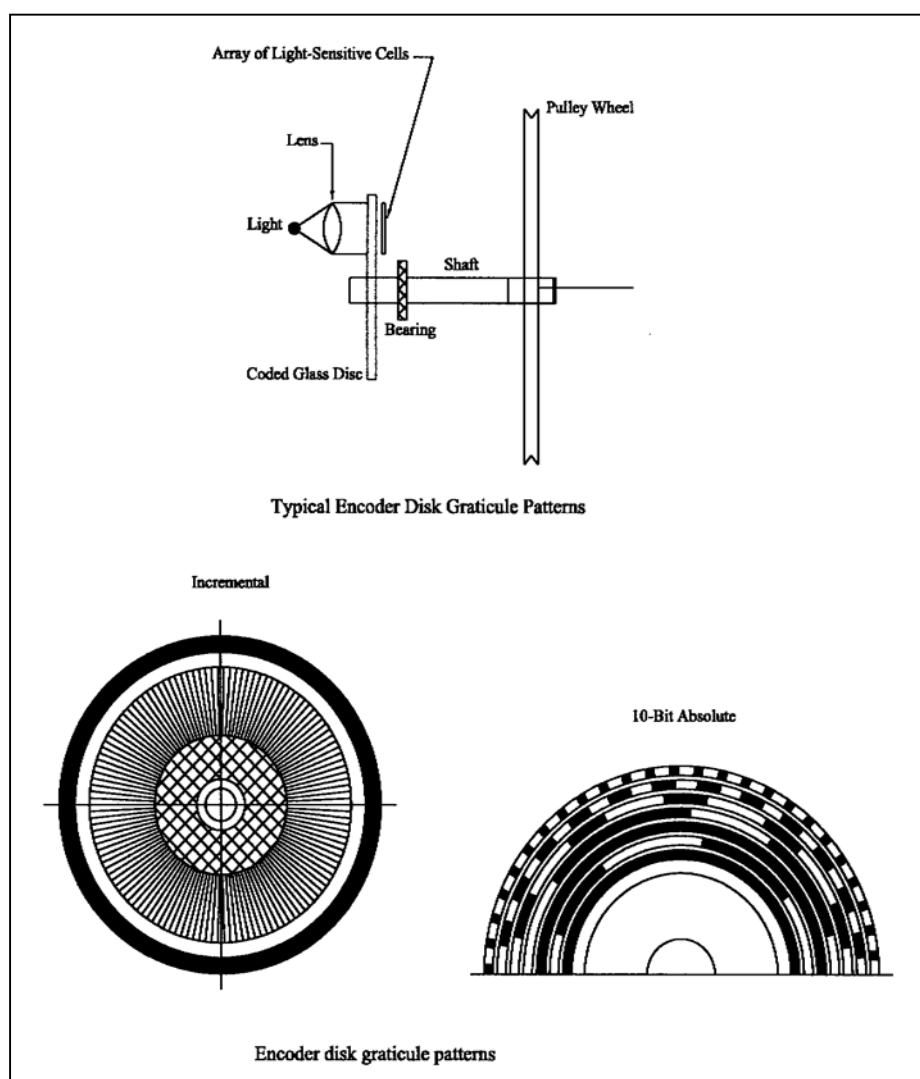*Basic features of the optical shaft encoder*

- The **Magnetically Switched Encoder** is a variant that uses a magnet or magnets located on the shaft-mounted disk to activate a series of reed switches, in sequence, as the shaft rotates. Linked to Up/Down Counting circuitry, and with a mechanism present whereby Direction of Rotation can be known, such devices have been known to perform with some success.

-   They suffer the predominant limitation of being purpose-built for hydrometry, thus directed at a relatively small market-place, as well as lacking in robustness - reed switches being relatively delicate devices, and multiple reed switch assemblies being particularly at risk in this regard.

The Optical Incremental Shaft Encoder is the type most widely used in hydrometric practice, though some specific proprietary data logging devices prefer to interface with **Optical Absolute Encoders,** for reasons of battery longevity.

The initial difficulty experienced with the Optical Shaft Encoder stemmed from its energy requirements. Early available variants used incandescent light sources, with heavy power requirements, and the introduction of the Light Emitting Diode (LED) as a more reliable substitute only relieved this shortcoming to a degree.

Flash Powering capability turned out to be the key factor in enabling the required marriage between Shaft Encoder-based water level movement detection and long-term battery-powered data logging and telemetry equipment. The concept of flash powering which is also applied to other water level sensors is described below:

-   LED illumination for 1 micro-second every millisecond of elapsed time, enables a 1,000-fold reduction in the energy requirements of this system element.

-   Given the relatively slow shaft-rotation rates associated with typical hydrometric applications, the fact that the Encoder's light source is dark for 99.9% of the time is rarely an insuperable operational difficulty - though attention does have to be paid to the potential for the occasional - perhaps unexpected - rapid water level change (or possibly a rapid float movement unrelated to water movement such as a release from a float snagging mechanism). If a change of Binary State occurs while the light source of an Incremental Encoder is dark, it will not be "seen" by the Up-Down Counting logic, and a level increment or decrement will be missed

-   Applications in which "missed Up/Down Counts" have been found to be problematic include Tidal Measurement, where inadequate stilling well damping has permitted relatively high frequency water level oscillations that have defeated the counting process.

Up/Down Counting is less of a problem if an Absolute Encoder is used, but cost escalates as a consequence.

In hydrometry, the typical operational Shaft Encoder is the Optical Incremental variant, float-driven by way of a float-and-counterweight assembly and a pulley wheel often of 400mm circumference. This combination delivers a system capability to register a 1mm change in water level, using an encoder that delivers 100 logic changes per 360° rotation of its shaft.

-   It does this through the mechanism of the **Quadrature Concept,** illustrated by Figure 3.2.

    -   Two Light-Detecting Components are deployed in the encoder's body that "see" the same On-to-Off or Off-to-On change created by shaft movement off-set by a difference in the Phase Angle of the related electrical wave form.

    -   This allows, firstly, direction of rotation to be resolved according to which light detector "sees" a change from light to dark or *vice versa* first - the two detectors do not "see" the change in illumination simultaneously but, always, one before the other.

    -   It allows, secondly, four combinations of light state to be detectable in respect of each of the 100 disc-graticule light transitions that are physically possible viz.

        Output 1 in Logical State 0   Output 2 in Logical State 0
        Output 1 in Logical State 1   Output 2 in Logical State 0
        Output 1 in Logical State 1   Output 2 in Logical State 1
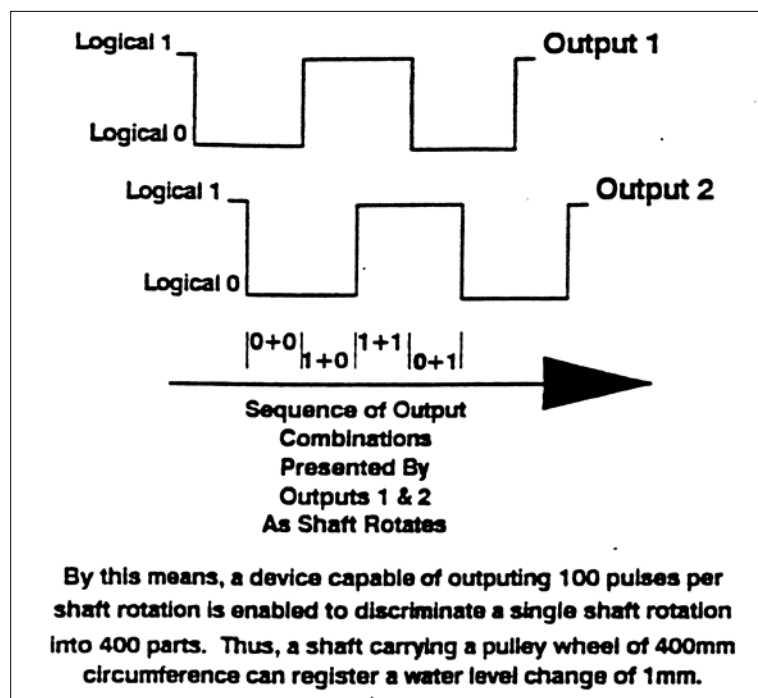   Output 1 in Logical State 0   Output 2 in Logical State 1

Figure 3.2:
Shaft encoders, illustration of
principle of quadrature
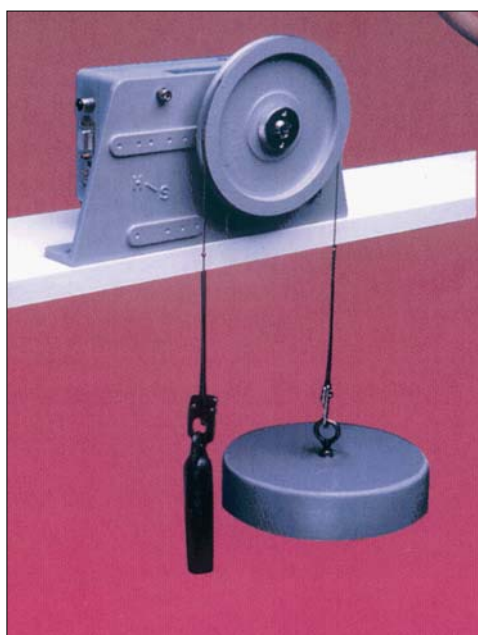
a) free-standing   b) parasitic

Figure 3.3:   Examples of shaft encoders

- The shaft encoder illustrated, in association with an appropriately-designed Flash-Powering and Up/Down Counting Electronic Module is capable of counting 400 "changes of state" in respect of each single 360° shaft rotation. Hence, a 400mm-circumference Drive Pulley can deliver 1mm record resolution.

- With other drive pulley circumferences, other record resolutions are possible, to accommodate, for example, extended water level range excursions at reduced resolution.

Sometimes shaft encoders are attached (parasitic) to the pulley shaft of a conventional chart recorder (AWLR), this means that a chart trace and a digital output can be obtained from the same float system. Figure 3.3 shows an example of both a free-standing (stand-alone) and parasitic shaft encoder.

3.3   ADVANTAGES AND DISADVANTAGES

The pro's and con's of shaft encoders relative to the float based autographic recorders are:

Advantages - they can provide as good an accuracy as the primary sensor i.e. the float system i.e. they do not have the errors associated with chart interpretation. For a well constructed float and pulley system they should be able to measure to +/- 3 mm. Shaft encoders can be added to some normal float and drum type water level recorders and as such, a back up chart can also be used in case of logger failure.

Limitations - Because they are operated by means of a float system they require the construction of a stilling well. Also, they can be subject to errors similar to those experienced with conventional float and chart recorders such as problems caused by submergence of the counterweight and line, displacement of the wires on the float or counterweight pulleys and kinks in the float suspension cables. Reference level adjustment is required when setting up i.e. the shaft encoder has to be set relative to staff gauge zero, which may be lost during operational use. Accidental fast rotations of the pulley wheel cannot be tracked.

4   GAS PURGE (BUBBLER) SYSTEMS

4.1   INTRODUCTION

This is basically an electro-mechanical technique. In simple terms, equipment of this type measures how hard it has to work to push out a stream of gas (often nitrogen) bubbles from a submerged orifice against the mass of water that overlies that orifice - mass being essentially a function of water depth in this instance (though also a function of the water's Specific Gravity - so water density changes can be relevant). As with a number of other types of water level/depth sensing equipment, the basic attraction is absence of need to construct a stilling well and, in days before modern designs of pressure transducer and other electronics-based devices became available, the "Bubbler Gauge" was widely used in river and tidal hydrometry as a "device of first choice" where a stilling well could not be economically constructed (or was undesirable for other reasons).

4.2   BASIS OF OPERATION

Air or an inert gas (often nitrogen) is allowed to bleed into a pipe, whose open end is submerged at the required location in the river, at a controlled rate and pressure (See Figure 4.1). The gauge measures the pressure of gas that displaces the liquid in the pipe; this is directly proportional to the head of the liquid above the mouth of the pipe.
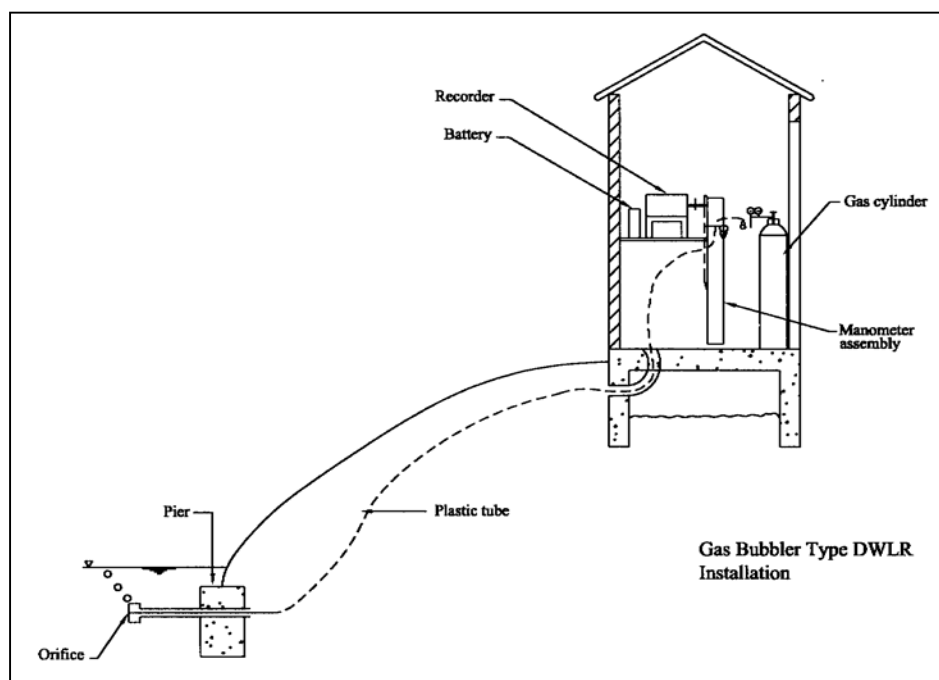
Figure 4.1: Sketch of a typical "bubbler" system installation

(Source: Ott advertising material)

The reaction pressure has traditionally been sensed by a mechanical balancing mechanism (See Figure 4.2) or by a mercury servo-manometer. However, these two equivalent system components for sensing reaction pressure apart, the devices are similarly designed so far as major modules are concerned viz.:

a)   A source of compressed gas - normally bottled "dry" nitrogen, in river applications, though some manufacturers' variants operate with built-in air compressors; "dry" nitrogen is normally used in preference to air to reduce the risk of moisture condensing in the bubble tube (see below).

1. Power supply
2. Servo amplifier
3. Strip-chart recorder
   or drum recorder
4. Sliding weight
5. Diaphragm bellows
6. Potentiometer or
   shaft encoder
10. Bubble sight glass
11. Dosing device
12. Measuring tube
13. Damping
14. Servomotor
15. Control counter
16. Nitrogen bottle
17. Pressure reducer
18. Safety valve
19. Quick-action coupling
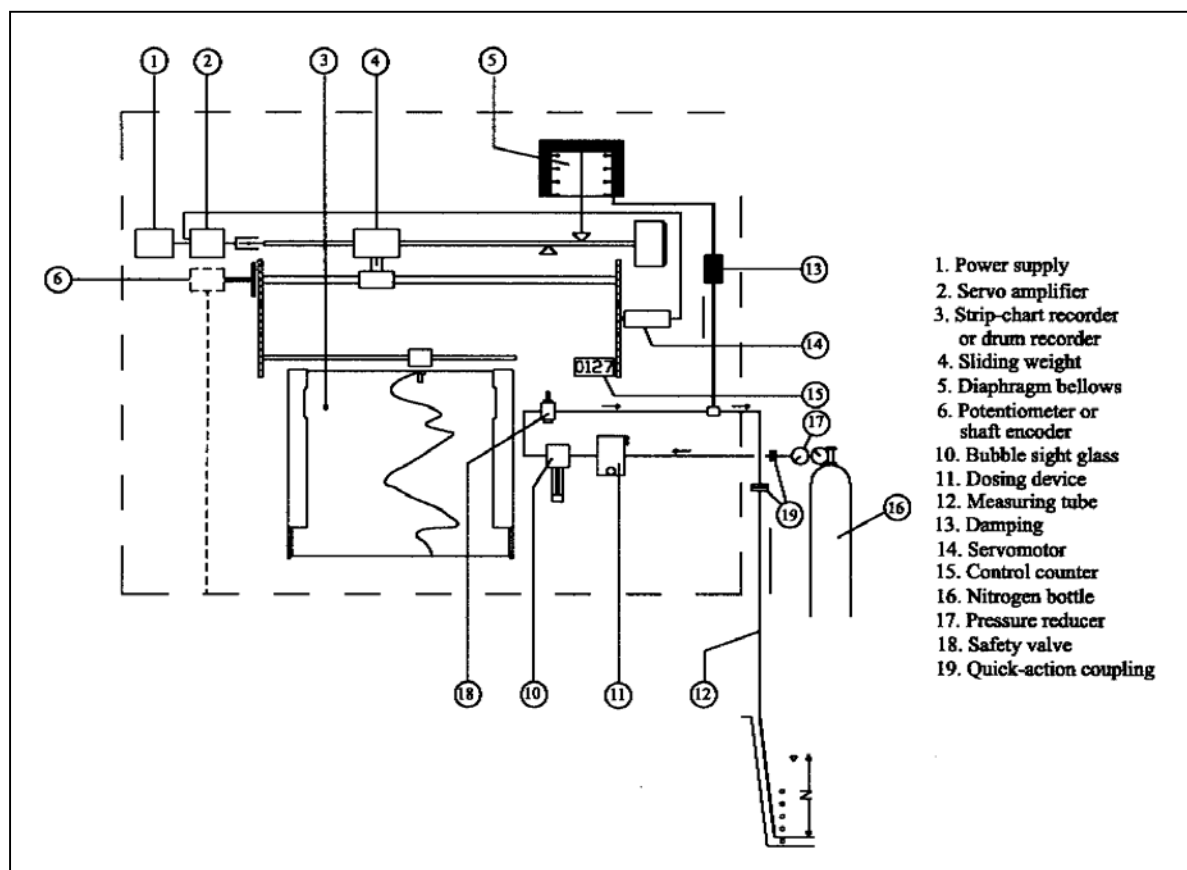
(Source: Ott)

Figure 4.2:   Schematic diagram of balance beam type of bubbler system

b) A pressure controller, whereby the rate at which gas bubbles are released into the water can be adjusted to deliver a sensible balance between device sensitivity and gas economy. At a constant rate of gas flow, the pressure in the system will rise at the same rate as the head of water above the orifice i.e. depending on the height of water a gas pressure is produced in the measuring tube of a magnitude equal to the hydrostatic pressure. At a constant density there is a linear relationship between the required height of water level and the pressure of air in the measuring tube.

c) A delivery tube, whose orifice is located below the lowest water level that is to be sensed, and the level of which becomes the effective datum for subsequent water depth measurement; this tube may extend to many metres length, but requires to be laid with some care (and protected thereafter) to ensure no depressions within which condensing moisture (which is a virtually inevitable occurrence sooner or later) can accumulate. Relevant to this component:-

- Clearly, the tube orifice needs to be secured adequately, so as to provide a stable reference datum.

- The proper placement of the orifice is essential. It is preferable to set the orifice at right angles to the flow and even better if it can be fitted flush to the wall of the mounting structure.

- Potential problems include trapped moisture, and outlet obstruction (by silt or debris deposition, or by algae growth).

- Purging of the tube by releasing gas under full pressure may, from time to time, be necessary, in order to clear any blockages.

d) A balancing mechanism, whereby the readjustment of the equipment to pressure equilibrium from states of imbalance caused by water level movements, can be utilised to drive autographic recorders and Shaft Encoders.

- Figure 4.2 (already referenced) shows a schematic of a Balance Beam Variant, and illustrates to a degree the relative complexity of the device and, hence, perhaps one of its less operationally-appealing features.

- The servo-manometer variant is similarly complex to the balance beam variant but operating its balancing mechanism in a vertical orientation, and suffering the additional disadvantage of including a mercury-filled component as an intrinsic part of the pressure balancing mechanism.

e) A small electric motor, that is activated by sensed imbalance in the balance beam or the manometer, and which rotates a lead screw (horizontally deployed in the beam balance, a sloping deployment in the manometer) to re-position a sliding weight for the beam balance and one end of a manometer tube for the servo-manometer version such that pressure equilibrium is re-established.

f) The lead screw referred to above that, as well as being the mechanism whereby pressure equilibrium is restored, also provides the rotational force for operating attached recording or signalling devices.

More modern devices of this class are able to make use of extremely sensitive, yet stable, pressure transducers - resident within the instrument housing itself - to sense pressure instability, and micro-processor (rather than electro-mechanical) logic to effect readjustment. For example one well-known manufacturer of hydrometric equipment uses an oil free ceramic cell to measure hydrostatic pressure of the water column and convert it into an electrical signal. These electrical signals are led via a controlling element to a high-precision stepping motor, which drives the reversing spindle with recording device for the record of water level. For the compensation of wave motions and short-time fluctuations, the reference cell has a superposed damping, which can be adopted to the conditions of the measuring site. These new bubbler systems do away with the need for the more complex traditional beam balance or mercury servomanometer type of devices. The latter in particular is no longer popular due to its use of mercury.

It is possible to convert the output from a "bubbler" type sensor into digital format for recording on a data logger.

4.3 ADVANTAGES AND DISADVANTAGES OF AIR REACTION (BUBBLER) SYSTEMS

Advantages and disadvantages of air reaction (bubbler) systems are:

Advantages - These gauges are particularly suited to measuring in water with a lot of suspended solids. Also, they do not require the provision of a stilling well. If properly set up and maintained the bubbler should be capable of measuring to at least within +/- 10 mm, and possibly to +/- 5 mm.

Limitations - the need to be able to replace gas cylinders, it is a fairly complex device needing skilled maintenance, can be affected by changes in density of the water column.

Some interest has been expressed in using the "bubbler" technology for certain applications in India. However, it is recommended that the use of pressure transducers (See Chapter 6 of Volume 4, Design Manual, Hydrometry) be given first consideration.

4.4 STATION DESIGN, CONSTRUCTION AND INSTALLATION ASPECTS

The advantage of gas purge or bubbler type of water level sensor is the fact that no stilling well is required. Bubbler systems can be located up to 100 m from the river being measured provided they are properly installed. However, at least one manufacturer quotes a maximum transmitting distance of 300 m. The following general guidelines should be adopted whenever possible:

a) The bubbler system consists of a regulated gas supply, usually nitrogen, a delivery tube which delivers the gas to the river and a pressure measuring system. It is essential that there are no significant leaks in the gas purge system.

b) The sensing orifice or dip tube is located in such a position that the head of water above it represents the stage to be measured. It should be positioned where it is unlikely to be silted over or where there is unlikely to be due turbulence or strong current action. The orifice should be orientated at right angles to the direction of flow.

c) From the source of gas supply the tube should be given a continuous down slope to the measurement point.

d) The delivery tube should be installed in a small diameter pipe to protect it from damage and buried below the ground down the side of the river bank. The protection pipe should be anchored to concrete foundation blocks. The gauge post foundation blocks could be used for this purpose. The protection pipe should be installed in a concrete foundation block below the minimum water level with an opening to allow the outlet of the sensing orifice. The sensing orifice should be installed at least 0.15 m below the minimum water level.

e) In unstable stream beds it is sometimes advantageous to place the bubble orifice in a vented well point driven into the unstable bed. The vent pipe should be extended back to the instrument shelter or above the highest water level.

f) There are several different types and variations of the bubbler type of AWLR. Therefore the manufacturers installation specifications should be followed and supersede these guidelines where differences occur.

g) A weatherproof instrument shelter of minimum internal dimensions of 1.2 x 1.2 x 2.2 m is normally needed to accommodate the equipment required for the bubbler type of AWLR.

h) An adequate supply of gas is required at all times. Therefore, this type of device should not be installed where in areas where it is not possible to obtain reliable refills of nitrogen cylinders. A spare cylinder should be kept at the site at all times to avoid loss of data.

i) A general, but not site specific specification for a bubbler (pneumatic) type of water level sensor is contained in the Volume: Equipment Specification, Surface Water'.

5   ULTRASONIC - IN AIR (LOOK DOWN) WATER LEVEL SENSORS

5.1   INTRODUCTION

This technology senses the position of a water surface relative to a known datum by measuring the time taken for a pulse of ultrasound to travel from the sensor itself (which is the effective datum) to the water surface and back again to the sensor (see Figure 5.1).   Its principle claimed operational attributes are that it is non-intrusive and non-contacting. A stilling well is not required for successful operation. Because such devices do not have to be deployed either in or on the water, installation can be straightforward and, hence, cost effective relative to alternative technologies. A typical ultrasonic - in air installation is illustrated in Figure 5.2.
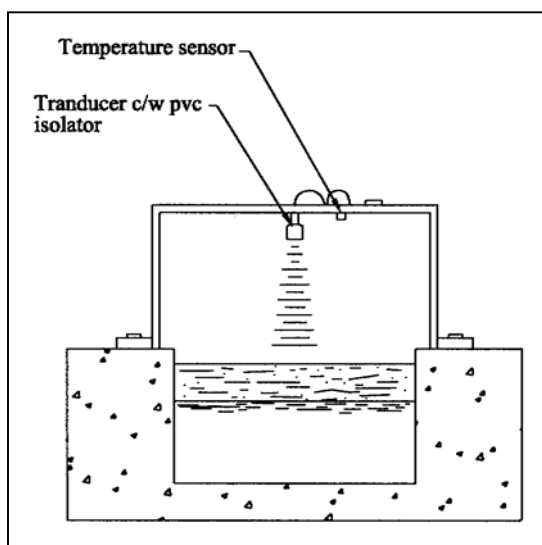

Figure 5.1:
Ultrasonic - in air (look down) water level sensor - General Operating Principle

Figure 5.2:   Typical Ultrasonic - in air (look down) water level sensor

5.2   PRINCIPLES OF OPERATION

The basis of the technique is that, since the speed of sound in air is known (331.45 m/s at 0°C), "there-and-back" ("round trip") travel time can be equated readily with "there-and-back" distance. A constraint is that the speed of sound in air varies with density. Hence temperature and humidity effect the accuracy of the method. Corrections can be made for temperature changes. Therefore, in order to make allowance for temperature changes (1.7mm/^{o}C/m change; warmer = faster), **temperature compensation** provision within the equipment design is required. Nearly all modern devices of this type do, either by direct temperature sensing or by "on-line calibration" in the form of a built-in reflecting target located at known distance from the transducer face.

Most (though by no means all) modern devices of this type claim sensing accuracy of the order of 1mm per metre range, with operation up to 3 m range in typical open air water level sensing applications. However, they have been used successfully over larger operating ranges (10 - 20 m).

The technique was developed originally for industrial applications and has been primarily used in environments with relatively small level ranges such as effluent discharge channels. Nevertheless, in principle, use of the technique could be considered for river-related applications in India and has been demonstrated to work effectively in similar physical conditions. However, there are several operational limitations/drawbacks of the technique, which the user should be aware of:

a)   Temperature compensation

Effective temperature compensation may be problematic when there is a large air gap to be spanned between sensor and water surface. Also, such other variables as air humidity and wind effect can come into play - to the potential detriment of either measurement accuracy, reliability, or both. Wind can result in loss of echo reception, as well as being one possible cause of a varying temperature profile in the path of the ultrasonic beam. Conversely in some situations wind can be a possible advantage since it can destroy temperature stratification.

b)   Proportional Error Effect

The sensor measures the air distance between itself and target (the water surface). It then goes on to infer the variable of (normally) true interest, stage by subtracting the measured variable (air distance) from a known, previously-surveyed constant - the distance between sensor and the stage board zero. The outcome of this process, in terms of the Absolute Accuracy of Depth Determination, will vary quite significantly depending upon the ratio of air distance to water depth. With system accuracy expressed (as it normally is) as a percentage of Range (which is often called "Span" in related literature), and a range of water level to be accommodated that extends to (say) 6 metres, a device that is accurate to 0.1% of range will deliver a determination of water level location that is within +/- 6 mm. If, at a given instant, the water body in question is 5 metres deep, that depth will be inferred plus or minus 6 mm - an absolute accuracy of 0.12%. If, on the other hand, the true water depth is 0.5 metres, that value will be assessed still to an accuracy of plus or minus 6mm - an absolute accuracy now of 1.2%. More exaggerated water level ranges will result in more exaggerated accuracy relationships. Therefore, **higher water levels can be measured more accurately than lower water levels**. However, there is minimum distance within which water level cannot be sensed at all (See para. (c) below).

c)   Dead Band Effect

There is a minimum distance between ultrasonic sensor and water surface within which the water level cannot be sensed at all. This is usually known as the Dead Band. This is because, when the sensor is energised and caused to transmit a pulse of ultra-sound, it does this by vibrating and, even after electrical excitation has ceased, the sensor continues to vibrate for a finite period. Only an echo from the water surface that returns after the sensor has ceased to vibrate (or "ring"), and which itself has the effect of causing the sensor to vibrate and generate an electrical signal as a

consequence, will be unambiguously distinguishable. Because of the Dead Band Effect, there is a system design compromise to be addressed where wide range excursion is to be accommodated. For any given claimed system measurement accuracy (1%; 0.5%; 0.1%, for example), best absolute accuracy of shallow depth measurement will be achieved by reducing the range over which the instrument is to operate. Reducing the range brings the sensor closer to the water surface and, inevitably, increases the risk of (possibly critical) data loss if the water level naturally encroaches within the Dead Band.

d)   Propagation Effects

The pulse of ultrasound that is generated by the Air Ranging device propagates as a cone which, at distance from its source may be wide enough to intersect channel banks or bridge abutments when water level is low - while performing acceptably when water level is high. A number of detrimental effects may result viz.:

- Spurious direct reflections, i.e. the sensor may range the obstacle rather than the water surface.
- Spurious indirect reflections (arriving late at the sensor head via a longer route than the most direct).
- On some sensor types, heavy rainfall may result in false echoes, and as a consequence give false echoes.

Also, there may be:

- energy dissipation to the extent that return signal detection is rendered uncertain.
- the way that surface water waves (e.g. ripples, wind waves, standing waves) are interpreted by the sensor will vary with range from the sensor to the water surface i.e. will the signal be reflected from peaks or troughs or an average of the two.

Ultrasonic level recorders have traditionally required mains power supplies owing to a higher power demand than other sensors. However, power requirements have been significantly reduced and the sensor is now a viable alternative at remote sites.

5.3   ADVANTAGES AND LIMITATIONS OF ULTRASONIC - IN AIR (LOOK DOWN)

The advantages and limitations of Ultrasonic - in air (look down) are:

Advantages - the sensor can be mounted above the water surface, say from an extender bar (a bracket cantilevered outwards) attached to a bridge pier, they are therefore easy to install and maintain and are ideally suited for interfacing with data logging systems.

Limitations - it is difficult to aim the acoustic beam tightly and thus transducer heads cannot be mounted flush with the side of the channels but need to be fixed on an extender somewhat. The compensation temperature sensor will only measure temperature at one place. Temperature gradients over the length of the ultrasonic beam will give rise to errors. Absolute errors will tend to increase the greater the distance from the sensor to the water surface.

5.4   STATION DESIGN, CONSTRUCTION AND INSTALLATION ASPECTS

The advantage of the look down ultrasonic water level sensor is the fact that it can be fixed above the water surface and there is therefore no need to install a stilling well or protection pipe or fix it to a wall or bridge pier. However, it is arguably not as accurate as the pressure sensor or float recorder. The main problem is the dependence of the speed of sound on air temperature. Therefore the more

reliable devices require good air temperature monitoring and compensation facilities. The inaccuracy as a percentage reading increases dramatically as water levels fall i.e. the measuring distance becomes longer. Despite these limitations it is believed that the look down ultrasonic could have a role to play in monitoring water levels at some difficult sites. Therefore, the following installation guidelines are suggested:

1. A method of installation could be to fix the sensor on a bracket attached to a horizontal, rigid fixing bar. This should be of sufficient length to position the sensor above the water away from the influence of the wall, bridge pier or other fixing structure. i.e. the acoustic signals should only be reflected from the water surface and not from surrounding objects. The length of the bar will depend on the maximum distance to be measured and the spread of the acoustic beam. It is envisaged that a length of 1 m out from the wall will generally be sufficient. Therefore, a fixing bar no longer than 2 m would be required. However, the manufacturer's advice should be obtained on this issue prior to installation. The non-sensor end of the bar should be securely bolted to the fixing structure.

2. The signal cable could be run back from the sensor to an instrument kiosk at a suitable location. The instrument/recorder kiosk design could be similar to that referred to in Volume 4, Design Manual, Hydrometry, Chapter 8 on pressure sensors. Alternatively, for a tower type installation the recorder box could be fixed on top of a platform. If the site is prone to human or natural interference then the signal cable should be installed in a protective duct. A sketch of a possible tower mounted arrangement is shown in Figure 5.3.

3. Some look down ultrasonic devices have calibration bars (see Figure 5.3). This is a means of self-calibration. Sound signals are reflected off the bar, which is a known distance from the sensor. The speed of sound for the environmental conditions prevailing at the time can then be estimated. This is then used by the instrument to estimate the distance from the water surface. When such calibration bars are part of an instrument set-up they should be installed in accordance with the manufacturer's instructions.

4. A general specification for this type of sensor is contained to in Equipment Specification, Surface Water.
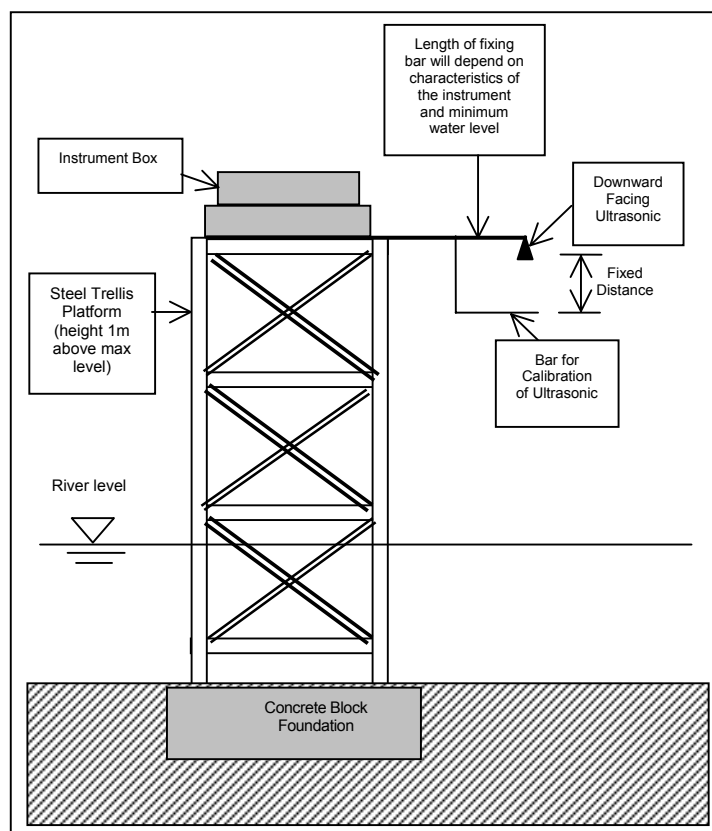
Figure 5.3:
Possible arrangement for a tower mounted look down in-air ultrasonic water level sensor

6   FLOW MEASURING STRUCTURES

6.1   INTRODUCTION

Flow measurement structures are often installed in smaller rivers (< 100 m wide) in order to stabilise the stage-discharge relationship and in some cases make it more sensitive. In view of the large size of most Indian rivers and the high capital cost of installing purpose built structures it is believed that their use should be limited to important, smaller river basins such as some of the western flowing systems and measurement of flows in irrigation systems for water balance studies.

Artificial controls include broad crested, triangular profile and thin plate (notches) weirs and flumes. In addition some types of spillways and other control structures can be used for flow measuring purposes. The most commonly used artificial controls have been the subject of intensive research and if they are correctly installed it is possible to estimate the stage - discharge relationship in accordance with appropriate theoretical formulae. This obviates the need to undertake frequent current meter gauging. However, it is important that the theoretical stage - discharge relationship is checked by current meter over the full range of flow.

Structures will only work in accordance with the theory of a control if they are operating in the **modular** flow range i.e. there is a change from sub-critical to super critical flow, through or over the structure. When the downstream water rises relative to the upstream level and starts to affect the upstream levels this ceases to occur and the flow is said to become **non-modular**. In such circumstances the theoretical formulae will overestimate the flow and it is necessary to apply an adjustment (reduction) factor. It is therefore very important to estimate the modular limit when designing flow measurement structures.

Structures create obstructions in the river channel. It is therefore important when designing a gauging structure to assess the impact on upstream water levels by undertaking open channel hydraulic backwater analysis.

At low flows, on wider rivers structures might not be very sensitive. In order to get round this sensitivity problem, compound structures are built with different crest levels or special derivatives of the basic structure are installed.

There are a variety of different flow measurement structures. The choice of structure will depend on a variety of factors including measurement objectives, flow range, afflux, size and nature of the channel, channel slope and sediment load, operation and maintenance, passage of fish and not least, cost. The applications and limitations of a structure will determine where its use is most appropriate. In this regard each type of structure has its own specific site selection criteria. General site selection criteria for structures are referred to in Volume 4, Design Manual, Hydrometry.

In order to be able to understand the basic principles of controls and in particular structures, it is necessary to have a basic understanding of simple open channel hydraulics.

6.2   BASIC PRINCIPLES

The hydraulic gauging structure can be considered as a natural development of the rated section to achieve a formal, stable and quantifiable hydraulic control with a rating relationship of the form:

$Q = f(h_u)$

Where: Q   =   gauged flow, and
       h_u =   upstream head or stage (static, not total head)

A relationship that is invariant with time

(i.e. $Q \neq f(h_u, t)$)   where t = time

and (as far as possible) invariant with downstream water level

(i.e. $Q \neq fn(h_u, h_d)$)   where h_d = downstream head

The stage - discharge relationship for most horizontal weir flow measurement structures takes the following form:

$$Q = C_d(\sqrt{g})bH^{\frac{3}{2}}$$

Where: Q =   discharge
        C_d =   discharge coefficient unique to the structure
        g =   acceleration due to gravity
        b =   width of structure (weir crest width or flume throat width)
        H =   total upstream head, where $H = h + \dfrac{v^2}{2g}$
        h =   static head (stage relative to weir crest)
        v =   velocity of approach to weir at stage monitoring section

The discharge coefficient is dependent on the geometry of the structure, the approach conditions and for some types of structure varies with head. For the determination of the discharge coefficients to be used for different standards reference should be made to the appropriate Indian and/or International Standard.

For triangular shaped weirs such as V notches and flat-V notches the formulae are similar except that the exponent (power) is 5/2 as compared to 3/2 for the horizontal weir.

6.3   MODULAR AND NON-MODULAR FLOW

The concepts of modular and non-modular flow have been briefly introduced in Section 6.1 above. These concepts are of critical importance in gauging station design. If the gauging station is considered as a hydraulic channel control, non-modular flow occurs as the gauging station starts to "drown out" (starts to become submerged) and the desirable $Q = f(h_u)$ relationship is superseded by the less desirable $Q = f(h_u, h_d)$ relationship as Q increases beyond the modular limit. The transition to non-modularity is not an abrupt one and takes place over a range of flow. Full (100%) non-modularity is achieved when the weir becomes completely drowned out i.e. when there is negligible difference between upstream and downstream water levels. Percentage non-modularity is expressed as $100\, h_d / h_u$ ($h_d \geq 0$) with the lowest point on the weir crest taken as datum. With a modern, hydraulically efficient weir design the modular flow rating persists with acceptable accuracy up to about 75 -80% non-modularity (the "modular limit") before the non-modular rating begins to take over.

The continuation of the rating relationship into the non-modular region requires the measurement and recording of both upstream and downstream water levels. This involves a doubling of data collection, processing and storage effort, but with the advent of automatic electronic data processing techniques this is not as much of an operating problem as it once was. Nevertheless, non-modularity introduces an unwelcome complexity into the rating relationship and an increased opportunity for error and inaccuracy. Maximisation of the modular range should therefore be a prime consideration of gauging station design.

References to weir crest tappings will be found in some textbooks and standards as an alternative to downstream water level measurement. The crest tapping was developed for use with triangular profile weirs.

6.4   ATTRIBUTES OF A GOOD ARTIFICIAL CONTROL

WMO (1980) list the following attributes as being desirable in an artificial control:

a) The control should have structural stability and should be permanent. The possibility of excessive seepage under and around the control should be considered, and the necessary precautions should be taken for its prevention by means of sheet-piling or concrete cut-off walls and adequate abutments.

b) The crest of the control should be as high as practical to eliminate if possible, the effects of downstream conditions or to limit those effects to high stages only.

c) The profile of the crest of the control should be designed so that a small change in discharge at low stages will cause a measurable change in stage, the profile of the crest should be designed to give a stage-discharge relationship of such shape that it can be extrapolated to peak stages without any error.

d) The shape of the control structure should be such that the passage of water creates no undesirable disturbances in the channel upstream or downstream of the control.

e) If the stream carries heavy sediment load, the artificial control should be designed to be self-cleaning. Flumes and triangular profile weirs have this attribute. Broad crested weirs can often be made self cleaning by a design modification in which the vertical upstream face of the weir is replaced by an upstream face that slopes gently from the apron to the weir crest.

6.5   TYPES OF PURPOSE-BUILT FLOW MEASUREMENT STRUCTURES

There are a large variety of different types of flow measurement structures in use throughout the world. Some of the more commonly used structures are summarised as follows:

a) Thin plate weirs:
   - rectangular
   - V-notch
b) Broad crested weirs:
   - round nose
   - rectangular
   - V-shaped
c) Triangular profile weirs
   - Crump
   - flat 'v'
d) Flumes
   - rectangular
   - trapezoidal
   - U - throated

The choice of a flow measurement structure and its design is very dependent on the purpose for which it is required and the physical conditions occurring at the location of interest..

The table 6.1 below which is based on a table in ISO 8368-1985 and BS 3680: Part 4H: 1986 presents some of the purposes for which some structures may be applicable, together with a guide to their limitations. This also makes reference to the appropriate International Standards.

Type	International Standard	Typical uncertainties in computed discharge %	Modular limit	Geometric conditions	Typical application
Thin plate weirs	ISO 1438/1	1 - 4	*	2**	Laboratory, pump tests, sediment free water, small irrigation channels. For best overall accuracy over a wide range of very small discharges the v notch is preferable to the rect. thin plate weir.
Broad crested weirs					Where economy and ease of construction are important factors. Irrigation channels with little fall available and wide range of flow.
a) rectangular profile	ISO 3846	3 - 5	66%	1.5**	
b) round-nosed horizontal crest	ISO 4374	3 - 5	80%	1.5**	
c) v shaped	ISO 8333	3 - 5	80%	1.5 - 3.0**	
Triangular					
Crump	ISO 4360/1	2 - 5	75%	3.5**	Low afflux, Hydrometric networks and principal irrigation channels.
Flat - V	ISO 4377	2 - 5	70%	2.5**	Hydrometric works with wide range of flow, sensitive at low flows, relatively low afflux
Long throated flumes	ISO 3847	2 -5	74%	0.7+	Relatively low afflux, sediment-laden channels, flow with debris, migratory fish, conduits and partially filled pipes, sewers.

* - Nappe to be fully aerated

** - Maximum H/P, where H is the total upstream head and P is the height of the weir

\+ - Maximum A_t/A_u are the cross-sectional areas of the throat and approach channel respectively

Table 6.1:   Flow measurement structures - applications and limitations

Brief descriptions of some of the more well known types of flow measuring structures are contained in Sections 6.5 to 6.10 below. However, it is not possible and beyond the scope of this manual to reproduce all the hydraulic theory and formulae, design criteria, limitations of use and other relevant information. For further information, the reader is referred to the appropriate Indian and/or International Standards and the other references referred to in Section 6.14.

6.6   THIN PLATE WEIRS (ISO 1438/1)

Thin plate weirs are relatively inexpensive to construct even though the manufacture of the crest requires particular care (See Figure 6.1). These are generally thought of as the typical laboratory weir; accurate, sensitive, and with well established rating relationships. They are nevertheless used to a considerable extent in outdoor hydrometry, albeit with a limited range of applications. There are two types of sharp edged weir - the rectangular notch (See Figure 6.3) and the V-notch (See Figure 6.4), usually a 90° V but other angles may be encountered. Their rating is influenced by the approach channel geometry and the appropriate value of velocity coefficient (given in the relevant standard) should be used. The V-notch is particularly accurate at very low flows.
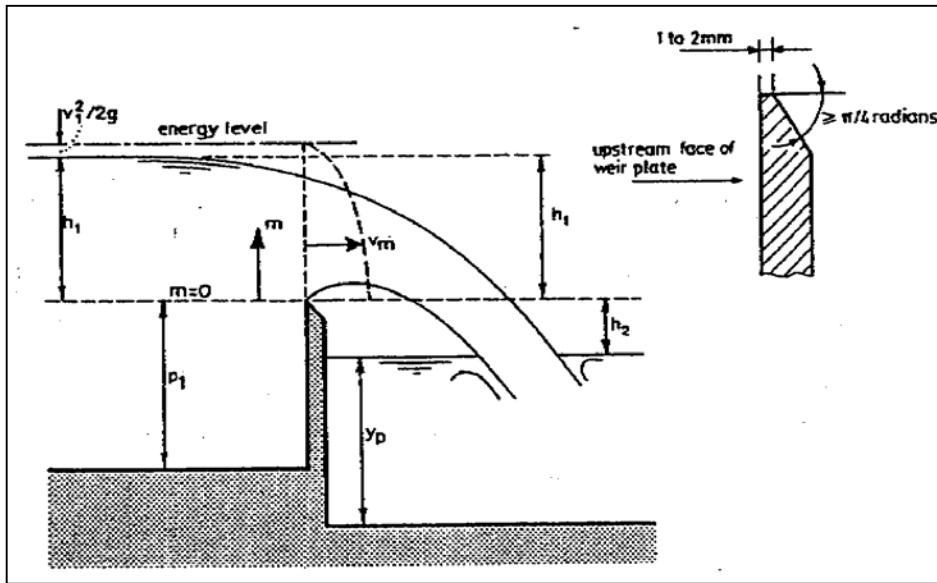
Figure 6.1:
Sketch illustrating
thin plate weir
nappe and crest
shape

Apart from the obvious limitations of scale, the significant afflux (increase in upstream water level) created by these weirs which is the requirement to obtain the sensitivity, makes them suitable only for locations where small, relatively constant flows are expected, e.g. springflows, reservoir leakage, small discharges etc. These are also applications for which the tendency of weir plates to trap floating debris is not critical. Weir plates are often used for temporary or experimental installations, but the difficulties of ensuring that the structure into which the weir plate is fixed is watertight are all too frequently overlooked.

Possibly their most common application is the borehole pump-test weir tank, where relatively constant discharges with pre-determined maxima are involved. The weir plate should be set in the short side of a rectangular tank, remote from the inflow. Baffle plates across the tank upstream of the weir are essential, as is the need to ensure that the tank is set level.
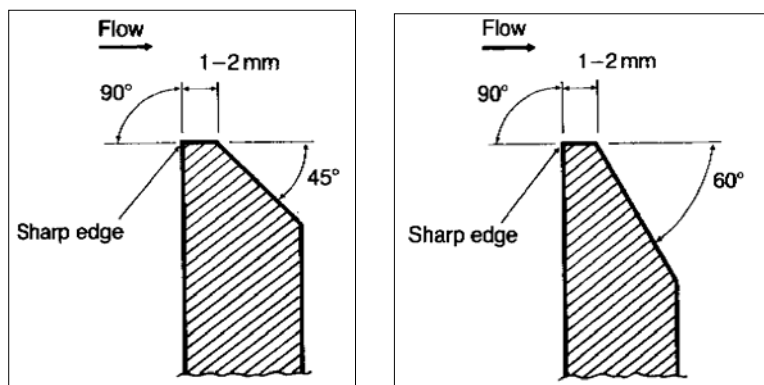
Figure 6.2:
Sketches illustrating cross-section
over a thin plate weir

(a) Rectangular notch   *(b) V - notch*

Traditionally, corrosion-resistant sheet metal has been used for weir plates, though rigid plastic, which is less attractive to thieves, has been successfully used ("thin" does not preclude the use of relatively thicker sheet plastic) but the edge of the notch must still be chamfered to the required dimensions. The angle between the horizontal and the line of the bevelled surfaces should be at least 45° for rectangular thin plate weirs and at least 60° for v-notches.

Thin plate weirs are dependent on the full development of the contraction below the nappe. Therefore the downstream water level should be low enough that it does not interfere with the air pocket below the nappe.
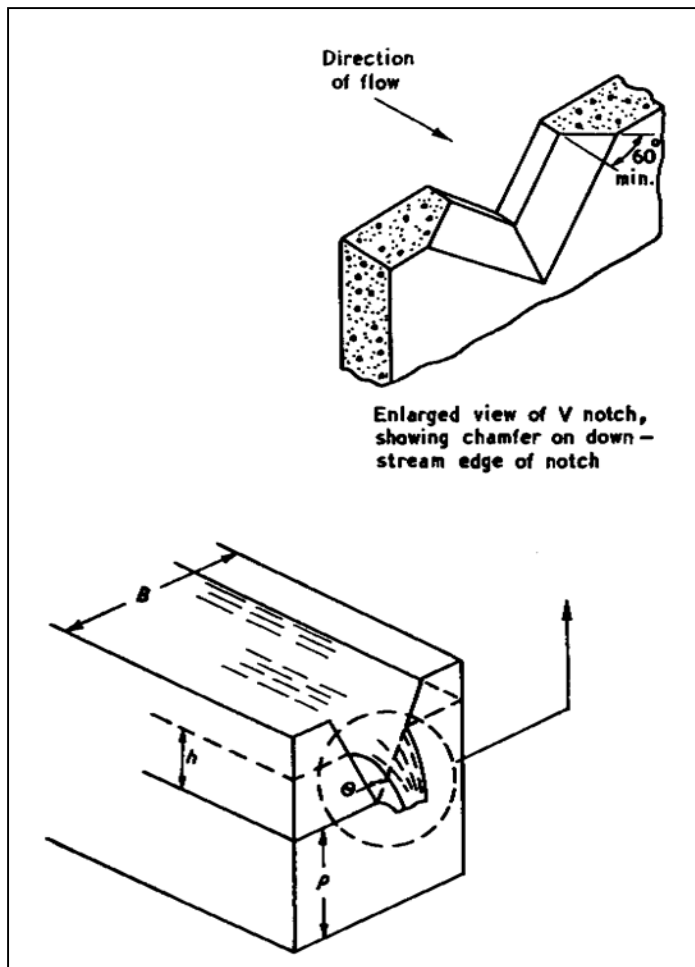
Figure 6.3:
V notch
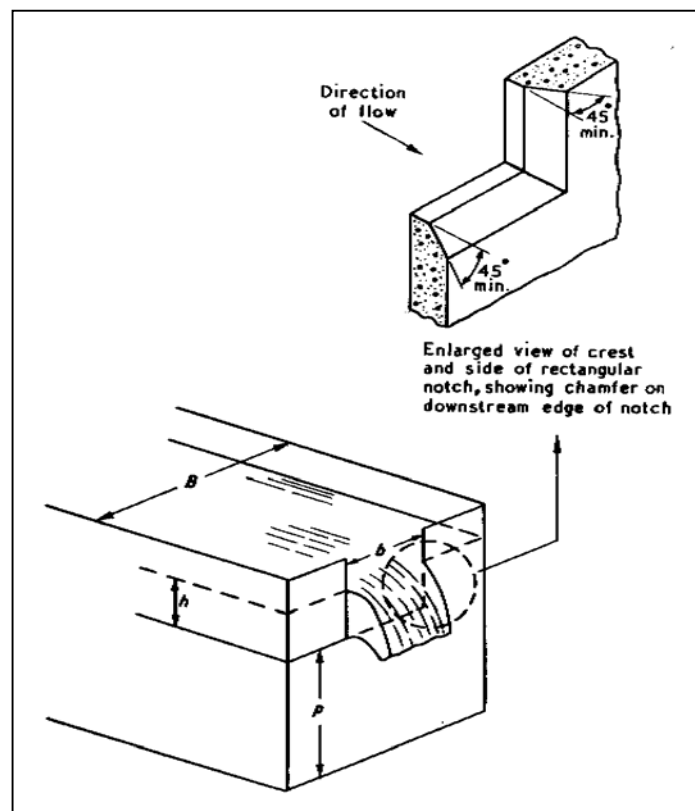
Figure 6.4:
Rectangular thin plate weir

6.7   BROAD CRESTED WEIRS -RECTANGULAR (ISO 3846), ROUND NOSED (ISO 4374), V- SHAPED (ISO 8333)

A weir is considered to be broad crested if the breadth of the weir is greater than 2.5 times the upstream head and the flow over the crest is parallel to it at some point, though other definitions of "broad" may still be found that include all weirs that are not sharp edged. For a rectangular broad crested weir (Figure 6.5), the crest must be horizontal and its upstream edge a sharp right angle. A rounded edge significantly increases the discharge coefficient and round-nosed weir criteria then apply. Broad crested weirs are unsuitable for use in steep channels where complications may arise with the formation of a standing wave upstream.

Figure 6.5:
Rectangular broad crested weir

Figure 6.6:
Round nosed horizontal broad crested weir

Broad crested weirs can be relatively inexpensive to construct and the round nosed version in particular is robust. The edges of the weir crest on the rectangular version are prone to damage.

Broad crested weirs are best used in rectangular channels where regular maintenance permits clearance of any deposition upstream and of algae from the crest. Round nosed broad crested weirs (Figure 6.6) have a good discharge range and submergence ratio and are appropriate for use in smaller and medium sized installations. The V-shaped broad crested weirs (Figure 6.7) are suitable where a large discharge range is expected. They are recommended for use in small rivers and artificial channels with little fall available. V-shaped broad-crested weirs can be constructed either with a fixed crest or a movable one in vertical slots.

Broad crested weirs are prone to siltation and have a variable discharge coefficient i.e. the coefficient varies with changing stage and discharge. At low flows, broad crested weirs can be badly affected by debris grounding on the flat crest, as well as by weed and algae growth.

Sometimes existing structures with a broad crested profile (e.g. a ford, canal overspill) are adapted for gauging purposes.
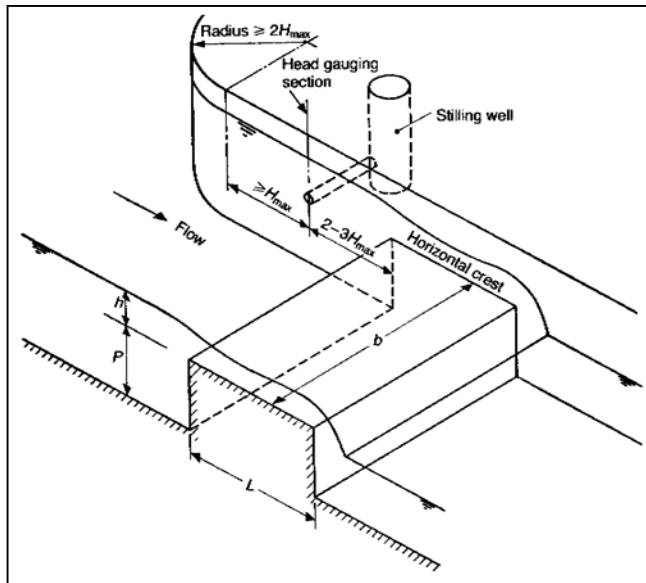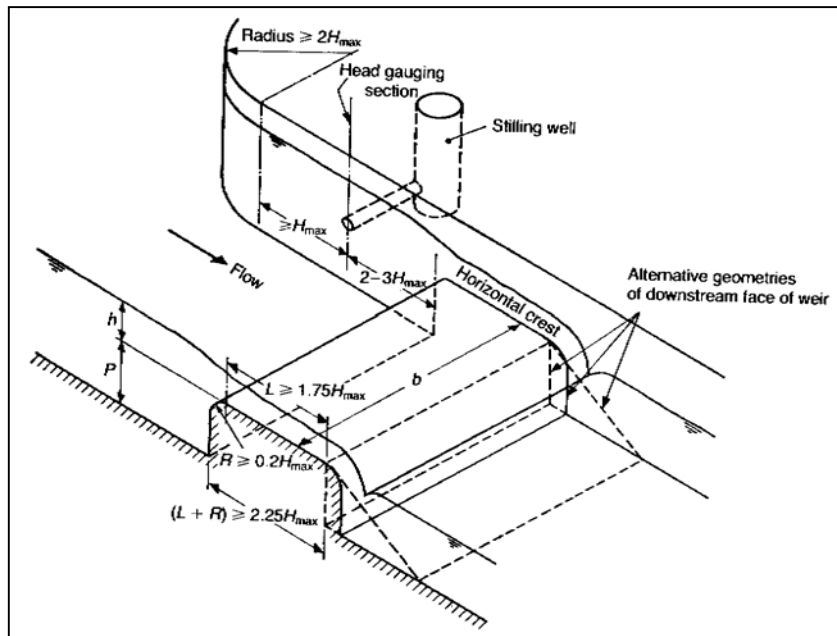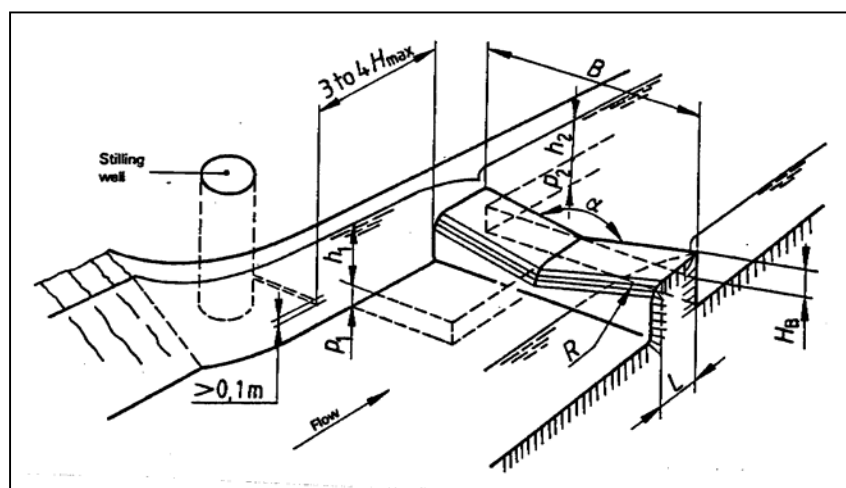

Figure 6.7:
Sketch of V-shaped broad crested weir

6.8   TRIANGULAR PROFILE (CRUMP) WEIRS (ISO 4380)

The introduction of the triangular profile weir was due to E.S. Crump in the USA in 1952, and this type is still commonly known as the Crump weir. Crump's intention was to produce a weir that combined hydraulic efficiency i.e. a high modular limit, with a discharge coefficient constant over a wide range of flow and good self-cleansing ability, at least for floating debris. Therefore, they are particularly appropriate for the measurement of flow in natural watercourses where minimum head losses are sought and where relatively high accuracy required. They have a good discharge range and modular limit, are robust, insensitive to minor damage and will operate even when the flow is silt laden. (It is interesting to note that in Crump's original proposals he intended using a Pitot tube on a central pier to measure total upstream head, not merely static head, but this proved impractical under field conditions).

The standard triangular profile has a 1 (vertical) to 2 (horizontal) upstream slope and a downstream slope of 1 to 5 (Figure 6.8) or 1 to 2. However, both slopes may be truncated within specified limits to reduce the overall breadth of the weir but in practice, and to avoid creating a debris trap, only the downstream slope is usually truncated. The Crump weir tends to have reduced accuracy at lower stages less than 60 mm. Its derivative the flat V weir (See section 6.8) providing greater sensitivity at low flows.
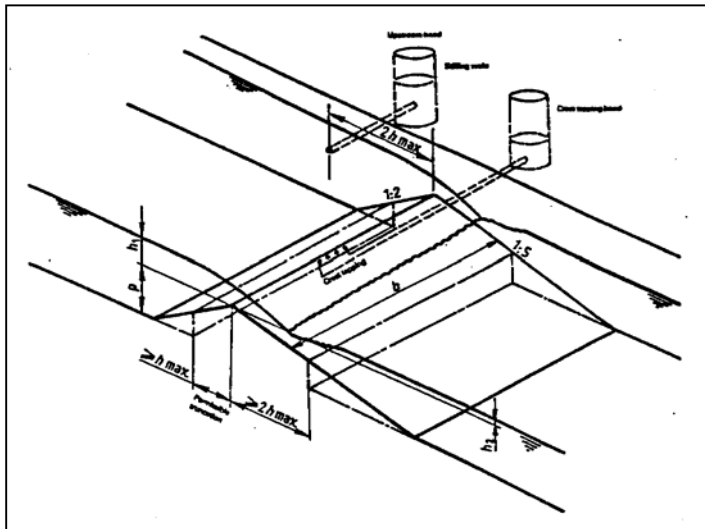
Figure 6.8:
Sketch of triangular profile (Crump) weir

The International Standard includes provision for a secondary crest tapping the data from which are used to estimate non-modular flows. It is located just downstream of the crest of the weir and measures the piezometric head in the separation (eddy) pocket of water formed just downstream of the weir crest. The crest tapping should consists of a sufficient number (usually 4 to 12) of 0.01 m diameter holes drilled in the weir crest block on 0.1 m centres, 0.019 m downstream of the weir crest as shown in Figure 6.9. The edges of the holes should not be rounded or burred. Preferably the crest tapping should be located at the centre of the weir, but may be well off centre provided that the sidewalls do not interfere with the pressure distribution in the separation pocket. a distance of about 1.2 m from the side walls should be sufficient. Weirs with a breadth (b) of less than 2.5 m should have the crest tapping in the centre.
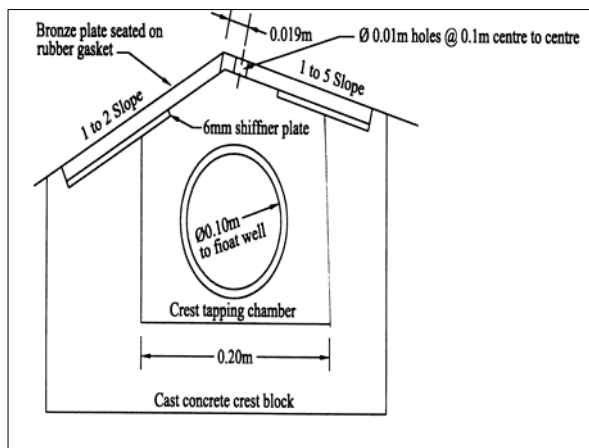
Figure 6.9:
Sketch showing the location of crest tappings and alternative design solutions for a Crump weir

(a) Crest tapping Alt 1
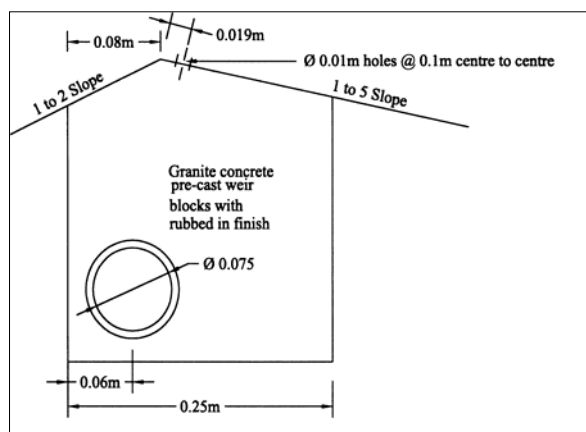
(b) Crest tapping Alt 2

The ratio of the crest tapping piezometric head (h_p) and the total upstream head (static plus velocity heads, H_u) can be used to obtain a reduction factor (f) which is incorporated into the standard weir formulae as follows:

$$Q = fC_d\left(\sqrt{g}\right) bH_1^{\frac{3}{2}}$$

where: Q = discharge
C_d = discharge coefficient unique to the structure
g = acceleration due to gravity
b = width of structure (weir crest length or flume throat width)
H_1 = total upstream head, where
f = reduction factor, which is a function of (h_p/H_1)

However, crest tappings are liable to become easily blocked in silt laden rivers and are not easy to maintain. The downstream head measurements are relatively easy to obtain and can be used to estimate the reduction factor f as a function of (H_2/H_1) where: H_2 = total downstream head.

However, the relationship between the ratio of the downstream and upstream heads and the reduction factor is relatively flat relationship and as such is insensitive. The relationships between reduction factor and the ratios of crest tapping pressure head to upstream head and downstream head to upstream head are illustrated in Figure 6.10
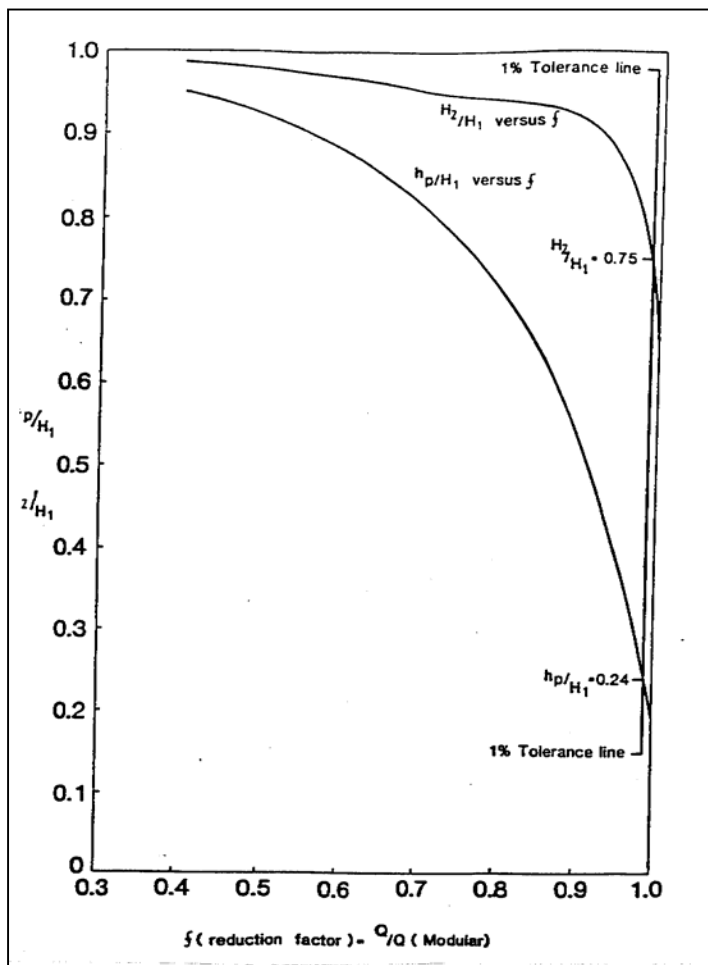
Figure 6.10:
Reduction factor characteristics

It can be seen from Figure 6.10 that Crump weirs start to become non-modular when the following ratios are exceed:

$h_p/H_1 \quad > \quad 0.24$

$H_2/H_1 \quad > \quad 0.75$

Crest tappings create additional construction complexities and can be difficult to maintain in an operational state under field conditions. In particular the crest tappings and the intake to the crest tapping stilling well are prone to blockage by silt. Also, historically there have been problems in synchronising head and crest-tapping readings when chart and/or mechanical punch taped recorders have been used. In order to avoid siltation, silt-flushing systems can be designed and constructed. Some of these systems can be made to work automatically using timer-controlled pumps. However, the use of crest tappings is generally not recommended for most Indian applications.

6.9 TRIANGULAR PROFILE FLAT V (V CRUMP) WEIRS (ISO 4377)

These are extremely sensitive and are recommended where low flows are such as to introduce unacceptable uncertainties if a horizontal (Crump) crest were to be considered. These weirs combine the good modular range and self cleansing capabilities of a Crump weir with the gauging sensitivity over a wide range of flows of the v shaped weir.

They were developed in the mid-1960s by W.R. White at the Hydraulics Research (HR), Wallingford, UK and their development is well documented in a series of HR publications from 1966 onwards, though their alternative name - the V-Crump weir - reveals their original derivation. The flat-V weir combines the 1 to 2 / 1 to 5 Crump weir crest geometry with a relatively shallow v section, in practice almost always with 1 to 10 side slopes. However, HR ratings have also been derived for the much less sensitive 1 to 20 and 1 to 40 crossfalls. The concentration of flow at the centre of the V further improves the modular performance (a stronger hydraulic jump) and debris-shedding characteristics of the Crump design, but the three-dimensional approach flow means that the positioning of the stilling well inlet pipe, normally at least 3 x h_u upstream of the weir, is important.
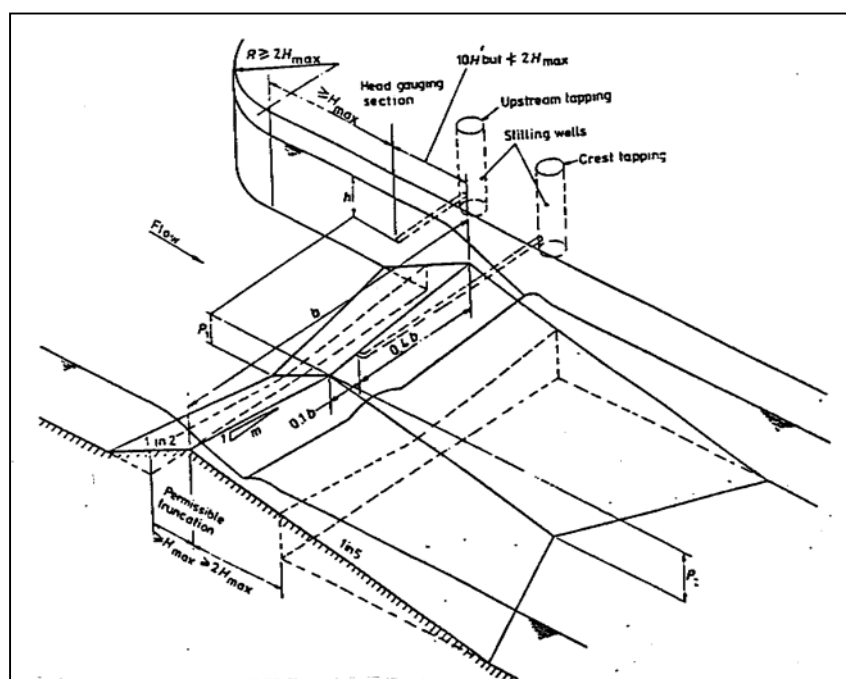
Figure 6.11:
Triangular profile flat-v weir

The weirs relatively complex geometry means that considerable care is required in setting out and fabricating of the formwork during construction. These structures are relatively expensive to construct, especially if erosion is liable to occur downstream and substantial protection works are required.

Crest tappings have been installed in Flat-V weirs. However, the relationships between the ratio of crest pressure to upstream head or downstream to upstream head and the reduction factor are not so easily definable as the Crump weir relationships. This is because three dimensional flow conditions occur in the approach channel due to the shape of the crest.

6.10   COMPOUND WEIRS (BS 3680/4D)

Although pre-dating the flat-V weir, compound weirs have not been entirely superseded by the flat-V design. This is because the flat-V has its limitations on large, broad, relatively shallow rivers where even a flat-V with the less common 1:20 crossfall would present problems with channel "fit" Defined in general terms as two or more individual structures operated in parallel and separated by dividing piers, their configuration is usually a central, low flow section flanked by a pair of medium/high flow weirs. The low flow portion may be either a Crump weir or flume; the flanking weirs, preferably (but not always) symmetrical about the centre line, are usually horizontal Crump weirs and constructed with a higher crest level. The purpose of the piers is to prevent crossflow (i.e. three-dimensional flow) between the sections. For the same reason it is recommended that the difference in level between adjacent crests (or inverts) should not exceed 500 mm. The piers must be at least 300 mm wide with rounded upstream cutwaters. They should extend from the upstream recording section to the downstream limit of the individual weirs or flume, and should be the same height as the structure's wing walls.
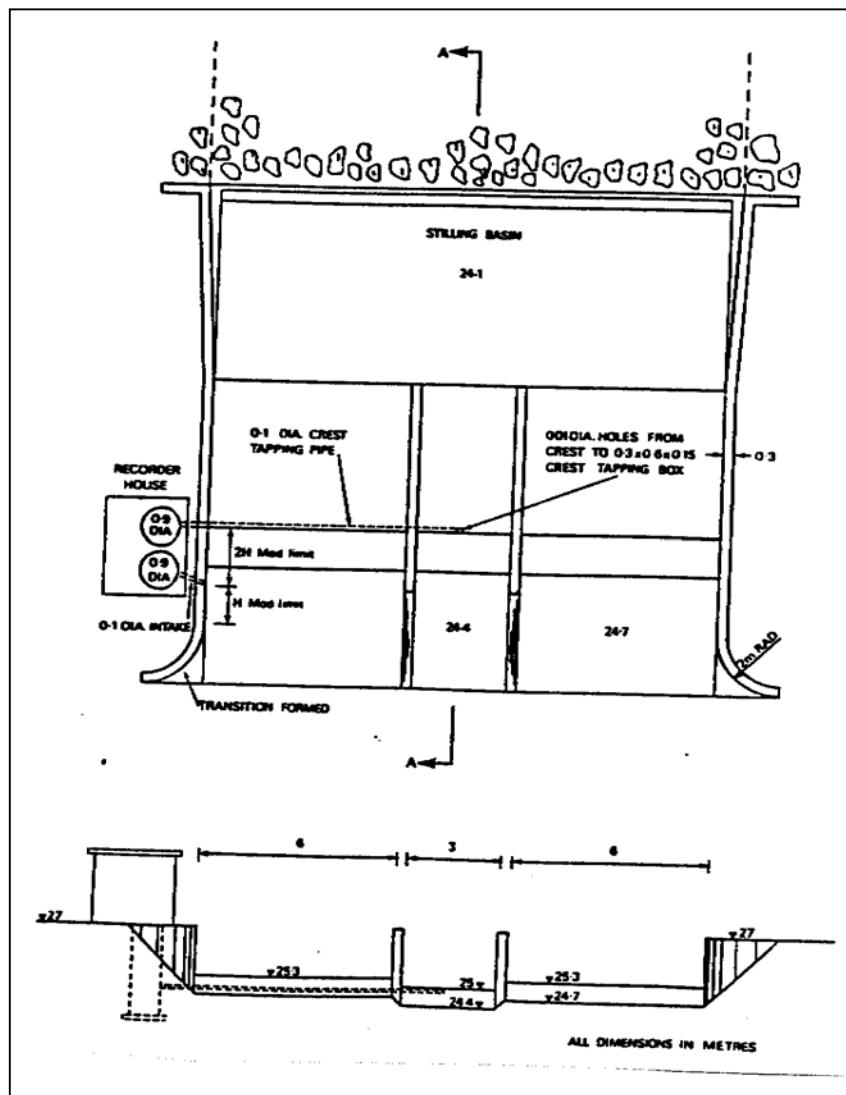
Figure 6.12:
Compound Crump weir

There are three main disadvantages of compound weirs:

a) Even though the flow is assessed as the sum of their individual components, their rating is complicated by the tendency of the low flow section, with its lower crest level, to become non-modular well before the flanking weirs.
b) Floating debris is liable to become wedged against the pier cutwaters.
c) With their complex layout, they are relatively costly to construct.

6.11   FLUMES (ISO 4369)

A flume can be defined as the narrowing of a channel to give critical flow conditions in the constriction (or throat), with a return to sub-critical flow at a standing wave formed downstream of the constriction. Under these conditions (and assuming modular flow) there is, for a given flume geometry, a fixed relationship between upstream head and flow. Because the invert (floor) of the flume is level with the bed of the natural channel, flumes have excellent self-cleansing properties and are therefore suitable for use in watercourses with a significant silt or bed load. Their inherently low-afflux characteristic is also a very attractive feature and, incidentally, minimises the need for foundation cutoff walls.

The following flumes are discussed:

*   Rectangular flume
*   U-Throated flumes
*   Trapezoidal flumes
*   Short throated flumes

Rectangular Flumes
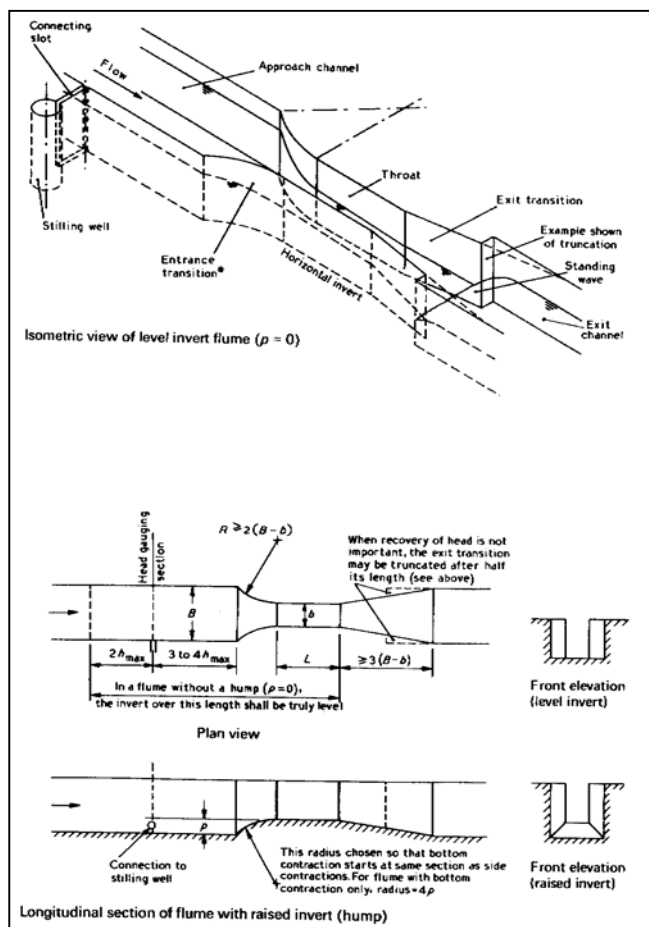
Figure 6.13:
Rectangular flume

This is the basic flume configuration, with vertical, parallel throat walls and a flat, horizontal floor. The International Standard gives the limiting geometry as L > 2 x h_u where L = the length of the parallel sided throat b > 1/3 x h_u where b = the width of the throat and the upstream head (h_u) should not exceed 2.0m. It is important for rating accuracy that the floor of the flume should be laid level and horizontal. The basic rectangular flume has three main disadvantages. Firstly, the formal structure is relatively long in relation to the size of watercourse. Then its shape gives it a limited gauging range so it can only be used on smaller watercourses but, most seriously, its accuracy diminishes markedly at low flows. To overcome these problems a number of variations to the basic design has been introduced. Another problem, common to all flumes, is that the downstream standing wave may scour the channel bed and banks. This may in turn create shoaling further downstream which could adversely affect modularity. Energy dissipation measures should therefore be given careful consideration, especially with larger flumes.

U-Throated Flumes

This is a rectangular flume with a U-shaped invert. Its principal use is for measuring flows in sewers or conduits running partially full. It is therefore unlikely to have any significant role to play for hydrological field measurements in India.
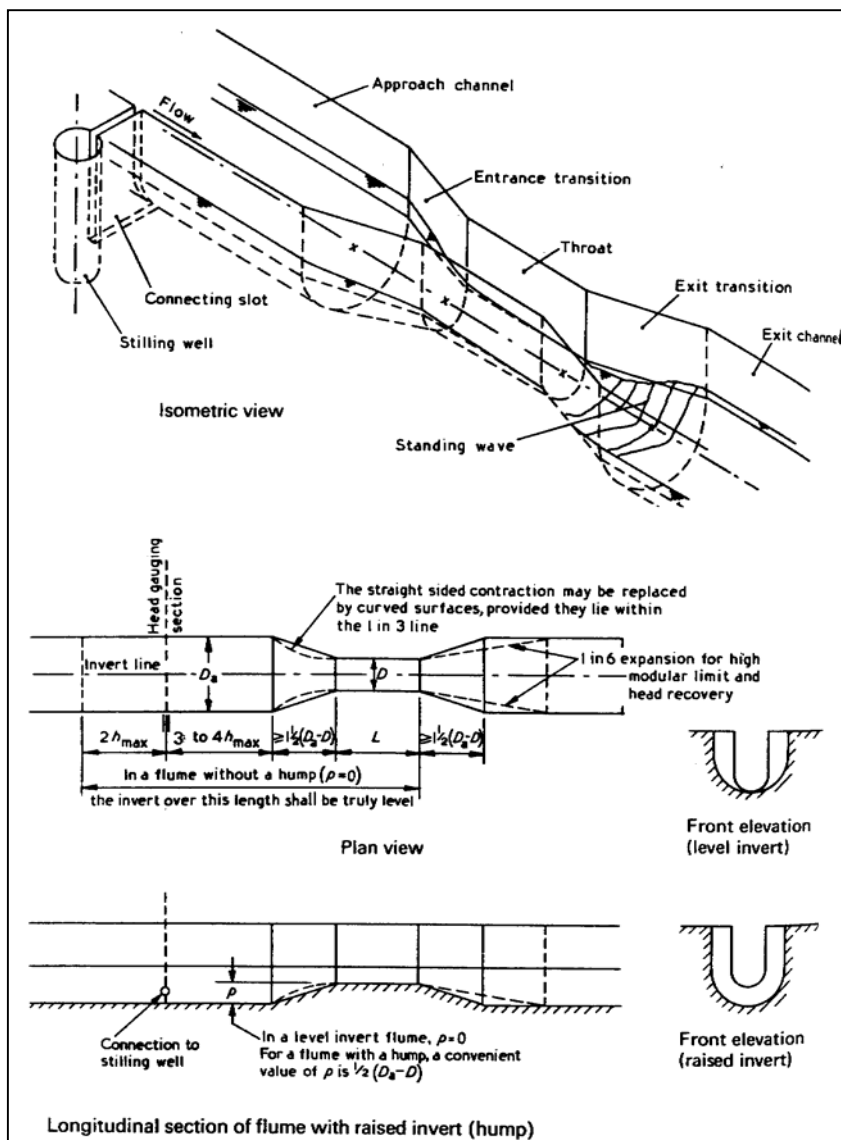
Figure 6.14:
U-throated flume

Trapezoidal Flumes

Here the shortcomings of the rectangular flume - limited capacity and poor low flow sensitivity - are addressed by introducing the "flat-V" solution into the flume context and making the throat of the flume trapezoidal in section.

Short Throated Flumes

In this variation the problem of the excessive length of the rectangular flume is overcome by placing a hump in the bed of the otherwise horizontal throat in order to encourage earlier formation of critical flow conditions. This however, prejudices to some extent the self-cleansing properties of the flume.
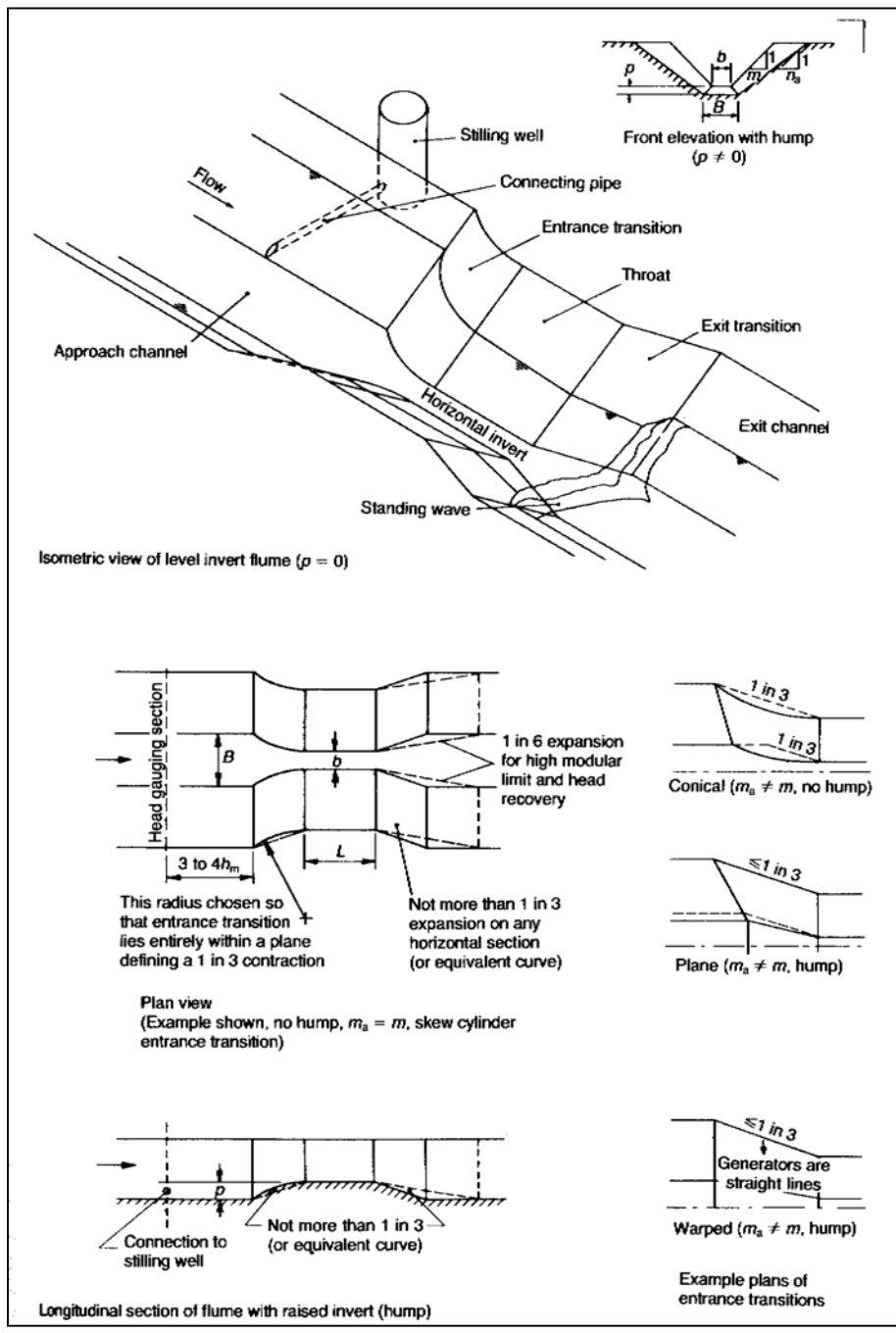
Figure 6.15
Trapezoidal flume

6.12   RESERVOIR SPILLWAYS, SLUICE GATES AND OTHER CONTROL STRUCTURES

The overflows of dam spillways, barrages, weirs and other control structures can often be utilised for flow measurement. In India it is often essential in terms of reservoir safety and river management to estimate the flow being discharged from a reservoir or barrage. These flow estimates can be incorporated into a hydrological data base for use in water resources planning, flood investigations and reservoir optimisation and operations studies.

The type of control structures and other devices that are often encountered include:

- Round crested spillways;
- Vertical sluice gates;
- Radial gates;
- Drop gates;
- Broad crested overspills;
- HEP off-take.

Some of the main problems with utilising non-purpose built structures for flow measuring purposes are summarised as follows:

- Many structures are required for safety reasons, to minimise level increases while at the same time maximising discharge. Therefore, they do not always make sensitive flow measuring devices;
- Many structures are complex and can have a variety of operating modes (gate openings/positions);
- Monitoring the position (amount of opening) of control gates can sometimes be problematic;
- HEP and water supply abstractions are sometimes difficult to measure. The former can sometimes be estimated on the basis of power production and worked backwards to a flow based on the level in the reservoir (head) and the most recent turbine efficiency curve;
- Even though theoretical formulae are available for standard control structures, the choice of appropriate discharge coefficients (C_d values) is sometimes problematic, particularly over time when deterioration of the structure can result in changing values.

If control structures are to be used for flow measuring purposes **it is recommended that reference is made to the latest published material on the type of structure concerned**. In this regard reference (2) in Section 6.14 contains some methods for estimating C_d values for some types of control structures. At important sites the C_d values selected should be whenever possible checked by field observation i.e. normally current meter gauging. However, finding suitable sites for field measurement downstream of large river structures is not always possible. Some indicative formulae are provided below for round and parabolic crested spillways/weirs and vertical sluice gates.

Round crested spillways/weirs

Many Indian dam spillways and barrages have crest sections of parabolic or circular form (See Figure 6.16) and can be adopted for flow measurement purposes.

$$Q = C_e \frac{2}{3} \left[\frac{2}{3} g \right]^{0.5} b H_1^{1.5}$$

The general equation takes the form:

Where Q = discharge (m^3/s)

C_e = effective discharge coefficient, it is the product of the standard discharge coefficient and the velocity of approach coefficient

g = acceleration due to gravity (m/s^2)

b = length of weir/spillway crest (m)

H_1 = Total upstream head above the crest (m)

$$Q = Cbh^{1.5}$$

The above equation is often simplified to the following general form:

where C = an overall discharge coefficient which has dimensions of the square root of gravity g

h = static head above the crest (m)

where:

For circular weirs with vertical upstream faces: $C = 2.03(h/R)^{0.07}$

and for parabolic weirs with vertical upstream faces: $C = 1.86h^{0.1}$

where R is the radius of the circular crest.
The equations therefore become:

- Circular weirs: $Q = 2.03(h/R)^{0.07}bh^{1.5}$ (m^3/s)

- Parabolic weirs: $Q = 1.86bh^{1.6}$ (m^3/s)
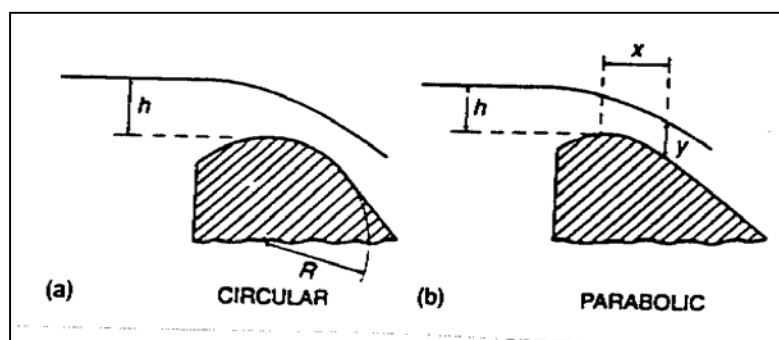
Figure 6.16:
Round crested weirs
a) circular b) parabolic

(Source: Streamflow Measurement by R. G. Herschy, 1995)

The limitations of applications for the above equations are approximately as follows:

a) The upstream head (h_1) should be measured a distance of 2 to 3 times maximum head (h_{1max}) upstream from the weir face.

b) The static head (h_1) should not be less than 0.05 m.

c) The ratio of upstream height of weir crest above bed level to static head (p_1/h_1) should be greater than 0.3.

d) The ratio of the height of the weir above downstream crest to static head (p_2/h_1) be greater than 0.75.

e) To reduce the influence of boundary layer effects at the side walls of the weir, the ratio b/h_1 should not be less than 2.0.

f) The modular limit $H_2/H_1 = 0.3$, provided that the tailwater channel does not interfere with the flow pattern over the weir.

g) The above equations are valid for weirs with vertical upstream faces. For sloping weirs approximate correction by which to multiply the correction factor are given as follows in Table 6.2.

h$_1$/p$_1$	Slope of upstream face (ratio of vert. to horiz. distances)		
	3:3	3:2	3:1
3	1.02	1.02	1.01
2	1.01	1.02	1.00
1	1.00	1.01	1.00
0.5	0.99	1.00	1.00

Table 6.2:   *Correction factors for the coefficient of discharge for circular and parabolic weirs for various upstream slopes (vert./horiz.) as a function of h$_1$/p$_1$*

Sluice gates

Free discharge:

The free discharge through a sluice gate (Figure 6.17) can be estimated using the following equation:

$$Q = C_d bw \sqrt{2g(h_1 - a)} \ (m^3/s)$$

Where: C_d = discharge coefficient
       h_1 = upstream water level relative to the sill (m)
       b = width of sluice gate (m)
       w = sluice gate opening (m)
       a = water level immediately below the sluice gate (m) or taken to be equal to $\delta \times w$, where δ is the coefficient of contraction
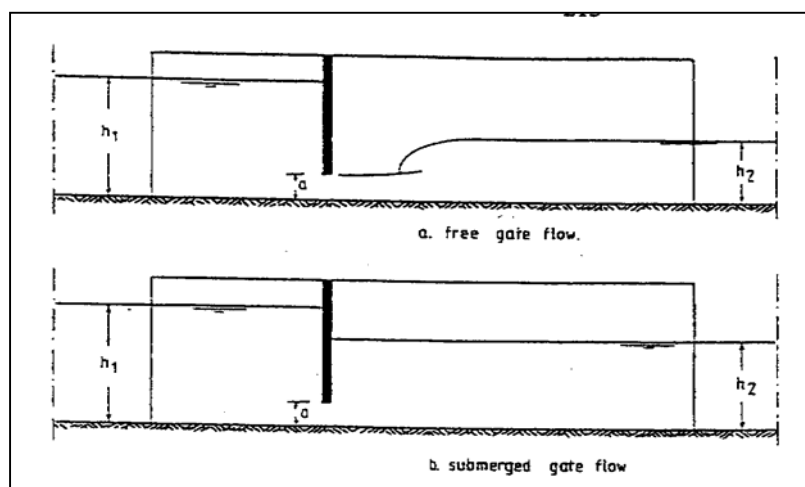
Figure 6.17:
Detail of a sluice gate

(Source: Sreamflow Measurement by R. W. Herschy, 1995)

The coefficients of discharge (C_d) and contraction (δ) vary according to the ratio of h$_1$/w. However, for field structures sufficient accuracy will be obtained between the following empirical values:

         δ = 0.63   for h_1/w = 2
         δ = 0.625 for h_1/w = 3
         δ = 0.62   for h_1/w = 10

         C_d = 0.60   for 1.5 < h_1/w < 3.5
         C_d = 0.605 for 3.5 < h_1/w < 5.0
         C_d = 0.61   for h_1/w > 5.0

Submerged sluices:

The basic discharge equation for submerged sluice gates is as follows:

$$Q = C_d bw \sqrt{[2g(h_1 - h_2)]} \qquad (m^3/s)$$

where $(h_1 - h_2)$ is the differential head across the sluice gate or the difference between upstream and donstream water levels

For a fully contracted, submerged rectangular orifice the discharge coefficient may be assumed to be 0.61.

6.13 DESIGN AND INSTALLATION CRITERIA

6.13.1 INTRODUCTION

The design of artificial controls will depend on the type of structure selected. Therefore for specific design requirements reference should be made to the appropriate Indian and International Standards and the other references. General site selection criteria for structures are discussed in Volume 4, Design Manual, Hydrometry, Chapter 4. The concept of structures and some types of structure are discussed in the previous sections.

6.13.2 CROSS-SECTIONAL AND LONGITUDINAL SURVEYS

Prior to undertaking the detailed design of a structure a cross-sectional survey shall be undertaken at the site. None of the hydraulic considerations outlined below can be properly evaluated without the benefit of surveys. Cross-sections should be undertaken at the site of the structure and both upstream and downstream of the proposed weir site. The extent of these surveys will be dependent on the physical characteristics of the river reach concerned. However, it shall be of sufficient distance to estimate the hydraulic behaviour of the channel both upstream and downstream of the site.

As a general guide the surveys would include the following:

1   Two or three cross-sections, 10m apart, at the proposed site
2   Cross-sections 25m apart for at least 50m upstream and 100 m downstream of the site.
3   Additional cross sections at any nearby bridges and other features.
4   Cross-sections at 100 m intervals for 500 m upstream and downstream of the site and then at 500 m intervals for the remaining survey reach in order to assess the modular limit and the upstream afflux effects (see Section 6.12.6). In larger rivers with low gradients it might be necessary to extend these surveys over several kilometres, maybe as much as 10 km.
5   A long section, derived from these cross-sections, showing channel bed (lowest point on the cross-section) and bank top levels.

It is also important to survey, record and plot the water level at each cross-section and, for consistency, to carry out the survey on a single day if possible, or when flow conditions are relatively steady.

The cross-sections and the longitudinal profiles shall be plotted along with a plan of the river reach. The preferred position for the structure should be plotted on the plan. A minimum length of straight approach channel of five times the maximum width of the water surface is recommended for most structures, except for thin plate weirs where ten times the maximum channel width is recommended. This is to ensure that the flow approaching the weir is aligned symmetrically with the weir and to

minimise turbulence. However, research has shown that for triangular profile weirs accurate results can be obtained even if the structure is only twice the channel width from an upstream bend.

Although of less importance, a reasonably straight channel immediately downstream of the station is also desirable, both to reduce downstream water levels, thereby maximising the modular range of the weir, and to avoid bank erosion. Downstream conditions should preferably be stable. Sites, which are influenced by downstream confluences with other rivers, river control structures, dams, tidal conditions or heavy weed growth, should be avoided. Such downstream conditions should be taken into account when designing the structure to assess the modular limit.

6.13.3 APPROACH CONDITIONS

The approach channel should be of uniform cross-section and free from irregularities and the flow in the approach channel shall have a regular velocity distribution. This can be most readily be provided by having a long, straight approach channel. Thin plate weirs are particularly sensitive to upstream velocity distribution.

The discharge coefficients of many structures vary when the velocity head in the upstream approach channel becomes large in relation to the depth of flow. A dimensionless number which describes this is the Froude number (Fr), which for a rectangular channel with parallel flow is defined as :

$$Fr = \frac{v}{\sqrt{g\bar{d}}}$$

where: v = mean velocity in the approach cross-section
g = acceleration due to gravity
$\bar{d}$ = mean depth of flow

To prevent water surface instability in the approach channel **the Froude number should generally not exceed 0.5**.

6.13.4 FLOW RANGE

The required discharge capacity of the structure and the range of flows to be measured should be estimated. A common hydrometric paradox is that at some locations, the flows of greatest interest to hydrologists are flood peaks, yet these are the very flows which most gauging stations are least capable of measuring with acceptable accuracy.

6.13.5 CHANNEL WIDTH

The channel width of the structure should be carefully tailored to fit the natural cross-section of the site. In practice, this is done by retaining an equivalent cross sectional area, as shown in the diagram below.

This criterion applies to the lined channel within the gauging structure, not to the weir profile. The invert level of the base slab should be set slightly (100 mm+) above the general bed level of the channel at the site to avoid creating a silt trap. If the structure is too narrow the gauging range will be unnecessarily limited and velocity heads increased; too wide and silt accretion and weed growth will occur upstream, obstructing the weir flank approaches.

Like all controls it is essential that the structure creates a sensitive stage-discharge relationship. In wider rivers, this can be a problem at low flows Therefore, sites should be selected where structures

which provide this sensitivity can be constructed. Structures such as the triangular profile flat "v" weirs have been developed to provide such sensitivity.
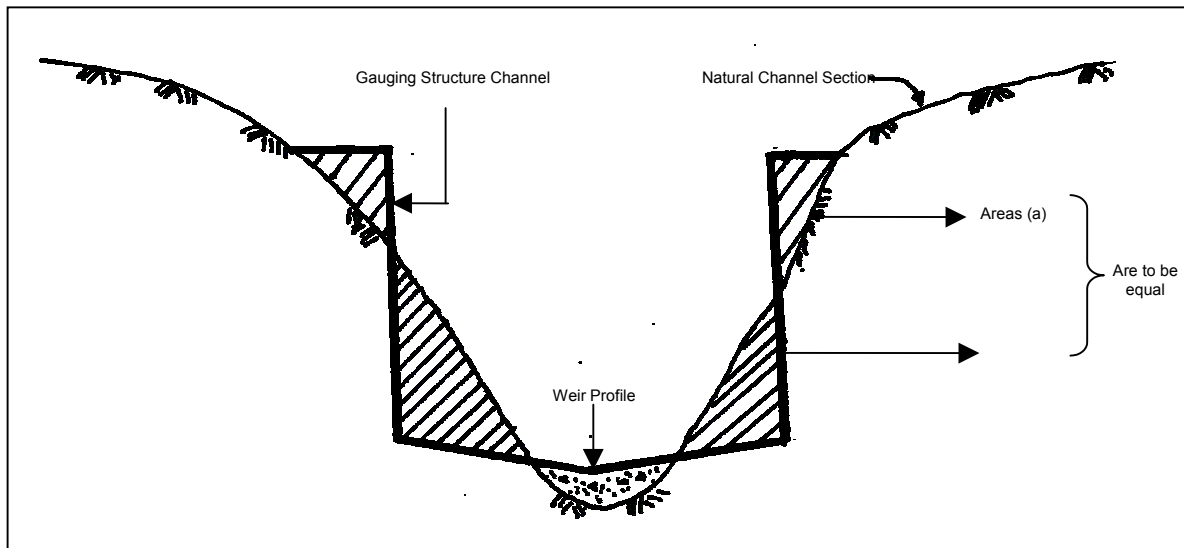
Figure 6.18:   Sketch to illustrate the fitting of a flow measurement structure into a river channel

6.13.6 MODULAR LIMIT AND AFFLUX

The final weir crest or flume invert level and flume throat width will ultimately be a compromise between maximising the modular limit while at the same time avoiding adversely raising upstream level conditions. In other words afflux should be maximised to achieve the greatest possible modular range at the site. However, do not build a small dam! Creating afflux may create unacceptable conditions upstream (e.g. submerged irrigation canal outfalls, reduced river bank freeboard etc.) and so limit the effect of afflux above the gauging weir and avoid compromising flood defence standards by:

1. locating the weir along a reach of river with a steeper bed gradient than that of the adjacent reaches.
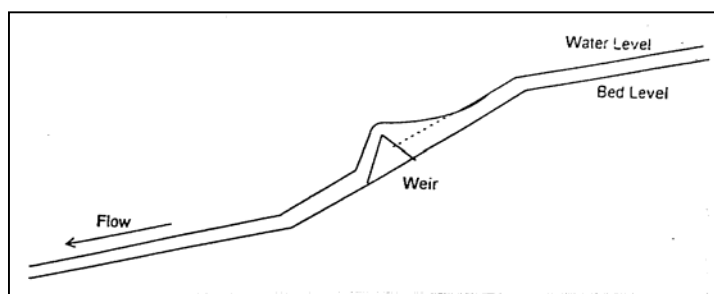
Figure 6.19:
Locating a structure to minimise the impact of afflux (1)

2. utilising existing afflux, natural or artificial, at a "step" in the channel bed, (e.g. below a bridge is a favourite location) but site the structure downstream of the "step" if possible to minimise the extent of the upstream backwater. This is illustrated in the sketch below.
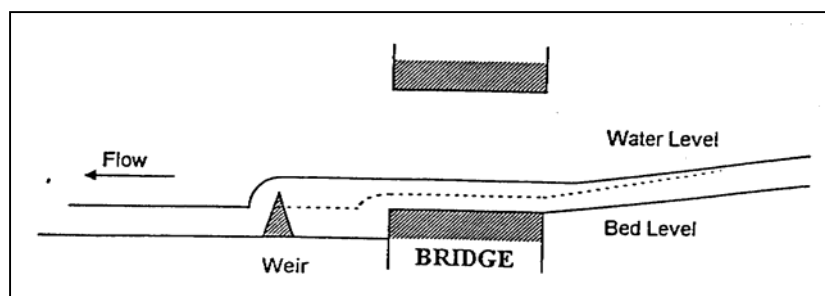
Figure 6.20:
Locating a structure to minimise the impact of afflux (2)

3. by constructing the gauging weir in a relatively deeply incised section of the channel.

In order to fix the optimum level of the weir crest or flume invert and the throat width the following should be undertaken.

1. Develop a rating curve for the natural site relative to a fixed datum. This will be assumed to be applicable to the channel downstream of the structure after construction and is referred to as the downstream or tailwater rating curve. This should be developed in terms of total head i.e. a relationship between stage plus velocity head vs. discharge should be derived.

    Total head is given by the following equation:

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

where: H = total head (m)
        h = stage (m)
        v = mean velocity in the cross-section (m/s)
        g = acceleration due to gravity (m/s^2)

    The rating curve can be derived from existing stage - discharge data if they are available. If no data are available then a relationship can be derived using the Manning formulae. The data obtained from the cross-sectional surveys should be used for this purpose i.e. for various stages the flow can be estimated using the cross-sectional data, the slope of the channel and assumed Manning's roughness coefficient (n) values. It is important that the data used to estimate the slope are extended sufficiently far downstream in order to obtain a true reflection of the possible impact of downstream conditions and not just some local change of slope. This might require surveying over several kilometres. The tailwater-rating curve (Curve 1) should be plotted on graph paper to a suitable scale.

2. Decide on preliminary dimensions and crest/invert levels of the structure. Use these dimensions and the appropriate hydraulic formulae for the structure(s) under consideration to estimate a total head (relative to structure crest level) vs. discharge curve for the weir. Modular limit is expressed in terms of the ratio of total downstream head to total upstream head. For example for a triangular profile (Crump) weir the modular limit is 0.75. Multiply the total head values used to derive the weir stage discharge curve by the modular limit for the type of structure concerned. Plot the reduced stage values against the discharge (Curve 2) to the same scale and relative to the same datum as that of the tailwater curve (Curve 1). The two head - discharge curves should be superimposed on the same graph or overlaid. Read from the graph, the total head reading corresponding to the point where the weir curve (Curve 2) cuts the tailwater curve (Curve 1). This will be the estimated modular limit.

3. The impact of the structure on upstream water levels should be estimated. Therefore, longitudinal water surface profiles should be estimated using standard, open channel hydraulics, backwater analysis for the upstream channel with and without the weir in place. The analysis should be extended far enough upstream to allow the "with weir profile" to become asymptotic with the "natural profile" i.e. the point where the backwater effect becomes negligible.

4. Once the modular limit and the impact on upstream water levels have been estimated the level of the crest (weir) or invert (flume) level should be reviewed. In the case of a flume the throat width should also be reviewed. If necessary new elevations and throat widths are selected and the analysis repeated until the optimum level(s) and throat widths are obtained. The afflux should be maximised to achieve the greatest possible flow range, while at the same time avoiding a detrimental increase in upstream water levels.

The installation of **a downstream water level sensor** is recommended at all important flow measurement structures so that periods when non-modular flow has occurred can be identified and non-modular flow estimates made.

Once the final crest elevation and the other weir or flume dimensions have been determined work can proceed with the structural design of the gauging station. This should be undertaken in accordance with the appropriate Indian design and construction standards e.g. concrete mixes etc. for this type of structure.

6.13.7 SEDIMENT

Sites where high sediment load or scouring occurs should be avoided if possible. The design of the structure should be such as to minimise upstream sediment deposition or downstream scouring. In rivers with high bed loads the use of structures which significantly reduce the upstream velocity is not recommended.

6.13.8 LEAKAGE/SEEPAGE

This refers to leakage or seepage underneath the foundations of the gauging structure, not the ungauged hydrogeological flow. Leakage beneath the structure is proportional to the hydraulic gradient in the underlying soil, i.e. the head differential per unit length of potential flow path. Because the head differential causing leakage under a gauging structure is relatively small (though often at its greatest at low flows) the hydraulic gradient along the structure is correspondingly small, helped by the length of channel structure needed to comply with the appropriate ISO for many types of gauging weir. The leakage flow path may be considerably lengthened by the use of short (<2m below bed level) lightweight steel sheet pile cut-off walls across the channel bed at either end of the structure. The cost of this may be offset by using the piles as permanent shuttering for the concrete base slab and as a cofferdam during construction, cutting the pile heads off at slab top level on completion of the work.

6.13.9 THE GAUGING STRUCTURE

The gauging structure should be designed in accordance with Indian Standards relating to this type of Hydraulic Structure. The fundamental requirement is to achieve a high degree of accuracy in attaining the correct crest line, levels and slopes (often in three dimensions) and this can create serious difficulties on site. The main problem arises from the fact that the contractor will probably not be familiar with the dimensional accuracy required for a gauging weir, or be capable of achieving it. Therefore, adequate supervision should be provided to ensure that the formwork is constructed to the required standard and dimensions. The length and elevation of the stilling basin should be estimated in accordance with the Indian Standards on weir basin stilling basin design.

6.13.10 POSITIONING OF INTAKE PIPES AND DESIGN OF STILLING WELLS

The design of stilling wells and intake pipes is discussed in Volume 4, Design Manual, Hydrometry, Chapter 8. The same general principles apply to stilling wells for structures. However, at gauging structures the location of the intake pipe relative to the crest of the weir or throat of the flume is an important consideration in terms of application of the hydraulic formulae for the structure. The position will depend on the type of structure. For example, for a Crump weir ISO and other standards recommend that the intake pipe should be located at a distance of the minimum of the following three distances:

1. Six times the weir height
2. Twice the maximum upstream head
3. Twice the upstream head at the modular limit

The appropriate ISO or Indian Standard should be referred to, to obtain the recommended position of the intake. Whenever possible the intake should be set in the wing walls at right angles to the direction of flow.

6.13.11 FINAL DIMENSIONS

On completion of construction the leading dimensions and levels of the structure should be accurately surveyed and compared with the design drawings. Any discrepancies should be recorded and, if necessary, the stage - discharge relationship re-calculated accordingly.

6.14   OBSERVATION PRACTICE

6.14.1   INTRODUCTION

The theory and applications of flow measurement structures and design and installation criteria are described in the previous sections. Some discussion on the use of river control structures for flow measurement purposes is also contained in Chapter 6 of Design Manual, Hydrometry.

In order to measure flows using structures requires the measurement of upstream water level or stage relative to the crest of the structure. Provided the structure is working within its limits of application then the discharge is a function of this stage. As such most of the observation practice is the same as that described in Field Manual, Hydrometry, Part II on stage observations.

The cost of installing purpose built structures is such that the additional cost of installing a stilling well is relatively small. Also, purpose built flow measurement structures are normally installed on smaller rivers and streams where the response times to rainfall events can be of relatively short duration. Therefore, manual measurement cannot normally provide the frequency of measurement required. As such at most purpose built flow measuring structures, stage will be measured by means of a continuous recording device, either an AWLR or a DWLR.

6.14.2   AS BUILT DIMENSIONS

The appropriate formulae relating stage to discharge for a structure are dependent on the dimensions of the structure. On construction it is essential that the structure is accurately surveyed and all relevant dimensions obtained. The following key dimensions are required:

*   Weir crest or flume invert level;
*   Crest length (width of structure);
*   Upstream apron/approach bed level;
*   Width of upstream approach channel;

- Side slopes - trapezoidal structures;
- Crest length in direction of flow e.g. broad crested weir. This is not relevant to sharp crested structures.
- Downstream apron/bed level.
- Width of downstream channel.
- Height of wing walls and banks.
- Modular limit, if known.

For river control structures adapted for flow measurement purposes similar information to the above is required. Additional information could include:

- Control gate dimensions and details of operating positions/levels if these are fixed.
- HEP off-takes details.
- Water supply off-takes details.
- Leakage, if significant.

6.14.3 ESTIMATION OF STAGE-DISCHARGE RELATIONSHIP

The derivation of the stage-discharge relationship for rated sections/natural controls is covered in Processing Manual. Stage-discharge relationships can be derived for hydraulic structures using the appropriate formulae as contained in the relevant ISO, Indian or other relevant standard. Some of these formulae are based on upstream static head, others are based on the total head. Once the as-built dimensions are known the stage discharge relationship can be derived in terms of measured head/static head (stage). This can be in the form of an equation, graph or a look up table. Most reputable hydrometric software packages have the facility to derive such theoretical relationships based on internationally accepted standards. Therefore, provided the structure is operating within the limitations of applicability e.g. within the modular range it is possible to start estimating flows immediately after commissioning.

6.14.4 CHECK CALIBRATION (VALIDATION) IN THE FIELD

Standard purpose built flow measurement structures should not require calibration. Provided a structure has been properly constructed and is working within its range of application then it should be possible to estimate discharges to within +/- 5%. In reality this does not always happen. Therefore, **it is important to undertake field checking to confirm the theoretical stage-discharge relationship**. About 6 - 10 measurements over the full range should be sufficient for this purpose, followed up by occasional periodic checks to make sure no significant changes have taken place.

The main sources of error when using structures are not generally due to any deficiencies in the theory, but are caused by:

- Incorrect gauge zero setting relative to the crest resulting in a systematic error in the head measurement;
- incorrect measurement of the important as-built dimensions such as crest width;
- incorrect design of intake pipe and stilling well resulting in an uncertainty in head measurements;
- uncertainty or malfunctioning of recording instruments;
- overestimation of modular limit i.e. weir becomes non-modular at a lower limit than estimated.

Many flow-measuring structures will become non-modular at higher flows. Therefore, even though they might provide some stability at the measuring section, at higher flows the site effectively becomes a channel control. As such current meter gauging or some other form of flow measurement is required in order to establish the upper end of the rating curve. This sometimes referred to as a **hybrid gauging station** i.e. where a combination of methods is used to define the full flow range.

There are formulae available for non-purpose built structures such as reservoir spillways and sluice gates which can be applied to estimate flows. However, the approach conditions and design of control structures is a lot more complicated than purpose built structures. In addition, control structures do not always make good flow measuring devices. In particular they are often designed to minimise water level increases while at the same time maximising discharge capacity. Therefore they are often very insensitive controls. Also, sometimes assumptions have to be made about certain draw-offs. For example sometimes the only way to estimate HEP draw-off is to work backwards form the amount of energy produced making assumptions on the efficiency of the turbines using the most recent turbine efficiency curves. Field checking is therefore very important when using control structures for flow measurement purposes. However, the nature of channels downstream of reservoirs and other large structures sometimes makes this difficult to achieve to an acceptable level of accuracy.

Many large hydraulic structures are studied using scale models during their design stage. These studies provide among several things estimates of the discharge coefficients (C_d values). They also provide a means of developing operational rules for a wide variety of gate openings and hydrological event scenarios. However, the use of scale models is not without its difficulties. While standard formulae are available for most outlet/control structures, in some cases they are only appropriate within laid down limits of application. The choice of C_d values is dependent on many factors including approach conditions, amount of contraction, stage/water level relative to crest, gate openings, downstream conditions and roughness. Also, C_d values tend to be based on steady state conditions. In unsteady state conditions their values can vary. It is sometimes quite difficult to reproduce all these characteristics totally effectively in a scale model. However, at many large, complex structures particularly where there is a need to develop optimal operational rules, model studies can be extremely useful in developing such management procedures and confirming/estimating C_d values. Nevertheless the results of model studies should be confirmed by field checking whenever possible.

6.14.5 MEASUREMENTS

1. The most important reading is the upstream water level or stage. In this regard the positioning of measurement device is important.

2. Stage readings should be observed according to the procedures outlined in Field Manual Hydrometry, Part II.

3. It is advisable to install a downstream water level monitoring device whenever possible so that periods of non-modular flow can be identified. This will assist with the estimation of the modular limit if it is constant. For some types of structures such as submerged orifices/sluice gates it is essential that downstream water level monitoring takes place.

4. For control structures all, or some of the following measurements will be required:

    - Upstream water level/stage;
    - Downstream water level;
    - Amount and timing of gate openings - the position of each gate will require to be recorded individually;
    - HEP power production;
    - Water supply metered draw-off;
    - Estimate of leakage, if significant.

- The measurement of gate openings is sometimes difficult, or poorly undertaken. The amount of gate opening can sometimes be effectively measured by installing inverted staff gauges on the wing wall adjacent to the gate. Some recent systems use modern sensing devices to automatically monitor the position of the gates.

- Special forms have to be designed for each specific control structure so that all the relevant information can be recorded in a form that it can be readily entered onto a computer for computational purposes.

6.14.6 COMPUTATION OF DISCHARGE

The computation of discharge for structures is covered the Manual on Data Processing at the Divisional Office - Stage-Discharge relationships.

6.14.7 MAINTENANCE OF STRUCTURES

Even though flow measurement structures are designed to operate unattended for significant periods of time they still require periodic maintenance. The procedures to be applied to the gauge posts, stilling wells, AWLR's or DWLR's installed at structures are the same as outlined in Field Manual, Hydrometry, Part VIII.

The following other routine maintenance should be undertaken on a regular basis:

1. Keep structure free of debris.
2. Keep the structure clean - remove algae growth.
3. Upstream and downstream channels should be kept clear of aquatic weed growth and other matter which could effect the accuracy and the performance of the structure. In particular it is important that weed growth does not occur to such an extent in the downstream channel that non-modular conditions occur.
4. Inspect the weir and surrounds for structural damage and bank erosion.

The above should be undertaken on each regular visit to change the chart or download the data logger.

It is important that structural repair works are undertaken as soon as possible after their need is identified so that the problem does not significantly worsen with resulting cost implications. To use the old expression " a stitch in time saves nine".

6.15 REFERENCES

International Standards

ISO 1438	1975	Thin plate weirs and venturi flumes
ISO 1438/1	1980	Weirs and venturi flumes, Part 1, Thin-plate weirs
ISO 3846	1987	Free overfall weirs of finite crest width (rectangular broad-crested weirs)
ISO 4359	1983	Flumes
ISO 4360	1984	Triangular profile weirs
DIS 4362	1990	Trapezoidal weirs
ISO 4374	1990	Round-nose horizontal broad-crested weirs
ISO 4377	1990	Flat-V weirs
ISO 8333	1985	V-shaped broad-crested weirs
ISO 8368		General guidelines for selection of flow gauging structures

Books

In addition to the International Standards already cited, the following two books are also useful reference material:

1. Bos M. G. (Ed.) (1978)
    Discharge measurement structures.
    ILRI, Publication No. 20, Working Group on Small Structures, Wageningen, the Netherlands.

2. Ackers, P., White W. R., Perkins, J. A. & Harrison, A. J. M. (1978)
    Weirs and Flumes for Flow measurement.
    John Wiley and Sons (pub).

3. Herschy, R. W., White, W. R. and Whitehead, E. (1977),
    The Design of Crump Weirs
    UK Dept. of Environment, Water Data Unit, Technical Memorandum No. 8.

4. Herschy, R.W. (1995)
    Streamflow Measurement
    2nd edition, E&FN Spon, London

7   ERRORS IN THE MEASURING PROCEDURE

In ISO 7178-1983, which is a Technical Report, the results of an investigation of the total error in velocity area methods is presented. Below a summary is given of the most relevant results.

Apart from instrumental error, the error in the mean flow velocity component can be considered as comprising three independent types of errors:

1. **Error Type I (σ_f):** (measuring time), due to restricted measuring time of the local point velocity in the vertical;

2. **Error Type II (σ_s):** (number of points (m) in the vertical), arising from the use of a restricted number of sampling points in the vertical.

3. **Error Type III (σ_d and σ_h):** (number of verticals n), of the same nature as error Type II, due to the restricted number of verticals in the cross-section. The horizontal velocity profile (σ_h) and the bed profile between two verticals (σ_d) have to be determined by interpolation and therefore errors will be introduced.

Error Type I (σ_f). Taking in a measuring time t = 30 seconds for high velocities or t = 60 seconds for low velocities, the error reaches an acceptable minimum value. The random fluctuation error σ_f depends furthermore slightly on the number of points in the vertical. Values for σ_f are therefore combined with those of error Type II, see Table 7.1.

Error Type II (σ_s). Methods for computing the average velocity in the vertical from point observations are discussed in Sub-section 6.4.9.of Volume 4, Design Manual, Hydrometry The errors involved in these methods as presented by ISO (1983) combined with the Type I error are summarised in Table 7.1. Further improvement can be obtained by increasing the total measuring time. It appears that the effect of Type I error relative to Type II error is rather small. It was concluded that the results presented in Table 7.1 had better validity for larger rivers (Q > 120 m^3/s) than for smaller ones.

No	Computational method	Number of points	Standard deviation of the mean error σ_s (%) Type II	Standard deviation of the mean error σ_{f+s} (%) Types I and II
1	$\bar{v} = v_{0.6}$	1	7.5	8.2
2	$\bar{v} = 0.96v_{0.6}$	1	4.8	6.5
3	$\bar{v} = 0.5(v_{0.2} + v_{0.8})$	2	3.4	4.9
4	$\bar{v} = 0.25v_{0.2} + 0.5v_{0.6} + 0.25v_{0.8}$	3	4.4	4.8
5	$\bar{v} = 0.25(v_{0.2} + v_{0.4} + v_{0.7} + v_{0.9})$	4	2.2	3.0
6	$\bar{v} = 0.1(v_{surf} + 3v_{0.2} + 3v_{0.6} + 2v_{0.8} + v_{bed})$	5	2.1	2.7

Table 7.1:   The error σ_{f+s} (Type I and II errors) as function of sampling procedure

Error Type III (σ_d and σ_h). Error Type III is due to approximation by interpolation of the bed profile σ_d and the horizontal velocity distribution σ_h. In practice, both factors occur simultaneously. The results, showing the standard deviation of error Type III, are given in Table 7.2 as a function of the number of verticals. In choosing the number and location of the verticals, for the preparation of Table 7.2 the following procedures have been applied:

1. Verticals equidistant: the number of verticals is decided beforehand and are equally spaced across the width
2. Sections of equal flow
3. Bed profile in the cross-section (location of verticals chosen based on irregularities in the bed profile read from an echogram)

Number of verticals	Relative standard deviation of error, %		
	Verticals equidistant	Sections of equal flow	Bed profile in cross-section
5	4.20		7.70
6	3.70	4.52	7.00
10	2.60	3.35	4.40
15	1.98	2.60	3.02
20	1.65	2.08	2.20
25	1.45	1.76	1.70
30	1.30	1.60	1.28
40			0.80
45			0.68

Table 7.2:   Error Type III as a function of the number of verticals

Some of the conclusions drawn by ISO on their analysis of Type III errors, not visible from Table 7.2, read:

- Calculating the discharge from a restricted number of verticals gives results which are systematically too low.
- For large rivers (Q > 120 m^3/s) the interpolation of the horizontal velocity profile affects the extent of the error more than the interpolation of the bed profile, though the difference is small.
- For small rivers (Q < 120 m^3/s) the interpolation of the bed profile influences the error much more than the interpolation of the horizontal velocity profile.
- Errors in discharge, caused by the interpolation of the velocity profile and depth respectively are related.
- The error in discharge can be reduced considerably by using knowledge of the continuous profile (an echogram) when determining the discharge, instead of using only the depth in the verticals where the flow velocity is observed!!

8   GUIDANCE ON THE DESIGN OF CABLEWAYS

8.1   INTRODUCTION

Cableway systems for stream gauging have been in existence for decades and in many variations. Bank-side operation, where the winch and operator is located on one bank, is the most common system. In India, the USA and Canada there has also been a tradition of using manned cable car systems, in particular for large spans in excess of 200 m. However, these have been phased out in Europe and are being eliminated where possible elsewhere for safety reasons and because of improvements in bank-side equipment.

Manned cable-car systems must also be engineered to a higher specification and be subjected to a more rigorous regime of safety checks producing higher installation and operating costs. Therefore, unless the river is otherwise inaccessible, bank-side operation is the preferred option and the only one considered here.

This guideline covers the design aspects of the cableway cables, support column and foundation. Examples are given of typical designs.

8.2   BANK-SIDE CABLEWAY SYSTEMS

8.2.1   GENERAL

Bank-side cableway systems are covered by the international standard ISO 4375 "Cableway systems for stream gauging". However, the existing standard has been thoroughly revised and is currently in circulation to ISO member bodies as a draft for comment. The design procedures described in this document take account of additional safety recommendations and the clearer definitions of working and ultimate loads for the purpose of selecting cables and calculating forces set out in the draft revision. A sketch of a typical cableway is shown in Figure 8.1.
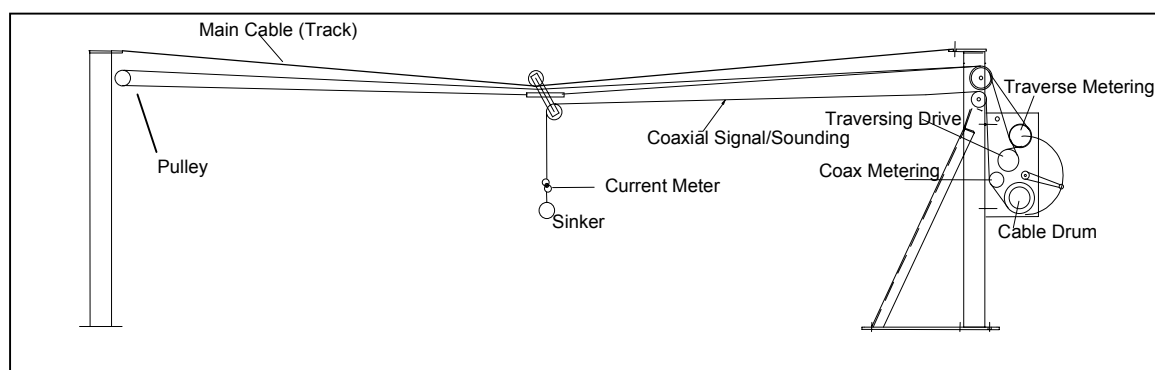

Figure 8.1:   Cableway system with loop traverse cable and spooled sounding cable

The main components of a bank-side cableway system are:

- support columns (towers)
- main cable (track cable)
- traversing cable (tow cable)
- meter suspension cable (sounding cable)
- hydrometric winch

(Alternative names in common use are shown in brackets)

8.2.2   SINKER WEIGHT

The design process begins with selecting the appropriate sinker weight as this affects cable selection, support design, foundations and winch. It is important not to over specify the sinker weight. The sinker weights rarely exceed 50 kg. If the river is deep and swift flowing, it may be necessary to specify a heavier sinker. In India practice specifies the mass of a sinker-weight needed in kg as approximately 5 x mean velocity of water in m/sec x depth of water in metres. It is noted that 100 and 150 kg sinker weights have been proposed and these values have been used for sample calculations. However, it is recommended that the sinker weight should not exceed 100 kg.

8.2.3   SELECTION OF CABLES

Selection of meter suspension cable

A FoS (Factor of Safety) of 5 is required by ISO 4375 for the meter suspension cable with respect to the maximum weight of the sinker. A 3.2 mm stainless steel coaxial signal cable has a breaking load 710 kg (6,965 N) and hence would be suitable for suspended loads up to 710/5 = 142 kg (1,393 N). If it is essential to use a 150 kg sinker, then it will be necessary to increase the size of the meter suspension cable, or accept a FoS of 4.7. This may be acceptable, if the winch is fitted with a load limiter (see Sub-section 8.2.5). The **breaking load** of the meter suspension cable is the **design load** for the main cable. It is important that this is not over-specified. For the purposes of providing examples, it has been assumed that sinker weight of 150 kg is being deployed on a 3.2 mm suspension cable and that a FoS of 4.7 is acceptable as the load is limited by a torque limiter.

Selection of main cable

Operational loadings on the main cable are a function of:

- the operating sag,
- the weight of the cable, and
- the suspended load.

ISO 4375 recommends an optimum working sag of 2% of the span. The worst case point loading is at mid-span. A main cable is selected so as to provide a factor of safety of 2 with a point loading at mid-span equal to the breaking load of the meter suspension cable. The tension in the cable when suspended between supports of equal height, under static conditions and neglecting wind loading, is given by:

$$T = T_h\sqrt{(1+(4D/S)^2)}, \quad \text{with:} \quad T_h = wS^2/(8D) + PS/(4D) \tag{8.1}$$

where: T =   actual cable tension, in Newton's
$\qquad T_h$ =   horizontal component of tension T, in Newton's
$\qquad w$ =   weight per meter run of wire rope or cable, in Newton's
$\qquad P$ =   the concentrated moving load, in Newton's
$\qquad S$ =   the horizontal span, in metres
$\qquad D$ =   cable sag (dip), in metres

From equation (8.1) it is observed that the cable tension is inversely proportional to the cable sag. In the computations of the cable tension for the design and FoS assessment, with respect to the cable sag three cases can be distinguished:

1. initial sag, i.e. the sag due to self-weight of the cable only, to be applied initially to arrive at the required sag of 2% of the span under working conditions with the suspended load at mid-span; the initial sag is a function of the span, cable diameter and suspended load to be applied

2. working sag of 2% of the span, i.e. the sag due to self-weight of the cable and the suspended load at mid-span, representing the working conditions, and

3. final sag, i.e. the sag due to self-weight of the cable and the breaking load of the suspension cable; consequently, the final sag is considerably larger than the working sag and is typically in the order of about 3.5 to 5% of the span, and is primarily a function of the cable diameter and the breaking load of the suspension cable and varies slightly with span.

From the above it follows, that for the computation of the tension in the main cable one has to apply equation (8.1) with the entries as presented in Table 8.1.

Cable tension	Sag (D)	Load at mid-span (P)
Initial	Initial sag	None
Working condition	2% of span	Sinker weight
Worst case: failure of suspension cable	Final sag	Breaking load of suspension cable

Table 8.1:   Entries for computation of cable tension

The tension in the main cable for the worst case point loading specified above is the design load for the cableway support columns.

8.2.4   DESIGN OF SUPPORT COLUMNS

Height of columns

Before columns can be designed to accommodate the loadings applied by the main cable, it is necessary to specify the required height of the columns. This can be calculated as, (see Figure 8.2):

$$H_{col} = D + C - H_{bfl} \tag{8.2}$$

where: H_{col} =   support column height above base
        D =   ultimate cable sag, which is 2% of cable span S
        C =   clearance for suspended equipment at highest expected flood level
        H_{bfl} =   height of column base above highest expected flood level.

Ground conditions and availability of space must be taken into account when deciding whether the support may be designed with a backstay or be self supporting. A self-supporting column is preferred on the winch operating side, particularly with columns of up to 3 m, as a backstay intrudes into the working space. Alternatively, a second lighter support may be provided for the winch and traversing cables, located with sufficient clearance in front of a main column.

In the absence of a cross section with information on highest expected flood levels, nominal figures have been use for the examples presented in Chapter 8.3.
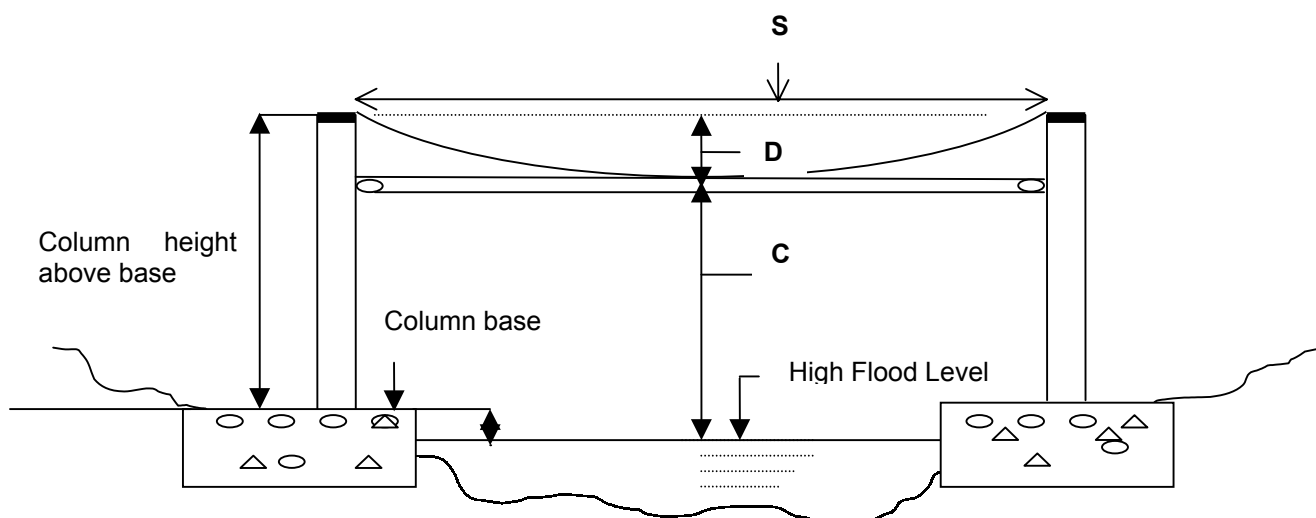
Figure 8.2:   *Definition sketch design of column height cableway*

Lateral loadings

Allowance should be made for lateral loading in the support columns parallel to the direction of flow, assuming that the worst case loadings would be applied horizontally to the top of the support.

In the examples with this document a drag on the current meter and sinker is used in the case of support without a backline stay.

8.2.5   TENSIONING SYSTEMS

Main cable

New wire rope will undergo permanent elongation of up to 0.25%, typically, over a period initially as it is subject to operational loadings. It will also be subject to elastic elongation and expansion and contraction due to temperature changes. It is necessary therefore to have some means of adjusting the tension in the main cable to avoid excessive sag on the one hand and excessive tension on the other. In most cases up to 200 m, a fixed main cable with provision for manual adjustment on the main cable should be adequate. Sighting bars or lines on the supports can be used to check that unloaded and working sags properly reflect the design tensions. For long periods between use, fixed cables may be slackened after use by a few turns of a screw adjuster and re-tensioned before use, using sighting bars to achieve the appropriate initial sag. It is important not to over tension the cable. Operating with a sag of 1% of span, for example, doubles the loading on the system compared with operation at 2%.

It is very convenient to have a system of automatic tensioning using counter weights, steel springs or gas springs, to deal with extreme conditions or in situations where sighting bars cannot be used. However, such systems are more appropriate if the installation will be in regular and continuous use and subject to adequate maintenance of the mechanical parts. Unfortunately, gauging sites are often unused for long periods and the more complex the installation the greater likelihood of malfunction after a period of inactivity. In this case steel springs or counterweights may be more appropriate than gas springs.
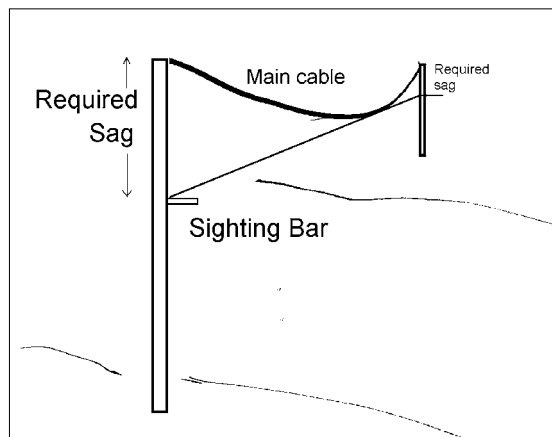
Figure 8.2a:
Sighting box to check cable sag

Traversing cable

A new traversing cable will also stretch during initial use and it may be necessary to shorten the cable at some stage and re-make the termination, depending on the tensioning system chosen. Spring tensioning built into the traveller, spring loaded deflector pulley or manual adjustment may be used to adjust the cable operationally.

Traveller

All track wheels, pulley wheels and guide rollers should be close fitting to prevent ropes from coming off the pulley and becoming snagged between pulley cheek and traveller side plates. The main track wheel and sounding cable pulley grooves should have a bottom radius to suit the cable. The track wheel radius should not be too small for the selected main cable diameter. If more than one track wheel is provided, track wheels should be arranged to equally share the load in operation. For example, if two small track wheels are used, the traveller should be articulated.

Electrically operated winch

An electrically operated winch must be capable of conversion to manual operation to allow recovery of equipment in the event of power failure. Electrical controls should provide "soft start" to prevent shock loading, "current limit" and "stall detect" to deal with accidental overload. A torque limiter should also be specified for the traverse drive system, so that the traversing cable will slip in a controlled manner should the sinker and current meter assembly become caught up in floating debris. This has the additional advantage of limiting the load on the cableway and causing the traveller and point of suspension to move towards the bank and out of the main current. This is preferable to inserting a shear link or weak link into the cableway system. A shear link parting may introduce a shock recoil loading with unpredictable results. It is important that the winch is able to deal with accidental overload without the intervention of the operator.

Manual winch operation

The gearing of the winch should be appropriate to the sinker-weight used. Heavy sinker-weights will require high gearing with correspondingly slow pay-out rates. An automatic load activated brake should be specified to prevent the winding handle from recoiling under the action of the load and the torque limiter should also continue to act in manual mode.

8.3   EXAMPLES

8.3.1   INTRODUCTION

In this Chapter examples are worked out, including:

- Cable design, (Sub-section 8.3.2)
- Support design, (Sub-section 8.3.3) and
- Foundation design (Sub-section 8.3.4)

In the examples the following design values have been assumed, (see for unit conversion Appendix 8.1):

- **Temperatures** range from 0 to 60^oC (as per maps in Appendix A and B of IS 800 – 1984)
- **Coefficient of expansion for steel** α_s = 1.25 x 10^{-5} K^{-1} ($^oC^{-1}$)
- **Wind load** (p):
  - recommended wind load p = 120 kg/m^2 (= 1177 Pa) as per IS:5228 – 1969
  - for cyclone prone areas in Orissa and Kathiawar (Gujarat) the wind load has to be derived from a wind velocity v_{wind} = 150 km/hour as follows (ref. V N Vazirani and M M Ratwani, "Steel Structures", Khanna Publishers, Delhi, 1997, page 337):

    $p = 0.06 \times v_{wind}^2$ (8.3)

    where: p   = wind load (Pa)

    v_{wind} = wind velocity (km/hour)

    From this it follows for the cyclone prone areas a design wind load of

    p = 1350 Pa (= 138 kgf/m^2), which is 15% higher than the value given above

- **Self weights of track cables** : (as per UK manufacturers)
- 16 mm cable, self-weight = 0.945 kg/m^1 or 9.27 N/m^1. (At the time of design values recommended by Indian manufacturers have to be used)
- 14 mm cable, self-weight = 0.723 kg/m^1 or 7.09 N/m^1. (At the time of design values recommended by Indian manufacturers have to be used)

8.3.2   CABLE SIZES

Some examples of the evaluation and selection procedure are given below, using nominal but realistic parameters to illustrate the procedure. Appendix 8.4 demonstrates the application to two sites for which some data has been made available.

The new (draft) ISO 4375 provides tables to guide the cable selection for various suspension loads, giving an estimate of the initial sag required to produce a working sag of 2% of span. The estimate of the load in the main cable at the breaking point of the suspension cable takes into account the elongation in the cable and the corresponding increased sag. The ISO tables have been augmented with estimates of loading, sag and factors of safety for working loads of 100 kg (Appendix 8.2) and150 kg (Appendix 8.3) over spans ranging from 20 to 200 m for ∅14 mm and ∅16mm wire rope. All calculations are based on the assumption that the cableway has no automatic tensioning device.

The following cases are considered:

a)   span = 200 m and sinker weight = 150 kg (1,472 N)
b)   span =150 m and sinker weight = 150 kg (1,472 N)
c)   span = 100 m and sinker weight = 100 kg   (981 N)

Case a: 200 m span - sinker 150 kg

Assumed design conditions

Span = 200 m
Working sag 2% of span = 4 m
Sinker weight = 150 kg (1,472 N)
Required factor of safety on main cable = 2
Height of column base above peak water level 2 m
Clearance required for equipment 1 m

Height of supports H_{col}

H_{col} = D+C-H_{bfl} = 4 + 1 - 2 = 3m   (see equation 8.2)

Current meter suspension cable

For a maximum sinker weight of 150 kg (1472 N), a 3.2 mm stainless steel coaxial suspension cable with a break strength 710 kg (6,965 N) will provide a factor of safety of 4.73 (see also Sub-section 8.2.2).

Main Cable

Given the design conditions following from the suspension cable, a $\varnothing$**16 mm**, galvanised steel, right hand ordinary lay wire rope, minimum break-strength 153,036 N, set up to an initial sag of 0.98% of the span to arrive at a working sag of 2% (see Appendix 8.3) will produce:

Initial tension

Cable tension due to self-weight of $\varnothing$16 mm cable
at an initial sag of 0.98% of span, see equation (2.1):
       Horizontal Tension T_h= $wS^2/(8D)$ (N)
       Actual tension   T = $T_h \sqrt{1+(4D/S)^2}$ (N)
where:
w = weight of cable $\varnothing$16 mm = 9.27 N/m
S = span is 200 m
D = sag at 0.98% of S =0.0098x200=1.96 m
   (see also Appendix 3, Table A1.b)

    T_h = 9.27x200^2/(8x1.96)=   23,648 N
    T = $T_h \sqrt{1+(4x1.96/200)^2}$=23,648x1.001=   23,666 N

FoS = break-strength/T = 153,036/23,666=   6.47

Working tension

Cable tension is due to self-weight ∅16 mm cable and
sinker weight at a working sag of 2% of span, see eq. (8.1)

Horizontal Tension $T_h = wS^2/(8D) + PS/(4D)$ (N)
Actual tension $T = T_h \sqrt{1+(4D/S)^2}$ (N)

where:
w = weight of cable ∅16 mm = 9.27 N/m
P = suspended load in N, 150 kg sinker weight = 1,472 N
S = span is 200 m
D = sag at 2% of S =0.002x200=4 m

T_h = 9.27x200²/(8x4)+1472x200/(4x4)=   29,988 N
T = $T_h \sqrt{1+(4x4/200)^2}$=29,988x1.003=   30,084 N

These values are also listed in Appendix 8.3.
(Small deviations with table values are caused by rounding off)

FoS = break-strength/T = 153,036/30,084=   5.09

Ultimate tension

Ultimate cable tension is due to self-weight ∅16 mm cable
and a load equal to the breaking-strength of suspension
at an ultimate sag of about 3.7% of span, see eq. (8.1)
4
Horizontal Tension $T_h = wS^2/(8D) + PS/(4D)$ (N)
Actual tension $T = T_h \sqrt{1+(4D/S)^2}$ (N)

where:
w = weight of cable ∅16 mm = 9.27 N/m
P = suspended load = break-strength of susp. cable = 6,965 N
S = span is 200 m
D = ultimate sag is 7.33 m (Appendix 3)

T_h = 9.27x200²/(8x7.33)+6965x200/(4x7.33)=   53,834 N
T = $T_h \sqrt{1+(4x7.33/200)^2}$=53,834x1.011=   54,409 N

These values are also listed in Appendix 8.3.
(Small deviations with table values are caused by rounding off)

Factor of safety for cable
FoS = break-strength/ultimate tension = 153,036/54,409 =   2.81

Permanent extension of rope
0.25%xS=0.0025 x 200 =   0.50 m

Thermal expansion of rope
α_sxSxΔT = 0.0000125x200x60 =   0.15 m

where: α_s = 1.25 x 10^{-5} $^oC^{-1}$, see Sub-section 8.3.1

ΔT= temperature range = 60 ^oC, see sub-section 8.3.1

For design of support column:

• Horizontal component of ultimate tension T_h =   53,834 N

• Moment on base of fixed support

H_{col}x$T_{h(ultimate\ tension)}$ = 3x53,834 =   161,502 Nm

Case b: 150 m span – sinker 150 kg

Assumed design conditions

Span = 150 m
Working sag 2% of span = 3 m
Sinker weight = 150 kg (1,472 N)
Required factor of safety on main cable = 2
Height of column base above peak water level 2 m
Clearance required for equipment 1 m

Height of supports H_{col}

H_{col} = D+C-H_{bfl} = 3 + 1 - 2 = 2 m   (see equation 8.2)

Current meter suspension cable

For a maximum sinker weight of 150 kg (1472 N), a 3.2 mm stainless steel coaxial suspension cable with a break strength 710 kg (6,965 N) will provide a factor of safety of 4.73 (see also Section 8.2.2).

Main Cable

Given the design conditions following from the suspension cable, a ∅**16 mm**, galvanised steel, right hand ordinary lay wire rope, minimum break-strength 153,036 N, set up to an initial sag of 0.87% of the span to arrive at a working sag of 2% (see Appendix 8.3 will produce:

(computational procedure is similar to case a; instead the values have now been taken from Appendix 8.3 where available)

Initial tension
With initial sag of 0.87% of S = 0.0087x150 = 1.31 m
    Horizontal Tension T_h= 9.27x150^2/(8x1.31)=   19,902 N
    Actual tension   T = 19,902x1.001=   19,914 N

FoS = break-strength/T = 153,036/19,914 =   7.68

Working tension

From Appendix 3
    Horizontal Tension T_h=   27,086 N
    Actual tension   T =   27,173 N

FoS = break-strength/T = 153,036/27,173 =   5.63

Ultimate tension

From Table
    Horizontal Tension T_h=   51,710 N
    Actual tension   T =   52,312 N

Factor of safety for cable
FoS = break-strength/ultimate tension = 153,036/52,312 =   2.93

Permanent extension of rope
0.25%xS=0.0025 x 150 =   0.38 m

Thermal expansion of rope

$\alpha_s \times S \times \Delta T = 0.0000125 \times 150 \times 60 =$ 0.11 m

For design of support column:

• Horizontal component of ultimate tension T_h =   51,710 N

• Moment on base of fixed support

$H_{col} \times T_{h(ultimate\ tension)} = 2 \times 51,710 =$ 103,420 Nm

Case c: 100 m span – sinker 100 kg

Assumed design conditions

Span = 100 m
Working sag 2% of span = 2 m
Sinker weight = 100 kg (981N)
Required factor of safety on main cable = 2
Height of column base above peak water level 2 m
Clearance required for equipment 1 m

Height of supports H_{col}

H_{col} = D+C-H_{bfl} = 2 + 1 - 2 = 1 m (in this case, H_{col} = 2 m would provide some headroom under the cableway)

Current meter suspension cable

For a maximum sinker weight of 100 kg (981 N), a 3.2 mm stainless steel coaxial suspension cable with a break strength 710 kg (6,965 N) will provide a factor of safety of 7.1 (see also Sub-section 8.2.2).

Main Cable

Given the design conditions following from the suspension cable, a $\varnothing$**14 mm**, galvanised steel, right hand ordinary lay wire rope, minimum break-strength 116,739 N, set up to an initial sag of 0.80% of the span to arrive at a working sag of 2% (see Appendix 8.2) will produce:

(computational procedure as for case a, instead the values have now been taken from Appendix 8.2 where available)

Initial tension
With initial sag of 0.80% of S = 0.0080 x 100 = 0.80 m and
self-weight of $\varnothing$14 mm cable of 7.09 N/m
Horizontal Tension T_h= 7.09 x 100^2/(8 x 0.80)=   11,078 N
Actual tension   T = 11,078 x 1.001 =   11,084 N

FoS = break-strength/T = 116,739/11,084 =   10.5

Working tension

From Table
Horizontal Tension T_h=   16,691 N
Actual tension   T =   16,744 N

FoS = break-strength/T = 116,739/16,744 =   6.97

Ultimate tension

From Table
     Horizontal Tension T_h=   43,349 N
     Actual tension   T =   44,006 N

Factor of safety for cable
FoS = break-strength/ultimate tension = 116,739/44,006 =   2.65

Permanent extension of rope
0.25%xS=0.0025 x 100 =   0.25 m

Thermal expansion of rope
α_sxSxΔT = 0.0000125x100x60 =   0.08 m

For design of support column:

- Horizontal component of ultimate tension T_h **=**   43,349 N

- Moment on base of fixed support
H_{col}x$T_{h(\text{ultimate tension})}$ = 2x43,349 =   86,698 Nm

NOTE: The breaking strengths and other information are typical for galvanised steel right hand lay wire rope of the diameters specified and may be used as a guide. **However, the cable/wire rope manufacturer's actual specifications should be used in the design or checked prior to construction to make sure they comply with the design assumptions.**

It can be seen from the above that a 16 mm diameter wire rope is suitable in examples (a) and (b), and a 14 mm is adequate in the case of example (c). In each case, once the permanent extension has been taken up, adjustment to allow temperature variation is small and could be accommodated by a screw adjuster alone. However, it depends upon being able to sight through between reference bars to determine the correct sag and it is not always possible to do this easily with long spans for many reasons including the support height. A spring or counterweight system would not be so limited and would remove the need to make regular adjustments. It will depend on site conditions whether this is a matter of convenience rather than necessity.

8.3.3   SUPPORT DESIGN

Assuming supports of height 3 m and all other parameters as shown above, the support could be made from a standard universal column, cast directly into the concrete foundation.

The force on the top of the support can be resolved into a horizontal force at right angles to the river and a force acting in the direction of the sag, i.e. with no deflection of the sinker, this component would be vertical, downwards. For the purpose of calculating the lateral loading on the support it is assumed that the suspension/ sounding cable is horizontal at breaking point and the force is applied horizontally, in the direction of flow as the worst case.

Example:

Continuation of case a: 200 m span – sinker 150 kg

Perpendicular to river

Horizontal loading applied by main cable = 53,834 N

Moment on support M_y = 161,502 Nm

Allowable stress for steel σ_s = 165 N/mm^2

Required section modulus

$Z_y = M_y/\sigma_s = (161,502 \times 10^2) / (165 \times 10^2)$ = 979 cm^3

Parallel to river

Assuming that the main cable is being pulled horizontally in the direction of flow under ultimate loading, a lateral loading of $1/2 \times 6,965$ N = 3,483 N ($1/2 \times 710$ kg = 355 kg) would be applied to the top of the support.

Moment on support $M_x = 3483 \times 3$ = 10,448 Nm

Required section modulus

$Z_x = M_x/\sigma_s = (10,448 \times 10^2)/(165 \times 10^2)$ = 63 cm^3

Hence suitable is :   Universal Column 254x254 @ 89 kg/m,
with Z_y= 1100 and Z_x= 378 cm^3

If access or handling is difficult, a lighter support could be designed to be bolted together on site. If backstays are permitted a much lighter and simpler support could be used. A simple strutted support is shown in Figure 8.3.

8.3.4   FOUNDATIONS

Design of the support foundations is highly dependent on ground conditions and therefore very site specific. However, a case design for 200 m span supports is further illustrated with no back stay. Tower and base are designed to withstand moments. A **254x254 @ 89 kgf/m Universal Column** is adopted, see Section 3.3.

The moment due to ultimate tension on yy-axis M_{yy} = 161.5 kNm

The moment due to hor. force parallel to flow on xx-axis M_{xx} = 0.4 kNm

Adopt foundation block 2 m wide (parallel to flow) x 4 m long x 2.5 m deep

Taking M20 concrete: weight = 2x4x2.5x24 kN/m3 = 480 kN

Load due to cableway = (89x9.81x3+0.5x6,965+ 9.27x100) =
(2,619+3483+927) = 7029 N = 7 kN

Vertical component of ultimate tension = $4D/S \times T_h$ =
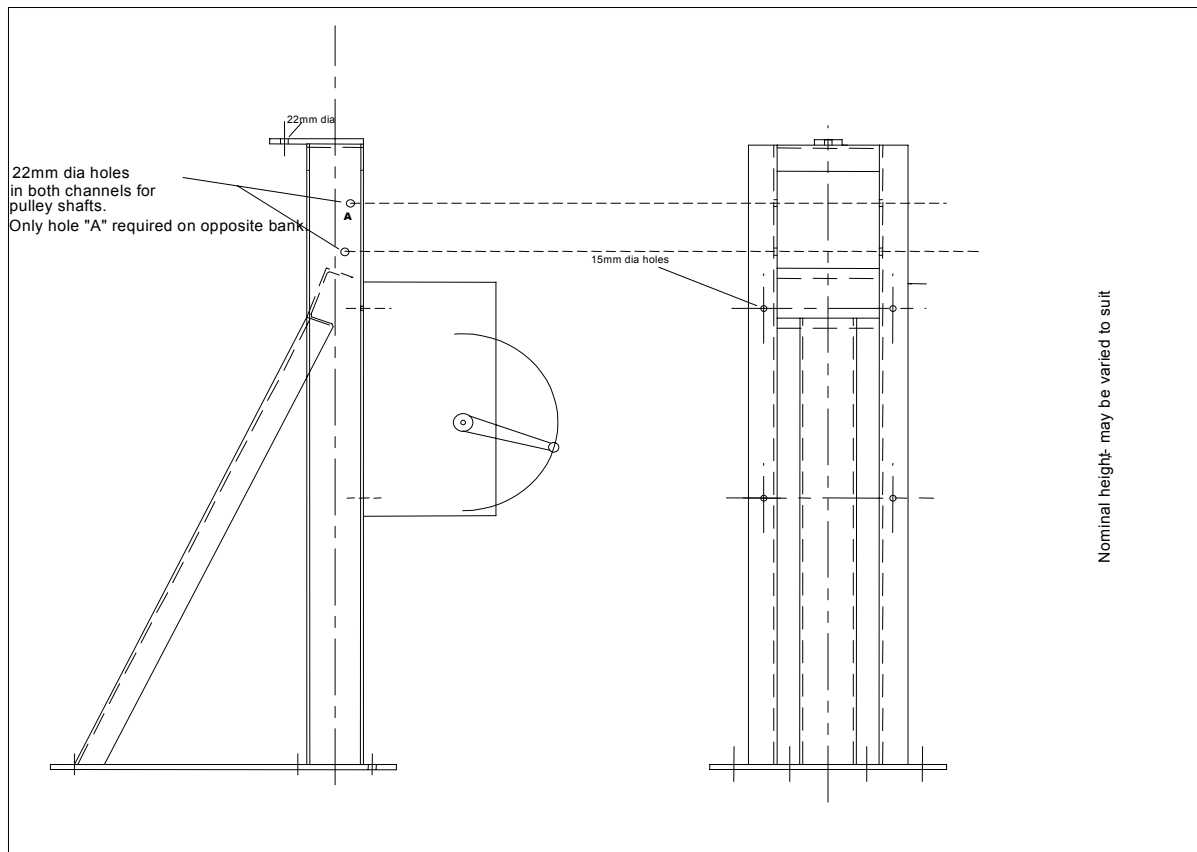
Figure 8.3   Strutted Column and baseplate for bolting on to

4x7.33/200x53,834 = 7,892 N =	8 kN
Total vertical load maximum = 480+7+8	495 kN
Average soil load = 495/8 =	62 kPa

$Z_{yy} = 1/6x2x4^2 = 5.33 \ m^3, \ Z_{xx} = 1/6x4x2^2 = 2.67 \ m^3$

Stress due to Myy is 161.5/5.33 =	30.3 kPa
Stress due to Mxx is 10.4/2.67 =	3.90 kPa
Maximum pressure on base soil = 62 + 30.3 + 3.9 =	96.2 kPa (9.8 tf/m^2)
Minimum pressure on base soil = 62 – 30.3 – 3.9 =	27.8 kPa (2.8 tf/m^2)

The resultant is within middle third and the block is stable and the bearing loads are as above and acceptable. (Sub-strata bearing capacity needs to match with permissible value of site)

Horizontal forces on the foundation block = 53,834 N = 53.8 kN

Vertical load including block weight (unsubmerged) =   495 kN

Neglect cohesion between base and soil and consider soil friction @ 0.3

Frictional resistance 495x 0.3 = 149 kN or FoS against sliding = 149/53.8 = 2.8

Consider the case when water covers the lower 0.5 metres of foundation block

Reduced weight (495 – 2x4x0.5x9.81) = 451 kN

Frictional resistance 451 x 0.3 = 135 kN or FoS against sliding = 135/53.8 = 2.5

The universal column and an adequately designed base have to be embedded into concrete for transferring moments by shear forces. This has to be designed as part of detailing.

Due consideration should be given to river training and bank protection.

Draft Specification

1. A cableway system with bank-side winch is required to operate over a span of (xxx m), with a sinker of yyy kg). *(values to be inserted as required)*

2. The working sag shall be 2% of the span.

3. The current meter suspension cable shall have a breaking strength of at least 5 times the sinker weight.

4. The main cable, fixings, supports and foundations shall be capable of withstanding a load at mid-span equal to twice the load on the current meter suspension cable or the release load of the torque limiter. Cable terminations shall be 100% efficient. Test certificates shall be provided for all ropes, rigging screws and shackles used. Shear links are to be avoided. The supports shall be designed on the assumption that there is no automatic tensioning device, so that the system may operate with or without automatic tensioning.

5. The winch shall provide interlocked traversing and sounding modes, with a loop traversing cable and a spooled sounding cable. It shall incorporate a load limiter to allow the traverse drive system to slip in the event of accidental overload. The winch shall include current meter connections to the coaxial signal cable via a slip ring or similar device and provide depth and distance counters, in metres, displaying 2 decimal places.

6. The winch should be electrically driven but fully convertible to manual operation in the event of power failure. The input power requirement shall not exceed (220V, 2kVA) and shall be 50Hz, single phase.

7. The motor control system shall provide "soft start" to minimise shock loading, current limiting and stall detection so that the motor will be shut down in the event of accidental overload.

8. Winch motor controls shall be interlocked push buttons, preferably on a pendant. It shall be necessary to maintain the pressure on the control button for the motor to run. Releasing the button shall cause the motor to stop. An emergency stop button, with key release, shall be provided in the power supply line.

9. The system shall fail safe in the event of power failure.

10. In manual operation, the operator shall be protected by a load activated brake, automatic in action, so that the handle cannot be driven by the load.

11. All traveller track wheels, pulley wheels and guide rollers shall be close fitting to prevent ropes coming off the pulleys and becoming snagged. The main traveller track wheel and sounding cable pulley grooves shall have a bottom radius to suit the cable. The track wheel radius shall be appropriate for the selected main cable diameter. If more than one track wheel is provided they should be arranged so as to equally share the load in operation. In operation, the traveller shall be captive on the main cable and the sounding cable shall be captive on the traveller. The return span of traversing cable shall pass through roller guides on the traveller.

12. Guidance on cableway design given has to be perused to appreciate the details from site by survey and investigation.

APPENDIX 8.1:   UNITS

In this guideline generally the metric or SI unit system is used. This includes:

Quantity	SI unit
Mass	kilogram (kg)
Force	Newton (N)
	kilo-Newton's (kN)
Weight	Newton (N) = mass (kg) x 9.81 m/s^2
Stress	Pascal (Pa) = Newton/m^2
	Mega-pascal (MPa) = 10^6 Pa

Besides the metric units for force and stress, the Newton (N) and Pascal (Pa) also the kilogram-force (kgf) and kgf/m^2 is used:

1 kgf = 9.81 N   1 N = 0.1019 kgf

1 kgf/m^2= 9.81 Pa   1 Pa = 0.1019 kgf/m^2

1 kgf/cm^2= 98,100 Pa   1 N/mm^2 = 1 Mega-pascal (MPa)

1 kgf/cm^2= 0.0981 MPa   1 N/mm^2 = 1 MPa = 10.19 kgf/ cm^2

APPENDIX 8.2:   CABLE WAY SET UP 1

Cableway set up to achieve a sag of 2% with a working load of 100kg

Rope Dia	Span	Initial sag	Initial sag	Factor of Safety	Tension	Horizontal Load	Factor of Safety	Tension	Horizontal Load
					Under normal conditions		Failure of suspension cable		
	metres	%	metres		N	N		N	N
14mm	20	0.29	0.06	8.9	13147	13105	2.8	41623	40979
14mm	30	0.40	0.12	8.6	13612	13568	2.8	41929	41284
14mm	40	0.48	0.19	8.3	14063	14019	2.8	42227	41580
14mm	50	0.56	0.28	8.0	14504	14457	2.7	42520	41871
14mm	60	0.62	0.37	7.8	14963	14915	2.7	42820	42170
14mm	70	0.67	0.47	7.6	15414	15365	2.7	43118	42467
14mm	80	0.72	0.58	7.4	15855	15804	2.7	43412	42758
14mm	90	0.76	0.69	7.2	16290	16238	2.7	43705	43049
14mm	100	0.80	0.80	7.0	16744	16691	2.7	44006	43349
14mm	110	0.84	0.92	6.8	17189	17134	2.6	44299	43641
14mm	120	0.87	1.05	6.6	17638	17582	2.6	44600	43940
14mm	130	0.90	1.17	6.5	18080	18022	2.6	44901	44239
14mm	140	0.93	1.30	6.3	18521	18462	2.6	45199	44536
14mm	150	0.96	1.43	6.2	18963	18902	2.6	45500	44834
14mm	160	0.98	1.57	6.0	19412	19351	2.5	45804	45137
14mm	170	1.00	1.71	5.9	19860	19797	2.5	46109	45441
14mm	180	1.02	1.84	5.7	20304	20239	2.5	46412	45742
14mm	190	1.05	1.99	5.6	20750	20684	2.5	46718	46046
14mm	200	1.06	2.13	5.5	21190	21122	2.5	47022	46349
16mm	20	0.50	0.10	11.4	13401	13358	3.4	44605	44002
16mm	30	0.61	0.18	10.9	14008	13963	3.4	45016	44411
16mm	40	0.70	0.28	10.5	14600	14554	3.4	45416	44808
16mm	50	0.77	0.38	10.1	15188	15140	3.3	45813	45203
16mm	60	0.83	0.50	9.7	15773	15723	3.3	46209	45596
16mm	70	0.87	0.61	9.4	16358	16305	3.3	46605	45990
16mm	80	0.92	0.73	9.0	16942	16888	3.3	47002	46385
16mm	90	0.96	0.86	8.7	17523	17467	3.2	47398	46779
16mm	100	0.99	0.99	8.5	18102	18044	3.2	47795	47174
16mm	110	1.02	1.12	8.2	18686	18626	3.2	48195	47571
16mm	120	1.05	1.26	7.9	19271	19209	3.1	48597	47971
16mm	130	1.07	1.40	7.7	19853	19789	3.1	48999	48371
16mm	140	1.10	1.54	7.5	20430	20365	3.1	49400	48769
16mm	150	1.12	1.68	7.3	21008	20941	3.1	49802	49169
16mm	160	1.14	1.83	7.1	21596	21528	3.0	50210	49575
16mm	170	1.16	1.98	6.9	22179	22108	3.0	50617	49979
16mm	180	1.18	2.12	6.7	22762	22690	3.0	51025	50386
16mm	190	1.20	2.28	6.6	23339	23264	3.0	51432	50790
16mm	200	1.21	2.43	6.4	23922	23845	3.0	51844	51199

APPENDIX 8.3: CABLEWAY SET 2

Cableway set up to achieve a sag of 2% with a working load of 150kg

Rope Dia	Span	Initial sag %	Initial sag m	Under normal conditions			Failure of suspension cable		
				Factor of Safety	Tension	Horizontal Load	Factor of Safety	Tension	Horizontal Load
	metres	%			N	N		N	N
16mm	50	0.46	0.23	7.2	21344	21276	3.2	48123	47543
16mm	60	0.52	0.31	7.0	21924	21854	3.2	48538	47955
16mm	70	0.58	0.40	6.8	22512	22441	3.1	48955	48370
16mm	80	0.62	0.50	6.6	23104	23031	3.1	49375	48788
16mm	90	0.67	0.60	6.5	23669	23594	3.1	49785	49196
16mm	100	0.71	0.71	6.3	24251	24173	3.0	50203	49611
16mm	110	0.75	0.82	6.2	24829	24750	3.0	50620	50026
16mm	120	0.78	0.93	6.0	25419	25338	3.0	51042	50446
16mm	130	0.81	1.05	5.9	26003	25920	3.0	51464	50866
16mm	140	0.84	1.18	5.8	26586	26502	2.9	51886	51286
16mm	150	0.87	1.30	5.6	27173	27086	2.9	52312	51710
16mm	160	0.89	1.43	5.5	27744	27656	2.9	52732	52128
16mm	170	0.92	1.56	5.4	28323	28232	2.9	53156	52549
16mm	180	0.94	1.69	5.3	28904	28812	2.9	53583	52974
16mm	190	0.96	1.83	5.2	29492	29398	2.8	54012	53402
16mm	200	0.98	1.96	5.1	30074	29978	2.8	54438	53825

APPENDIX 8.4:   SAMPLE CALCULATIONS

Sample calculations have been carried out using data contained on cross sections supplied by Maharashtra State Surface Water Department. Column heights have been calculated in relation to ground level, making no allowance for working platforms or other building works. The minimum support height relates to the higher of the two supports.

Maximum sinker weight has been estimated using mass (kg) = 5 x mean velocity (m/s) x depth (m), although this is a little artificial for the River Shivan as no slope information was provided to allow a proper estimate of velocity.

Steel sections have been selected from a table of British Standard sections, assuming a permissible stress of 165 N/mm^2 in compression and tension.

The support height may be affected by other factors not considered here, such as the site of the winch installation.

In the absence of detailed information, assumptions have been made to allow the sample calculations to proceed.

River Telenadu at Parsewada

Estimates of discharge and velocity using slope-area method
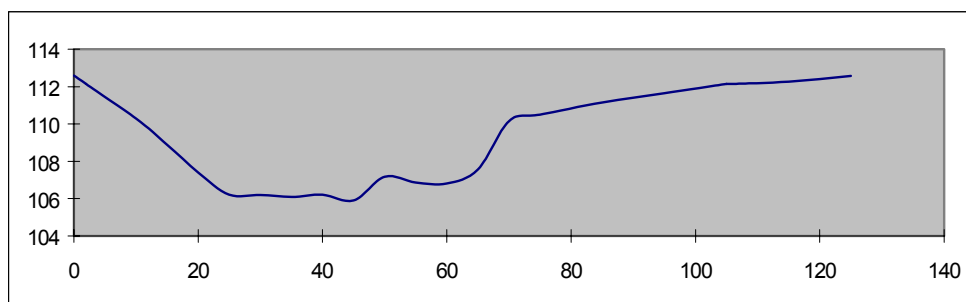
Cross section up to max flood level

Chainage	Elevation	delta x	delta y	Depth	Sigma a calc A	Sigma pw calc P
0	112.6					
2	112.15	0	0	0		
10	110.3	8	1.85	1.85	7.44	8.25
15	108.88	5	1.42	3.27	12.80	5.20
20	107.4	5	1.48	4.75	20.05	5.21
25	106.21	5	1.19	5.94	26.73	5.14
30	106.19	5	0.02	5.96	29.75	5.00
35	106.1	5	0.09	6.05	30.03	5.00
40	106.22	5	-0.12	5.93	29.95	5.00
45	105.91	5	0.31	6.24	30.43	5.01
50	107.17	5	-1.26	4.98	28.05	5.16
55	106.87	5	0.3	5.28	25.65	5.01
60	106.81	5	0.06	5.34	26.55	5.00
65	107.56	5	-0.75	4.59	24.83	5.06
70	110.15	5	-2.59	2	16.48	5.63
75	110.5	5	-0.35	1.65	9.13	5.01
85	111.15	10	-0.65	1	13.25	10.02
95	111.65	10	-0.5	0.5	7.50	10.01
105	112.14	10	-0.49	0.01	2.55	10.01
105.8	112.15	0.8	-0.01	0	0.00	0.77
115	112.27					
125	112.57					
		103.81271		Area	341.14	105.50
				R	3.23	
				n	0.035 assumed	
				s	0.00204	
				Q	963 cumecs	
				velocity	2.8 m/s	

Estimated peak discharge   963 cumecs
Estimated mean velocity, peak discharge   2.8 m/s
Estimated Maximum sinker weight   88 kg (5 x mean velocity x depth)

River Telenadu at Parsewada

Span	150	m
Max Sinker Weight	100	kg

From Appendix 2 for 100kg sinker, a 14mm rope would provide a factor of safety of 2.6

Peak water level	112.15	m
Assuming Winch installed on right bank		
Level of support base (right bank)	112.57	m
Level of support base (left bank)	115.27	m
Height of right bank Base above peak water leve	0.42	m
Clearance for equipment	1	m
Working sag @ 2%	3	m
Minimum height of RB support	3.58	m	working sag + clearance - height of base above peak level
Level of cableway	116.15	m
height of support (left bank)	0.88	m

For Right Bank Support:

Maximum horizontal loading (from Table 1)	44834	N
Moment on RB support	160506	Nm	height of support * max horizontal loading
Required section modulus Zx	973	cm^3	Moment / allowable stress (165N/mm2)
Horizontal force in the direction of Flow	3482	
Moment on support parallel to flow	12466	
Required section modulus Zy	76	cm^3

Universal Column

254x254 x 89kg/m with Zx= 1099 cm^3 and Zy= 379 cm^3 would be suitable.

OR

Universal Beam

406x178 x 60kg/m with Zx= 1059 cm^3 and Zy= 125 cm^3 would be suitable.

For Left Bank Support:

Maximum horizontal loading (from Table 1)	44834	N
Height of left bank support	0.88	m
Moment on LB support	39454	Nm	height of support * max horizontal loading
Required section modulus Zx	239	cm^3	Moment / allowable stress (165N/mm2)
Horizontal force in the direction of Flow	3482	
Moment on support parallel to flow	3064	
Required section modulus Zy	19	cm^3

Universal Beam

203x133 x 30kg/m with Zx= 278 cm^3 and Zy= 52 cm^3 would be suitable.
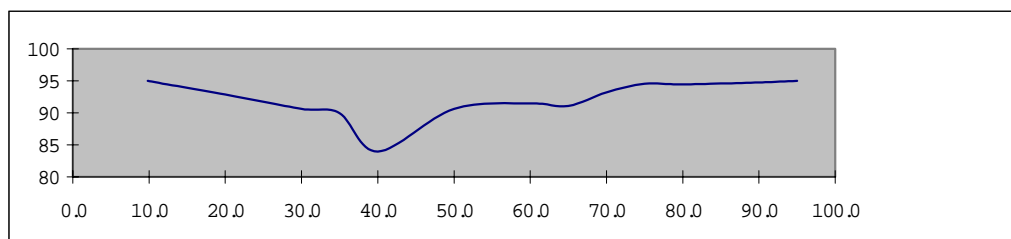
Range of Tensioning Device

Temperature Range - 0 to 60 deg C	60	Temperature range
Permanent extension of rope	0.375 m	0.0025 x Span
Thermal expansion of rope	0.1125 m	0.0000125 x span x temperature range

Rigging screw would be adequate

<u>River Shivan near Khamgaon</u>

Estimates of discharge and velocity using slope-area method

Cross section up to max flood level.

Chainage	Elevation	delta x	delta y	Depth	Sigma a calc A	Sigma pw calc Pw
0	97.85					
9.8	95	0	0	0		
10	94.95	0.2	0.05	0.05	0.00	0.18
20	92.895	10	2.055	2.105	10.78	10.21
30	90.66	10	2.235	4.34	32.23	10.25
35	89.94	5	0.72	5.06	23.50	5.05
40	83.94	5	6	11.06	40.30	7.81
50	90.62	10	-6.68	4.38	77.20	12.03
60	91.49	10	-0.87	3.51	39.45	10.04
65	91.07	5	0.42	3.93	18.60	5.02
70	93.23	5	-2.16	1.77	14.25	5.45
75	94.55	5	-1.32	0.45	5.55	5.17
80	94.45	5	0.1	0.55	2.50	5.00
90	94.75	10	-0.3	0.25	4.00	10.00
95	95	5	-0.25	0	0.63	5.01
100	95.25			Area	268.98	91.21
110	95.86			R	2.95	
120	97.66			n	0.035	
130	98.35			s	0.00204	
140	98.45			Q	714 cumecs	
				velocity	2.7 m/s	

NOTE:

No slope information was provided for this site. The slope for the Telenadu was used to complete
        the calculations and give a general indication of peak values.

Estimated peak discharge   714 cumecs
Estimated mean velocity, peak discharge   2.7 m/s
Estimated Maximum sinker weight   147 kg (5 x mean velocity x depth)

River Shivan at Khamgaon

Span	140 m	
Max Sinker Weight	150 kg	
From Appendix 3 for 150kg sinker, a 16mm rope would provide a factor of safety of 2.9		
Peak water level	95 m	
Assuming Winch installed on left bank		
Level of support base (left bank)	97.85 m	
Level of support base (right bank)	98.45 m	
Height of left bank Base above peak water lev	2.85 m	
Clearance for equipment	1 m	
Working sag @ 2%	2.8 m	
Minimum height of LB support	0.95 m	working sag + clearance - height of base above peak lev
To provide headroom under cable, use	1.8 m	
Level of cableway	99.65	
height of support (right bank)	1.2 m	

For Left Bank Support:

Maximum horizontal loading (from Table 2)	51286 N	
Moment on LB support	92315 Nm	height of support * max horizontal loading
Required section modulus Zx	559 cm^3	Moment / allowable stress (165N/mm2)
Horizontal force in the direction of Flow	3482	
Moment on support parallel to flow	6268	
Required section modulus Zy	38 cm^3	

Universal Column.

203x203 x 60kg/m with Zx= 581 cm^3 and Zy= 199 cm^3 would be suitable.

OR

Universal Beam

305x165 x 40kg/m with Zx= 559 cm^3 and Zy= 83 cm^3 would be suitable.

For Right Bank Support:

Maximum horizontal loading (from Table 2)	51286 N	
Height of right bank support	1.2 m	
Moment on RB support	61543 Nm	height of support * max horizontal loading
Required section modulus Zx	373 cm^3	Moment / allowable stress (165N/mm2)
Horizontal force in the direction of Flow	3482	
Moment on support parallel to flow	4178	
Required section modulus Zy	25 cm^3	

Universal Column.

203x203 x 46kg/m with Zx= 449 cm^3 and Zy= 151 cm^3 would be suitable.

OR

Universal Beam

254x146 x 37kg/m with Zx= 433 cm^3 and Zy= 72 cm^3 would be suitable.

Range of Tensioning Device

Temperature Range - 0 to 60 deg C	60	Temperature range
Permanent extension of rope	0.35 m	0.0025 x Span
Thermal expansion of rope	0.105 m	0.0000125 x span x temperature range

Rigging screw would be adequate

9 QUALITY ASSURANCE PLAN FOR DISCHARGE MEASUREMENTS USING BROADBAND ACOUSTIC DOPPLER CURRENT PROFILERS

9.1 ABSTRACT

The recent introduction of the Acoustic Doppler Current Profiler (ADCP) as an instrument for measuring velocities and discharge in the riverine and estuarine environment promises to revolutionize the way these data are collected by the U.S. Geological Survey. The ADCP and associated software, however, compose a complex system and should be used only by qualified personnel. Standard procedures should be rigorously followed to ensure that the quality of data collected is commensurate with the standards set by the Water Resources Division for all its varied activities in hydrologic investigations.

Note: this Chapter was taken from U.S. Geological Survey, Open File Report 95-701, by Stephen W. Lipscomb.

9.2 INTRODUCTION

The Acoustic Doppler Current Profiler (ADCP) is an electronic instrument developed and manufactured by RD Instruments of San Diego, California, that is used to measure water velocities. The instrument transmits acoustic signals into the water column. When the frequency of the transmitted signals is compared with the frequency of backscatter signals reflected off particles in the water, the velocity of the particles and, hence, the water, can be calculated.

ADCP's have been widely used by oceanographers to measure currents in the deep-sea environment. More recently, the technology has been adapted to the riverine and estuarine environment and promises to revolutionize the way many streamflow discharge measurements are made by the U.S. Geological Survey (USGS).

As ADCP technology has been refined, it has begun to gain acceptance as a viable method for obtaining velocity and discharge data. The primary advantages of making discharge measurements using the ADCP compared with a Price or other point-current meter are that (1) the time required to complete a measurement is reduced; (2) data can be collected throughout the water column and cross section rather than at discrete points; (3) taglines or other stationing devices are unnecessary because the instrument keeps track of distance traveled, provided the bed is stable; and (4) the instrument can be boat-mounted, thus eliminating the installation, maintenance, and liability of costly cableways.

The primary disadvantages of using the ADCP compared with a standard Price current meter are its high initial cost; inability to function in shallow water; complexity, requiring an in-depth understanding of the physics, electronics, and software of the system prior to use; and the frequent revisions to hardware, firmware, and software due to the newness of the technology. These revisions generally result in improvements in the accuracy of the data collected; however, they pose problems in dealing with data collected using earlier systems and software.

The Office of Surface Water has recognized the utility of the ADCP for many of the tasks performed by various Water Resources Division offices. However, as with any technology or methodology adopted by the Division for the collection and analysis of water resources data, it is critical that strict measures be implemented to ensure the quality and reproducibility of the data throughout our agency. The purpose of this report is to provide a quality assurance plan that, when implemented, will ensure that data collected with ADCP's meet existing accuracy standards and that the procedures used to collect the data are consistent throughout the Division. Specific objectives of the plan are to ensure that:

Field and office procedures associated with the use of ADCP's adhere to applicable Division policies and standards governing the collection, analysis, and reporting of surface-water data.

1. Personnel who collect and analyze the data are adequately trained in proper and acceptable ADCP operational procedures.
2. All activities related to the collection, analysis, and reporting of data collected using ADCP's are reviewed regularly for accuracy, completeness, and conformance to Division standards.
3. Any deficiencies discovered during review are addressed immediately and corrective action is taken.

9.3   TRAINING REQUIREMENTS

Because of the complex nature of the ADCP and associated software, at least one person in the field crew is required to have formal training in the operation and maintenance of ADCP's. Acceptable training is available through RD Instruments and the USGS National Training Center in Lakewood, Colorado, or equivalent training can be provided by personnel within the Water Resources Division. Eventually, qualified personnel within Districts will be designated as trainers.

Reference materials on the theory, operation, and maintenance of the ADCP are available from the manufacturer in the form of user's manuals for both the ADCP and the associated software package "Transect" (RD Instruments, current versions). These manuals are the primary source of information for the setup and operation of the instrument and peripheral equipment. The manuals also contain sections dealing with configuration file setup and the use of Transect for system configuration, data acquisition, playback, output, and data manipulation. A report on field procedures for ADCP's (M.R. Simpson, U.S. Geological Survey, written commun., 1995) is the primary source of information on accepted Divisionwide procedures for collecting velocity data and making discharge measurements.

Field personnel responsible for data collection using the ADCP must be completely familiar with these reference materials. Procedures for data collection and ranges of acceptable conditions described in these documents must be strictly adhered to.

9.4   PRE-FIELD OFFICE PROCEDURES

To avoid delays in the field and to ensure that the data collected are complete and of the highest quality, some preparations prior to departure are recommended. These preparations should include a determination as to whether the instrument is utilizing the most recent software and firmware upgrades and that the complete system, including the field computer and power supply, is operational.

Software and firmware revisions

Because of the newness of ADCP technology, there have been, and likely will continue to be, frequent upgrades to both software and firmware associated with the system. Many of these upgrades will result in only minor improvements to internal processing speed or data output capabilities and will not substantively affect the quality of discharge measurements made with the instrument.

Less frequently, major upgrades could affect the quality of measured velocity or discharge data. Upgrades of this type will require field testing at one of the validation sites described in the report by Morlock (in press) prior to Division acceptance, to ensure that the instrument is performing up to required standards. The Office of Surface Water will determine, prior to implementation of all software and firmware upgrades, whether field testing is necessary.

Before the ADCP is taken to the field, the most recent Division-accepted version of Transect should be installed on the primary and backup field computers. In addition, a backup of this software should

be copied to a diskette and kept with the computer in the event that the copy on the hard drive is inadvertently erased. In every office having an ADCP, a contact person will be assigned to obtain and distribute the most recent copies of Transect from the manufacturer. This person will be responsible for notifying users of software and firmware revisions and for installing firmware upgrades as they become available from the manufacturer. These upgrades can be installed either by computer download from RD Instruments via modem or by manually replacing chips within the instrument's circuit boards. If the latter approach is required, an electronic technician, trained in the procedures for opening the ADCP pressure case, should install the firmware upgrades in a static-free environment.

Pre-field inspection

A pre-field inspection must be made to ensure that the instrument is functioning correctly. This inspection consists of connecting the ADCP to a computer, preferably the one that will be used in the field, connecting a power supply to the deckbox, and powering the system up. The system then can be checked by entering the "Acquire" mode of the Transect software and attempting to "wake the system up." If any problems are encountered at this point, the ADCP and Transect user's manuals (RD Instruments, current versions) should be consulted.

Inclusion of all required cables and connectors should be ascertained and any spare parts that might be difficult to obtain in the field, such as fuses for the deckbox, batteries for the ADCP and the computer, cables, and diskettes for making backup copies of data files should be assembled as part of the pre-field inspection.

9.5   FIELD PROCEDURES

General field procedures for making discharge measurements with the ADCP are described in a report by M.R. Simpson (U.S. Geological Survey, written commun., 1995) and therefore will not be discussed here. However, certain precautions should be taken that will ensure the accuracy, precision, and completeness of the data collected.

9.5.1   VESSEL INSTALLATION

The ADCP is typically mounted on either side of a boat or in a well through the hull. The instrument should be rigidly attached in a vertical position so that the transducers are submerged at least 3 inches (8 cm) below the water surface. In rough water, the transducers may have to be lowered further to ensure that their heads stay submerged and that no cavitation occurs in their vicinity during the entire measurement. The mount used to attach the ADCP to the boat should be designed to withstand the combined forces of water velocity resulting from the boat's movement and the ambient stream velocity.

The mount should be designed so that the instrument can be quickly raised or rotated out of the water for moving from one site to another or for quickly traversing the cross section. The ADCP must not be mounted near steel or any other ferrous material that would affect the functioning of the internal flux-gate compass. For this reason, a boat with a steel hull should not be used and the instrument should be mounted as far as possible from any ferrous objects on the boat, such as an engine with a cast-iron block or heads. A rule of thumb is to keep the compass, located near the top of the ADCP pressure case, away from any ferrous object on the boat by at least the longest dimension of the object. For instance, if the longest dimension of a steel davit mounted on the boat is 4 feet, the ADCP should be mounted no less than 4 feet from the davit to avoid interference with the compass.

9.5.2   INSTRUMENT CHECK

After the ADCP is mounted and the required cables to the computer and power supply are connected, the instrument must be checked to ensure that all circuits and sensors are operating properly. Several

subroutines in BBTALK, which is a program provided as part of the ADCP software package, perform internal diagnostic and calibration tests on the ADCP and display its internal setup. These subroutines will test the circuits and sensors of the ADCP and, upon request, will write the results to a user-designated file. Specific procedures for initializing the self-test subroutines in BBTALK are detailed in a predeployment test procedures document (RD Instruments, current version) provided by the manufacturer with each ADCP or upon request and must be performed prior to each deployment of the ADCP. If the instrument fails to pass any of the self-test subroutines, the ADCP technical manual and Transect user's manual (RD Instruments, current versions) should be consulted. If the problem cannot be corrected, the manufacturer's field service representative should be contacted and corrective action taken. If the ADCP has failed any of the self-tests and data are collected, the data should be clearly marked as suspect and the nature of the test failure documented in the remarks section of the ADCP fieldnote sheet.

During the initiation of communication between the computer and the ADCP, a comparison of the configuration file and the ADCP setup is performed. If the ADCP fails to respond, or if any error messages are displayed, the ADCP and Transect user's manuals (RD Instruments, current versions) should be consulted. Many times, the problem can be solved by ensuring that all connections, including the power supply to the ADCP, are properly made.

Sometimes an error message will warn that the configuration file is not in agreement with the ADCP setup. Again, these problems usually can be solved by rechecking the configuration file for proper communication settings, transducer frequency, or head orientation (upward or downward looking). If the problem cannot be discerned by consulting the user's manual, a manufacturer's field service representative should be contacted and all problems related to failure of self-tests or error messages should be resolved before proceeding.

9.5.3   CONFIGURATION FILE SETUP

The configuration file must be matched to the physical conditions of the cross section. Such parameters as bin size, mode, bottom track and water pings per ensemble, and blanking distance require setting by a trained user to optimize the quality of data collected for the existing conditions. Proper setup of the configuration file is beyond the scope of this report but is addressed in a report describing discharge measurement procedures using ADCP's (M.R. Simpson, U.S. Geological Survey, written commun., 1995). That report provides specific details on selecting cross sections and setting up configuration files for a variety of conditions and should be consulted prior to collecting data.

Currently, the configuration file is stored on the computer separately from the files that contain the raw velocity data. Therefore, each raw data file must be linked in some way to the configuration file used during data collection so that the parameters used during playback and postprocessing of the raw data files are the same as those that were used during data acquisition. The raw data files and configuration file used for a specific measurement must be saved on the hard drive of the computer and copied to a diskette for a backup as soon as possible following completion of the discharge measurement. Space on an ADCP fieldnote sheet adopted by the Office of Surface Water has been designated for listing raw data filenames and associated configuration filenames.

9.5.4   DISCHARGE MEASUREMENTS

The ADCP discharge measurement procedures guide (M.R. Simpson, U.S. Geological Survey, written commun., 1995) describes procedures for making discharge measurements, including site selection criteria, configuration file setup, and postprocessing of raw data. That reference provides details for field use of ADCP's and postprocessing of data as accepted by the Office of Surface Water and should be adhered to rigorously. Any variation from the procedures described in that publication must be documented and reviewed before the data can be released.

Before discharge data are collected using the ADCP, preliminary information describing the site, date, personnel, equipment, and versions of software and firmware used should be entered on the ADCP fieldnote sheet. The remainder of this sheet should be completed as data are collected. Information related to the distance to riverbanks at the beginning and ending of individual transects and raw data and configuration filenames must be entered immediately. This information is critical to the accurate calculation of discharge, and any delays in entering it on the fieldnote sheet will increase the likelihood of errors.

Individual transects can be viewed as instantaneous discharge at a particular site. It is therefore necessary to average multiple transects to reduce variation due to turbulence and velocity surges. Averaging is analogous to the Division's policy of measuring velocity with a Price current meter over at least 40 seconds to minimize instantaneous fluctuations in stream velocities. In general, at least four transects must be made at each site to ensure a valid determination of discharge. More transects will be needed under certain conditions, such as extremely turbulent water. If any one of the first four transects differs from the mean discharge by more than 5 percent, it should first be evaluated to determine if there is any reason to justify discarding it. A determination to discard a single transect might be made on the basis of a bad ensemble; a "Made Good" distance that differs significantly from the total distance; anything that might have affected the velocity profiles during the measurement, such as boat traffic in the vicinity; or any number of other factors. If a transect is discarded, another should be made so that the discharge measurement is calculated from the average of at least four transects. If there is no justifiable reason for discarding the transect, four additional transects should be made and all of them, including the outlier, should be averaged to determine discharge.

During a transect, the ADCP measures the velocity in the water column relative to the movement of the vessel to which it is attached. The vessel velocity relative to the channel bed is also measured and is used to calculate the actual water velocity. This calculation assumes a fixed bed. If, however, the bed is in motion, the calculation of actual water velocity will be in error. Therefore, assessment of bed movement is necessary before a discharge measurement can be made. Bed movement can be assessed by anchoring the vessel to the bed or to a fixed object such as a bridge, or by holding the boat at a fixed location within the channel while a series of ensembles is collected. If the bed is stable, the Shiptrack display will indicate no significant movement of the vessel after a period of a few minutes. If, however, the bed is moving, it will be reflected in the Shiptrack display by an apparent gradual movement of the vessel in an upstream direction. If this movement is significant relative to the average water-column velocity, an alternate site should be sought. Often, bedload movement varies considerably from one point to another across a channel. As a result, it may be necessary to check for bed movement at several locations across the channel to ensure that the bed is stable throughout.

After the discharge measurement has been made, each raw data file should be reviewed using the Playback mode in Transect to ensure that the data are complete and do not include any bad ensembles, that depths and velocities do not exceed the prescribed limits set for the instrument in use, and that no velocity spikes are recorded because stream velocity equaled the ambiguity velocity. The instrument limitations and ambiguity velocity for specific instruments and configurations are discussed in a report by M.R. Simpson (U.S. Geological Survey, written commun., 1992). Measurements of conditions that exceed instrument limitations or that contain velocity spikes should not be included in the determination of a final discharge value. If a raw data file contains bad velocity ensembles or velocity spikes, the measurement should be repeated until at least four complete measurements with no bad data have been obtained.

Once a sufficient number of transects have been made and reviewed for completeness, the nearshore discharge estimate section on the back of the fieldnote sheet must be completed to determine total discharge, including the unmeasured sections near each bank. The Transect software contains a utility for making the nearshore estimates on the basis of the first and last valid velocities and depths measured. Procedures for using this function are provided in the Transect user's manual (RD Instruments, current version). Care should be taken to determine the direction of boat movement during collection of the ensemble that is used for determining the nearshore discharge. Boat direction determines whether a positive or negative sign should be assigned to the estimated discharge before the total discharge can be calculated.

A gage height from a staff or some other reference should be obtained before and after each measurement and entered into the space provided on the front of the fieldnote sheet. Otherwise, the gage heights corresponding to the time of each measurement should be obtained from the gage recorder and entered. A mean gage height and discharge then can be calculated and entered into the space provided on the upper part of the fieldnote sheet.

9.5.5   MEASUREMENT ASSESSMENT

An overall assessment of the mean discharge measurement should be made after completion of the transects composing the measurement. This assessment is based on a qualitative judgement of conditions encountered in making the measurement and a quantitative evaluation of the individual transects. Completeness of the measurement, in terms of the percentage of the total cross-sectional area measured, and overall measurement conditions must be assessed. Conditions such as turbulence, eddies, reverse flows, surface chop, and proximity of the instrument to ferrous objects, which, under certain circumstances, might affect the results of the measurements and velocity profiles, should be noted under the appropriate sections on the fieldnote sheet and used in assigning a rating for the measurement.

The mean and standard deviation of the discharge measurement and the coefficient of variation (CV), which is the ratio of the standard deviation to the mean discharge (s/Q), should be calculated. The CV then should be entered as a percentage in the space provided on the fieldnote sheet. The CV is a measure of the grouping of individual transects around their mean and is a useful statistic for making a quantitative assessment of the measurement. If the CV is greater than 5 percent, additional transects should be made. In some cases, one or more transects will be apparent outliers and will result in a large CV. If there is an obvious reason for the variation, the transect can be eliminated from the mean discharge calculation after the circumstances are noted on the fieldnote sheet. Additional transects may be needed if one or more is eliminated so that at least four are used for computing the average discharge. If the mean discharge varies from the applicable rating or shift curve by more than 5 percent, the measurements should be repeated.

The final discharge rating is assigned by circling the appropriate entry on the front of the fieldnote sheet. This rating is based on the qualitative evaluation of the measurement and the CV, which provides a quantitative measure of precision.

After the fieldnote sheet is completed, all raw data and configuration files resulting from the discharge measurements should be copied onto a disk-ette as a backup in case of damage to the computer's internal hard drive.

9.6   POST-FIELD OFFICE PROCEDURES

The ADCP should be inspected after it has been returned from the field to determine the condition of the transducers, pressure case, connectors, cables, and deckbox. Damage or undue wear to any of the components should be reported to the person responsible for maintenance of the instrument.

9.6.1   DISCHARGE MEASUREMENT REVIEW

All discharge measurements must be reviewed by a second qualified person in the office. Special attention should be given to the calculation of total discharge, including nearshore estimates. All calculations used to derive the mean discharge and gage height should be checked and the fieldnote sheet should be reviewed for accuracy and completeness. The reviewer then must initial the front of the fieldnote sheet.

9.6.2   DOCUMENTATION AND ARCHIVAL OF DISCHARGE MEASUREMENTS

Eventually, Transect will include an option to generate a standard output file that will contain the information in the configuration file and a summary of the discharge measurement. This file will include information similar to that on the standard 9 - 275 discharge measurement form -- width, depth, area, mean velocity, and discharge at incremental locations across the section. Until this option is available, the raw data should be processed and an ASCII output file generated that contains a summary of stationing and velocity data at approximately 30 equally spaced locations across the section.

Copies of the configuration, subsystem self-test, and the discharge measurement summary files should be printed and attached to the fieldnote sheet for filing as soon as practical after returning from the field. Information from these documents should be used to enter the measurement on the 9 - 207 discharge measurement summary form and into the computer database of discharge measurement summaries (HT - 1). In both the paper and computer summaries of the measurement, it should be noted in the remarks section that the measurement was made using an ADCP.

Generation of a paper copy of the raw data files is not practical because of their size and binary format. However, contained in these files is a significant amount of information, such as horizontal and vertical velocity vectors calculated for each ensemble, that should be retained. At this time, there is no accepted method for permanent archival of digital data files within the Water Resources Division. Until an acceptable archival method is approved, the raw data and configuration files should be copied to as permanent a storage medium as is available. Preferably, the files should be copied to some type of semipermanent storage medium such as CD-ROM. Eventually, these files could be transferred to a more permanent storage medium when the technology becomes available.

9.7   ANNUAL INSTRUMENT CHECK

Each ADCP must be checked annually by making a discharge measurement at a site where the ADCP-measured discharge can be compared with a known discharge derived from some other source. An example of such a site would be one where a stable stage-discharge relation with no significant shifting has been established over a period of several years. The site ideally would be chosen to minimize the amount of unmeasured sections near the banks or in shallows and should not be near any large steel structures, such as bridges, that might affect the ADCP's compass. The discharge obtained using the ADCP must be within 5 percent of the known discharge. If these measurements fail to agree with the known discharge, the ADCP must be returned to the manufacturer for further evaluation and calibration if necessary. These check measurements must be fully documented and a summary log of the results kept on file in the District or Field Office and noted in the applicable station analysis.

9.7.1   PERIODIC REVIEW

Every 3 years, each District's procedures for ADCP measurements, documentation, and data archival will be reviewed by Office of Surface Water personnel or their designees during the District Surface Water Review. This review will include an evaluation of site selection, suitability of configuration files, measurement completeness and accuracy, documentation of discharge and check measurements, and archival of data files. In addition, the review will evaluate the maintenance procedures followed by each office to ensure that the most current firmware and software upgrades have been implemented and that the instrument and peripheral equipment are being properly maintained. Recommendations by the Surface Water Review team will be immediately addressed by the appropriate District personnel.

In summary, it is the primary objective of this plan to ensure that all data collection and analyses performed in conjunction with the use of ADCP's be conducted in a professional manner commensurate with the standards set by the Water Resources Division for all its varied activities in

hydrologic investigations. Because of the complexity of the ADCP and associated software and the dynamic nature of its application to the riverine and estuarine environment, the ADCP should be used only by qualified personnel and standard procedures should be followed rigorously.

9.8   SELECTED REFERENCES

1. Morlock, S.E., in press, Evaluation of Acoustic Doppler Current Profiler measurements of river discharge at 12 sites within the continental United States: U.S. Geological Survey Water-Resources Investigations Report 95 - 4218.

2. RD Instruments, 1989, Acoustic Doppler Current Profilers -- principles of operation: a practical primer: San Diego, Calif., RD Instruments, 36 p.

3. RD Instruments, current version, Broadband direct reading ADCP pre-deployment test procedures: San Diego, Calif., RD Instruments, 10 p.

4. RD Instruments, current version, Direct-reading broadband Acoustic Doppler Current Profiler technical manual: San Diego, Calif., RD Instruments, 49 p.

5. RD Instruments, current version, User's manual for RD Instruments Transect program (for use with broadband ADCP): San Diego, Calif., RD Instruments, 88 p.

6. Simpson, M.R., and Oltmann, R.N., 1992, Discharge-measurement system using an Acoustic Doppler Current Profiler with applications to large rivers and estuaries: U.S. Geological Survey Water-Supply Paper 93 - 2395, 32 p.