2 1 1
7 7 L E

ire notes of the Unesco/NORAD Third Regional Training Course for Hydrology Technicians, Kenya, 1977

Editor K.D. Gray

unesco

Distribution: limited
SC-79/WS/32

211-77LE-10646

211
77LE

PREFACE

The training of hydrology technicians has been recognized as one of the major requirements for the development of water resources and their rational utilization in developing countries. Unesco has organized a number of regional training courses for hydrology technicians as well as assisted in the organization of several national training courses.

When preparing these courses, the lack of appropriate lecture notes often become apparent. Unesco, therefore, decided to compile suitable material and has collected together here the lecture notes which were used at the Unesco/NORAD Third Regional Training Course for Hydrology Technicians, Nairobi, 1977.

These lecture notes are reproduced in the hope that future courses will benefit from them. They are intended to serve as an illustration of the type and level of material relevant for hydrology technicians. They do not represent a comprehensive view of the entire field of hydrology and should, therefore, be used in conjunction with basic textbooks.

This publication does not constitute a recommended manual for the training of hydrology technicians but should be considered as a first step towards the compilation of such a manual. The reader is, therefore, invited to make proposals for the improvement, completion or re-orientation of the text and/or illustrations.

Within the framework of the educational component of the International Hydrological Programme (IHP), Unesco will take such proposals into account when preparing comprehensive textbooks for technician training.

[illegible]
[illegible]
[illegible]

LIBRARY IRC
PO Box 93190, 2509 AD THE HAGUE
Tel.: +31 70 30 689 80
Fax: +31 70 35 899 64
BARCODE: 18646
LO:
211 77LE

CONTENTS

A. MATHEMATICS AND STATISTICS

1. Algebra, exponential and logarithmic equations *

In the number a^n the letter a is the base and n is the index. The rules for dealing with indices may be summarized as follows:

a) Multiplying: When multiplying numbers which are powers of the same base, add the indices.

$$a^n \times a^m = a^{n+m}$$

b) Dividing: When dividing numbers which are powers of the same base, subtract the index of the denominator from the index of the numerator.

$$\frac{a^n}{a^m} = a^{n-m}$$

c) Powers of Powers: When a power of any base is raised to a further power, the index numbers are multiplied.

$$(a^n)^m = a^{nm}$$

From these rules arise the following points worth noting:

(i) Any base to the power 0 is equal to 1. $a^0 = 1$

(ii) Fractional indices indicate roots. $a^{\frac{1}{2}} = \sqrt{a}$, $a^{\frac{1}{3}} = \sqrt[3]{a}$

(iii) Negative indices indicate reciprocals. $a^{-n} = \frac{1}{a^n}$

Using these rules, any number can be expressed as a power of any other number.

Examples:

(1) As powers of 2: $8 = 2^3$ $\frac{1}{4} = 2^{-2}$ $3 = 2^{1.585}$

(2) As powers of 8: $64 = 8^2$ $2 = 8^{\frac{1}{3}}$ $\frac{1}{8} = 8^{-1}$

(3) As powers of 10: $1000 = 10^3$ $0.001 = 10^{-3}$ $20 = 10^{1.301}$

In equations containing unknown values to certain powers or roots, the powers and roots have to be removed.

---

* Basic algebra has not been reproduced.

Examples:

(1)
$$x^2 = 9$$
$$x = \pm 3$$

Observe that a square root can be either positive or negative.

(2)
$$\sqrt{x - 3} = 2$$
$$x - 3 = 4$$
$$x = 7$$

(3)
$$x^2 + 5 = 30$$
$$x^2 = 25$$
$$x = \pm 5$$

(4)
$$4\sqrt{x + 3} = 12$$
$$\sqrt{x + 3} = 3$$
$$x + 3 = 9$$
$$x = 6$$

When a number is expressed as a power of a certain base, the index is known as the logarithm of the number to that base. Thus with the previous examples:

(1) To base 2: $\log_2 8 = 3$ $\quad \log_2 \frac{1}{4} = -2$ or $\bar{2}$ $\quad \log_2 3 = 1.585$

(2) To base 8: $\log_8 64 = 2$ $\quad \log_8 2 = \frac{1}{3}$ $\quad \log_8 \frac{1}{8} = -1$ or $\bar{1}$

(3) To base 10: $\log_{10} 1000 = 3$ $\quad \log_{10} 0.001 = -3$ or $\bar{3}$ $\quad \log_{10} 20 = 1.301$

Note that 2 is actually 2^1, and thus $\log_2 2 = 1$. This is true for any base, i.e., $\log_a a = 1$. For most purposes logarithms to the base 10 are used and these are known as common logarithms. The tables most frequently used give the values of common logarithms to four decimal places, but it should always be remembered that the fourth decimal place cannot be relied upon, so that answers to calculations should only be given to a number of figures that may reasonably be expected to be accurate.

If the unknown appears as an index, see if it is possible to take logarithms of each side of the equation.

Examples:

(1) If $2^x = 16$ or
$$2^x = 2^4$$
$$x = 4$$

Taking logarithms, $x \log 2 = \log 16$
$$x = \frac{\log 16}{\log 2}$$
$$x = \frac{1.204\ 12}{0.301\ 03}$$
$$x = 4$$

(2) If $3^x = 5$

taking logarithms, $x \log 3 = \log 5$
$$x = \frac{\log 5}{\log 3} = \frac{0.698\ 97}{0.477\ 12}$$
$$x = 1.465$$

(3) If $100^{2-7x} = 12$

$$\begin{aligned} \text{taking logarithms, } (2-7x)\log 100 &= \log 12 \\ (2-7x)2 &= 1.0792 \\ 2-7x &= 0.5396 \\ 7x &= 2 - 0.5391 = 1.4600 \\ x &= 0.2086 \end{aligned}$$

If the unknown appears as a logarithm, put it back into index form.

Examples:

(1)
$$\begin{aligned} \log_{10} x &= 3 \\ x &= 10^3 \\ x &= 1000 \end{aligned}$$

(2)
$$\begin{aligned} 2\log_{10} x &= 3 \\ \log_{10} x^2 &= 3 \\ x^2 &= 10^3 \\ x &= \sqrt{1000} = 10\sqrt{10} \\ x &\quad 31.62 \end{aligned}$$

Equations with unknown values to any power can be solved using logarithms.

Examples:

(1)
$$\begin{aligned} 2x^{\frac{2}{3}} + 5 &= 25 \\ 2x^{\frac{2}{3}} &= 20 \\ x^{\frac{2}{3}} &= 10 \\ \tfrac{2}{3}\log x &= \log 10 \\ \log x &= \tfrac{3}{2}\log 10 = \tfrac{3}{2} \\ x &= 31.62 \end{aligned}$$

or
$$\begin{aligned} 2x^{\frac{2}{3}} + 5 &= 25 \\ 2x^{\frac{2}{3}} &= 20 \\ x^{\frac{2}{3}} &= 10 \\ x &= 10^{\frac{3}{2}} = \sqrt{10^3} = \sqrt{1000} \\ x &= 31.62 \end{aligned}$$

(2)
$$\begin{aligned} 3x^{\frac{2}{5}} - 4 &= 17 \\ 3x^{\frac{2}{5}} &= 21 \\ x^{\frac{2}{5}} &= 7 \\ \tfrac{2}{5}\log x &= \log 7 \\ \log x &= \tfrac{5}{2}\log 7 = 2.113 \\ x &= 129.718 \end{aligned}$$

Exercises:

Solve the following equations:

(1) $3\sqrt{x + 2} = 6$ (2) $2\sqrt{2x + 5} = \sqrt{2 - x}$

(3) $2 + \frac{x^3}{4} = \frac{x^3}{2}$ (4) $x(x - 2) = 2(8-x)$

(5) Express the following numbers as powers of 10:

(a) 0.001 (b) 0.003 (c) 0.01

(6) Solve the equations:

(a) $4^x = 512$ (b) $(\frac{1}{4})^x = 8$

(7) Solve the equations:

(a) $2 \log_{10} x = 3$ (b) $\log_{10} \frac{1}{3} = 0.45$

2. Elementary trigonometry

2.1 Fundamental facts

Consider carefully the right-angled triangle ABC shown in fig. 1. Although the letters A, B and C are put there to label the points where the sides of the triangle intersect (called vertices), the angles of the triangle are also often referred to by letters. Thus angle A, or $\angle A$, may be used to denote the angle at vertex A. If this is likely to cause confusion, e.g., should there be more than one angle at a particular point, then it is better either to refer to it as angle BAC, i.e., the angle between BA and AC, or else to place a small letter such as x, y, z, α, β, θ or ϕ, in the angle so that the particular angle referred to may be easily distinguished.

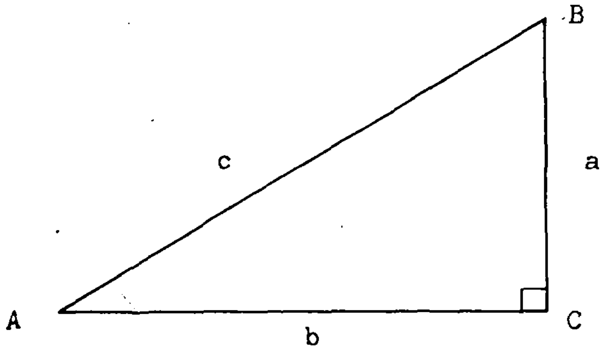


Fig. 1

In any right-angled triangle the side opposite to the right-angle is called the <u>hypotenuse</u>, the other two sides being called the opposite and the adjacent in accordance with their position relative to the angle under consideration. Thus, in fig. 1, relative to angle A the side BC is opposite and AC is adjacent, whereas it is the reverse relative to angle B. It is usual to denote the side opposite angle A as a, the side opposite angle B as b, etc.

The three most important trigonometrical ratios are defined as follows:

(1) the <u>sine</u> of angle A: $\sin A = \frac{\text{opposite}}{\text{hypotenuse}}$

(2) the <u>cosine</u> of angle A: $\cos A = \frac{\text{adjacent}}{\text{hypotenuse}}$

(3) the <u>tangent</u> of angle A: $\tan A = \frac{\text{opposite}}{\text{hypotenuse}}$

The reciprocals of these are also very useful, and these are as follows:

(4) the cosecant of angle A: $\operatorname{cosec} A = \frac{1}{\sin A}$

(5) the <u>secant</u> of angle A: $\sec A = \frac{1}{\cos A}$

(6) the <u>cotangent</u> of angle A: $\cot A = \frac{1}{\tan A}$

Let us now state these ratios for angles A and B of fig. 1 in terms of the sides of the triangle.

$\sin A = \frac{BC}{AB} = \frac{a}{c}$ $\quad\quad$ $\sin B = \frac{AC}{AB} = \frac{b}{c}$

$\cos A = \frac{AC}{AB} = \frac{b}{c}$ $\quad\quad$ $\cos B = \frac{BC}{AB} = \frac{a}{c}$

$\tan A = \frac{BC}{AB} = \frac{a}{c}$ $\quad\quad$ $\tan B = \frac{AC}{BC} = \frac{b}{a}$

$\operatorname{cosec} A = \frac{AB}{BC} = \frac{c}{a}$ $\quad\quad$ $\operatorname{cosec} B = \frac{AB}{AC} = \frac{c}{b}$

$\sec A = \frac{AB}{AC} = \frac{c}{b}$ $\quad\quad$ $\sec B = \frac{AB}{BC} = \frac{c}{a}$

$\cot A = \frac{AC}{BC} = \frac{b}{a}$ $\quad\quad$ $\cot B = \frac{BC}{AC} = \frac{a}{b}$

As the angles of a triangle add up to 180°, and angle C is a right-angle, it follows that $A + B = 90^\circ$. A and B are said to be <u>complementary</u> angles.

The following are some of the more important relationships arising from the above:

$\tan A = \frac{\sin A}{\cos A}$ $\quad\quad$ $\cot A = \frac{\cos A}{\sin A}$

$\sin A = \cos (90 - A)$ $\quad\quad$ $\cos A = \sin (90 - A)$

$\tan A = \cot (90 - A)$ $\quad\quad$ $\cot A = \tan (90 - A)$

2.2 <u>The theorem of Pythagoras</u>

This theorem gives us the relationship that the square of the hypotenuse is equal to the sum of the squares of the other two sides. Thus in fig. 1, for the triangle ABC, $a^2 + b^2 = c^2$.

Proof:

Consider the following areas in fig. 2:

large square = small square + 4 triangles.

$$(a + b)^2 = c^2 + 4(\tfrac{1}{2}ab)$$

$$a^2 + 2ab + b^2 = c^2 + 2ab$$

$$\therefore \underline{a^2 + b^2 = c^2}$$

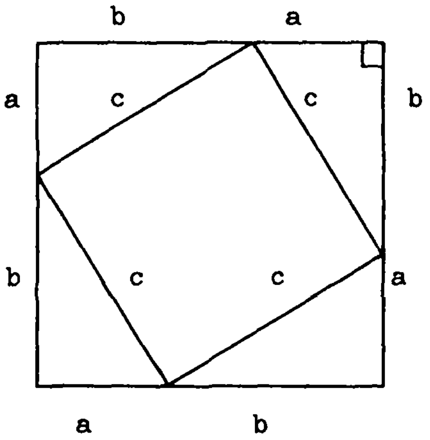


Fig. 2

If now we divide each side by c^2 we get,

$$\left(\frac{a}{c}\right)^2 + \left(\frac{b}{c}\right)^2 = 1$$

$$\therefore \underline{\sin^2 A + \cos^2 A = 1}$$

If each side of the equation had been divided by b^2, we would get

$$\left(\frac{a}{b}\right)^2 + 1 = \left(\frac{c}{b}\right)^2$$

$$\therefore \underline{\tan^2 A + 1 = \sec^2 A}$$

If each side of the equation had been divided by a^2,

$$1 + \left(\frac{b}{a}\right)^2 = \left(\frac{c}{a}\right)^2$$

$$\therefore \underline{1 + \cot^2 A = \operatorname{cosec}^2 A}$$

2.3 Some important triangles

Set squares are usually triangles with angles either 45°, 45° and 90°, or 30°, 60° and 90°, i.e., either half a square or half of an equilateral triangle.

In fig. 3(a) we have a square with a side of unit length, and it is required that we should find the length of the diagonal AC. Calling this length d units, we have by the Pythagorean Theorem:

$$d^2 = 1^2 + 1^2$$

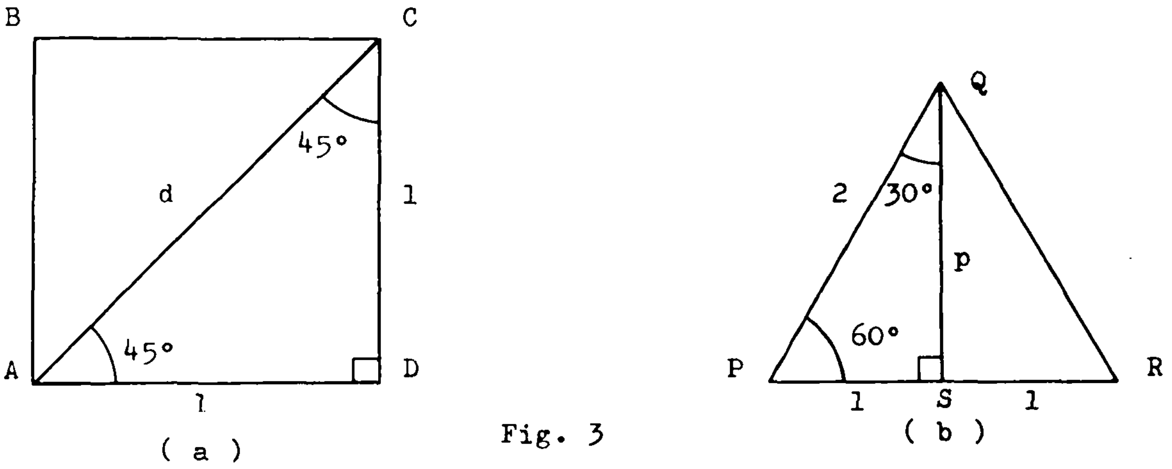
$$d^2 = 2$$

$$d = \sqrt{2}$$

Therefore, we can now write down our trigonometrical ratios for 45°:

$$\sin 45^\circ = \frac{1}{\sqrt{2}}, \quad \cos 45^\circ = \frac{1}{\sqrt{2}}, \quad \tan 45^\circ = 1$$

In fig. 3(b) we have an equilateral triangle of side 2 units symmetrically divided into two equal parts and it is required that we should find the length of the perpendicular QS.



(a)

Fig. 3

(b)

Calling this length p units, we have by the Pythagorean Theorem:

$$2^2 = 1^2 + p^2$$

$$p^2 = 3$$

$$p = \sqrt{3}$$

Therefore we can now write down our trigonometrical ratios for 30° and 60° :

$$\sin 30^\circ = \frac{1}{2}, \quad \cos 30^\circ = \frac{\sqrt{3}}{2}, \quad \tan 30^\circ = \frac{1}{\sqrt{3}}$$

$$\sin 60^\circ = \frac{\sqrt{3}}{2}, \quad \cos 60^\circ = \frac{1}{2}, \quad \tan 60^\circ = \sqrt{3}$$

2.4 Solution of right angled triangles

(a) Given two sides, the remaining side may be found by applying the Pythagorean Theorem, and the angles by using sine, cosine or tangent as appropriate.

(b) Given one side and one angle, the remaining angle may be found from the fact that the angles are complementary, and the sides by using sine, cosine or tangent as appropriate.

Example: In the triangle ABC, right-angled at C, AB is 40m and BC is 9m. Find AC and the angles at A and B.

(a) We first draw a small diagram and insert all the given information (fig. 4).

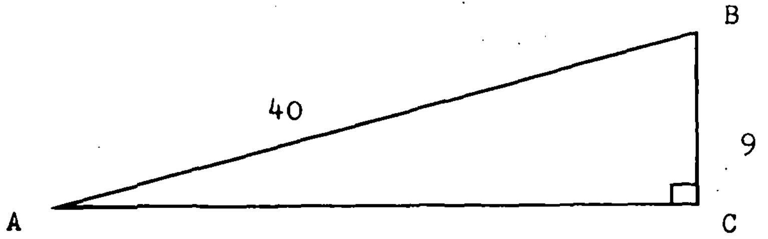


Fig. 4

From the Pythagorean Theorem:

$$AC^2 = AB^2 - BC^2$$

$$AC^2 = 40^2 - 9^2$$

$$AC^2 = 1519$$

$$AC = 38.97\text{m}$$

(b) To find angle A, note first that the two given sides are in the positions of hypotenuse and opposite side relative to ∠ A, so we use the sine:

$$\sin A = \frac{\text{opposite}}{\text{hypotenuse}} = \frac{BC}{AB} = \frac{9}{40} = 0.225$$

Using a table of natural sines we see that

$$\sin 13^\circ = 0.2250$$

$$\therefore \angle A = 13^\circ$$

To find angle B, note that the two given sides are in the positions of hypotenuse and adjacent relative to ∠ B, so we use the cosine:

$$\cos B = \frac{\text{adjacent}}{\text{hypotenuse}} = \frac{BC}{AB} = \frac{9}{40} = 0.225$$

From a table of natural cosines we have:

$\cos 77^\circ = 0.225$

$\therefore \angle B = 77^\circ$

Check: A and B should be complementary:

$13^\circ + 77^\circ = 90^\circ$

Note that we could have used the calculated length of AC to find either of the angles, but then, had there been an error in calculating AC, it would have affected the result for the angles. It is always better to work as long as possible directly from the given information.

<u>Example</u>: Triangle PQR has an angle of $38\frac{1}{2}^\circ$ at P and a right-angle at R. Given that PQ is 16m, find the lengths of the other two sides.

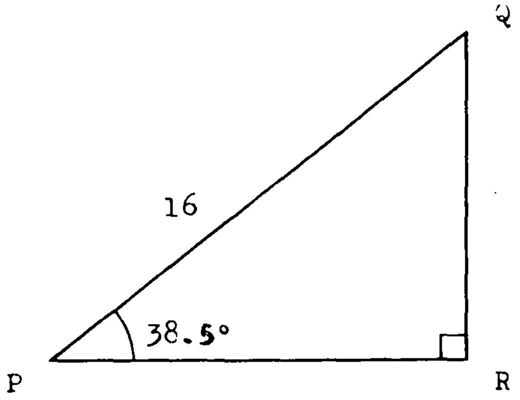


Fig..5

We first draw a small diagram (see fig. 5) and insert all the given information. To find QR, note that QR and the given side PQ are in the positions hypotemuse and opposite side relative to the given angle P, so we use the sine:

$$\sin P = \frac{\text{opposite}}{\text{hypotemuse}} = \frac{QR}{PQ}$$

$$QR = PQ \times \sin P$$

$$= 16 \times 0.6225$$

$$= 9.96\text{m}$$

Similarly, to find PR we use the cosine:

$$\cos P = \frac{\text{adjacent}}{\text{hypotemuse}} = \frac{PR}{PQ}$$

$$PR = PQ \times \cos P$$

$$= 16 \times 0.7826$$

$$= 12.52\text{m}$$

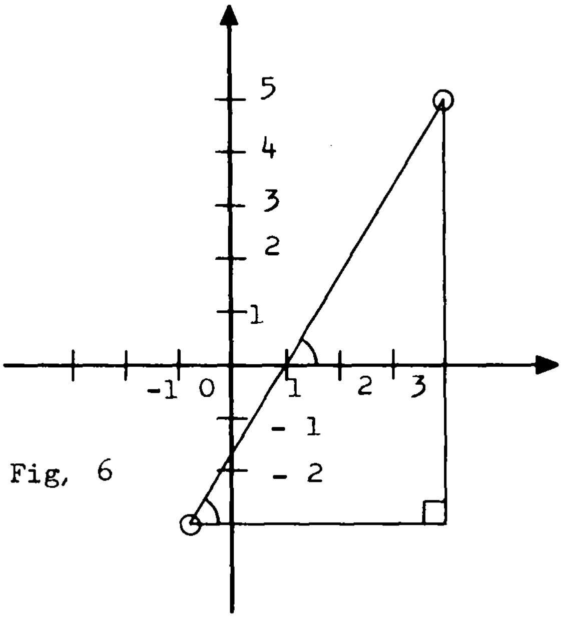
2.5 Gradients

If two points are selected on a straight-line graph, then the gradient (or slope) is given by the difference in the y-values (ordinates) divided by the difference in the x-values (abscissas).

$$m = \frac{y_1 - y_2}{x_1 - x_2}$$

and this is equal to tan θ, where θ is the angle the line makes with the x-axis. This is easily seen, as the ratio is $\frac{\text{opposite}}{\text{adjacent}}$.

Example: Find the angle made with the x-axis by the line through the points (4,5) and (-1, -3).



Fig, 6

The line and the angle are shown in fig. 6.

$$\tan\theta = \frac{5-(-3)}{4-(-1)} = \frac{8}{5} = 1.6$$

$$\therefore \theta = 58^\circ$$

2.6 Deduction of trigonometrical ratios one from another

2.6.1 Method 1: Look up the given ratio in the tables to find the angle, then refer to the appropriate tables to find the new ratio.

Example: Given sin x = 0.55, find tan x.

From sine tables: $\sin 33^\circ\ 22' = 0.5500 \quad \therefore x = 33^\circ\ 22'$

From the tangent tables: $\tan x = 0.6586$

2.6.2 Method 2: Draw a small diagram, insert the given ratio, then by the Pythagorean Theorem, find the third side and hence the required ratio.

Example: Given $\tan x = 1\frac{7}{8}$, find cos x.

$$\tan x = \frac{\text{opposite}}{\text{adjacent}} = \frac{15}{8}$$

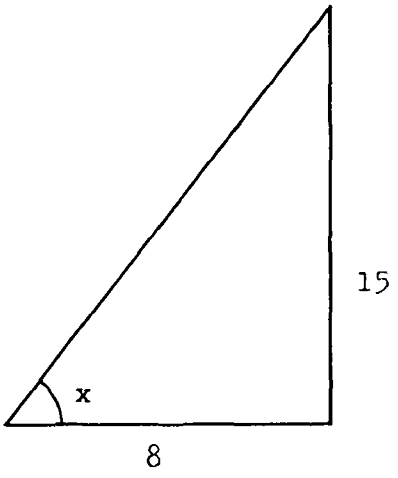


Fig. 7

These values are put in as shown in fig. 7.

$$\text{Now } 15^2 + 8^2 = 225 + 64 = 289 = 17^2$$

$\therefore$ the hypotenuse is 17, and $\cos x = \frac{8}{17}$

2.6.3 **Method 3:** Use the relationships $\sin^2 A + \cos^2 A = 1$ and $\tan A = \frac{\cos A}{\sin A}$.

Example: Given $\sin A = \frac{3}{5}$, find cos A and tan A.

$$\cos^2 A = 1 - \sin^2 A$$

$$= 1 - \frac{9}{25} = \frac{16}{25}$$

$$\therefore \cos A = \frac{4}{5}$$

$$\tan A = \sin A \div \cos A = \frac{3}{5} \div \frac{4}{5}$$

$$\therefore \tan A = \frac{3}{4}.$$

2.7 Exercises

(1) ABC is a triangle right-angled at C. Given that AC is 12 m and BC is 5m, find AB and angles A and B.

(2) Find the smallest angle and the hypotenuse of a right-angled triangle in which the other two sides are 35m and 12m.

(3) Find the smallest side and smallest angle in a right-angled triangle given that the remaining sides are 40m and 41m.

(4) If the foot of a ladder is placed on level ground $3\frac{1}{2}$m from a vertical wall and it just touches the wall at a point 12m above the ground, find the length to which the ladder is extended.

(5) Two straight lines on a graph cross one another at (1, 2). One line also passes through (-4, -6). The other line passes through (-9, -5). Find the angle each line makes with the axis and hence prove that the angle at which they cross another is 23^{o}.

(6) If $\sin x = \frac{1}{3}$, find the value of cos x and tan x without using trigonometrical tables.

3. Formulas - problems in hydrology

3.1 Evaporation

$$E = R + (0.5 \times C) \text{ mm}$$

where E = evaporation in mm
R = rainfall in mm
C = numbers of standard cups added (+) or taken out (−) from the pan.

Example:

Estimate the evaporation during 24 hours if the measured rainfall is 31.7 mm and 52 cups have been taken from the pan.

$$R = 31.7$$
$$C = -52$$
$$E = 31.7 - (0.5 \times 52) = 31.7 - 26$$
$$E = \underline{5.7} \text{ mm}$$

Exercise:

Estimate the evaporation during 24 hours if the measured rainfall is 0.6 mm and 7 cups have been added to the pan.

3.2 Automatic water level recorders

$$R = n \times C \text{ and}$$
$$S = \frac{L}{R}$$

where R = range of change in water level in m.
n = maximum number of rotations of the float wheel required to move the pen from one end of the "useable drum length" to the other.
C = circumference of float wheel in m.
S = scale or height reduction ratio.
L = useable drum length in m.

Example:

Determine the range and scale for a recorder with a float wheel circumference of 0.25m and a maximum number of float wheel rotations of 10. Useful drum length is 0.5m.

$$n = 10$$
$$C = 0.25$$
$$L = 0.5$$
$$R = 10 \times 0.25 = 2.5\text{m (range)}$$
$$S = \frac{0.5}{2.5} = 0.2 = \underline{1 \text{ to } 5} \text{ (scale)}$$

Exercise:

A recorder is to be installed in a river, which has a range of 5m between floods and low flows. The useable drum length is 0.3m. The maximum number of rotations of the float wheel is 10. What is the circumference of the float wheel required for the recorder? Determine also the scale (height reduction ratio).

3.3 Mean velocity in a cross-section of a river

$$Q = A \times v$$

where Q = measured discharge in m^3/s
A = measured cross-section area in m^2
v = mean velocity in the cross-section in m/s

Example:

$$Q = 2.9$$
$$A = 4.1$$
$$2.9 = 4.1 \times v$$
$$v = \frac{2.9}{4.1} = \underline{0.71 \text{ m/s}}.$$

Exercise:

Determine the mean velocity in a river if the discharge is $3.8 m^3/s$ and the area is $5.7 m^2$.

3.4 Rating equation for current meter

$$v = 0.0534\ n + 0.044$$

where v = velocity in m/s
n = number of revolutions per second

Example:

Determine the measured velocity if the number of revolutions of the propeller is 2.2 per second.

$$n = 2.2$$
$$v = 0.0534 \times 2.2 + 0.044$$
$$v = 0.117 + 0.044 = \underline{0.161 \text{ m/s}}.$$

Exercise:

Estimate the velocity if the number of revolutions of the propeller is 2.8 per second.

3.5 Discharge through a V - notch weir

$$Q = \frac{8}{15} \times \mu \times \sqrt{2g} \times \tan\frac{\theta}{2} \times h^{\frac{5}{2}}$$

where Q = discharge in m^3/s
μ = discharge coefficient
g = $9.81\ m/s^2$
θ = angle of the V
h = head in m.

Example:

40 l/s shall be discharged through a V - notch weir with an angle of 80°. $\mu = 0.6$. What is the required head?

$$\begin{aligned}
Q &= 0.040 \\
\mu &= 0.6 \\
\theta &= 80^\circ \\
0.040 &= \frac{8}{15} \times 0.6 \times \sqrt{2g} \times \tan\frac{80}{2} \times h^{\frac{5}{2}} \\
0.040 &= \frac{8}{15} \times 0.6 \times 4.429 \times 0.839 \times h^{\frac{5}{2}} \\
h^{\frac{5}{2}} &= \frac{0.040 \times 15}{8 \times 0.6 \times 4.429 \times 0.839} = 0.0336 \\
\frac{5}{2}\log h &= \log 0.0336 \\
\log h &= \frac{2}{5}(-1.4737) = -0.5895 \\
h &= 0.257\text{m} = \underline{25.7\text{ cm}}
\end{aligned}$$

Exercise:

Determine the required head for a discharge of 50 l/s if the angle of the V - notch weir is 53°. $\mu = 0.6$.

3.6 Storage reservoirs

$$I = O \pm \Delta S$$

where I = net inflow to the reservoir in m^3/s.
O = net outflow from the reservoir in m^3/s.
$S = \frac{\Delta H \times A}{t}$ in m^3/s.
ΔH = change in storage elevation in m during a particular period of time.
A = mean area of the reservoir in m^2
t = time in seconds.

Example:

The inflow to a tank is 3.3 l/s and the outflow is 4 l/s during 16 hours. What is the required storage volume?

$I = 3.3$
$O = 4.0$
$t = 16 \times 3.600 = 57,600$ seconds

$$3.3 = 4.0 - \frac{\Delta H \times A}{57,600}$$

Volume required is $= \Delta H \times A$ litres

$\Delta H \times A = 0.7 \times 57,600 = 40,320$ litres $= \underline{40.3 m^3}$

Exercise:

Determine the required storage volume if the inflow to a tank is 4.5 l/s and the outflow is 3.7 l/s during 15 hours.

4. Graphs

In hydrology a large amount of data is collected from gauging stations. This data is presented either in tables or on graphs of various types. There are tables for stage and discharge values. Graphs can show stage and discharge variations with time (hydrographs), duration of flow, frequencies, correlation between stage and discharge (rating curves), minimum and maximum discharges, etcetera.

The most common type of graph consists of two axes drawn at right angles to each other and intersecting at the origin 0; the horizontal axis 0X is often referred to as the x-axis, and the vertical axis 0Y as the y-axis. The position of any point on the graph is then fixed if its distance from each axis is given, such distances being known as co-ordinates. The distance of the point from the x-axis is called its ordinate, and its distance from the y-axis is called its abscissa.

4.1 Plotting a graph

It is advisable to observe the following details:

(1) Give the graph a title to explain what it is all about.

(2) Make full use of the graph paper; the larger the graph the more accurate it should be.

(3) Choose scales which are easy to work with; it is far better to let one unit on the graph paper represent 1, 2, 5 or 10, say, than 3, 7 or 9.

(4) It is often possible to start one or both scales from zero, but never do so if it will result in crowding the points into one small corner of the graph.

(5) Write sufficient figures along each axis to help in reading the graph, but avoid overcrowding.

(6) Label each axis to explain what the numbers represent, not forgetting to put in the units where appropriate.

(7) Mark the points as clearly as possible. One method of doing this is to place a very small dot in the correct position with a small fine circle around it to make it easy to find.

(8) If the graph is a straight line, use a ruler or other straight edge. If a curve has to be drawn, French curves or other devices may be used, but often the curve has to be sketched in freehand. In this case always sketch the curve from the inside with as long a stroke as possible. The ideal is a single fine pencil line through the points, but practice is needed to achieve this, and at first it may be necessary to erase an attempt with gentle strokes of a soft rubber.

4.2 The straight line graph

Any equation which can be expressed in the form $y = mx + b$ will give a straight line graph. The quantity in the position of y in the equation is plotted on the vertical axis, and the other variable along the horizontal axis.

The quantity m represents the gradient or slope of the line, and thus in fig. 8:

$$m = \frac{RS}{QS} = \tan \theta$$

while b represents the length OP, i.e., the intercept on the y-axis. It follows from this that if $b = 0$ the graph passes through the origin, and equations with the same value of m have graphs which are parallel straight lines.

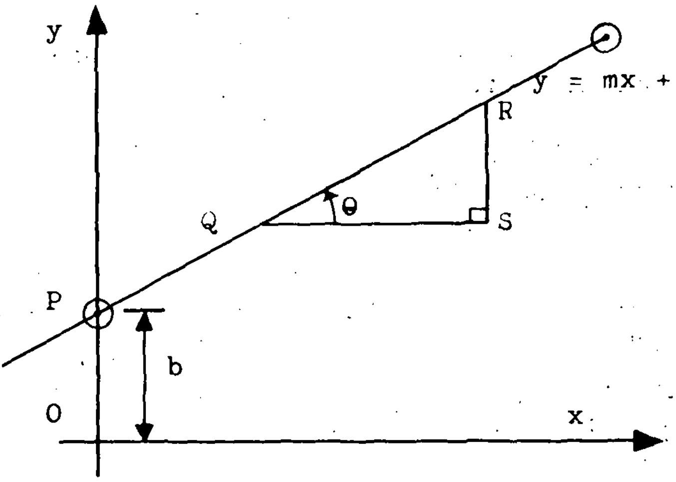


Fig. 8

Although only two points are really essential to fix the position of the straight line, it is advisable to plot three or possibly four as a check that all the points are correctly placed.

Example:

Plot the graph of $3x + 2y + 1 = 0$ for values of x from -3 to $+2$. The given equation is not in the form $y = mx + b$, so it must first be transposed. Thus

$$y = -\frac{3}{2}x - \frac{1}{2}$$

from which it can be seen that this straight line graph will have a slope of $-\frac{3}{2}$ and an intercept on the y-axis of $-\frac{1}{2}$.

From the equation, values of y corresponding to the values of x from -3 to +2 are determined and set out in a table:

x	-3	-2	-1	0	1	2
y	4	$2\frac{1}{2}$	1	$-\frac{1}{2}$	-2	$-3\frac{1}{2}$

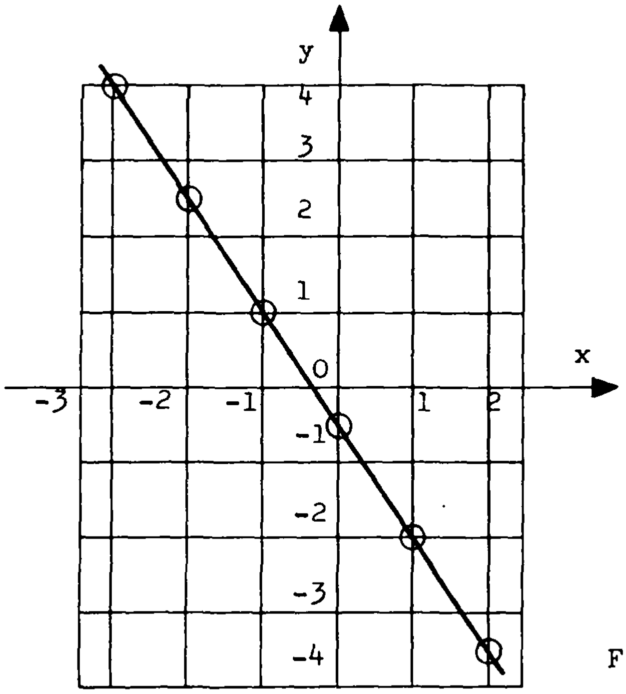


Fig. 9

Fig. 9 shows these points plotted on graph paper; the straight line drawn through them represents the given equation.

Exercises:

(1) Find the gradients and intercepts on the y-axis for each of the following without plotting the graphs:

(a) $2y = 4x + 3$ (b) $y + 5x = 7$ (c) $4y - 6x = 9$
(d) $2x - 8y = 5$ (e) $9x = 3y - 2$ (f) $7 - x - y = 0$

(2) Draw the straight line graph through the points (4, 2) and (1, -3). Measure the intercept and gradient and so find the equation of the straight line. Check by substituting the values in the equation $y = mx + b$ and solving the simultaneous equation for m and b.

4.3 Plotting the rating curves

The rating curve or stage-discharge relation is obtained by plotting numerous measured discharges against their corresponding gauge heights. They are plotted on sheets divided either by an arithmetic scale or a logarithmic scale. The vertical axis or ordinate is the gauge staff height, while the horizontal axis or abscissa is the discharge.

In order to establish the rating curve, discharge measurements must be made over the entire range of gauge heights. When an adequate number and range of measurements become available a smooth curve is drawn through the points.

The curve itself is drawn either by eye or by calculating the line by the "method of least squares". In drawing the line by eye the line is drawn through the average of all points. As an aid in drawing the line the average of clusters of points are calculated and the line drawn through the averages points.

4.3.1 Rating curve on arithmetic paper

Fig. 10 shows a rating curve drawn on a arithmetic graph paper.

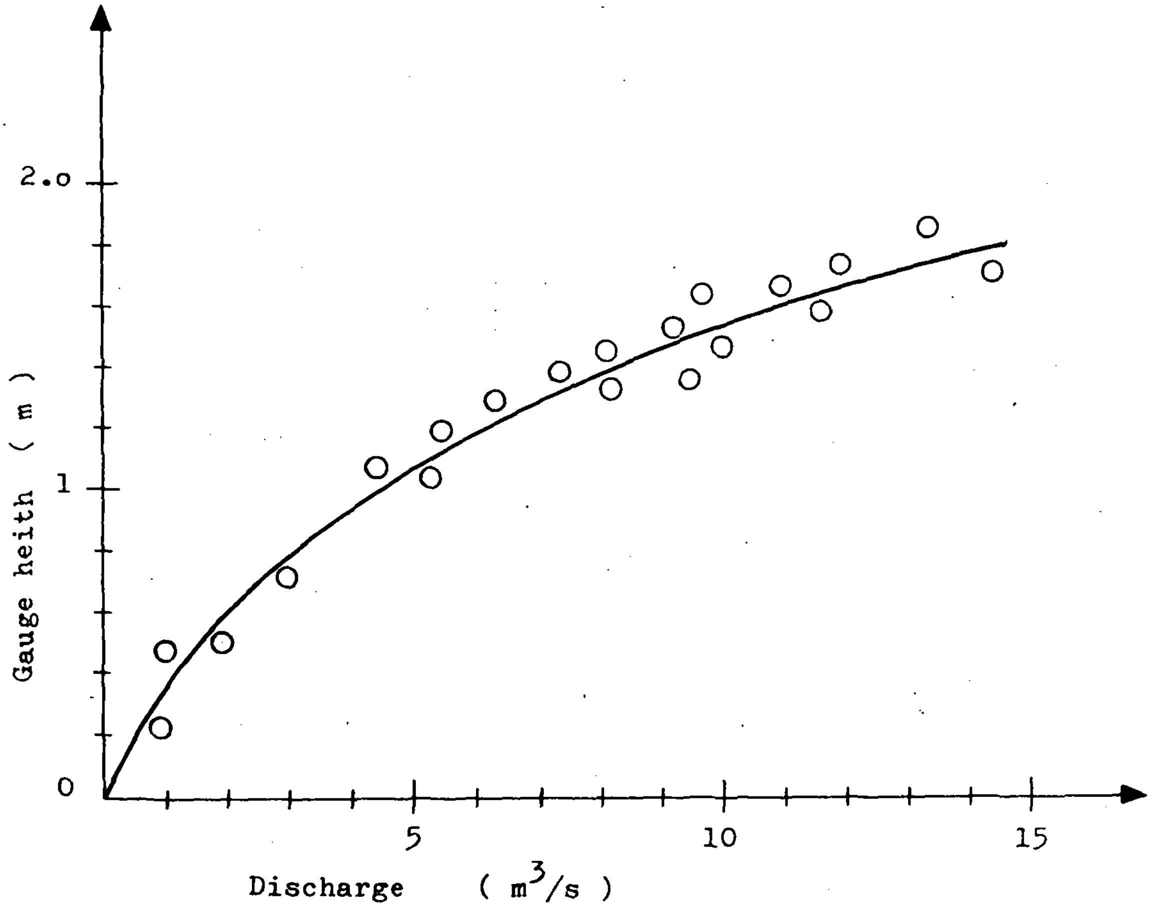


Fig. 10

4.3.2 Rating curve on logarithmic paper

When stages and corresponding values of the measured discharge for a regular gauging station are plotted on a logarithmic paper, a rating curve can be drawn as a straight line. The equation for the rating curve is:

$$Q = a \times H^{b}$$

where A = discharge, m^3/s
H = stage, m
a and b = coefficients

The equation represented on a logarithmic paper is a straight line because

$$\log Q = \log a + b \times \log H$$

In order to draw the line of best fit, the "method of least squares" may be used. This means that the coefficients a and b shall be solved from the following two equations:

$$\Sigma \log Q = n \times \log a + b \times \Sigma \log H$$

$$\Sigma (\log Q \times \log H) = \log a \times \Sigma \log H + b \times \Sigma (\log H)^2$$

The solution of these two equations gives:

$$b = \frac{n \times \Sigma (\log Q \times \log H) - \Sigma \log Q \times \Sigma \log H}{n \times \quad (\log H)^2 - (\quad \log H)^2}$$

$$\log a = \frac{\Sigma \log Q - b \times \Sigma \log H}{n}$$

4.3.3. Example: plotting a rating curve

In a river 12 discharges and corresponding stages have been measured. The values are shown in the first two columns of the table below. In order to calculate b and log a the following values must first be found:

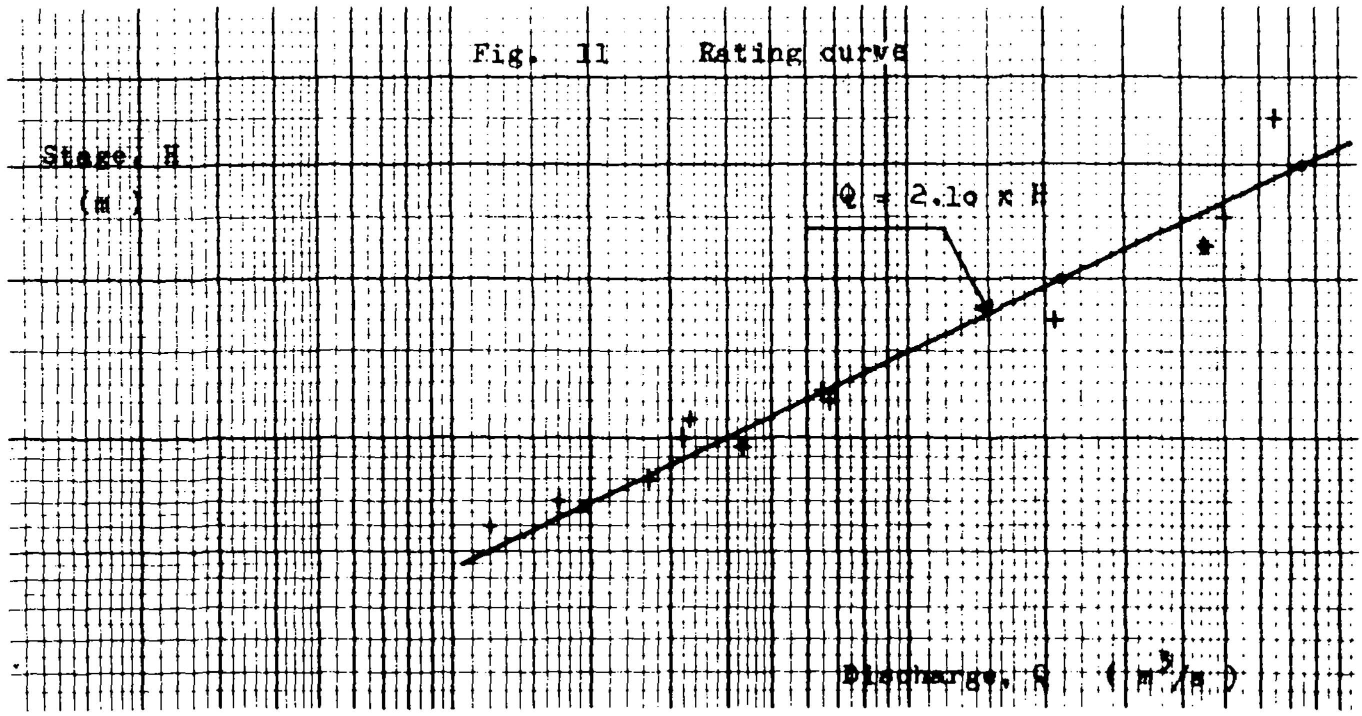


Fig. 11 Rating curve

$\Sigma \log H$
$\Sigma \log Q$
$\Sigma (\log Q \times \log H)$
$\Sigma (\log H)^2$

These values are also shown in the table.

Table: Fitting of a Curve by the Method of Least Squares

	H	Q	log H	log Q	(log Q x log H)	$(\log H)^2$
1.	3.24	451.32	0.5105	2.6545	1.3551	0.2606
2.	2.10	33.65	0.3222	1.5269	0.4920	0.1038
3.	1.59	11.78	0.2014	1.0712	0.2157	0.0406
4.	1.69	16.64	0.2279	1.2212	0.2783	0.0519
5.	2.20	68.54	0.3424	1.8360	0.6286	0.1172
6.	2.02	31.98	0.3054	1.5049	0.4596	0.0933
7.	2.66	211.78	0.4249	2.3259	0.9883	0.1805
8.	1.94	43.40	0.2878	1.6375	0.4713	0.0828
9.	1.80	27.11	0.2553	1.4332	0.3659	0.0652
10.	3.43	494.72	0.5353	2.6943	1.4423	0.2865
11.	2.26	67.91	0.3541	1.8320	0.6487	0.1254
12.	4.55	649.63	0.6580	2.8126	1.8507	0.4330
			$\Sigma = 4.4252$	$\Sigma = 22.5502$	$\Sigma = 9.1965$	$\Sigma = 1.8408$

As can be seen from the table

$\Sigma \log H = 4.4252$
$\Sigma \log Q = 22.5502$
$\Sigma (\log Q \times \log H) = 9.1965$
$\Sigma (\log H)^2 = 1.8408$

These values can now be inserted in the formulas and we obtain:

$$b = \frac{12 \times 9.1965 - 22.5502 \times 4.4252}{12 \times 1.8408 - 4.4252^2} \doteq 4.22$$

$$\log a = \frac{22.5502 - 4.22 \times 4.4252}{12} = 0.3230$$

$$a = 2.10$$

The equation for the rating curve is therefore

$$Q = 2.10 \times H^{4.22}$$

This curve as well as the twelve H, Q points are plotted on fig. 11.

4.3.4 Exercise

In a river 8 measurements of discharge and corresponding stages have been done. The values are the following:

Stage H (m)	Discharge Q (m^3/s)
2.20	15.1
4.15	89.5
6.20	278
3.34	61.5
5.41	173
2.52	28.1
4.55	136
5.85	221

Plot the values on logarithmic paper, calculate the equation for the rating curve and draw the straight line corresponding to the equation.

5. Statistics

5.1 Frequencies

Hydrological data is often recorded in tables in chronological order. This data can thereafter be ordered according to frequencies. For instance, discharge values have been obtained during a period of 75 days and might be recorded as follows:

Discharge (m^3/s) x		Number of days f	f in %
2		1	1.3
3		3	4.0
4		7	9.3
5		15	20.1
6		20	26.7
7		13	17.4
8		8	10.6
9		4	5.3
10		2	2.7
11		1	1.3
12		1	1.3
	Total	75	100.0

The first column gives the value of x, the variate, the next column gives the value of f, the frequency, showing how many times (days) each particular value of x occurs and the last column gives the frequency in percent (%).

5.2 Histogram

The histogram provides an immediate visual comparison of the respective frequencies. The histogram for the values above is shown in fig. 12.

This comparison is really based on the relative areas of the columns to allow for cases where the base line cannot conveniently be divided into equal intervals, but in the great majority of cases the intervals are made equal, and the length of each column is proportional to the frequency. The total frequency of the distribution is represented by the total area of the figure. For a well-balanced figure, the greatest height should be roughly $\frac{2}{3}$ to $\frac{3}{4}$ of the width.

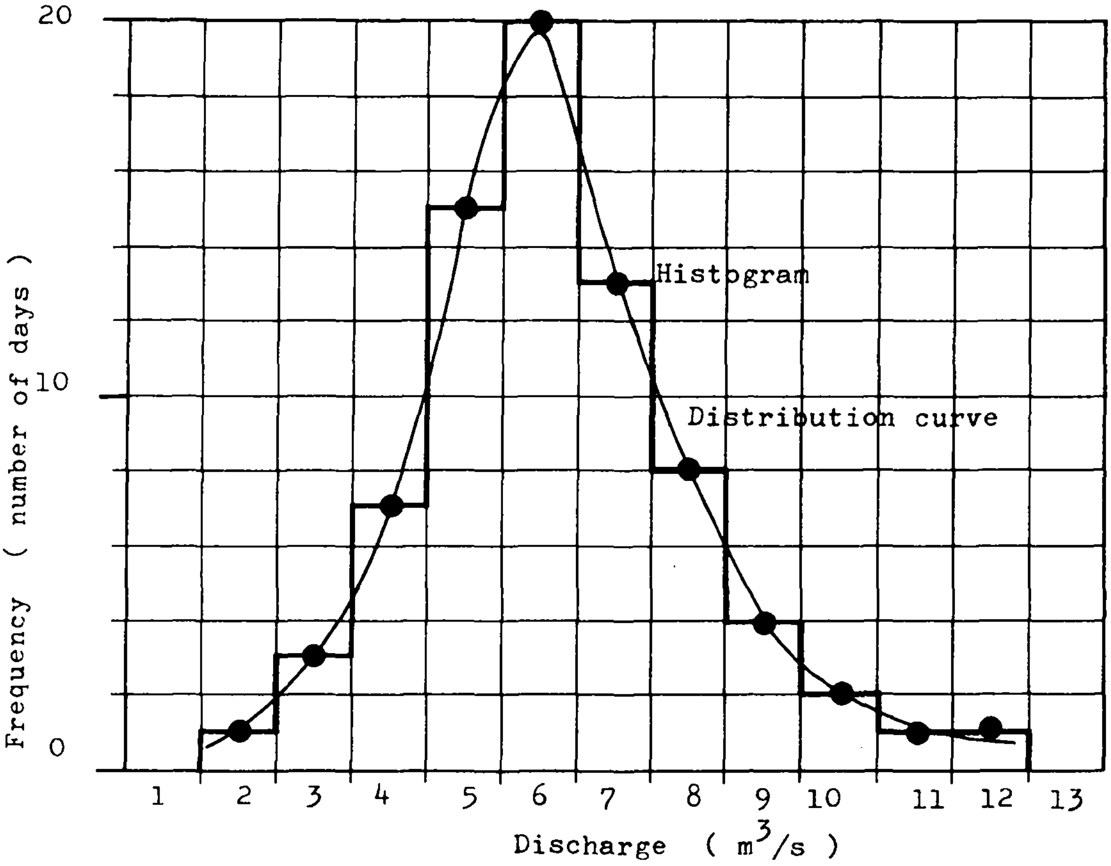


Fig. 12 Distribution curve and histogram

5.4 Cumulative frequency curve

Another way of showing a distribution is by a cumulative frequency curve (fig. 13). This curve illustrates how the total frequency is built up and is useful in that it facilitates estimates of the proportion of the distribution between set limits. To plot the cumulative frequency curve for the previous example, the table of values must be extended to include also the cumulative frequencies, obtained by adding together the preceeding frequencies after having re-listed the values in order of size. It is often an advantage to express the cumulative frequencies in percent. The values in the table above can be re-listed as follows:

Discharge m^3/s	Number of days f	f %	Cumulative f %
12	1	1.3	1.3
11	1	1.3	2.6
10	2	2.7	5.3
9	4	5.3	10.6
8	8	10.6	21.2
7	13	17.4	38.6
6	20	26.7	65.3
5	15	20.1	85.4
4	7	7.3	94.7
3	3	4.0	98.7
2	1	1.3	100.0

The cumulative frequency curve for the values in the table above is shown on fig. 13.

5.5 Flow duration curve

In hydrology work the cumulative frequency curve is used as a flow duration curve, which expresses the duration (often in percent of the total time) specified discharges are equalled or exceeded. The above cumulative frequency curve in fig. 13 is therefore also a flow duration curve. It can be seen from fig. 13 that the flow with a duration of 95% in $3.8 m^3/s$. It means that during 95% of the total time 75 days or during $\frac{95}{100}$ x 75 = 71 days the discharge is $3.8 m^3/s$ or more. The 95 percent duration discharge is often regarded as the low flow parameter.

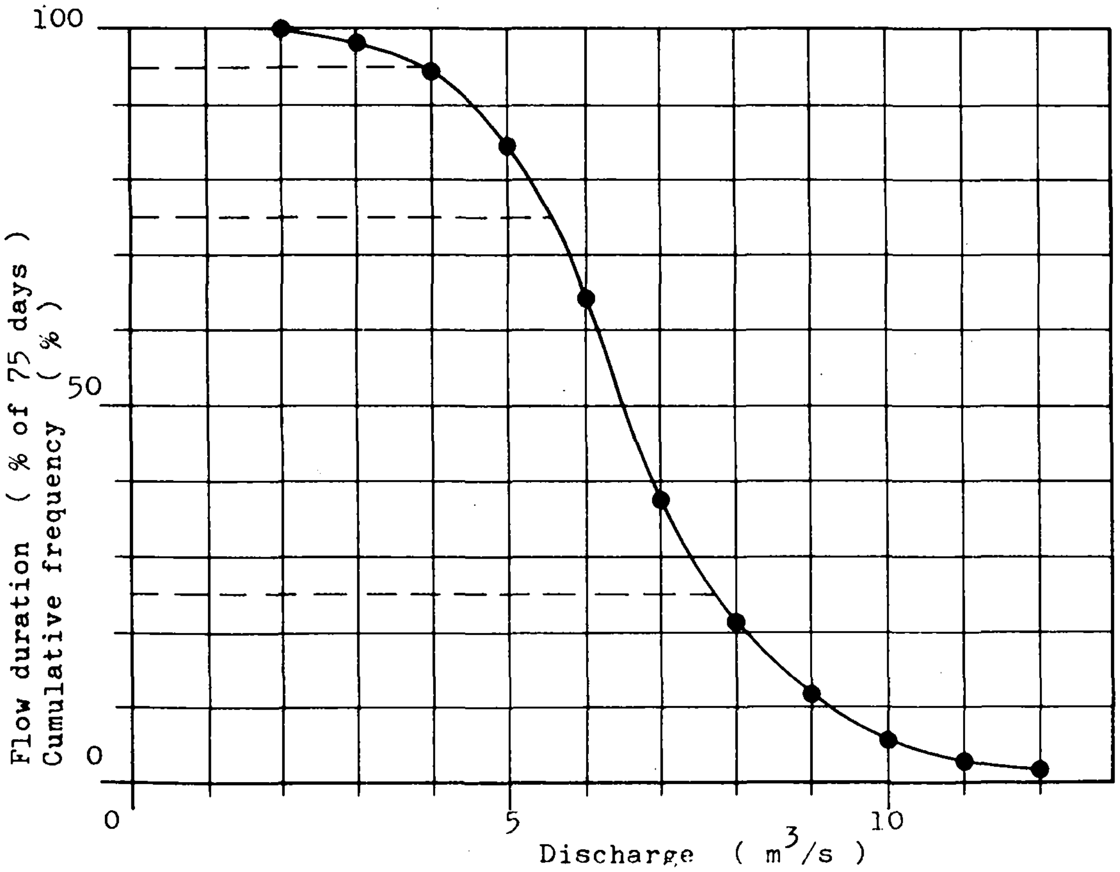


Fig. 13 Cumulative frequency curve ; flow duration curve

5.5.1 Maximum value

The maximum peak value (discharge) is the highest value among all observations within a particular period of time. This value is also the discharge with the lowest duration (frequency) on the flow duration curve. The maximum discharge in our example is $12m^3/s$.

5.5.2 Upper quartile value

The upper quartile value is the value of the discharge, which corresponds to the duration (frequency) of 25%. This means that 25% of all values are equal or higher than the upper quartile value. In our example this value is $7.6m^3/s$.

5.5.3 Median value

The median value is the value of the discharge, which corresponds to the duration (frequency) of 50%. This means that 50% of all values are equal or higher than the median value. In our example this value is $6.5m^3/s$.

5.5.4 Lower quartile value

The lower quartile value is the value of the discharge, which corresponds to the duration (frequency) of 75%. This means that 75% of all values are equal or higher than the lower quartile value. In our example this value is $5.6m^3/s$.

5.5.5 Minimum value

The minimum (low-flow) value (discharge) is the lowest value among all observations within a particular period of time. This value is also the discharge with 100% duration (frequency) on the flow duration curve. The minimum discharge in our example is $2m^3/s$.

5.6 Types of averages

5.6.1 Arithmetic mean

The arithmetic mean is the average most frequently in use and most widely understood. It is obtained by adding together all the values of the items in the distribution and dividing the total by the number of items. This is conveniently expressed as

$$\bar{x} = \frac{1}{n} \sum x$$

where $\bar{x}$ is the mean, n is the number of items in the distribution, $\sum$ is a mathematical symbol meaning "the sum of all the values of" and x is the variate.

For a grouped distribution each value of the variate, x, must be multiplied by the corresponding frequency, f, and in this case the mean is given by

$$\bar{x} = \frac{1}{n} \sum fx$$

5.6.2 Median

The median is the central value. To find the median, the complete distribution must be arranged in order of size; then, with an odd number of items, the central value will be the median: with an even number of items the median is the average of the central pair. With a grouped distribution, first pick out the group containing the median by summing the frequency column as far as the value n/2 and then evaluate the median by proportion.

5.6.3 Mode

The mode is the most frequently occurring value. The mode corresponds to the peak of the distribution curve. It is the variate with the greatest frequency. For a grouped distribution the modal class can be defined as the class with the greatest frequency.

5.6.4 Weighted mean

The weighted mean is calculated through the formula

$$\bar{x} = \frac{(x \cdot w)}{w}$$

where $\bar{x}$ = weighted average value

$x \cdot w$ = various values multiplied with the respective weights

w = weight for the various values.

This formula can be used, for instance, when the average rainfall will be calculated for an area, where the various raingauge stations do not represent the same portion of the total area.

5.7 Standard deviation

The two sets of numbers 3, 4, 5, 6, 7 and 1, 3, 5, 7, 9 each have the same mean but it is obvious that the spread is different in the two cases. In this simple comparison it would be sufficient to use the range as a measure of dispersion 3 to 7 and 1 to 9, but in general the range is not a very satisfactory guide as it is entirely dependent upon the two extreme items. The measure of dispersion which takes account of all the items in the distribution and is used as the basis for further statistical calculations is the standard deviation.

Basically, the standard deviation is the root-mean-square value of deviations from the mean. To calculate it, first find by how much each value of the variate, x, differs from the mean, then square these deviations from the mean, then calculate the mean of the squares of deviations, and, finally, take the square root.

The symbol σ is used for standard deviation, and its mathematical determination may be expressed as

$$\sigma = \sqrt{\frac{1}{n} \sum (x - \bar{x})^2}$$

where n is the total number of items and $\bar{x}$ is the arithmetic mean.

For the two sets of numbers 3, 4, 5, 6, 7 and 1, 3, 5, 7, 9 the calculation of the standard deviations may be set out conveniently as follows:

	SET I			SET II		
	Variate x	Deviation d	(Deviation)^2 d^2	Variate x	Deviation d	(Deviation)^2 d^2
5 times	3	−2	4	1	−4	16
	4	−1	1	3	−2	4
	5	0	0	5	0	0
	6	1	1	7	2	4
	7	2	4	9	4	16
	$5 \lfloor 25$		$5 \lfloor 10$	$5 \lfloor 25$		$5 \lfloor 40$
	$\bar{x} = 5$		2	$\bar{x} = 5$		8

$\sigma = \sqrt{2}$ $\sigma = \sqrt{8} = 2\sqrt{2}$

Thus the standard deviation for the second set is double that for the first.

5.8 Normal distribution curve

Very few distributions follow the "normal" curve exactly, but unless the departure is pronounced it is useful to assume that many of the continuous distributions follow it approximately.

For a normal distribution curve there should be complete symmetry and it follows that the mean, the mode and the median should all coincide. For purposes of comparison it is convenient mathematically to take the value of the mean as the origin of co-ordinates and measure along the horizontal axis in intervals of σ as shown in fig. 14.

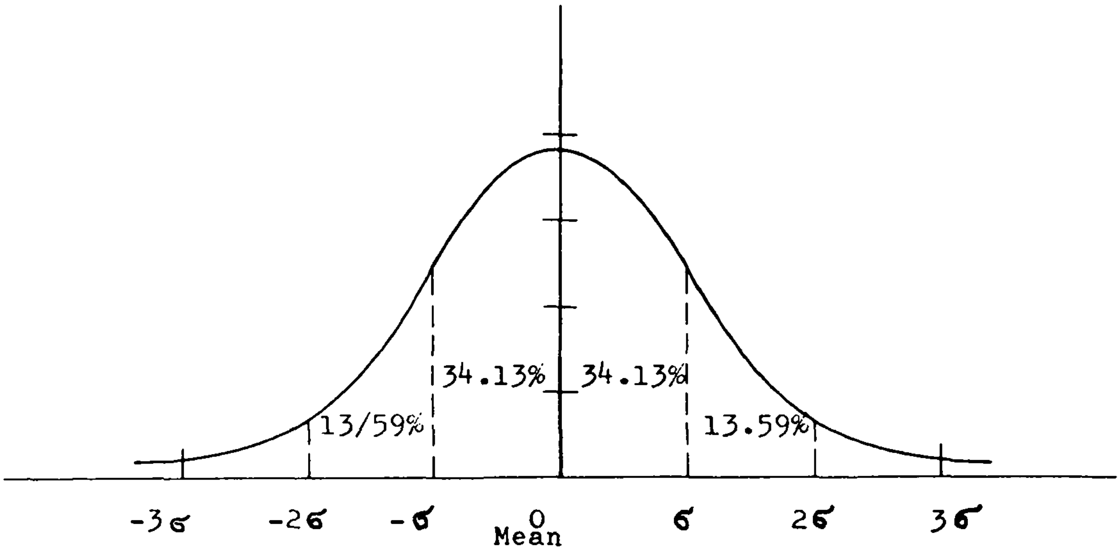


Fig. 14

Although theoretically the normal curve extends indefinitely each way, it can be shown that:

50 per cent of the items come between the limits ± 0.6745
68.26 " " " " " " ±
86.64 " " " " " " ± 1.5
95 " " " " " " ± 1.96
95.44 " " " " " " ± 2

The equation of the normal curve can be expressed in various forms, but the exponential functions involved are beyond the scope of these notes. However, the following facts are helpful when sketching a normal curve (fig. 18):

(i) The curve is symmetrical about the mean, and is often shown with the y-axis at this central value.

(ii) The curve never meets the x-axis at an angle, but approaches closer and closer to the x-axis at either end.

(iii) The places on the curve where the curvature changes (points of inflexion) occur where x has the values $\pm\sigma$.

(iv) The slope of the curve at these points is such that the tangent to the curve where $x = \sigma$ should cross the x-axis where $x = 2\sigma$, and the corresponding tangent on the other side where $x = -2\sigma$.

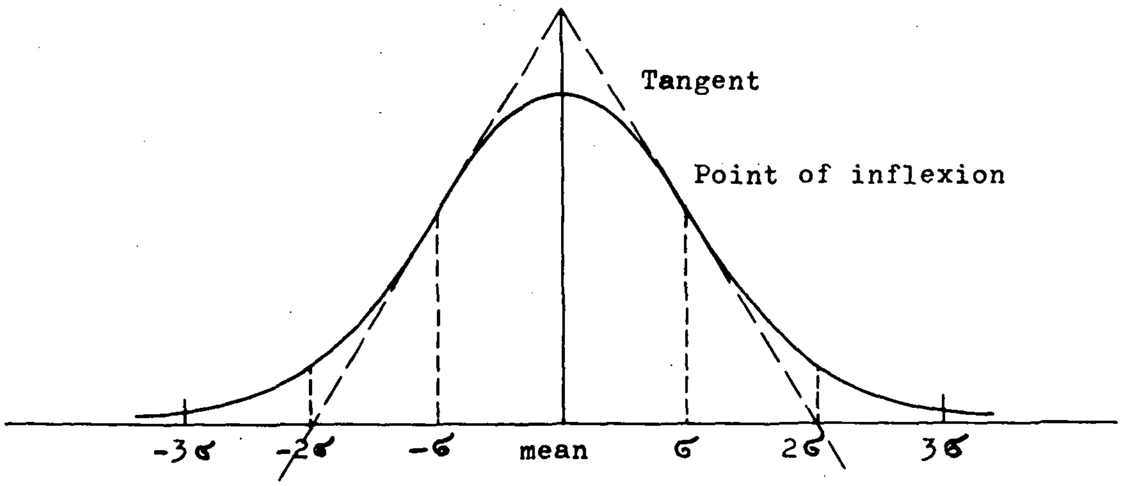


Fig. 15

5.9 **Exercises**

(1) The following table gives the marks awarded to 100 students on an examination:

x	1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80	81-90	91-100
f	1	4	8	17	24	21	14	6	3	2

Draw the histogram and the distribution curve of this distribution.

(2) Draw the cumulative frequency curve for the distribution in exercise (1).

(3) In the same examination, 22 students in one class gained an average of 46 marks, while 28 students in another class had an average of 51 marks. Calculate the overall average for the two classes together.

(4) The labour expenditure (man-hours per house) of twenty skilled bricklayers is given below. Calculate the mean and the standard deviation.

603	625	559	703	549	658	618	589	638	597
512	645	544	611	565	583	678	574	667	482

6. Probability

What do we mean by expressions such as "nine times out of ten" or "odds of four to one"? How does an insurance firm calculate its premiums? How can a builder estimate the chance of wet weather delaying certain stages of a construction project? All these are aspects of probability, i.e. the branch of mathematics which enables us to calculate the likelihood of any particular outcome.

6.1 Basic definitions

In any problem or experiment, each separate result is called an outcome. The particular happening we are looking for will be called the event. We shall define the probability of the event by $p = r/n$ where n is the number of observations during which the event was the outcome on r occasions.

Two events are complementary when together they include very possible outcome without any overlap or duplication. When all possible outcomes are known, the set of outcomes can be referred to as the sample space.

Example: When a couple have their first child, obviously it must be either male or female. Hence, for one child, the complete set of outcomes is (M,F).

Assuming equal probability for each possible outcome, the probability of the first child being male is $\frac{1}{2}$ and the probability of it being female is equally $\frac{1}{2}$. Note that the probabilities of the complementary events add together to give 1.

Now consider a family with two children. The sample space is (MM, MF, FM, FF). By symmetry, it is obvious that each of these possible outcomes, being equally likely have a probability of $\frac{1}{4}$, (i.e. 1 in 4).

If we consider the possibility of such a family with two children having at least one boy, we can either add the individual probabilities for MM, MF, FM to get $\frac{1}{4} + \frac{1}{4} + \frac{1}{4} = \frac{3}{4}$, or we can use our basic definition from which $r = 3$, $n = 4$, and thus $p = \frac{3}{4}$.

6.2 Non-occurrence

If the probability that a certain event may happen is p, then the probability that the event will not happen is $1 - p$.

A probability of zero implies that there should be no chance at all of the event occurring.

Note that negative probabilities are impossible, and that p can never be greater than 1 since p = 1 implies a certainty.

Example: The probability of having a year in England during which there is no rainfall is almost certainly zero and, conversely, the probability is therefore 1 that it will rain at least once.

6.3 Normal probability distribution curve

In the form of the normal frequency distribution curve, the area between the curve and the x-axis is equal to the total number of items, observations or results.

For probability estimations an alternative form is used for the normal probability distribution curve in which the total area under the curve is equal to unity (fig. 16). Table A gives figures for the area under this curve between the mean and an ordinate at z such that

$$z = \frac{x - \bar{x}}{\sigma}$$

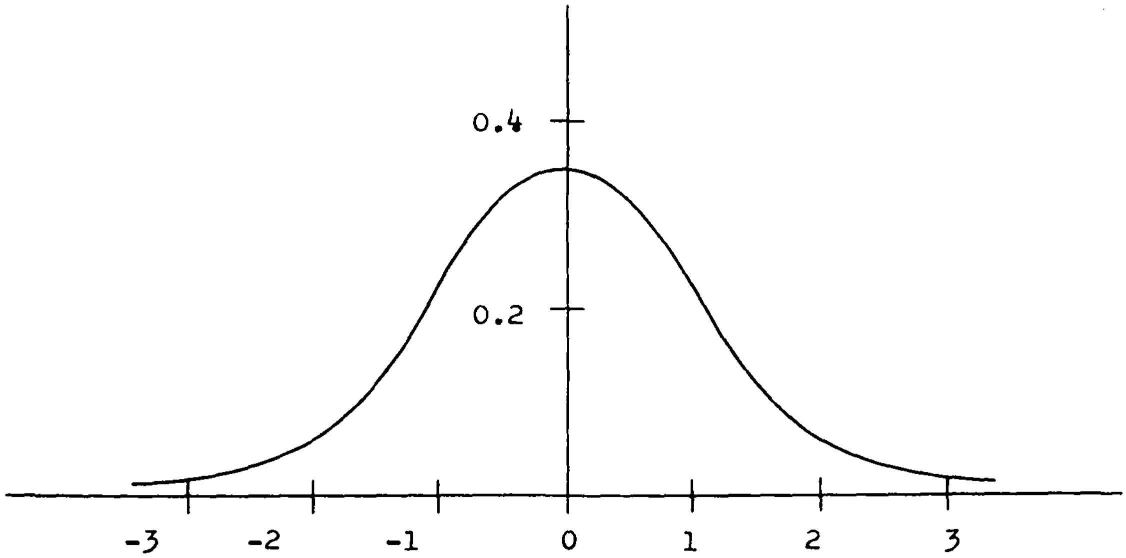
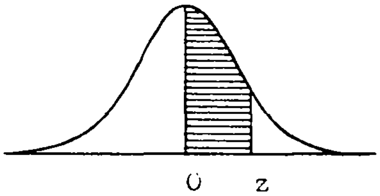


Fig. 16

This form of the table is in common use for estimating probabilities.

Table A: Areas under the normal curve
(to three decimal places)



z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.004	0.008	0.012	0.016	0.020	0.024	0.028	0.032	0.036
0.1	0.040	0.044	0.048	0.052	0.056	0.060	0.064	0.067	0.071	0.075
0.2	0.079	0.083	0.087	0.091	0.095	0.099	0.103	0.106	0.110	0.114
0.3	0.118	0.122	0.126	0.129	0.133	0.137	0.141	0.144	0.148	0.152
0.4	0.155	0.159	0.163	0.166	0.170	0.174	0.177	0.181	0.184	0.188
0.5	0.191	0.195	0.198	0.202	0.205	0.209	0.212	0.216	0.219	0.222
0.6	0.226	0.229	0.232	0.236	0.239	0.242	0.245	0.249	0.252	0.255
0.7	0.258	0.261	0.264	0.267	0.270	0.273	0.276	0.279	0.282	0.285
0.8	0.288	0.291	0.294	0.297	0.300	0.302	0.305	0.308	0.311	0.313
0.9	0.316	0.319	0.321	0.324	0.326	0.329	0.311	0.334	0.336	0.339
1.0	0.341	0.344	0.346	0.348	0.351	0.353	0.355	0.358	0.360	0.362
1.1	0.364	0.367	0.369	0.371	0.373	0.375	0.377	0.379	0.381	0.383
1.2	0.385	0.387	0.389	0.391	0.393	0.394	0.396	0.398	0.400	0.401
1.3	0.403	0.405	0.407	0.408	0.410	0.411	0.413	0.415	0.416	0.418
1.4	0.419	0.421	0.422	0.424	0.425	0.426	0.428	0.429	0.431	0.432
1.5	0.433	0.434	0.436	0.437	0.438	0.439	0.441	0.442	0.443	0.444
1.6	0.445	0.446	0.447	0.448	0.449	0.451	0.452	0.453	0.454	0.454
1.7	0.455	0.456	0.457	0.458	0.459	0.460	0.461	0.462	0.462	0.463
1.8	0.464	0.465	0.466	0.466	0.467	0.468	0.469	0.469	0.470	0.471
1.9	0.471	0.472	0.473	0.473	0.474	0.474	0.475	0.476	0.476	0.477
2.0	0.477	0.478	0.478	0.479	0.479	0.480	0.480	0.481	0.481	0.482
2.1	0.482	0.483	0.483	0.483	0.484	0.484	0.485	0.485	0.485	0.486
2.2	0.486	0.487	0.487	0.487	0.487	0.488	0.488	0.488	0.489	0.489
2.3	0.489	0.490	0.490	0.490	0.490	0.491	0.491	0.491	0.491	0.494
2.4	0.492	0.492	0.492	0.493	0.492	0.493	0.493	0.493	0.493	0.494
2.5	0.494	0.494	0.494	0.494	0.494	0.495	0.495	0.495	0.495	0.495
2.6	0.495	0.495	0.496	0.496	0.496	0.496	0.496	0.496	0.496	0.496
2.7	0.497	0.497	0.497	0.497	0.497	0.497	0.497	0.497	0.497	0.497
2.8	0.497	0.498	0.498	0.498	0.498	0.498	0.498	0.498	0.498	0.498
2.9	0.498	0.498	0.498	0.498	0.498	0.498	0.499	0.499	0.499	0.499
3.0	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499
3.1	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499
3.2	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499	0.499
3.3	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
3.4	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500

Example 1: Find the area under the normal curve between $z = 0$ and $z = 0.70$.

In Table A, follow down the first column until you reach the value $z = 0.7$. The figure for 0.70 is 0.2580 and this is the required area. It represents the probability that z lies between 0 and 0.70.

Example 2: Find the probability of a value of z between 0.95 and 3.05. From Table A

0 to 3.05 has probability 0.4989
0 to 0.95 has probability 0.3289

subtracting 0.1700

0.95 to 3.05 has probability 0.17

Exercises:

(1) Find the probability of a value of $z < -0.51$.

(2) Find the area under the normal curve between the limits of $\pm 0.44\sigma$.

6.4 Flood estimation

In many cases - for instance for design of spillways in dams - it is necessary to predict peak floods in the future. If annual peak discharges are available from a long period of years the recurrence intervals (return period in years) can be estimated and plotted on a diagram. A curve can then be extended in order to find the recurrence interval of a certain discharge.

The recurrence interval is often calculated according to the Weibull formula:

$$T_R = \frac{n + 1}{r}$$

where T_R = recurrence interval (return period)

n = number of years of records

r = the rank of the peak discharge
($r = 1$ for the highest value and
$r = n$ for the lowest value).

The exceedance probability is the reciprocal of recurrence interval:

$$P_e = \frac{r}{n \text{ x } 1} \text{ x } 100$$

where P_e = exceedance probability in percent (%).

The calculated recurrence intervals are then plotted on a diagram with a linear distribution on the discharge scale and with the Gumbel distribution on the time scale.

6.4.1 Example

Below is a table with the annual momentary-peak discharges from the period 1960-68 in chronological order.

Year	Date	Annual Peak Discharge (m^3/s)	Order (r)	Recurrence Interval (Year)	Exceedance Probability (%)
1960	6 April	1.6	5	2	50
1961	20 November	7.3	1	10	10
1962	5 January	3.2	4	2.5	40
1963	28 April	5.2	2	5	20
1964	24 April	1.4	6	1.7	60
1965	1 May	1.2	9	1.1	90
1966	4 May	3.8	3	3.3	30
1967	10 May	1.3	7	1.4	70
1968	9 June	1.3	8	1.2	80

The values for discharge and recurrence intervals are plotted on a diagram with the Gumbel distribution on the time scale (see fig. 17). The curve is extended and it can be seen that the 25 year return flood expected is of the magnitude of 10 m^3/s.

In this case the number of observations is too small to predict the 50 year and 100 year return flood. With a larger number of observations the curve can be extended to 50 and 100 yards. It should also be stressed that with a small number of observations, which may not be quite reliable, the result ought tobe checked with other methods.

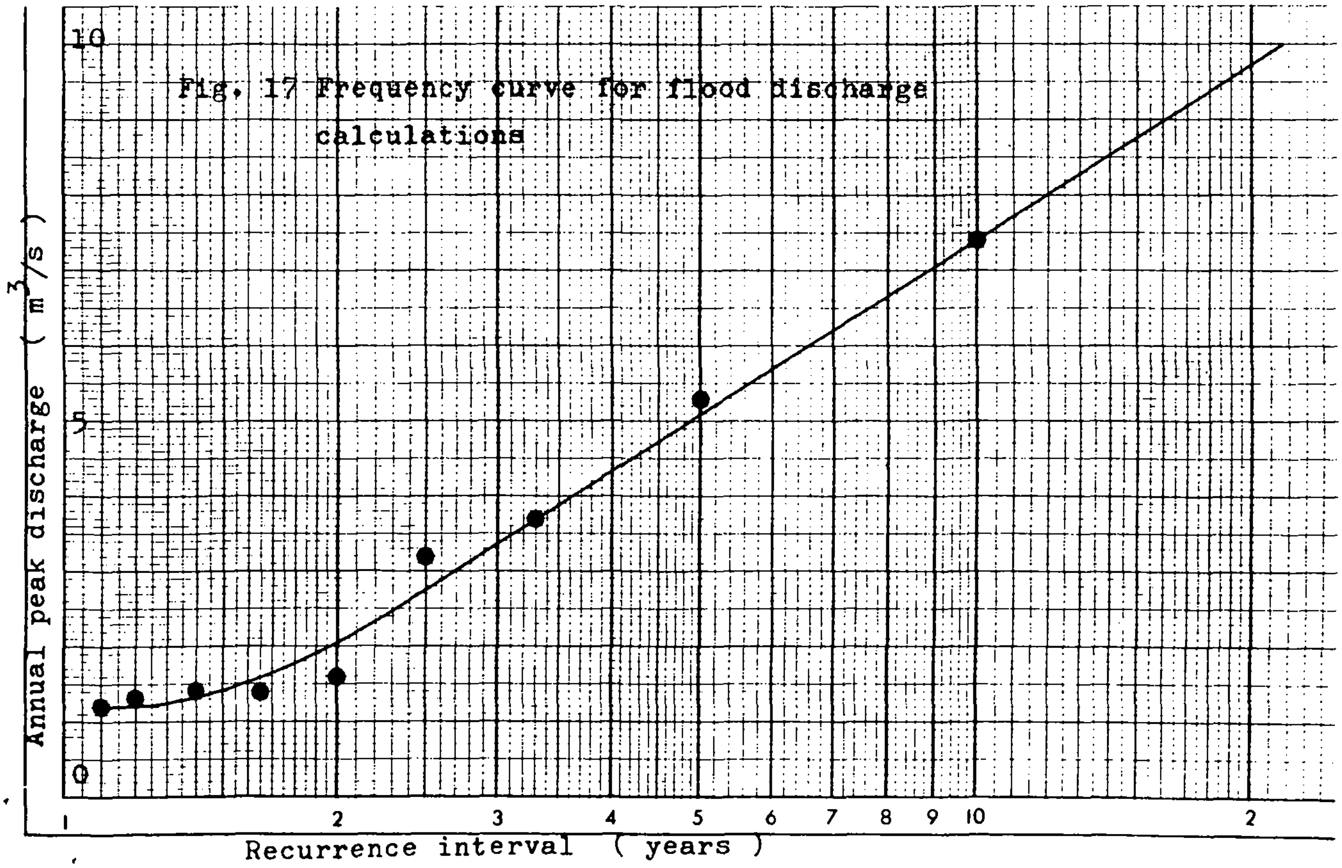


Fig. 17 Frequency curve for flood discharge calculations

6.4.2 Exercises

Below is a table with the annual momentary-peak discharges from the period 1960-69 in chronological order.

Year	Annual Peak Discharge (m^3/s)
1960	3.2
1961	6.4
1962	4.1
1963	5.8
1964	3.0
1965	2.2
1966	4.6
1967	2.8
1968	2.4
1969	8.2

Calculate the recurrence intervals (years) and the exceedance probability (%). Plot the values on a diagram with the Gumbel distribution and determine the 25 year return flood.

B. MAP READING AND AIRPHOTOS

1. Introduction

The following notes were accompanied by a map and airphotos of the area outside Nairobi, Kenya where the Third Regional Training Course for Hydrology Technicians was held. Future courses should modify them to illustrate local conditions.

Map provided: SK 66 sheet 148/4/6NW scale 1:5000. This covers the lecture centre and the area to the south of it. It is dated 1969 and does not show later features such as the dam on the Kerichwa Kubwa river. It has a 2000 ft grid and a 5 ft vertical contour interval.

Airphotos provided: Kenya 74/04 frames 004 and 006 scale 1:10,000 approximately giving stereo cover of the map area. Pocket stereoscopes will be provided.

2. Maps

A map is a vertical picture of the ground but shows only selected information to make it easier to read than an airphoto. Maps use a special language of conventional signs or symbols. Well-chosen signs do not need a reference box to interpret them. Abbreviations are in common use, e.g., Sp for spring, W for well.

As map scales get smaller, more information has to be squeezed into the same area of paper and more signs are necessary. There is exaggeration, generalisation and selection of detail shown. Note that maps include information which is not visible on the ground, e.g., heights, boundaries.

2.1 Scales

Map paper sheet size is fairly constant; therefore the bigger the area to be shown on a single sheet, the greater the reduction (i.e., the smaller the scale). A room 10 m square can be shown on a plan 1 m square at scale 1/10 but to show all Africa on the same size of paper the scale must be 1/10,000,000. Note that as the scale gets smaller the figure in the denominator of the scale fraction gets bigger. For convenience, scales are usually written as a ratio: 1:10 or 1:10,000,000 in the above examples. These are called Representative Fractions (or R.F.) and can be used to convert map distances to or from ground distances.

Example: On a map of scale 1:5000, ten centimeters on the map equals 10 x 5000 cm on the ground (i.e., 500 m or 0.5 km). Maps have line-scales printed on them by which ground distances can be read.

Large hydrological projects are normally planned on medium-scale maps in the range 1:20,000 - 1:100,000. Detailed design will be done on large-scale maps in the range 1:2000 - 1:10,000.

2.2 Distortion

The earth is a sphere, and large areas cannot be shown on a flat paper surface without distortion. However, the ground area covered by one map sheet at the above scales is small enough for the distortion to be scarcely measureable and it may be ignored.

2.3 Direction and Position on a Map

By convention the edges of a map usually run N-S and E-W, with North at the top.

Small-scale maps have a geographic graticule of meridians (lines of longitude) and parallels (lines of latitude). These lines are usually curved and not at equal spacing. So medium and large scale maps have a rectangular grid which is easier to use. The grid lines run approximately N-S and E-W.

Grid lines are numbered on a local system usually in multiples of a power of 10 (e.g., 2000, 3000, 4000). Grid squares vary in size from 1 cm to 20 cm square (on the map supplied 2000 ft on the ground equals approximately 12 cm on the map). Grids are numbered from west to east and from south to north (but note that on this map the numbers are all negative).

2.4 Grid Reference

A grid reference is a set of figures used to describe an exact position on the map. It is made up of two halves: easting/northing. This means that measurements are considered positive eastwards and northwards.

Example: TANK on the map is 930 ft east of grid line -90000; its easting is therefore + 930 - 90000 = -89070 ft. It is 1350 ft north of grid line -476,000. Its northing is therefore +350-476000 = -474650.

Its 12-figure grid reference is -089070/-474650. This figure is unique in this grid zone (which happens to be the area between 36° and 38°E longitude and the N and S boundaries of Kenya).

If we are going to work only on this map sheet we can omit the first two figures of easting and northing and the minus signs and write 9070.4650. Only one point on this sheet has this reference. If we decided to work to the nearest ten feet we can reduce the reference to 907.465. This is then a 6-figure grid reference. No other point in this zone can have this same reference.

2.5 Bearings

Direction on a map is best stated in the form of grid bearings. The direction of point B from point A is the angle measured from grid north at A clockwise to the line AB. Its value is between 0 and 360° and is always positive. The reverse bearing from B to A will be the forward bearing A to B plus or minus 180°.

2.6 Azimuth

Azimuth is a similar direction measured from true north (meridian) instead of grid north. Reverse azimuth is not exactly $\pm 180^\circ$ from forward azimuth because meridians are not parallel lines. The difference between an azimuth and a grid bearing (i.e., between true north and grid north) is usually very small (not more than a few minutes of arc) and may be neglected.

The magnetic compass does not point to true north nor to grid north. Magnetic north continuously changes direction at varying rates. Most topographic maps have a diagram giving the magnetic declination for the area covered by the map at a given date.

Example: "Mag. Decln. $3^\circ 20'$W in January 1970 annual change $10'$E" means that by January 1978 the declination should be $8 \times 10' = 1^\circ 20'$ less and the compass needle will point $2^\circ 00'$W of true north. Hence, if we read at point A on the ground the compass bearing to point B, we must subtract 2° from that reading before plotting the line as a grid bearing on the map.

3. Map Trigonometry

Note that if line AB has bearing α and length ℓ then dE equals $\ell \cdot \sin \alpha$ and dN equals $\ell \cdot \cos \alpha$ where dE and dN are the differences in the east and north grid references of B and A. Also dE = dN$\cdot\tan \alpha$. Be very careful of signs: ℓ is always positive but for example if B is southeast of A then dE and $\sin \alpha$ are positive but dN, $\cos \alpha$ and $\tan \alpha$ are negative.

4. Surface and its form

4.1 Areas

If we know the area of a piece of ground we obtain the paper area on the map by multiplying by the square of the scale.

Example: 5 hectares on the map scale 1:5000 becomes $5 \times (1/5000)^2$ hectares on the map. One hectare is $(10)^8$ sq cm so we get 20 sq cm on the map.

There are many methods of measuring area on a map. Two of the most common, by planimeter and by counting squares on a transparent overlay, will be illustrated by the instructor.

4.2 Relief

The previous lecture considered only problems in two dimensions: (distance/direction/area). But the ground also has height, elevation or relief. The map is flat paper with only two dimensions so various devices have to be used to show elevation.

If there is no accurate height survey of the ground, hachures and form lines can be sketched on the map to give a rough visual impression of relief (topography). Where there is full altimetric survey, accurate spot heights and contours can be plotted on the map. To give a better visual impression of relief, layer tints and hill shading may be added.

The only useful map for designing hydrological works will have contours and spot heights. The map provided has spot heights stated to four figures in feet. Near this building, at the north edge of the map by grid line -88,000 there is a brown cross and figures 5894. This means that the height of the ground at the centre of the cross is 5894 ft ± 1 ft above mean sea level (at Mombasa).

The contours are also shown in brown and are at 5 ft constant vertical interval. Thus contour 5850 is a real line on the map representing the position of an imaginary line on the ground whose height is 5850 ft ± 2.5 ft (the error in a contour must not exceed half the vertical interval between successive contours).

As the map scale gets smaller, the vertical interval between contours must increase to avoid overcrowding. At this scale 1:5000, V.I. is 5 ft. For a map at 1:50,000, the vertical interval would be 50 ft to get the same frequency of contours.

The distance between contours is an indication of slope. Compare the flat ground along Cemetery Road with the steep slopes of the river valleys. Contour values indicate whether the slope is up or down.

The line of greatest slope at any point is a line cutting the adjacent contours at angle 90°. Heights of points between contours may be estimated by linear interpolation along such a line.

The gradient of a line joining two points is obtained by calculating the difference in height of the two points and dividing by distance between them (the distance may be along a straight line, or measured along the course of a road, railway or river). A gradient may be expressed as a fraction, a percentage, or per mille (thousand), e.g., 1 in 50 is 2% or 20 ‰ which means a height change of 2 m in a distance of 100 m.

4.3 Profiles of Vertical Sections

If we take a line on the map (e.g., along the deepest part of a river valley) and plot a graph of height against distance or various points along the line we shall have a profile or section of the ground surface along the line.

Since height usually changes much more slowly than distance (except down a cliff or other steep slope) it is usual to exaggerate it by using a larger vertical scale. If we want to plot a profile along the Kerichwa River on the map we may use a 1:5000 horizontal scale with perhaps a 1:1000 vertical scale.

However, the slopes of the valley sides on this map are steep enough to dispense with exaggeration. For a cross-section of a valley we can use 1:5000 both horizontal and vertical.

5. Airphotos

(Note: for this lecture students will work in pairs. Each pair will use one stereoscope and one pair of overlapping photos).

For very small areas without much surface detail, the cheapest way to make a map is still by ground methods. However, nearly all mapping is now done from airphotos, usually by means of expensive plotting machines.

We are accustomed to looking at the ground horizontally from ground level. The survey airphoto however looks at it from above, vertically.

The airphoto records everything that is visible; it is not selective like a map. It does not record the invisible, e.g., names, descriptions of buildings, ground detail under trees, etc.

For economy, airphotos are usually monochrome, not in natural colour. There is distortion of scale and shape (high objects are nearer the camera and appear larger; they are also relatively displaced away from the centre of the photo).

All these considerations make photos more difficult to read and use than a map.

The standard size of an airphoto is 23 cm^2. They are usually exposed in north-south strips with forward overlaps of more than 50% (to give stereo cover - see below). Adjoining strips must also overlap to get full ground cover.

Photos can be cut and joined to make a mosaic, or print-laydown. This is rarely of any great value. We can also obtain enlargements which will give a larger paper size and larger scale.

When a map is made from airphotos the positions of the centres of the airphotos used are shown on the map.

Examples: At grid reference -88,800/-474,800 (near the TANK) there is a cross and number 7/112. This means frame no. 112 in film or flight no. 7. If we want to look at a photo of this area, this is the one we must ask for. (Note: the centre of the next frame in the strip (7/113) is about 3600 ft south of 7/112).

5.1 Scale

For economy in material and plotting, photo-flights are run as high as possible to get maximum area into each photo and minimum number of photos for a given area. Plotting machines can produce maps at scales up to five times larger than photo-scale. Hence this 1:5000 map may have been made from airphotos at a scale as small as 1:25,000. However, we are using some later airphotos of the area at scale 1:10,000.

The approximate scale of an airphoto can be determined by the following methods:

(a) Find on the photo two well-defined points (e.g., road junctions) and the same two points on the map. Compare the distances. If the photo distance is half the map distance the photo scale is half the map scale, e.g., $\frac{1}{2} \times 1/5000$ give 1/10,000 photo scale.

(b) On some photos the camera lens focal length (f) and the flying height (h) are printed at the edge of the photo. Then photo-scale is f/h. Note: h must be height of camera above ground surface, not above sea-level, and f and h must be same unit, e.g., both expressed in metres or both in feet. f is usually 6 inches (152 mm).

5.2 Photo-interpretation

Efficient photo-interpretation requires more skill and experience than map reading. There are many clues in size, shape, pattern, tone and shadows of photo-detail to aid the expert.

Of hydrological interest we may note that wet ground appears darker than dry ground, muddy water is lighter than clear water, deep water is darker than shallow.

5.2.1 Stereo-viewing

Like a map an airphoto is only two-dimensional; however, with two overlapping photos we can actually see the third dimension (height, relief). This greatly aids interpretation.

The human eyes are like two camera lens recording a stereo-pair of images from which the brain can compute shape, distance, etc.

If we view each of a pair of overlapping photos with separate eyes we get the same effect. Stereo-viewing is possible without a stereoscope but is difficult because we need to focus the eyes on a close object (the photos) while keeping the optic axes converging at a much greater distance (the eye naturally tends to equate focal distance to axis convergence distance).

The pocket stereoscope uses lenses to allow close photos to be viewed with a far focus.

A mirror stereoscope allows the photos to be set further apart but viewed with a close convergence. However, it is too bulky for field use.

Note that the human eye-base is only 6 or 7 cm while the airphoto base is much longer and may even be several kilometers. The stereo-view is thus exaggerated, and hills appear higher and steeper than they would if viewed by eye from the aircraft.

5.2.2 Classroom exercises

(a) Given a feature on the map find it on the photo.

(b) Given a feature on the photo find it on the map.

(c) Given a feature of recent construction on the photo which does not appear on the map, sketch it on the map.

5.2.3 Field exercises (map)

Map reading must be practised on the ground until the student can confidently transfer data from ground to map and vice versa. On the ground the first task is to place the map in correct orientation. This can be done by

(1) using magnetic compass,

(2) direction of sun (this requires some elementary knowledge of the sun's apparent motion),

(3) by directions to identified objects.

Exercises:

(a) given a feature on the ground find it on the map

(b) given a feature on the map find it on the ground

(c) write down co-ordinatees of (a)

(d) given two points on the ground find from the map the grid bearing and distance btween them

(e) given a point on the ground, find on the ground another point whose bearing and distance from the first point is given

(f) location of a featureless point by compass resection

(g) location on ground of a boundary shown on map.

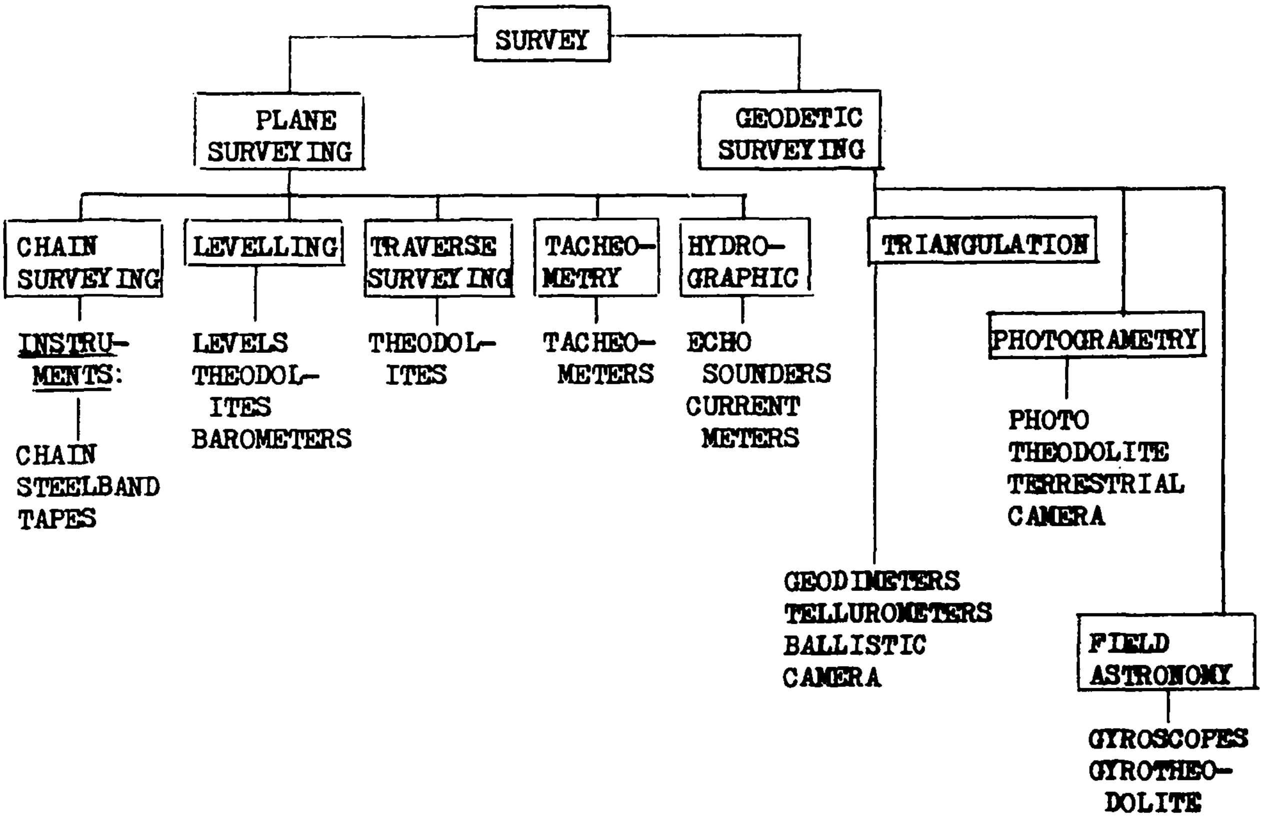
5.2.4 Field exercises (airphoto)

(a) and (b) above using airphoto instead of map.

C. SURVEYING

1. Introduction

Surveying is the operations connected with representation of ground features in plan.



Most surveying for ordinary purposes is in the branch of plane surveying. Plane surveying is used for the following purposes:

(a) measurement of areas
(b) making of plans in connection with legal documents
(c) making plans for the work of civil engineers, architects, builders and city planners
(d) making of maps and plans for particular applications - military, geographical, etc.

2. Chain surveying

This branch of surveying derives its name from the fact that the principal item of equipment used is the measuring chain. It is the simplest and oldest form of land surveying still extensively used.

2.1 Chain surveying equipment

The equipment falls under three broad headings:

(a) Linear measurement equipment
(b) Right angle measuring equipment
(c) Other equipment

2.11 Equipment for linear measurement

2.111 Chain

The chain, usually made of steel wire, consists of long links joined by shorter links (see fig. 1). It is designed for hand usage, sufficiently accurate for measuring chain lines and offsets of small surveys. The main disadvantage of the chain is that it is liable to change in length due to wear on metal-to-metal surfaces, bending of the links, thermal expansion, etc.

2.112 Surveyor's band

The surveyor's band is made of a steel strip approximately 1 cm in width, carried on a four-arm open frame winder. It may be 30 m, 50 m or 100 m long and has handles similar to those on a chain. The band is divided into tenths and hundredths of metres with the first and last metre subdivided in millimetres. The operating tension and temperature for which it was graduated should be indicated on the band.

The surveyor's band is much more accurate than the chain and with careful use maintains its length and therefore can be used as a standard measure. The main disadvantage is its lack of robustness and difficulty in doing repair in the field.

2.113 Tapes

Tapes may be made of linen, synthetic material (glass fibre), coated steel or plain steel. Tapes are used where greater accuracy of measurements is required. They are usually 15 or 30 m long and of three types:

(1) Linen or linen with steel wire woven into the fabric. These are liable to stretch and should have frequent tests for lengths - never used on work demanding great accuracy.

(2) Fibreglass tapes are much stronger than linen and will not stretch in use.

(3) Steel tapes are much more accurate, usually used for setting out buildings and always used for setting structural steelwork.

2.12 Equipment for right angle measurements

2.121 Cross staff

This instrument is used for setting out lines at right angles to the main chain line. The cross staff consists of two pairs of vanes set at right angles to each other with a wide and a narrow slit in each vane. It is mounted on a short ranging rod (see fig. 2).

Method of use: Establish chain line DB (see fig. 3). Set up on line DB at point A - the point at which it is desired to set off the new line. With a ranging rod at B on line DB some distance from A, sight the instrument upon it and then, without moving the cross staff sight through the second pair of vanes in the direction of C and direct an assistant with a ranging pole until this pole appears in sight. Line AC will be at right angles to line DB.

2.122 Optical square

2.123 Clinometer

2.13 Other equipment

2.131 Ranging rods are poles, circular in section, 2 m, 2.5 m or 3 m long - may be made of wood or steel. They are painted in alternate bands of red and white; the bands generally are 0.5 m thick so that the pole may be used for measuring offsets.

2.132 Arrows are pieces of steel wire about 0.5 m long used for marking temporary stations. A piece of coloured cloth fixed at the top of the arrows enables them to be seen more clearly. These are used when chaining a long line where the chain has to be laid down a number of times.

2.133 Station pegs are made of wood 50 mm x 50 mm and some length; they are used for marking stations.

2.2 Records

A field book is used to record measurements. Bookings should start at the back (from the bottom) of the page, continuing to the front. During the measurements, the surveyor should sketch in the detail and book the offsets as they are called to him.

2.3 Method of measuring a chain line

Insert ranging pole close to station peg so that position of peg may be located from a distance. In long lines poles are placed immediately between the stations and lined by eye so that straight line can be measured. To measure line AB, surveyor's assistant (chainman) takes 10 arrows, holding one end of the chain, walks from A towards B; the surveyor remains at Station A. When chain is fully extended, surveyor "lines" in chainman with Station B, who inserts an arrow at point A1.

Surveyor then walks along chain, measuring any offsets required. After completion of these measurements, chainman moves to point A2 dragging chain with him, while the surveyor now remains at point A1. When arrow is fixed at A2, surveyor removes arrow at A1, proceeds to take further offset measurements. The process is repeated until end of line, or chainman's arrows are finished. Collection of arrows by the surveyor is a check on the number of chains measured.

2.4 Applications in hydrology - sloping sites

Since the length of a line measured on the slope is greater than the horizontal distance between the stations required to plot the survey, any slope greater than 3° is adjusted in one of the following ways:

(a) Stepping the chain: Short lengths of chain held horizontally and plumbed down to ground surface, repeated until second station is reached. (See fig. 4).

(b) Measure angle (B) of slope by use of clinometer; calculate horizontal length (AB) of line by trigonometry: (See fig. 5)

AB = AC cos β, where AC = measured distance on slope.

Stepped chaining can be used in determining the bed profile of a river (see fig. 6). The following limitations should be observed:

(1) The stream should be at its dry weather flow (i.e., river almost dry, minimum flow).

(2) Select a section where the currents are not strong.

(3) The water depths are less than 5 m (use of graduated wooden poles possible).

The chain must be stretched taut with a specified tension to ensure negligible or no sag.

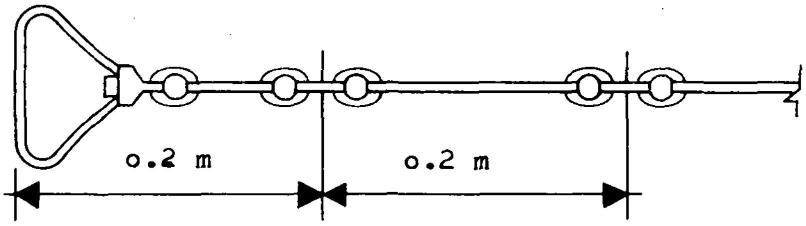


Fig. 1 Engineer's chain

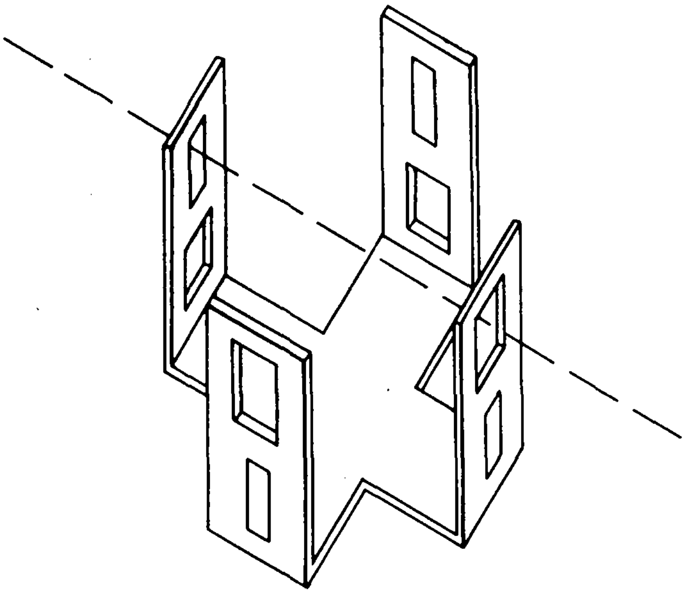


Fig. 2 Cross staff

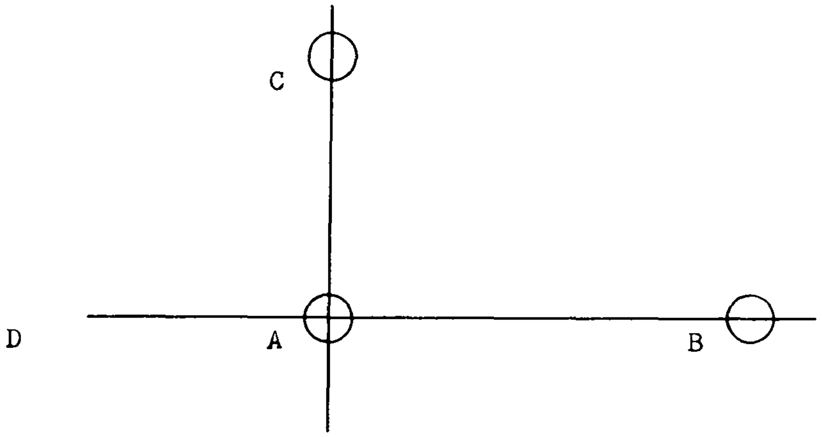
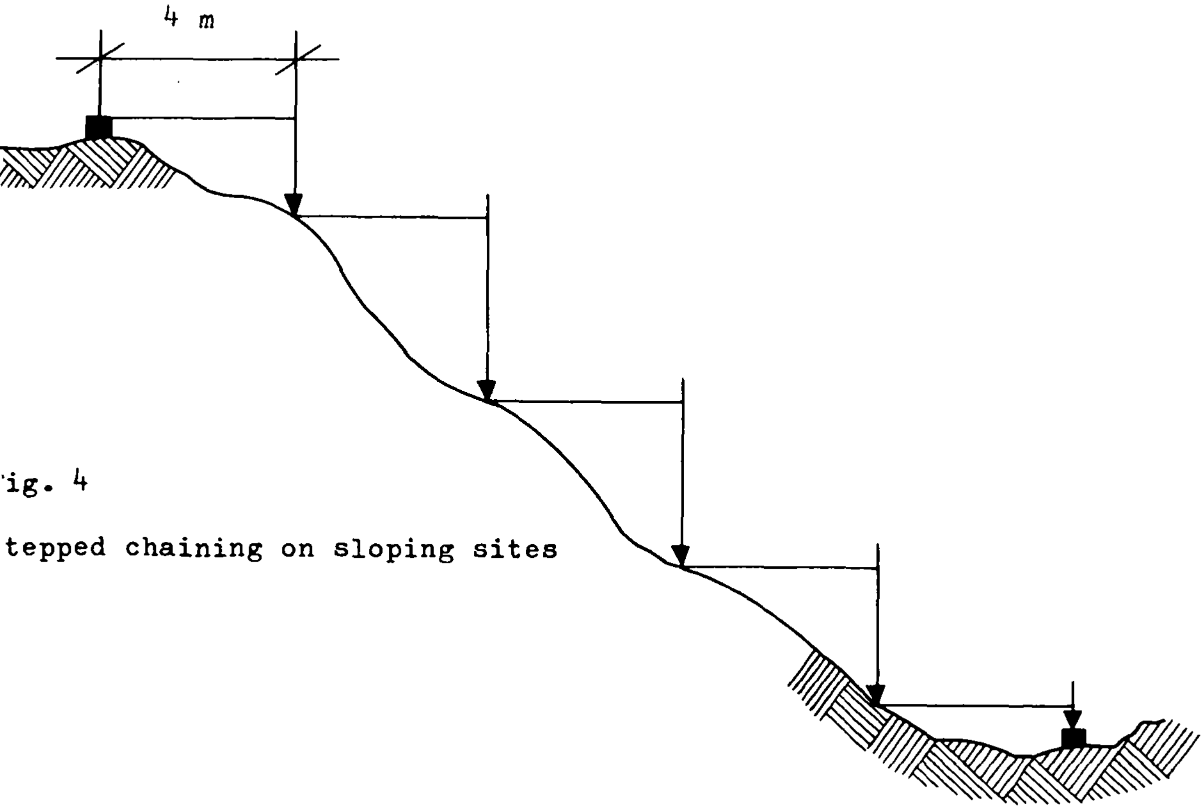


Fig. 3



ig. 4

tepped chaining on sloping sites

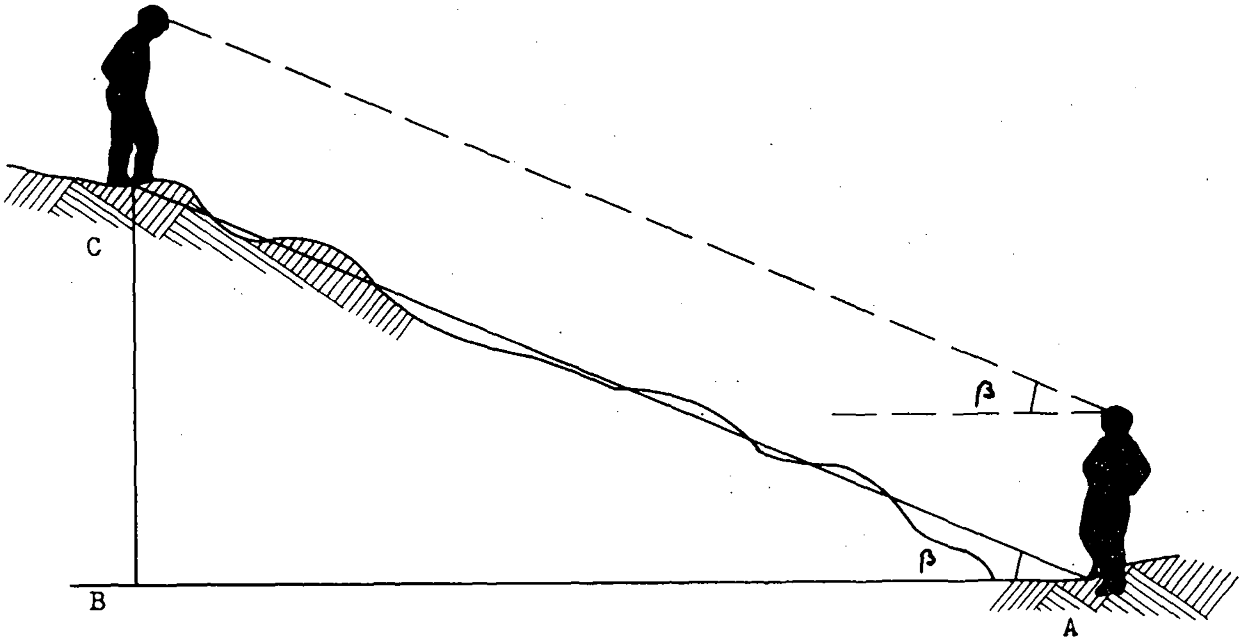


Fig. 5 Angle measurement

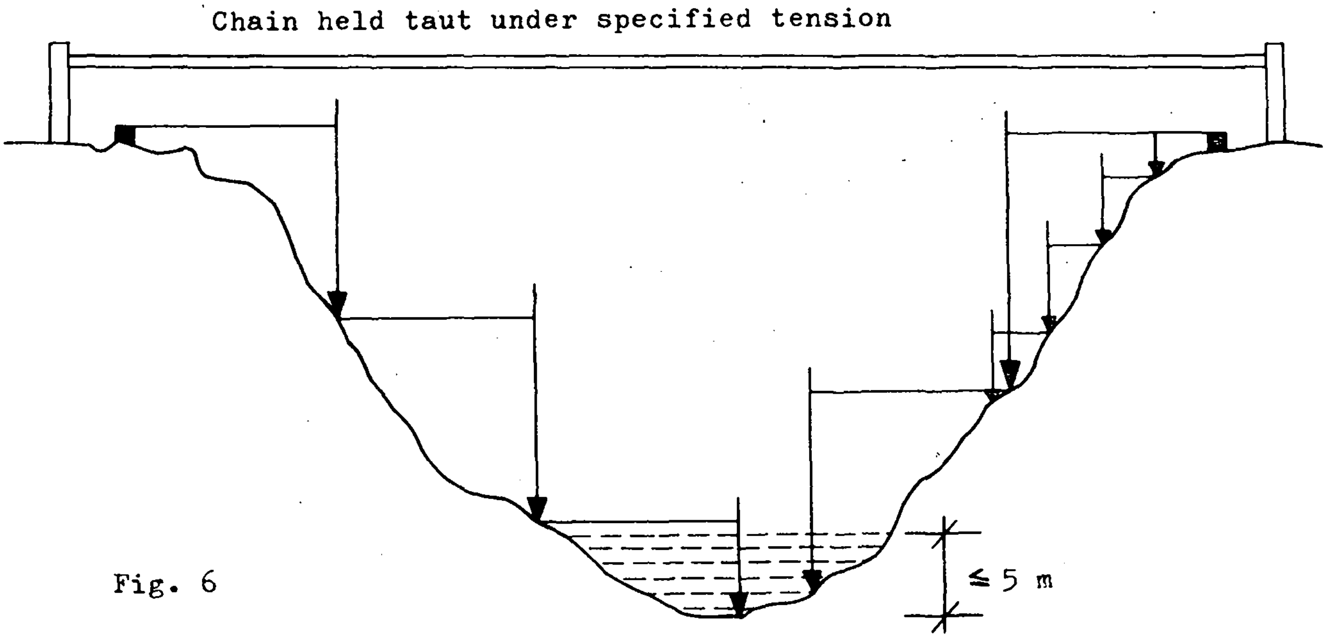


Fig. 6

Bed profile of a river

3. Levelling

3.1 Introduction

Levelling is the operation required for the comparison of points on the surface of the earth. The height of one point can be given only relative to another point or plans. If a whole series of heights is given relative to a plane, this plane is called a datum. In topographical work the datum used is the mean level of the sea.

The simplest type of level consists of a spirit level resting on top of a flat surface which is supported in such a way that the bubble of the level is centered. A line of sight may then be taken on to a graduated staff.

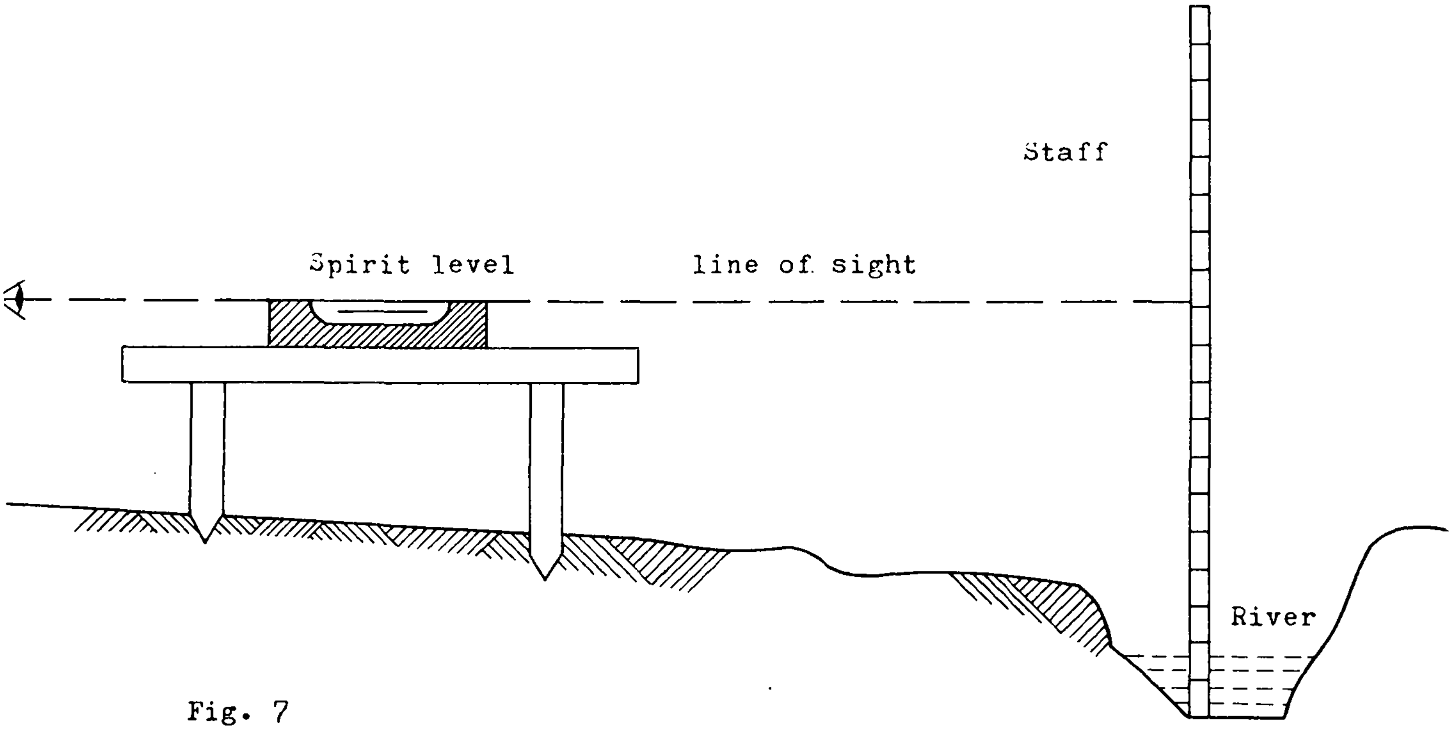


Fig. 7

This method has its limitations and is suitable only for very short lengths of sights (e.g., small building sites, paving work, etc.). For more accurate measurements, some better means of extending the horizontal line is needed.

The general features of the conventional level instrument are:

(a) Telescope to give extended lines of sight in horizontal plane, and

(b) Bubble tube to enable telescope to be brought horizontal.

3.2 Instruments and setting up

3.21 Dumpy level

- The telescope barrel and vertical spindle are cast in one piece (rigidly fixed).
- Bubble tube mounted on the telescope.
- Small circular bubble mounted on the upper plate of the levelling head enables a preliminary levelling up of the instrument to be made.
- Ideal for building purposes, very simple to use.

Setting up:

(1) Screw lower plate of level on the head of the tripod, whose legs have been opened and firmly fixed on the ground. Circular bubble should be nearly central; any excessive tilt on the instrument at this stage may lead to foot screw threads running out before accurate levelling is achieved.

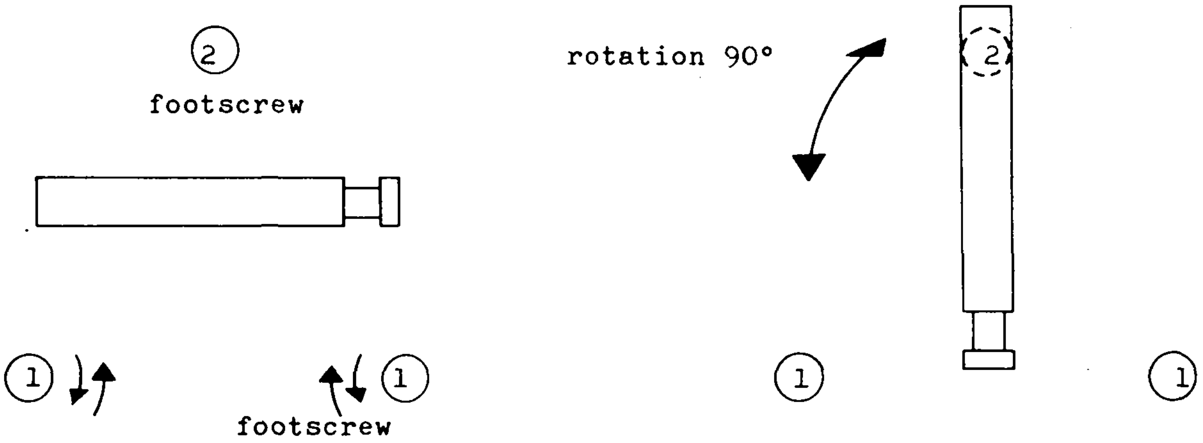


Fig. 8

(2) Telescope now turned so that it is parallel to screws 1-1, which are turned so that one moves clockwise and the other counter-clockwise simultaneously (i.e., both thumbs of the surveyor move towards each other or away from each other). The bubble is thus brought to the centre of its run.

(3) After turning telescope through 90°, the bubble is again brought to the centre of its run by the third screw. The bubble should remain centred in whatever direction the telescope is pointed if the instrument is in adjustment.

(4) Parallax between cross-hairs and the image of the levelling staff must be eliminated. To eliminate parallax: turn telescope to white background (paper, etc.) and focus the eyepiece so that cross-hairs appear clear and distinct. The eyepiece setting depends on the characteristics of the surveyor's eye, so that it will vary from one person to another.

Sight levelling staff and focus its image with the focusing screw so that when the eye is moved slightly, there is no relative movement between the image and the cross-hairs.

3.22 Tilting level

- The telescope is not rigidly fixed to the vertical spindle, but is capable of a slight tilt in the vertical plane. This movement is governed by a fine setting screw at the eyepiece end.

- The bubble is brought to the centre of its run for each reading of the level.

- Level may be mounted on the conventional three-screw base or on a ball-and-socket base (often described as 'quickset' level).

- The level base has a small circular bubble to enable a preliminary levelling up to be made.

- Most modern instruments are fitted with a prismatic coincidence bubble reader to eliminate excessive movement around the instrument. Both ends of the bubble are reflected into an eyepiece. Image split down the centre, one half of each end is seen next to the other half in the bubble reader eyepiece. Bring bubble into position ⓑ before each reading.

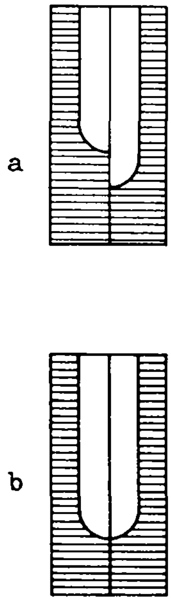


Fig. 9

Prismatic coincidence bubble reader

3.23 Automatic level

- Bubble tube is absent, preliminary levelling carried out using the conventional 3-screw head.

- Further levelling up is unnecessary, correction for any slight tilt being automatically made by a prismatic compensator (internally fixed).

- Ideally suited for all extensive levelling tasks because of its high accuracy and speed of observation.

3.3 Principle of levelling

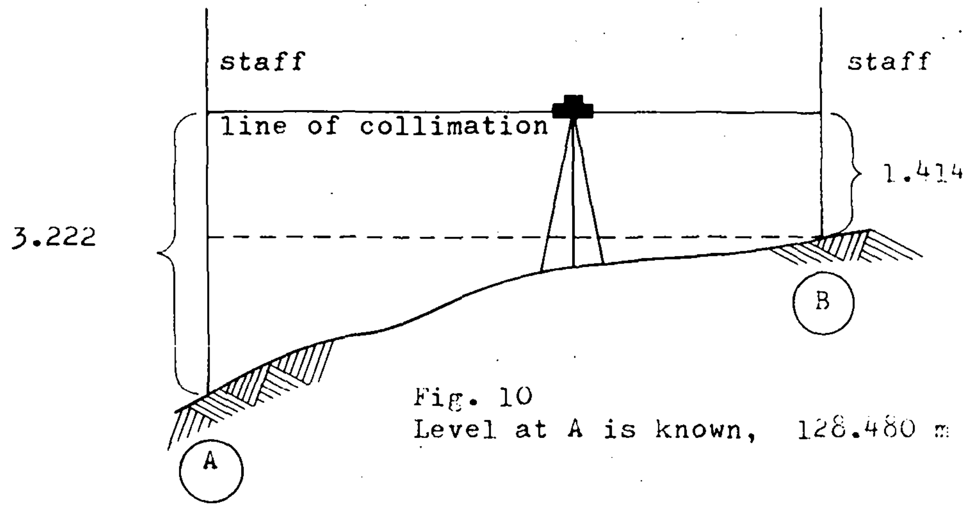


Fig. 10
Level at A is known, 128.480 m ; B = ¿

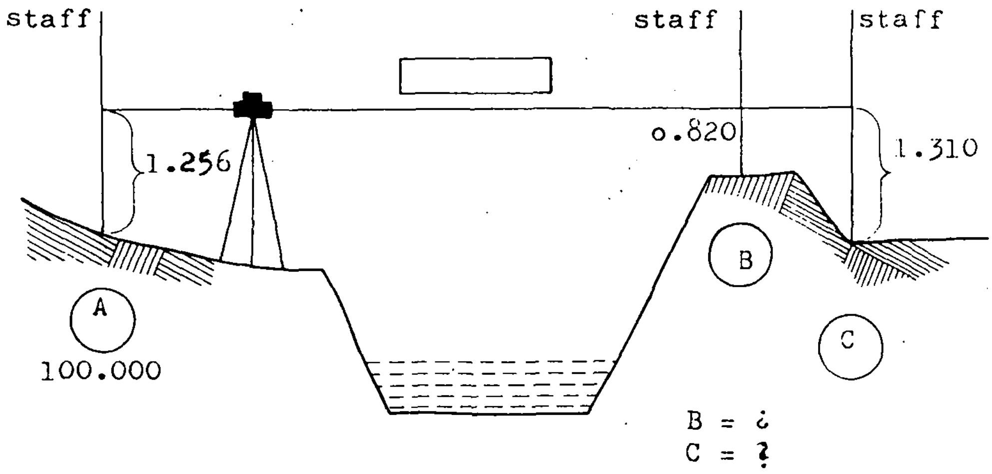


Fig. 11

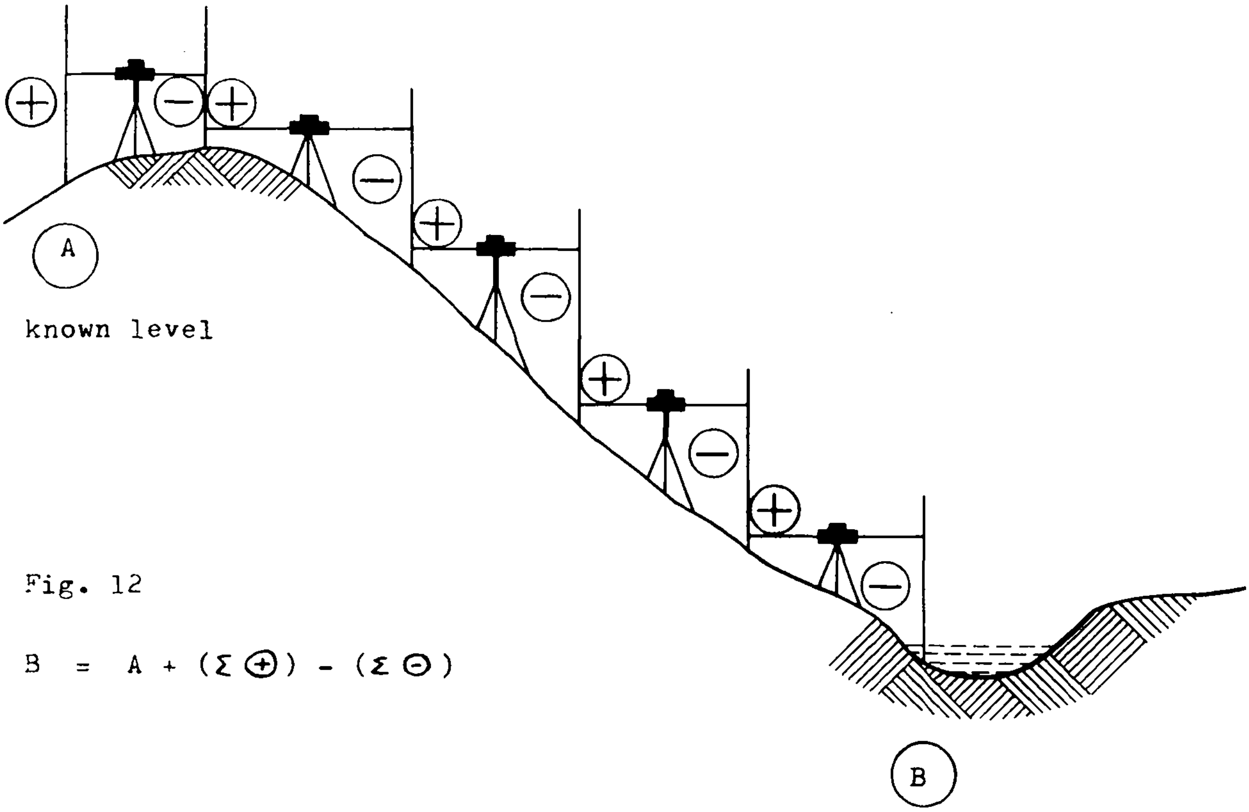


Fig. 12

B = A + (Σ ⊕) - (Σ ⊖)

Example

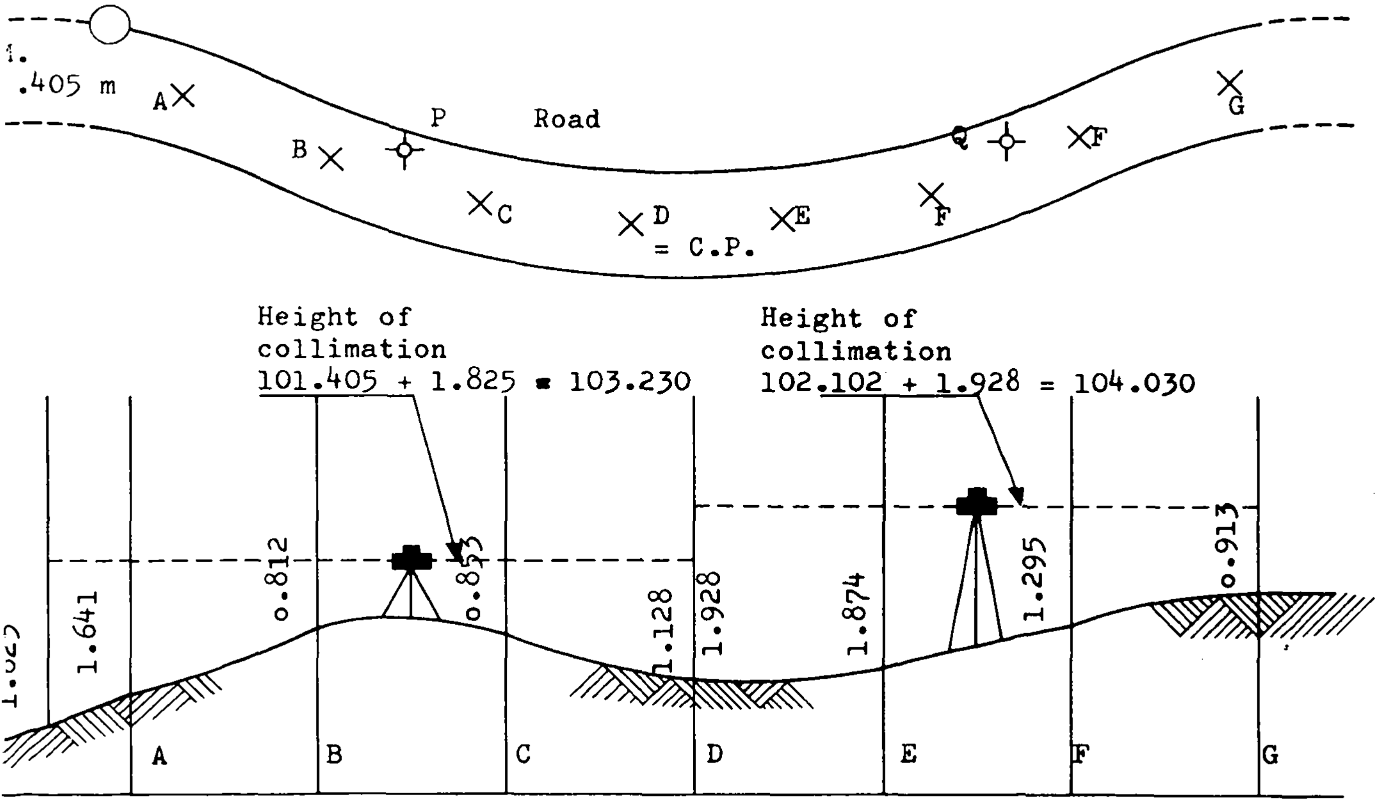


Fig. 13

General procedure

(1) Set up instrument at a convenient position P such that a benchmark (BM) may be observed.

(2) This first reading made with the staff on a point of known reduced level (which need not be a bench mark) is known as a backsight (BS). This term to be used to denote that reading taken immediately after setting up the instrument, with the staff on a point of known level.

(3) The staff is now held at points A, B, C, in turn, and readings, which are known as intermediate sights, are taken.

(4) It is found that no readings after D are possible, due either to change in level of ground surface, obstructions to the line of sight or long distances; and it is necessary therefore to change the position of the instrument.

(5) The last reading on D is known as foresight (FS) and is the final reading taken before moving the instrument. The point D itself is known as change point (CP) because it is the staff position during which the position of the level is being changed.

(6) The instrument is moved to Q, set up and levelled and the reading, a backsight, taken on the staff at the change point D, followed by intermediate sights with the staff on points at which levels are required, until further change becomes necessary, resulting on a foresight on point G.

(7) This procedure is repeated until all the required levels have been obtained.

3.4 Booking

Station	Target	Back-sight	Fore-sight	Inter. sight	Height of collimation	Reduced level	Lower stadia upper	Distance (m)	angle observed	Remarks
						101.405				
P	B.M.	1.825					2,225 1.425			
	A			1.641			1.851 1.430			
	B			0.812			1.312 0.310			
	C			0.853			1.453 0.250			
	D = C.P.		1.128				2.028 0.228			
Q	D = C.P.	1.928					2.428 1.428			
	E			1.874			1.994 1.750			
	F			1.295			1.499 1.091			
	G = C.P.		0.913				1.813 0.013			

Remarks

(1) Read accurately, estimate to 1 mm when making the staff reading.

(2) Enter each reading on a different line.

(3) Read both lower and upper stadia (for control and distance)

(4) Height of collimation is obtained by adding the staff reading, which must be a backsight to the known reduced level (RL) of the point on which the staff stands.

(5) All other readings are deducted from the height of collimation until instrument setting is changed.

(6) The new height of collimation is determined (by adding backsight to the RL at the CP.

(7) For lengthy levelling operations where pages of readings are involved, book the last reading on each page as a foresight, then as a backsight on the top of the next page.

(8) All bookings in ink only - no pencil.

3.5 Accuracy in levelling

The main errors affecting accuracy in levelling are:

- errors in reading the staff
- errors due to the bubble not being centred
- errors due to the instrument not being in adjustment
- errors due to differential settlement of the tripod
- errors due to tilting and settlement of the staff.

3.6 Uses of levelling

- determining the difference in level between 2 points
- taking of longitudinal sections
- cross sections
- contouring
- setting out levels
- measuring distances

3.61 Measuring distances with a level instrument

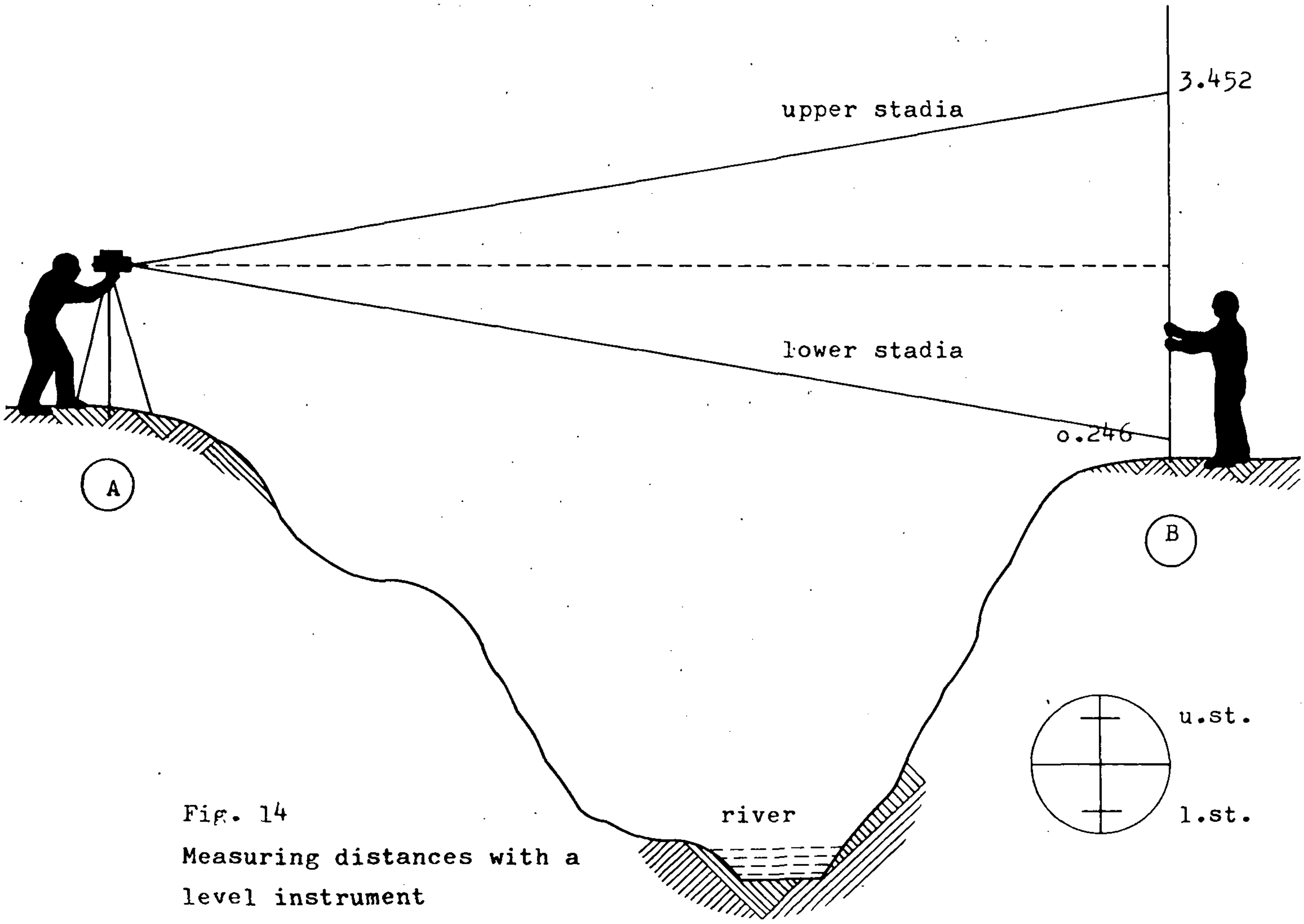


Fig. 14
Measuring distances with a level instrument

The difference in the readings between lower stadia and upper stadia gives the distance between the level instrument and the staff. The reading difference in cm gives the real distance in m.

Example:

Reading upper stadia = 3,452 m
Reading lower stadia = 0,246 m

difference = 3,206 m
= 320,6 cm

Distance in m between A and B = 320,6 m

3.7 Levelling practice

3.71 Closed level circuit

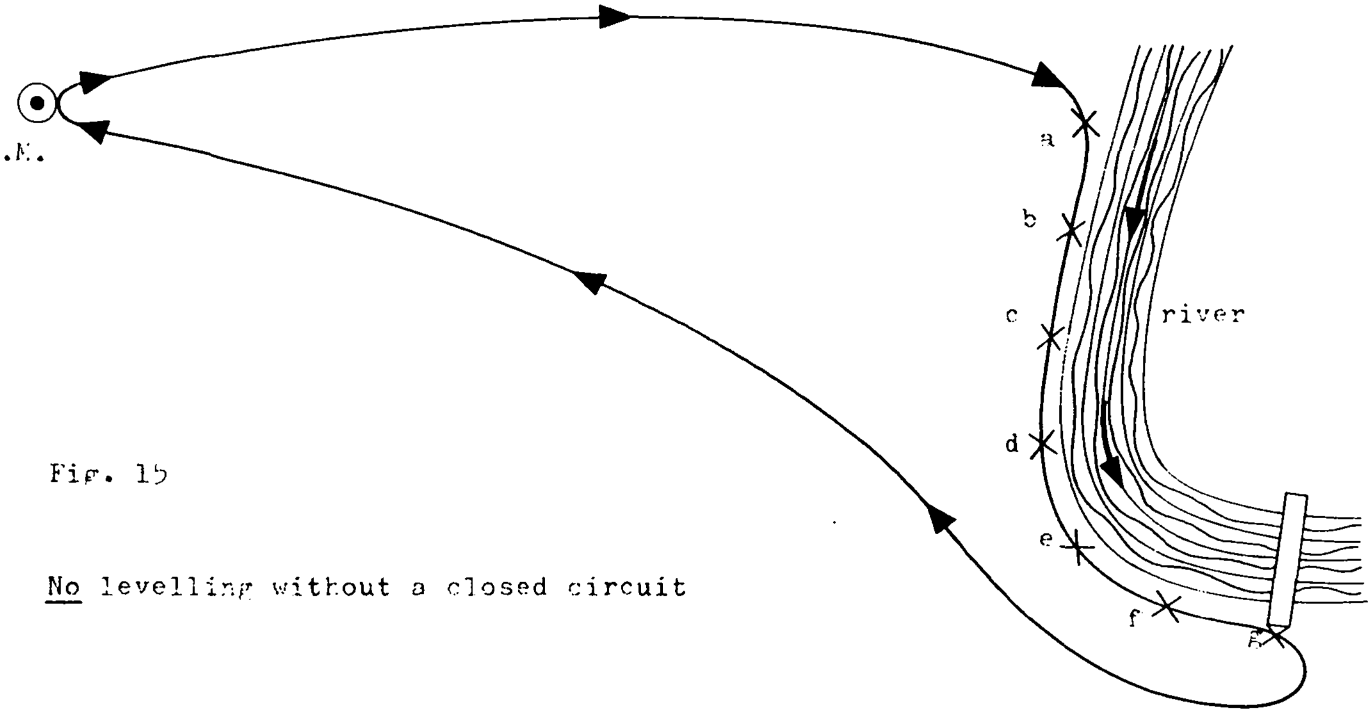


Fig. 15

No levelling without a closed circuit

3.72 Line of levels

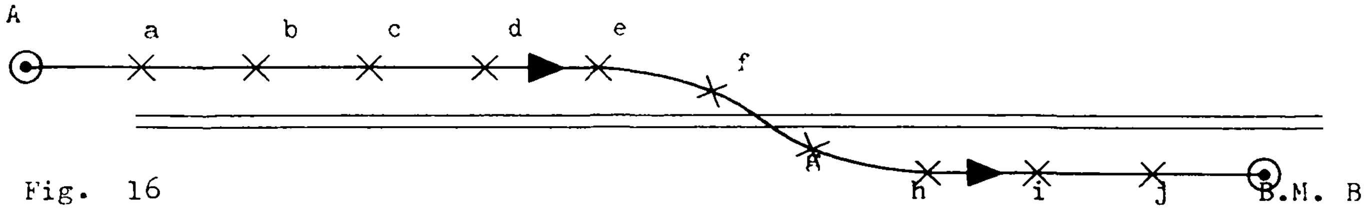


Fig. 16

If B.M. A and B.M. B have known elevations, the circuit is considered closed

3.73 Closing errors

Closing errors are inevitable and occur in any survey. The limit of closing error depends on the order of accuracy demanded.

3.74 Remember:

- Set tripod on firm ground.
- Sighting distances of about 50m to 60m may be regarded as normal.
- Set instrument approximately midway between change points.
- Change points: choose a solid base (a sharp stone, etc.) or a special ground plate.
- Hold staff always vertically.
- Readings: estimate to mm; read, note in pen (no pencil) on your fieldsheet, read again and check your booking. Note as well lower and upper stadia.
- Accidental errors may be either negative or positive - they partly cancel each other.
- Never leave your level instrument without watching it. Avoid bumping, rain or too much sun (get sunshade).
- Calculate line of collimation and elevation of change points directly on your field sheet.
- Check in the field and before you return to your office whether your level operation was carried out correctly.

3.8 Exercises

(1) Measure and level a given profile near the training course location. Draw the profile in scale 1:100.

(2) Level several given points near the training centre.

(3) Calculate the gradient of a nearby stream channel between two given points.

(4) Level and draw a given cross-section on the stream.

D. PRECIPITATION

1. Precipitation and its processes

1.1 Clouds and their formation

Clouds are a mass of tiny water droplets of various sizes which are so light that they remain suspended in the air or, otherwise, through which an upward air current keeps passing and hence suspends them. Movement of clouds in the horizontal plane means that there is a wind in the same direction but blowing faster than the speed of the cloud.

A sample of air containing any amount of moisture will release it if cooled to a temperature where condensation sets in, i.e., its dew point temperature, T_d. However, sometimes cooling does not produce condensation and the air becomes super cooled. This is due to lack of condensation nuclei, i.e. a particle on which condensation starts. If there are no particles then the cooling continues until a tiny ice particle is formed, then condensation starts.

The particles which condensation starts on come from chemical smokes, combustion, dust, and sea-salt spray which have radii ranging from 10 μm down to 10^{-3} μm (where μm = 10^{-6}).

1.2 Precipitation and its formation

All clouds can precipitate if the conditions are favourable. These conditions are, generally, in terms of temperature, moisture, air movement (especially upwards) and rate of droplets formation.

Water droplets grow mainly by coalescence into a water cloud resulting from collisions. The collisions are mainly due to different droplet sizes with different terminal velocities and a strong upward current which moves small droplets up against the downcoming larger droplets. This process occurs in shallow layer clouds and produces precipitation called drizzle. It is characterized by small drops of size slightly above 100 μm. Deep layer clouds produce rain which has drop size up to 2 mm in radius, and the deepest cumulonimbus clouds, extending for about 10 km, produce very heavy rainfall with all dropsizes represented and including those much above 2 mm.

The process of coalescence only is too slow for production of rain or heavier precipitation, hence another method results from so called mixed clouds. These contain both water droplets and ice-crystals. The moist air near the ice-crystal is super saturated hence condenses easily on the ice surface making it heavier and the ice-crystal starts falling colliding and coalescing with more droplets. As the drops fall below the 0^{o}C level they melt fully or partially producing rain or rain mixed with ice, i.e., sleet. Precipitation in form of hail (hailstorm) is produced from deep anvil shaped cumulonimbus clouds which have very strong upcurrents and extend several kilometers in height and reach well above the 0^{o}C level. The repeated up and down circulation of the drop or ice crystal produces the big hailstones.

1.3 Measurement of precipitation

The commonest method of rainfall measurement is by 13 cm or 20.5 cm diameter funnel raingauge. The raingauge measures the rain that has fallen on an area in depth of millimeters (mm). In order to ensure that the raingauge has "caught" not more nor less than what is falling elsewhere, the following points should be considered during its installation:

- deficiencies in the collecting instruments
- unsatisfactory siting to represent an area
- insufficient or poorly planned network

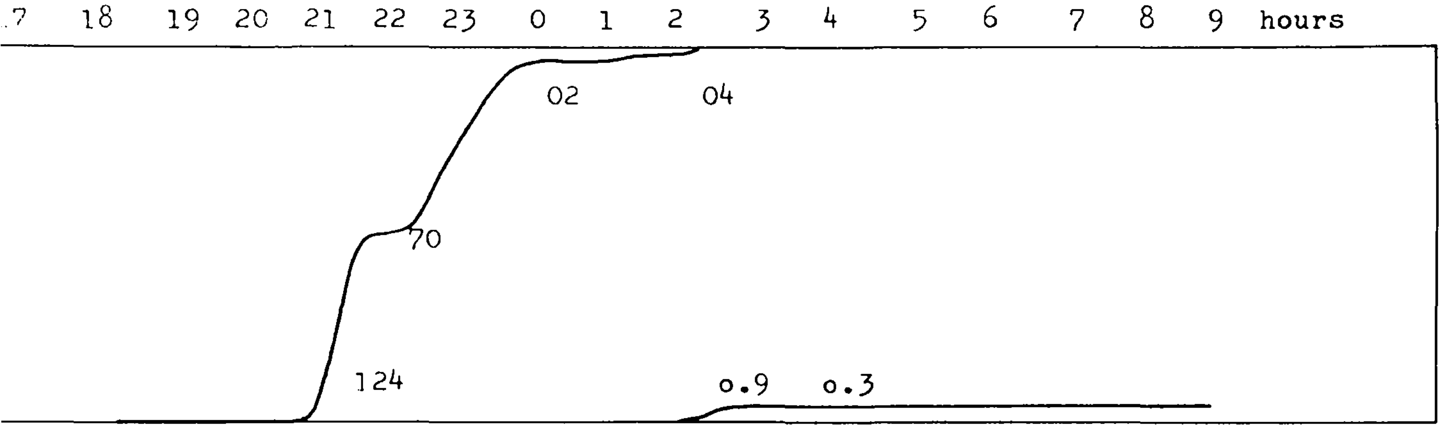
Some of the rainfall is lost by missing the gauge due to winds which cause the rain to come at an angle. Shielding and special construction of the gauge can increase the "catch" by about 20% in high winds, however for low winds the error is usually negligible. For East African networks the standard gauge is the 13 cm diameter mounted 30 cm above the ground. The gauge should be away from obstructions which can influence its catch, such as buildings, trees, depressions or raised ground. Proper siting should ensure that a gauge is placed 30 cm above ground and at least a distance twice the tallest obstruction from it. It is equally detrimental to put a gauge in an open field where most of the surrounding area was scattered with trees and bushes. For such a case local turbulence in the open field can cause it to catch less rainfall. The network planning is important as it directly affects the areal representation of the data collected. Generally the number of gauges is directly proportional to the proper areal representation of data collected, however for economical reasons maximum areas per gauge according to the type of terrain have to be determined, i.e. the minimum network density (see Table 1).

Table 1: Raingauge network of minimum density

Region or type	Normal tolerance area per station	Minimum tolerance in difficult conditions
Flat areas of temperate Mediterranean and tropical zones	600 - 900 km^2 about 30 km to next station	900 - 3000 km^2 about 50 km to next station
Same as above but mountainous	100 - 250 km^2 about 20 km to next station	250 - 2000 km^2 about 40 km to next station
Small mountainous islands with irregular precipitation	25 km^2	- - -
Arid and polar zones	1500 - 10000 km^2	- - -

For rainfall pattern studies it is necessary to have rainfall rates, i.e. the rate of precipitation with time. The most common is the daily rate which is available from ordinary raingauges. For longer periods of, e.g. one month or more, accumulator gauges (totalizers) are used. Some automatic recorders give data for longer than one-day periods. For shorter than one-day periods it is necessary to have continuous recorders. These recording gauges use a continuously rotating chart specially marked. The recorded rainfall is in cummulative mode, i.e. records the total rainfall up to that time.

A type of recorder used in East Africa produces a chart shown in fig. 1. If the pen reaches the top it moves down to zero and starts over again. The common types are a tipping-bucket recording gauges and syphoning recording gauges.



Recording raingauge chart (note the "full-up" condition at é am and the movement of the pen down to zero)

1.4 Representing rainfall on maps and graphs

Rainfall can be represented on a map by direct plotting of the amounts against the station (synoptic maps). This can then be joined by lines of equal amounts - isohyets. Also lines of equal intensities derived from pluviographs analysis can be drawn. Areal rainfall representation is the determination of the rainfall amount which best represents the rainfall over that area. The following methods are the most common ones used to determine the average or representative rainfall over an area:

Averaging. The rainfall as reported from raingauges within the area is added up and divided by the number of raingauges. Good for flat areas with evenly distributed rainfall.

Thiessen. The area under consideration is divided into subareas around each raingauge and these subareas given fractional areas of the whole catchment. Each rainfall amount reported is weighted by this fractional area, "weight", and then added. The subareas are determined by the lines joining the perpendicular bisectors of the lines joining it to the nearest surrounding raingauges (see fig. 3). Good for large flat areas with uneven distribution of rainfall.

Isohyetal. The plotted rainfall amounts are joined by lines of equal amount taking into consideration the topography of the area. The subareas cut within the "isohyets" are treated as in Thiessen method; however, in this case the rainfall amount weighted is the mean of the enclosing isohyets (see fig. 4). The most accurate for all conditions.

Graphical methods are used to represent rainfall, especially if it is going to be analysed and the means standard deviations and other significant parameters evaluated. The two common graphical methods are: mass curve and hyetograph. The former is a plot of the accumulated rainfall vs. time (hours, days, months or years) and the latter is the rainfall intensity (mm/hr) vs. time. This is normally used for periods of less than a day - sometimes down to minutes. Examples of both graphs can be seen in fig. 2.

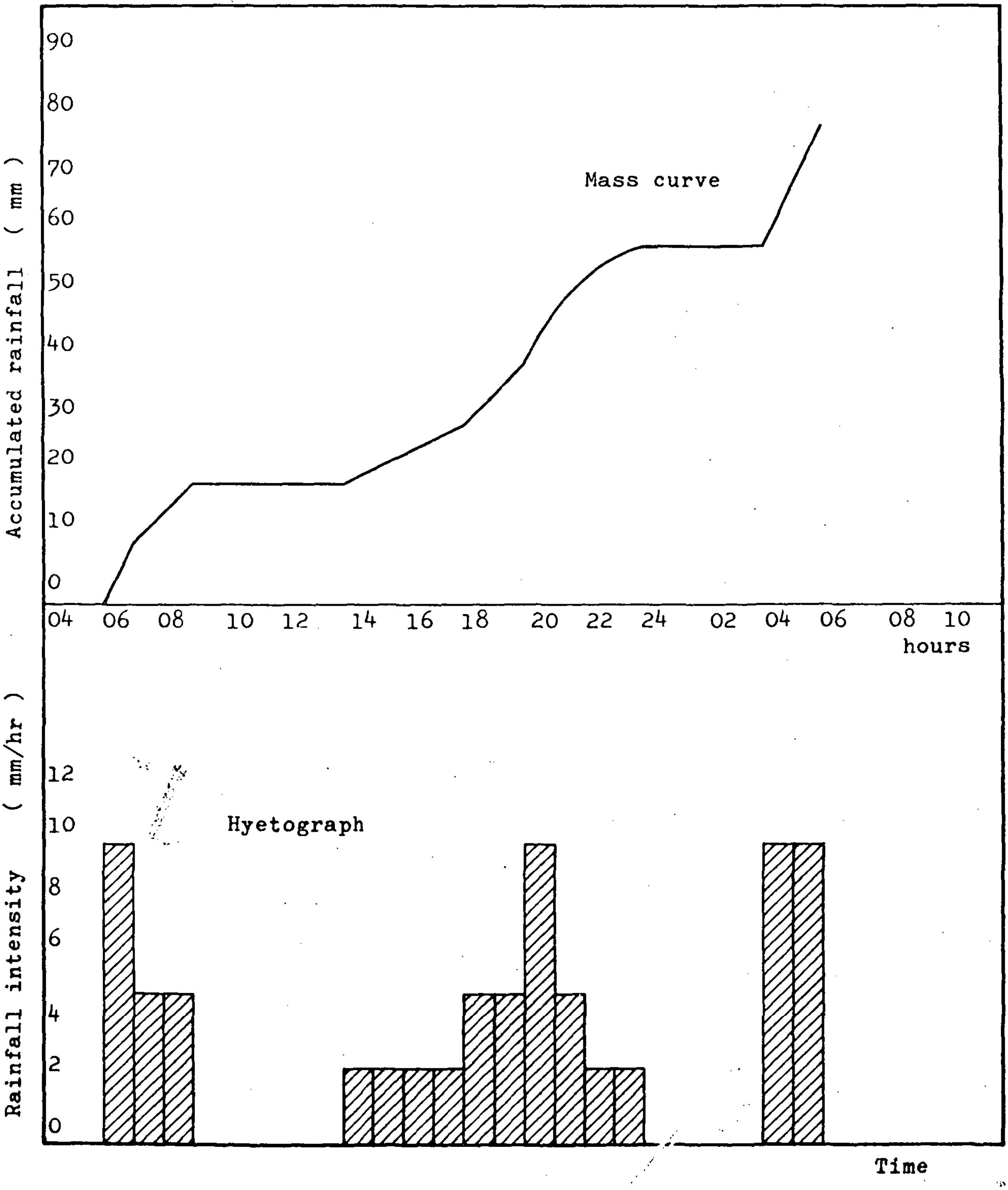


Fig. 2 Mass curve and hyetograph

1.41 Thiessen method (refer to fig. 3)

Basin precipitation is determined by the following steps:

(1) Draw the map of the basin which you are interested in and find its area using the scale on the map and planimeter. Express the area in either km^2 or m^2.

(2) Put on the map raingauge-stations which have data or those where data is reliably obtained. (Remember that if there is no rainfall report from one station, the whole procedure has to be repeated).

(3) Also include raingauge-stations which are nearby but outside the basin where data can be obtained.

(4) According to fig. 3, raingauge-stations (1), (2), (3), (9), (12), (13), (14) are outside and (3), (4), (6), (7), (8), (10), (11) are within the basin.

(5) Join the raingauge-stations with straight lines making sure that none cross each other, i.e. these lines should be to the nearest surrounding stations.

(6) Divide these lines into two equal parts, i.e. bisect these lines.

(7) Through the point of bisection, draw lines which are perpendicular, i.e. at 90° to the lines joining the stations.

(8) If this is done properly every three perpendicular bisectors should meet at a point. Make sure that these lines meet at a point. Slight adjustment of the line is not detrimental in order to meet at one point.

(9) Draw thick lines along these lines right around the station ignoring the lines which join the stations. Now each station should be within an enclosure of these perpendicular bisectors (drawn in thickly).

(10) Continuation outside the boundary of the basin is not necessary. Notice that the stations outside the basin also have areas within the basin to which they will contribute rainfall. Note also that Station (1) has not contributed at all to the basin precipitation.

(11) Find the sub-areas by planimetering or using graph paper and counting squares. If we call each sub-area by the number of station, then the areas to be planimetered are A2, A3, A4, A5, A6, A7, A8, A9, A10, A11, A12, A13, A14.

(12) The Thiessen weight of each sub-area is the area of the sub-area divided by the total area of the basin. If we denote the weight of each sub-area by W, then weight of A2 is

$W2 = A2/(A2 + A3 + A4 \ldots + A13 + A14)$

$W3 = A3/(A2 + A3 + A4 \ldots + A13 + A14)$

……………………………………

……………………………………

$W14 = A14/(A2 + A3 + A4 \ldots + A13 + A14)$

Make sure that $W2 + W3 + W4 + \ldots + W14 = 1.00$

(13) If the rainfall recorded by each station is P1, P2, P3 ... P14, then the mean basin precipitation is:

$(P1 \cdot W1 + P2 \cdot W2 + P3 \cdot W3 + P4 \cdot W4 + \ldots + W14 \cdot P14)$

For this basin $W1 = 0$, therefore our basin precipitation P is

$P = P2 \cdot W2 + P3 \cdot W3 + \ldots + P14 \cdot W14$

(14) If one of the stations does not report any rainfall whereas it did rain there, then new Thiessen weights have to be worked out excluding the non-reporting station.

1.42 Isohyetal method (refer to fig. 4)

Follow steps as under Thiessen method till step (5).

(5) Plot the rainfall amounts as reported by each raingauge station.

(6) Draw isohyets, i.e. lines of equal rainfall at convenient intervals. In this case intervals at 0.5 were used, but 1.0 can be used. Do not choose too big or too small intervals.

(7) Find sub-areas between these lines within the basin boundaries only.

area between 0 − 1.0 = A_0
" " 1.0 − 1.5 = $A_{1.0}$
" " 1.5 − 2.0 = $A_{1.5}$
" " 2.0 − 2.5 = $A_{2.0}$

……………………………………

……………………………………

" " 6.0 − 6.5 = $A_{6.0}$
" above 6.5 = $A_{6.5}$

Note that some sub-areas are in two places, so make sure both are put together.

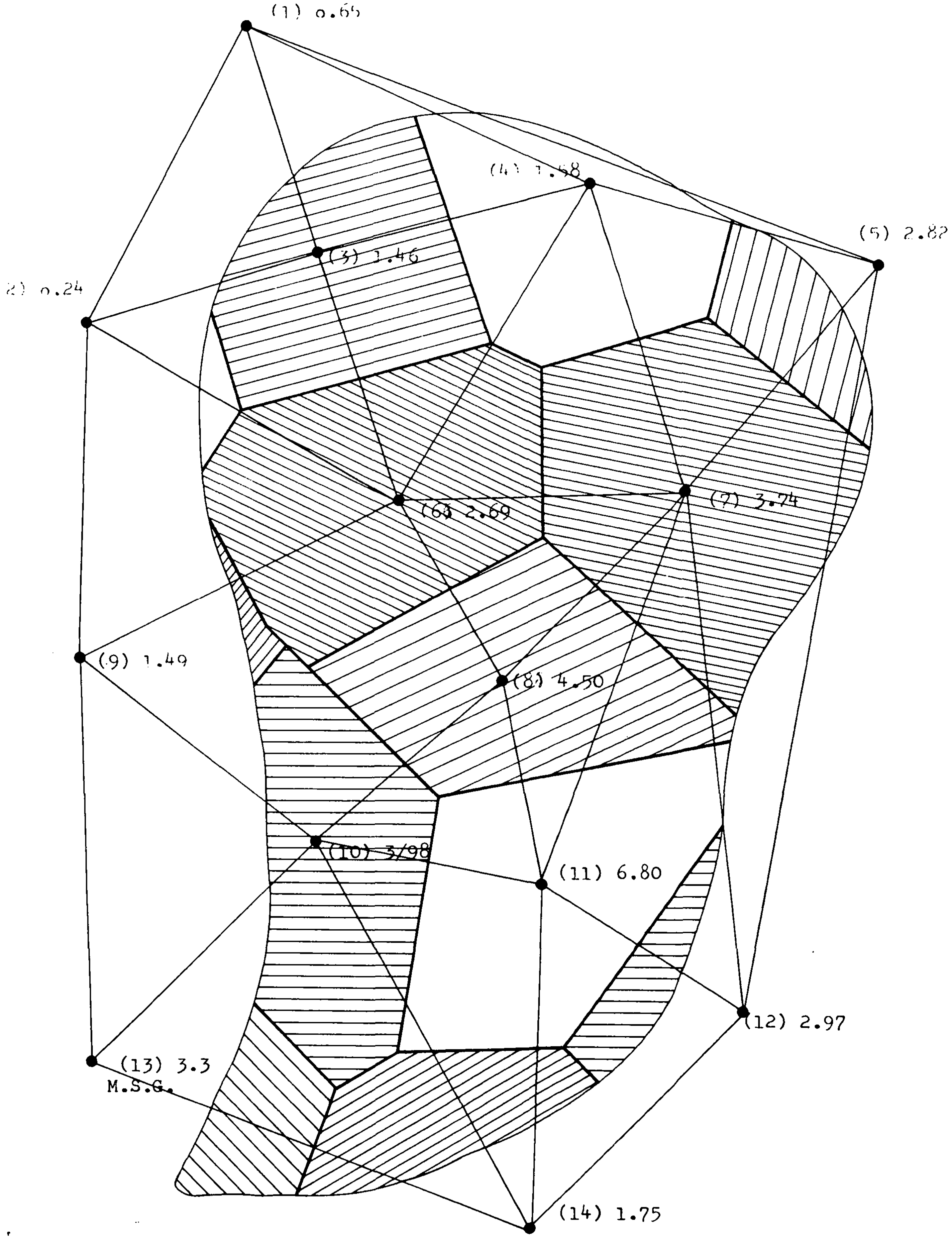


Fig. 3 Rainfall map for Uhuru River basin storm (in inches)

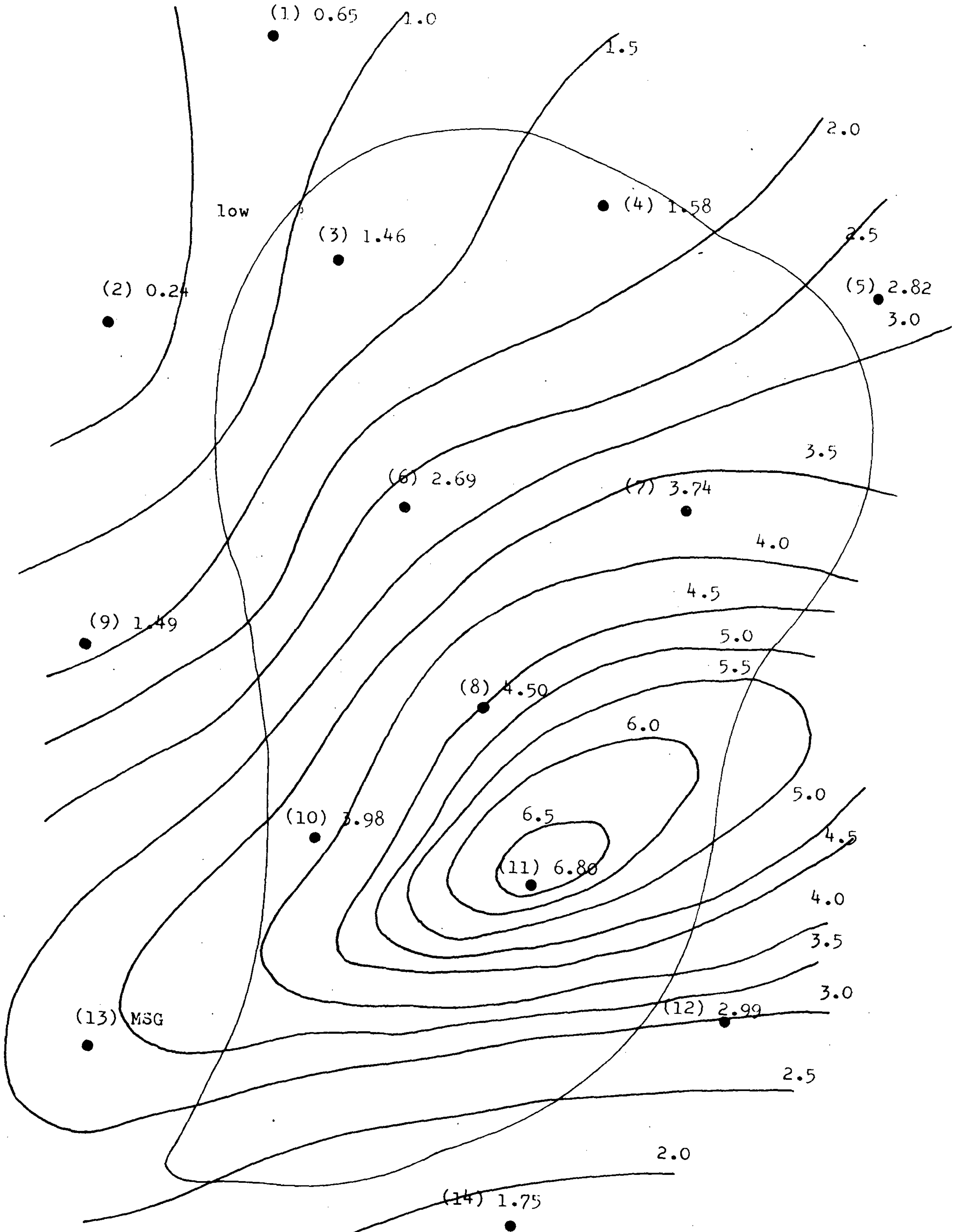


Fig. 4 Rainfall map for Uhuru River basin storm (in inches)

(8) If the total area of basin is A, then

$$A_0 + A_{1.0} + A_{1.5} + A_{2.0} + \dots + A_{6.0} + A_{6.5} = A$$

and the weights of the sub-areas are

$$W_0 = A_0/A$$
$$W_{1.0} = A_{1.0}/A$$
$$W_{1.5} = A_{1.5}/A$$
$$W_{2.0} = A_{2.0}/A$$

....

....

Make sure that $W_0 + W_{1.0} + W_{1.5} + \dots + W_{6.5} = 1.00$

(9) The average rainfall within each pair of isohyets is:

Sub-area	isohyets	average	
A_0	0 - 1.0	0.5	P_0
$A_{1.0}$	1.0 - 1.5	1.25	$P_{1.0}$
$A_{1.5}$	1.5 - 2.0	1.75	$P_{1.5}$
$A_{2.0}$	2.0 - 2.5	2.25	$P_{2.0}$
	....		
	....		
$A_{6.5}$	above 6.5	6.5	$P_{6.5}$

(10) The average basin rainfall P is given by:

$$P = W_0 \cdot P_0 + W_{1.0} \cdot P_{1.0} + W_{1,5} \cdot P_{1.5} + \dots + W_{6.5} \cdot P_{6.5}$$

(11) These isohyets have to be redrawn whenever different rainfall amounts are received. This is the best method but very tedious for repeated application.

(12) Note that the precipitation values obtained are not necessarily the same as in the Thiessen method.

2. Meteorological station

The collection centre of meteorological data is a meteorological station. This is distinct from a raingauge station because the meteorological station contains many instruments including raingauges.

For economic reasons the meteorological stations are not as many as rainfall stations and become even fewer for upper air observations. For example there are only four meteorological stations in East Africa which do upper air observations.

The meteorological station which will be considered is one which contains all possible surface data measuring instruments. Accordingly, a detailed description of instruments and their uses in a meteorological station will be given both diagramatically and verbally.

2.1 Visiting a meteorological station

For the purpose of seeing the instruments and how readings are made. Sample readings to be made. The station to be visited is the Dagoretti Met. Station at the Meteorological Department's Headquarters.

2.2 Instruments

Diagramatic description of instruments follows:

Wind	- standard wind-vane (fig. 5) - cup-counter anemonmeter (fig. 6) - pressure tube anemograph (fig. 7)
Atmospheric pressure	- Fortin barometer and Kew barometer (fig. 8) - precision aneroid barometer (fig. 9)
Temperature	- thermometer screen (fig. 10) - thermohygrographs (fig. 11)
Soil temp.	- bent stem soil thermometer (fig. 12)
Rainfall	- standard raingauge (fig. 13) - graduated measuring cylinder (fig. 14) - tilting-siphon rain recorder (fig. 15)
Evaporation	- standard Class A pan (fig. 16) - Piche evaporimeter (fig. 17)
Sunshine	- Campbell-Stores sunshine recorder (fig. 18)
Radiation	- Gunn-Bellani radiometer (fig. 19)

2.3 Upper air instruments

The commonly used upper air instrument is called a radiosonde which simultaneously measures:

- atmospheric pressure
- relative humidity
- temperature
- wind direction and speed by electronic circuitry

This instrument is attached to a balloon which floats it up into the air while it transmits back the information to a receiving centre.

Another instrument used in a pilot balloon which can give the direction and speed of upper wind only.

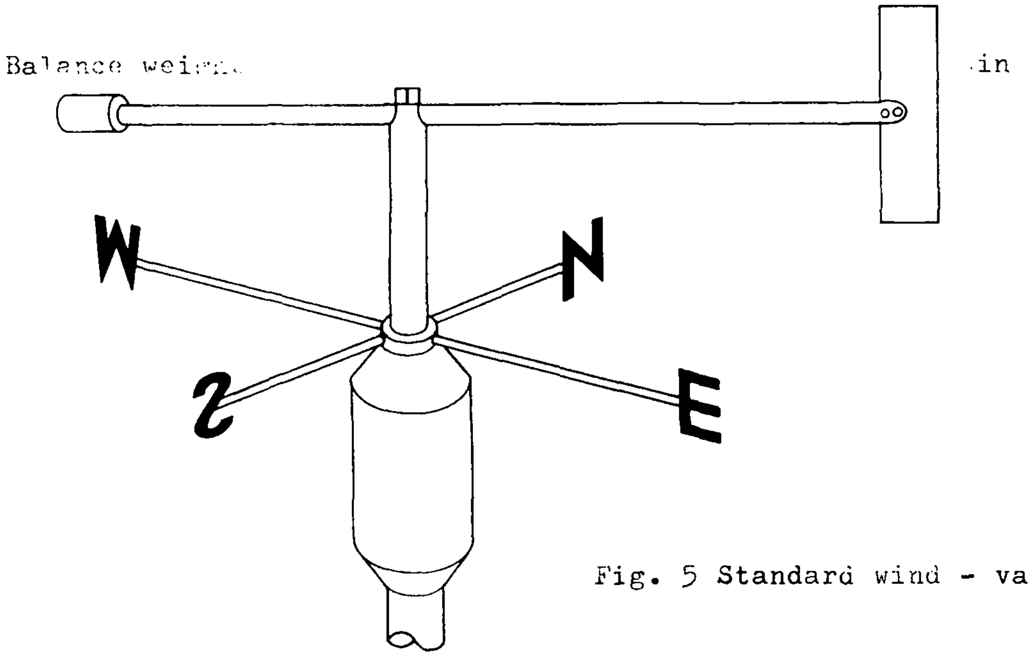


Fig. 5 Standard wind - vane

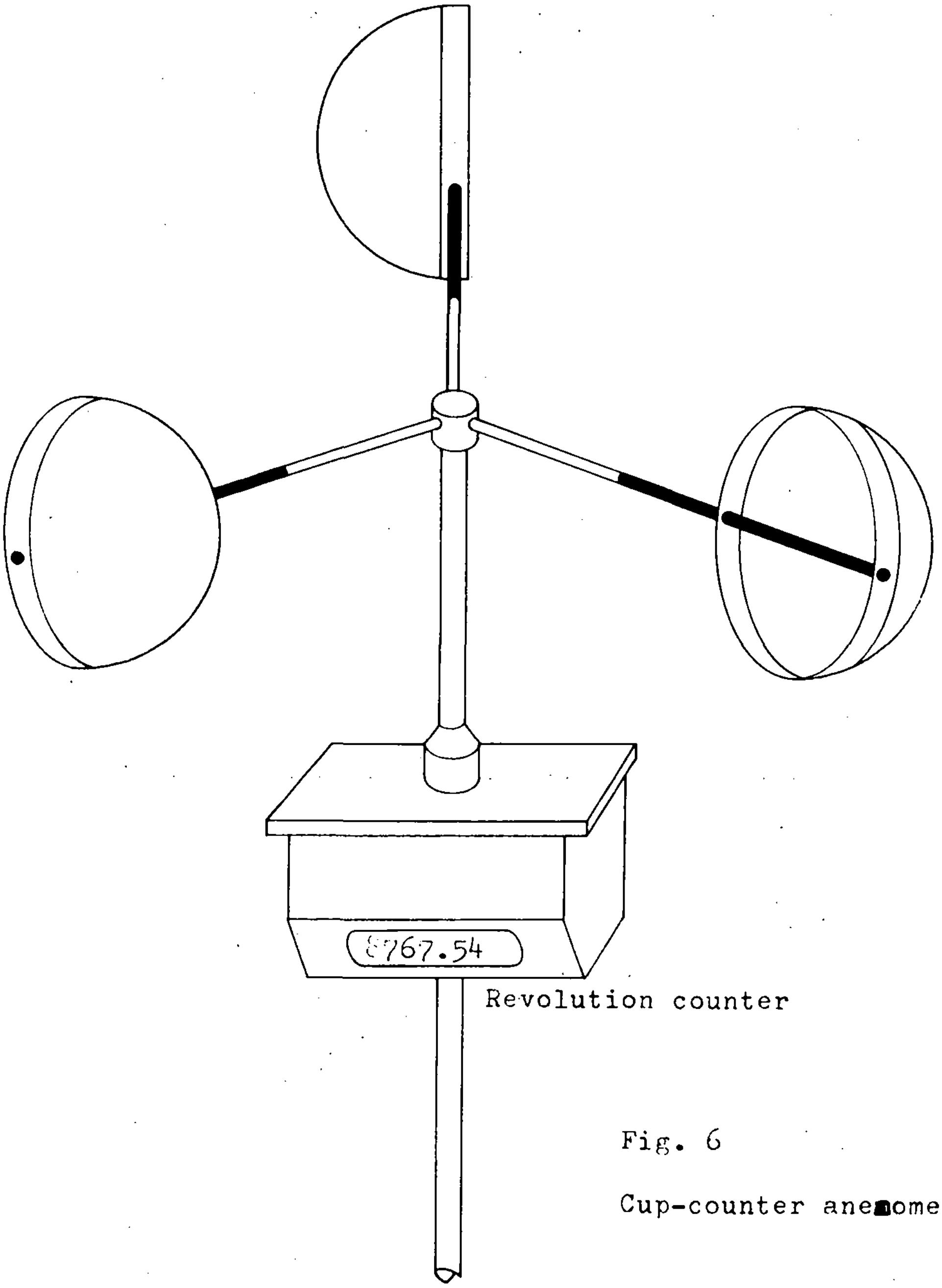


Fig. 6

Cup-counter anemometer

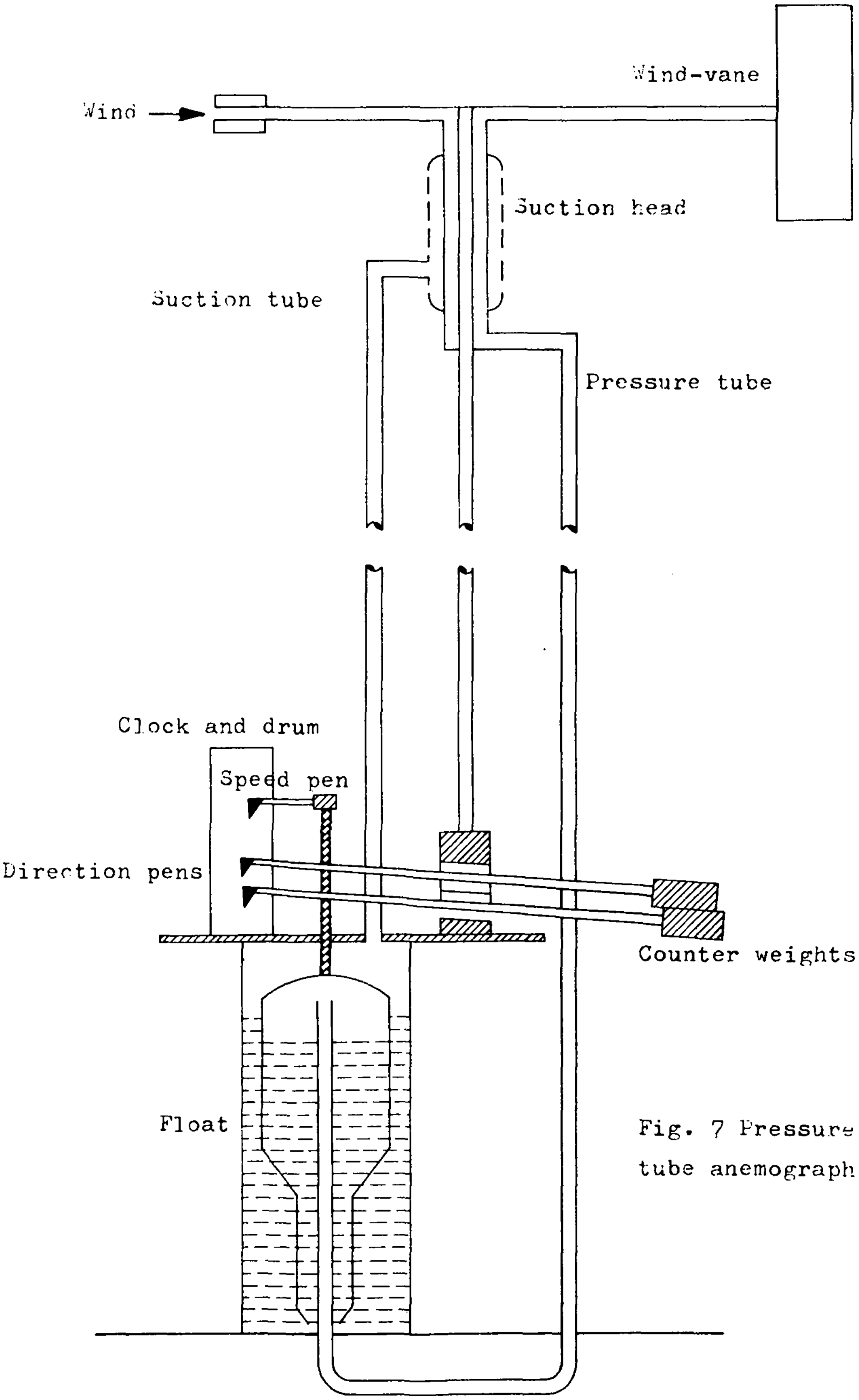


Fig. 7 Pressure tube anemograph

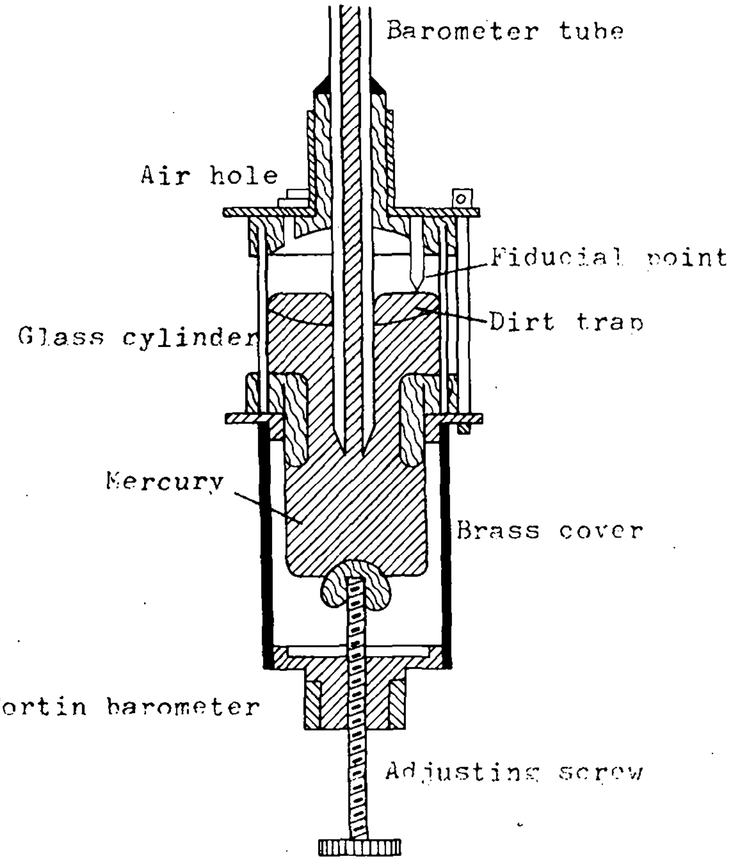


Fig. 8a

Cistern of a Fortin barometer

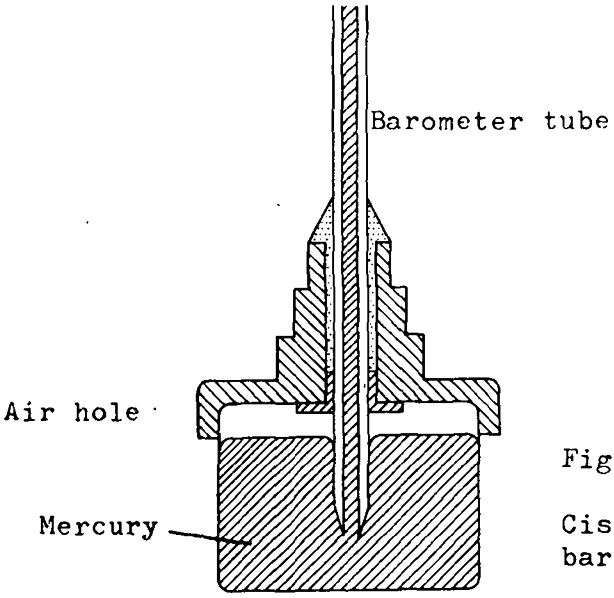


Fig. 8b

Cistern of Kew barometer

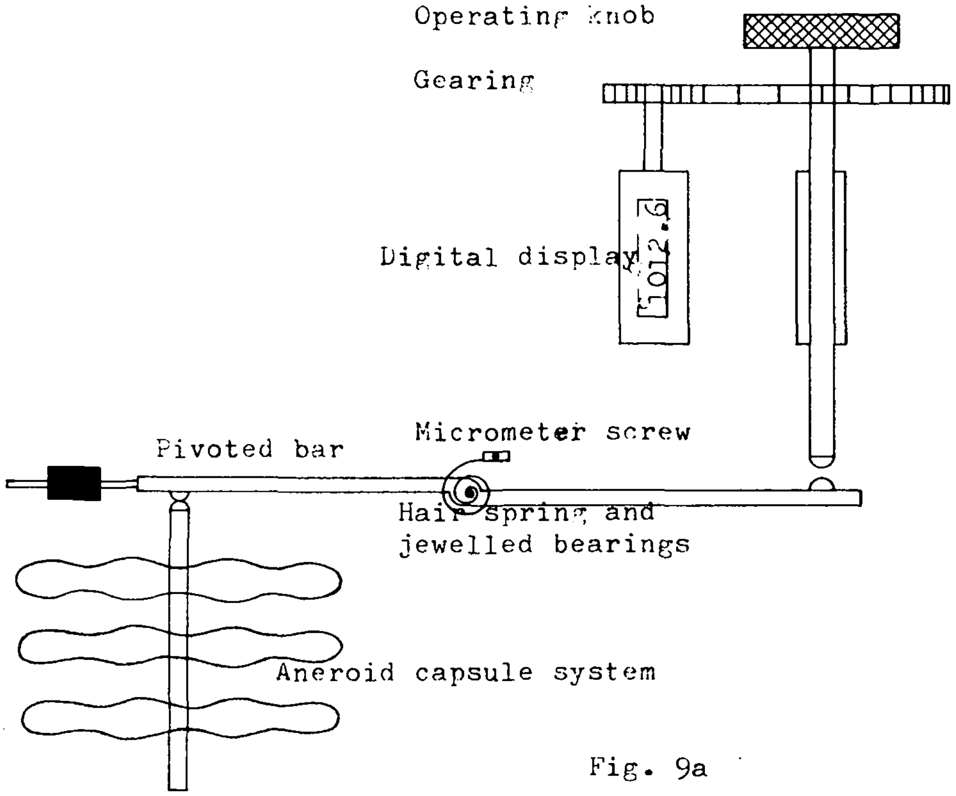
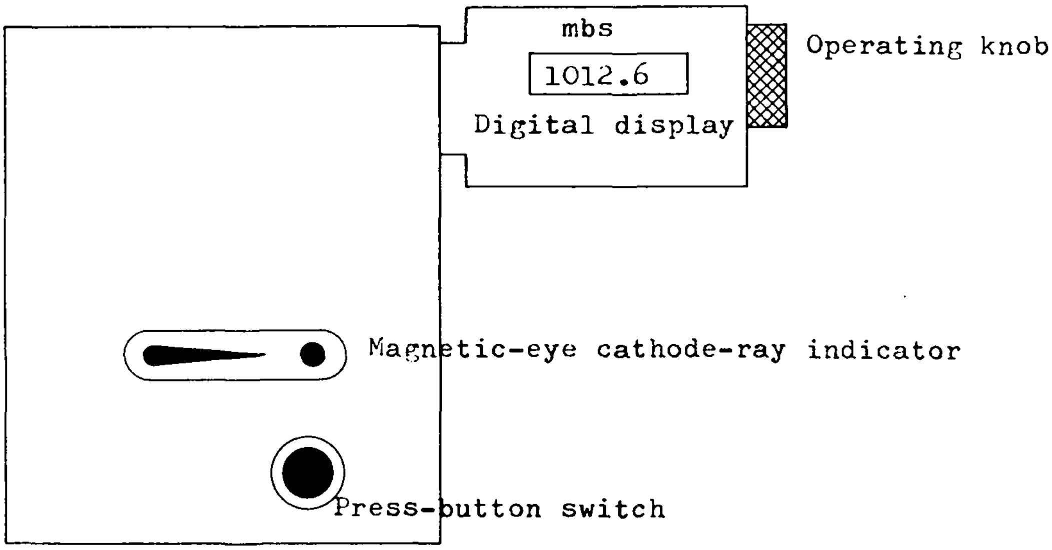


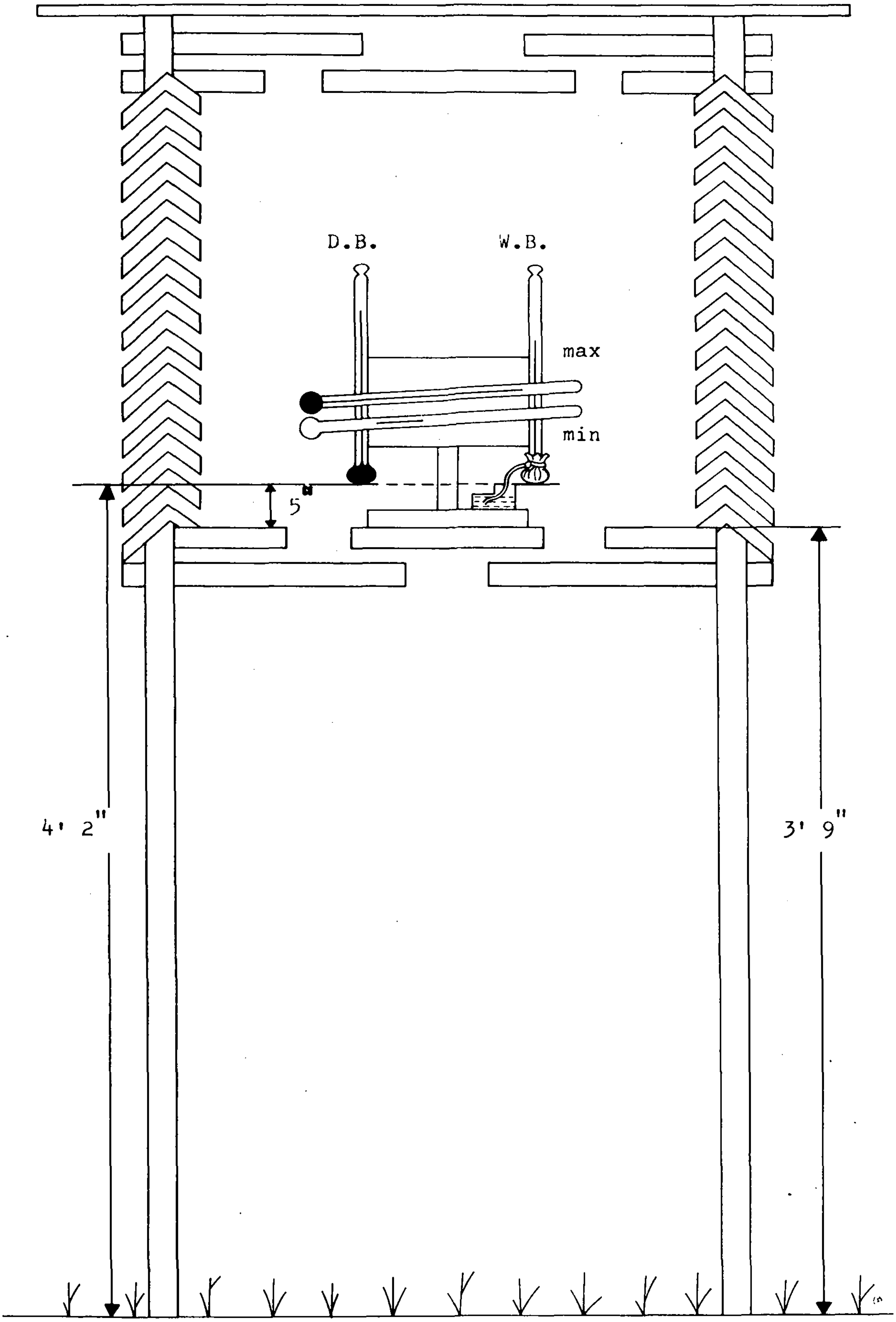
Fig. 9a
diagrammatic view of working parts



b) General view from above

Fig. 9 Precision aneroid barometer

Fig. 10 The thermometer screen



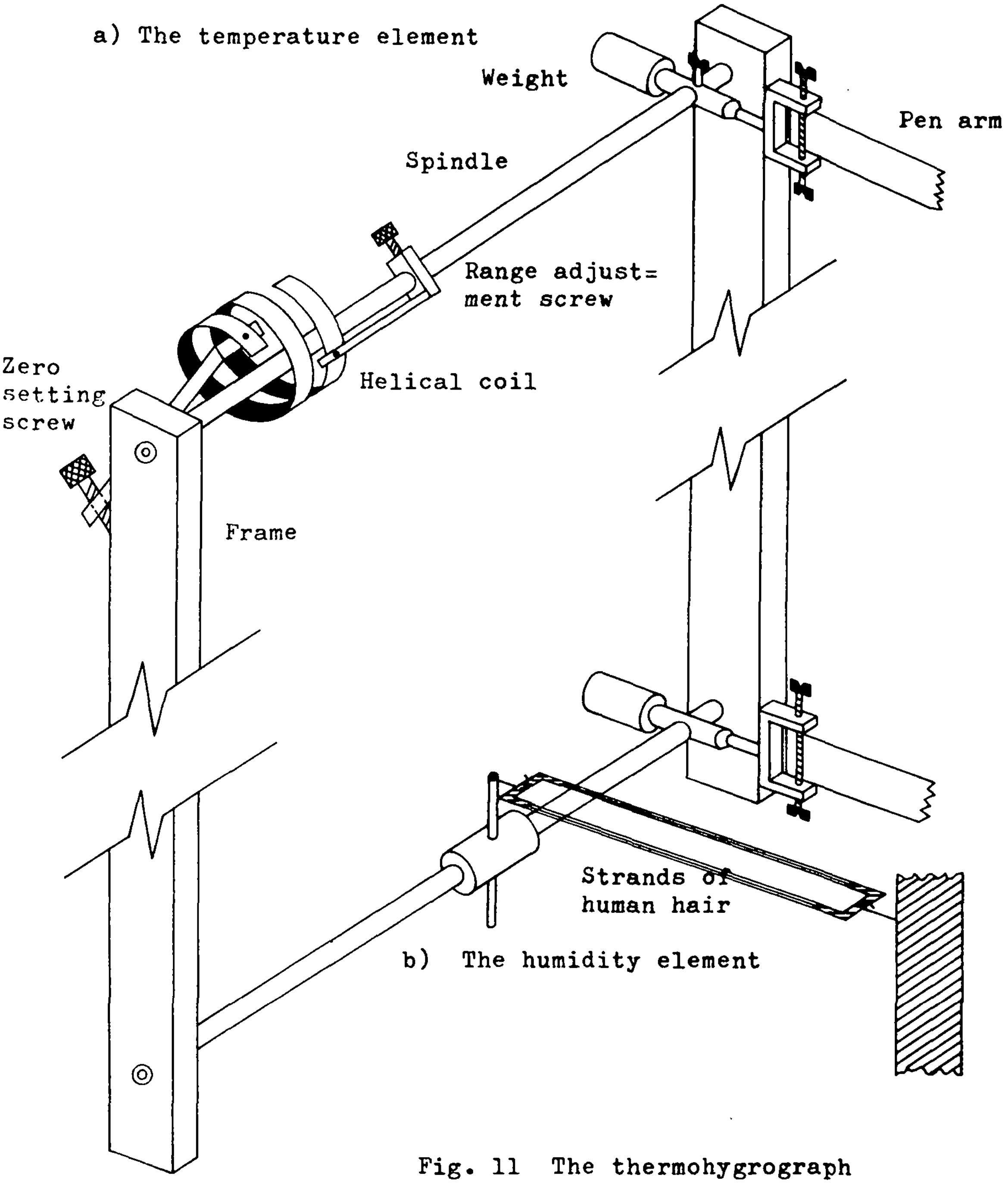
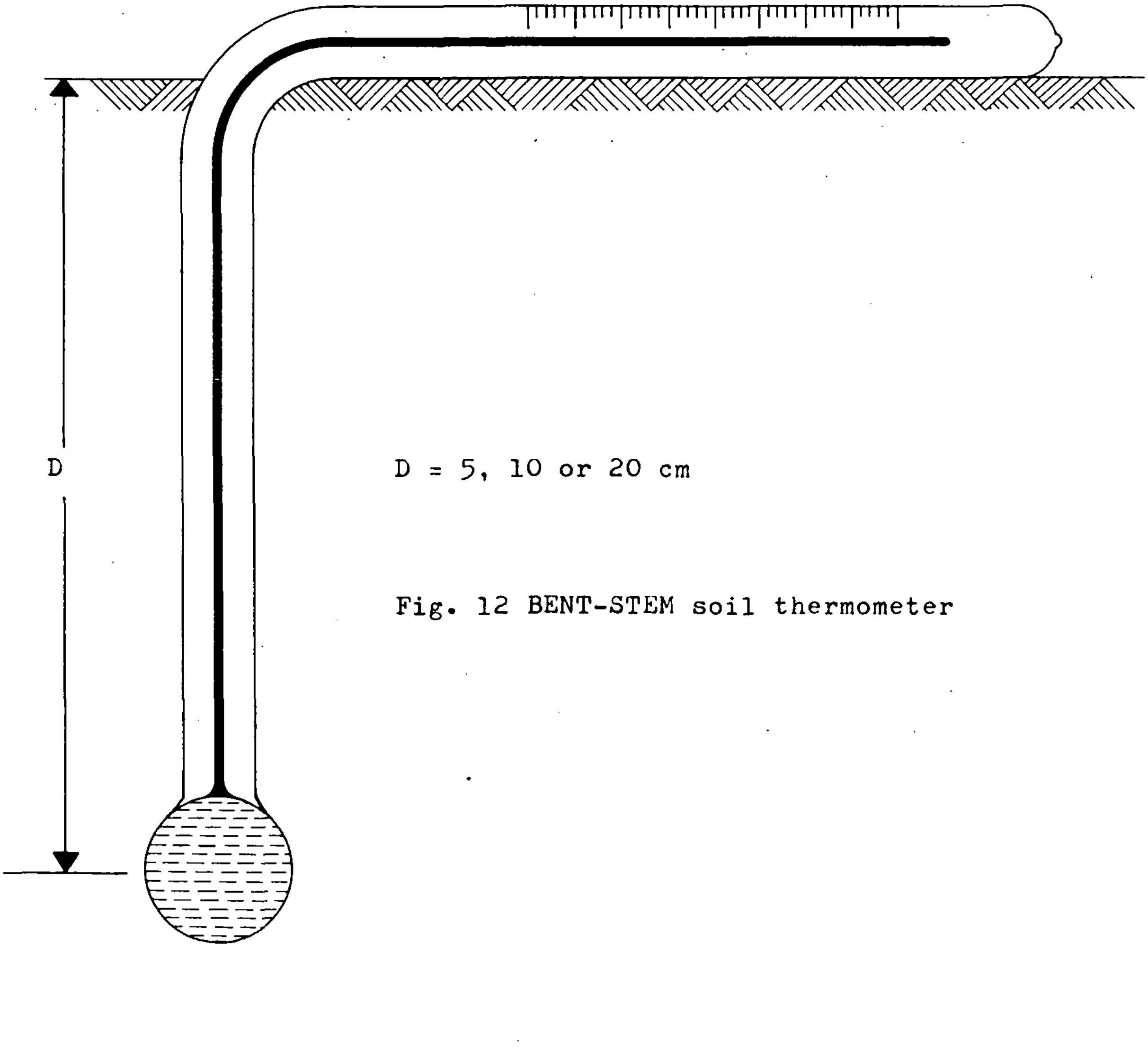


Fig. 11 The thermohygrograph
(NEGRETTI-ZAMBRA type)



D = 5, 10 or 20 cm

Fig. 12 BENT-STEM soil thermometer

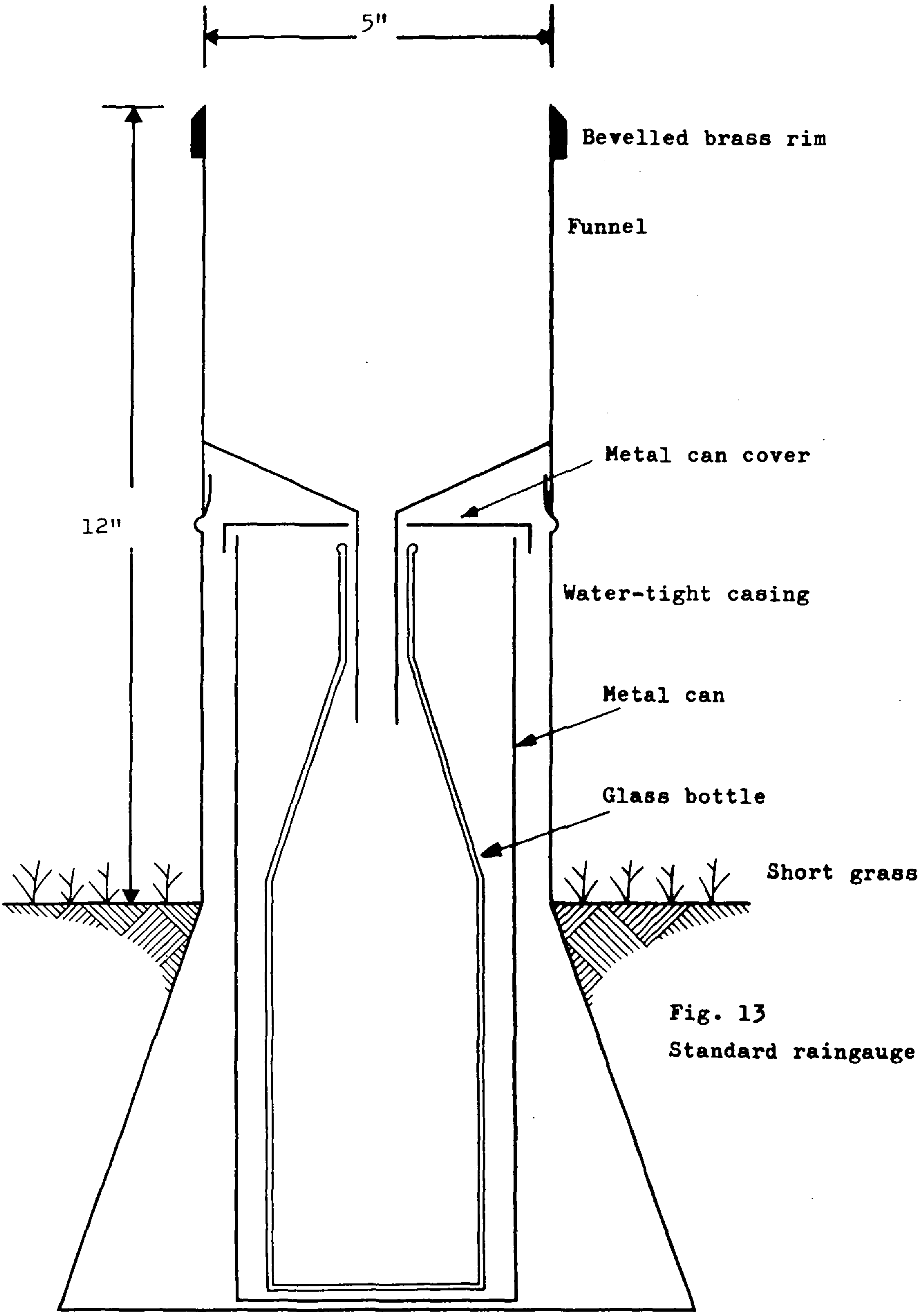


Fig. 13
Standard raingauge

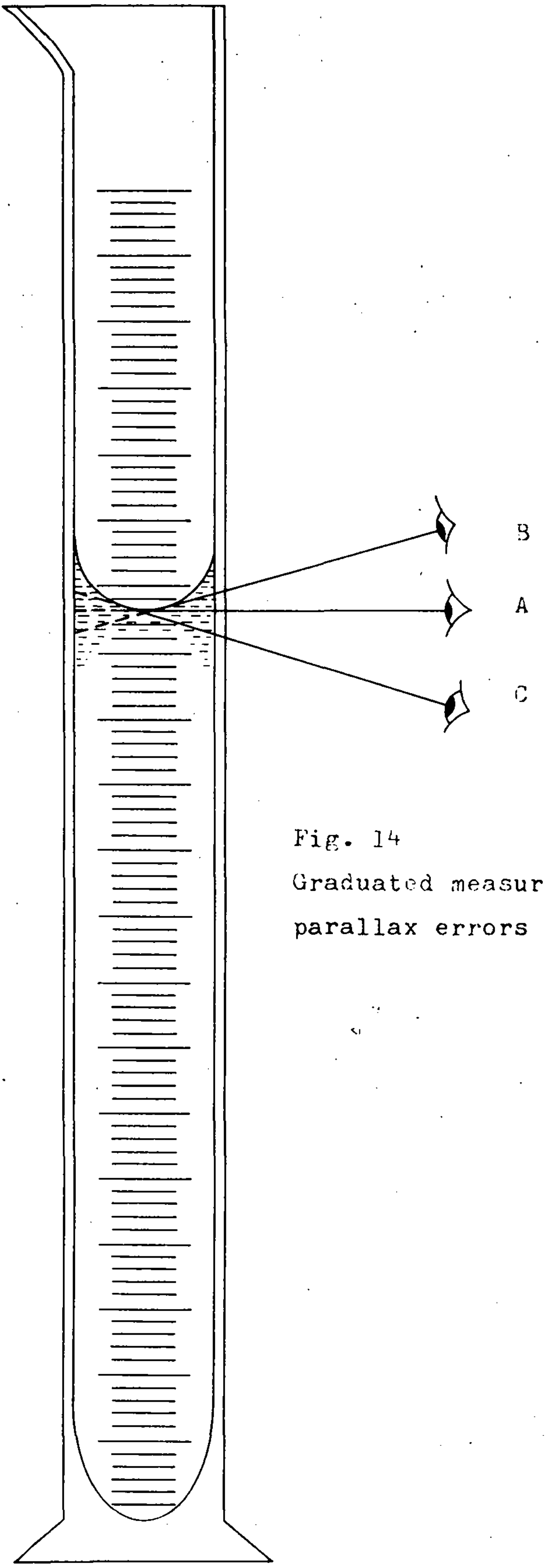


Fig. 14
Graduated measuring cylinder
parallax errors

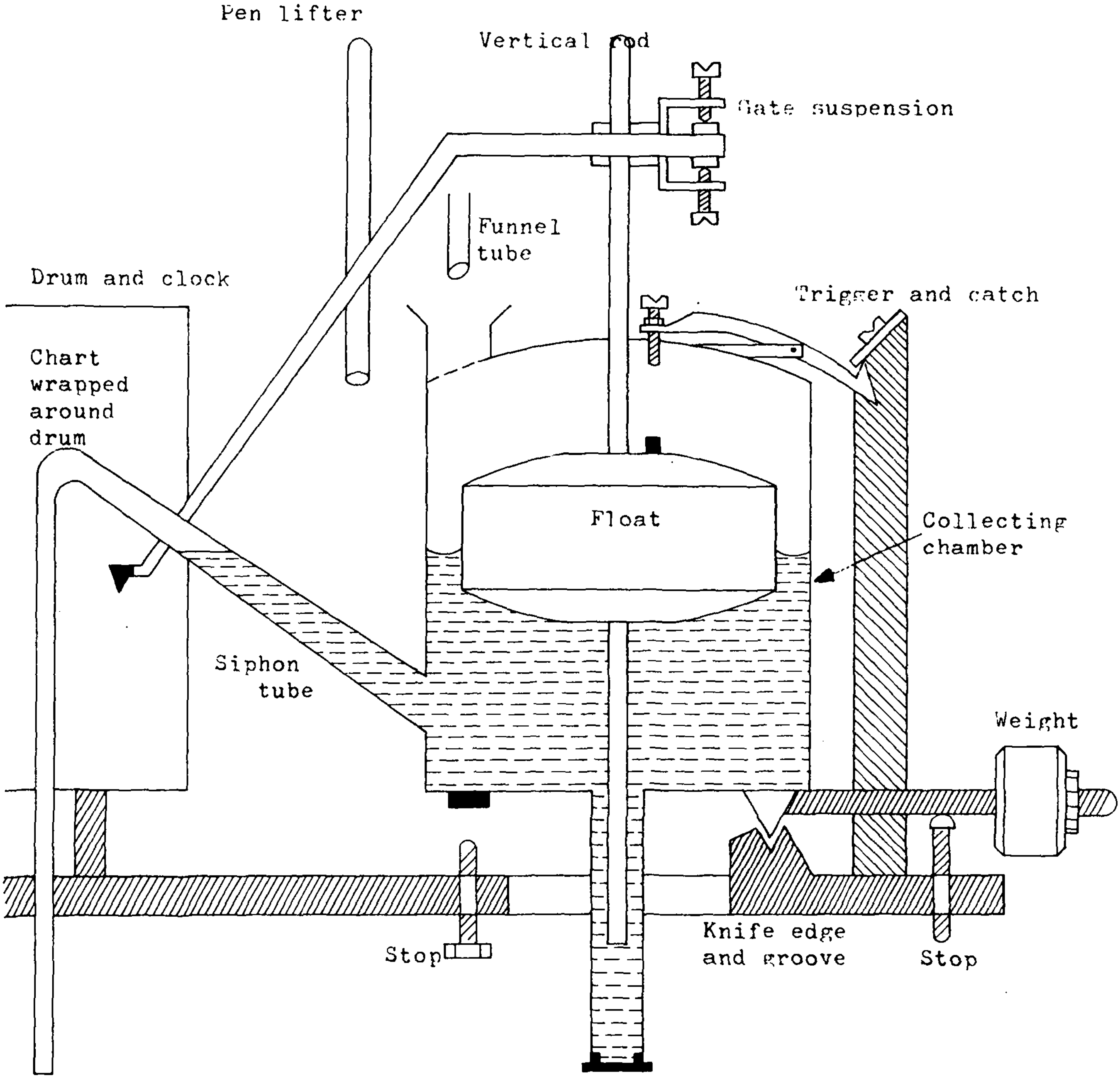
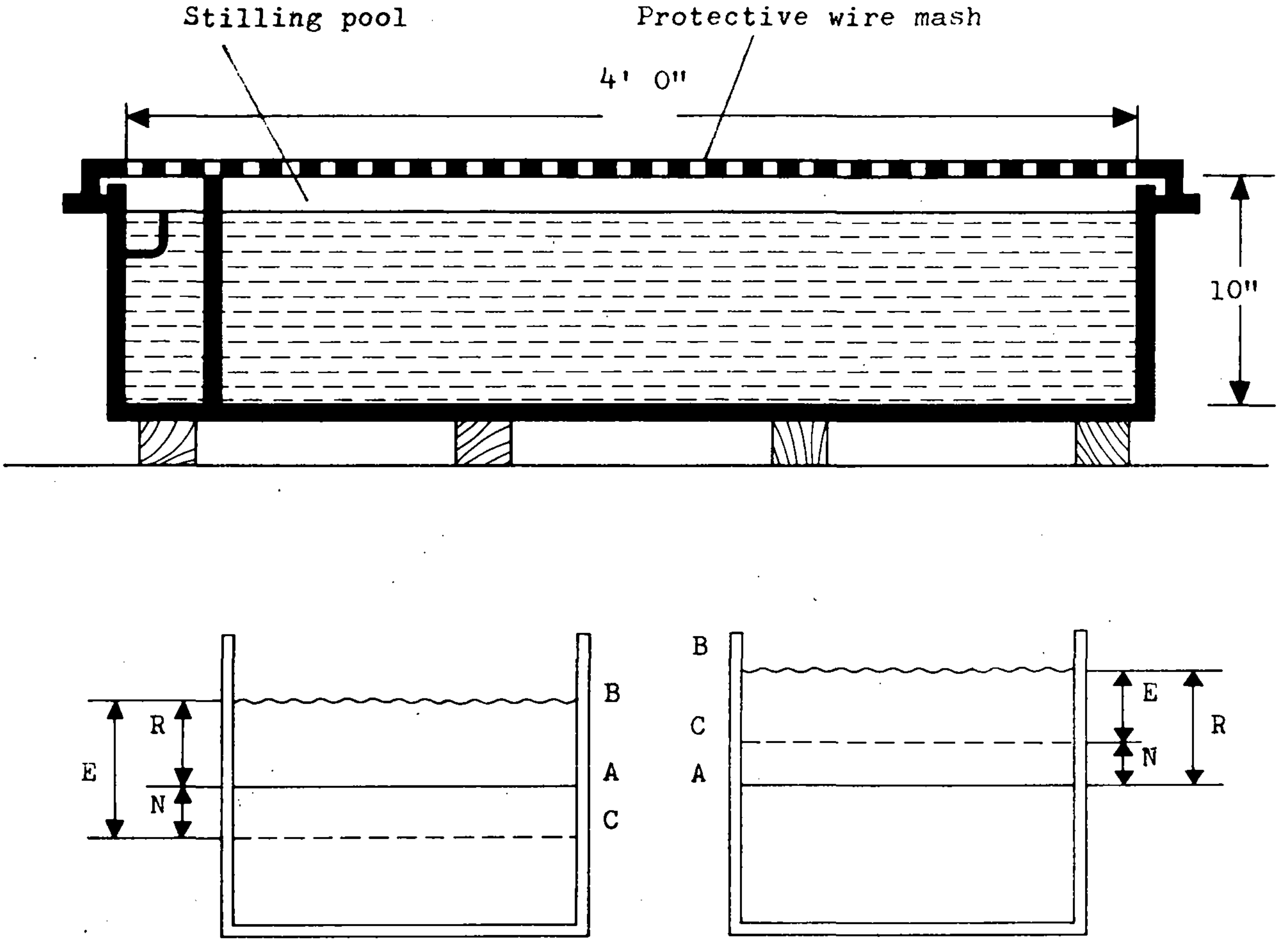


Fig. 15

Tilting-siphon rain recorder

Fig. 16 The standard evaporation pan



(i) n cups added
assume rainfall raises level from datum A to level B. Evaporation lowers level from B to C. Level is now below datum. n cups are added to bring level back to datum A.

E R

E = R + o.5 N

(ii) n cups taken out
assume rainfall raises level from datum A to datum B. Evaporation lowers level from B to C. Level is now above datum. n cups are taken out to bring level back to datum A

R E,R = E + o.5 N

E = R - o.5 N

(iii) no cups added

E = R

(iv) no cups taken out

E = R

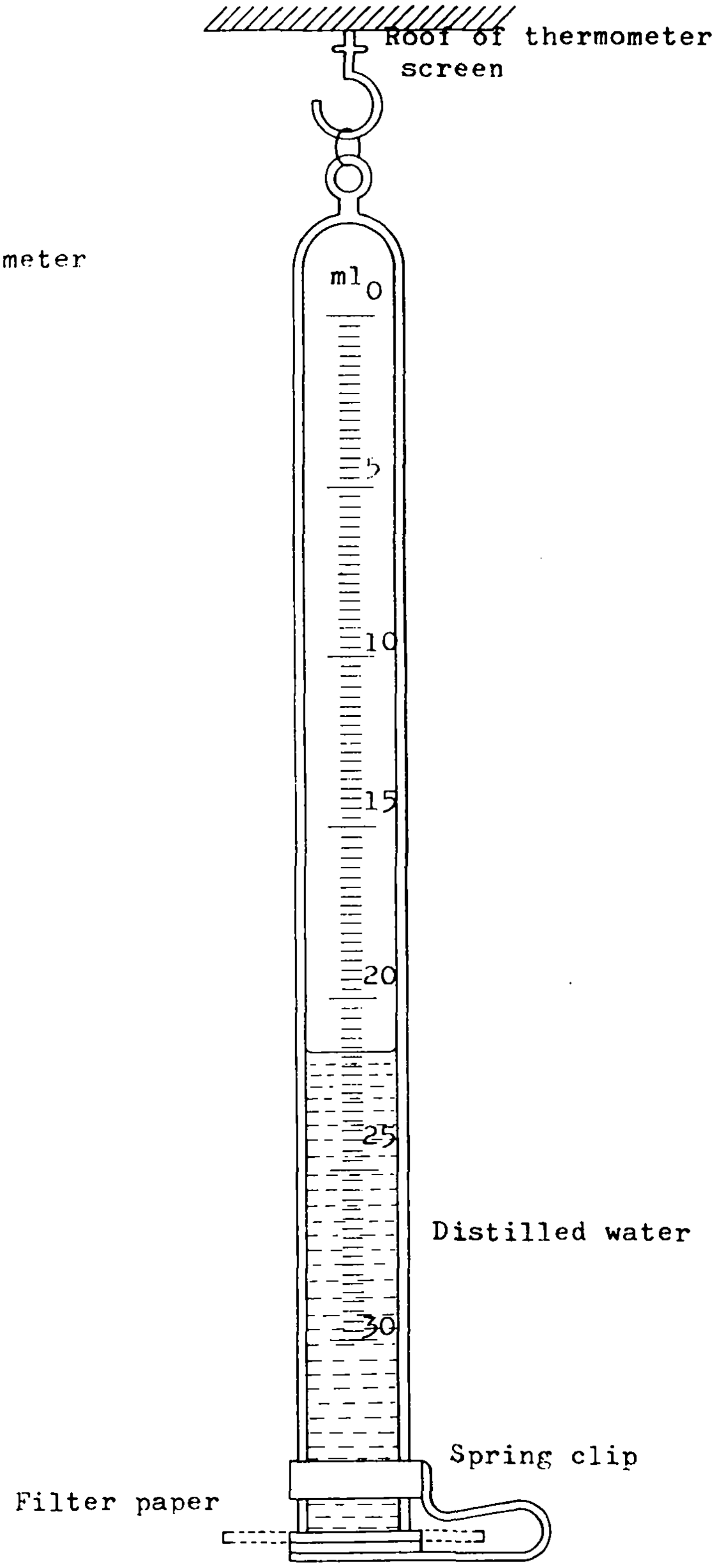


Fig. 17

Piche evaporimeter

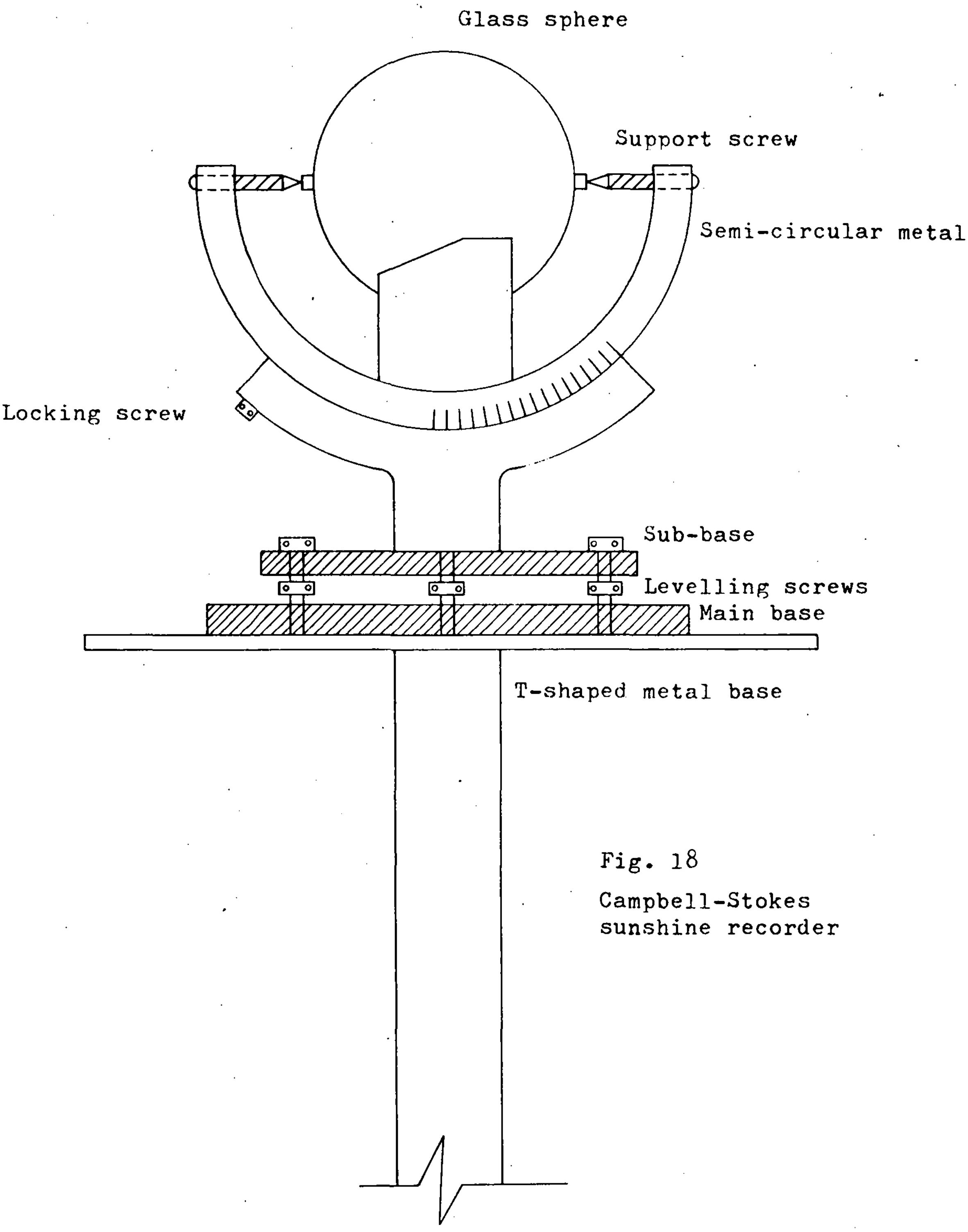


Fig. 18

Campbell-Stokes sunshine recorder

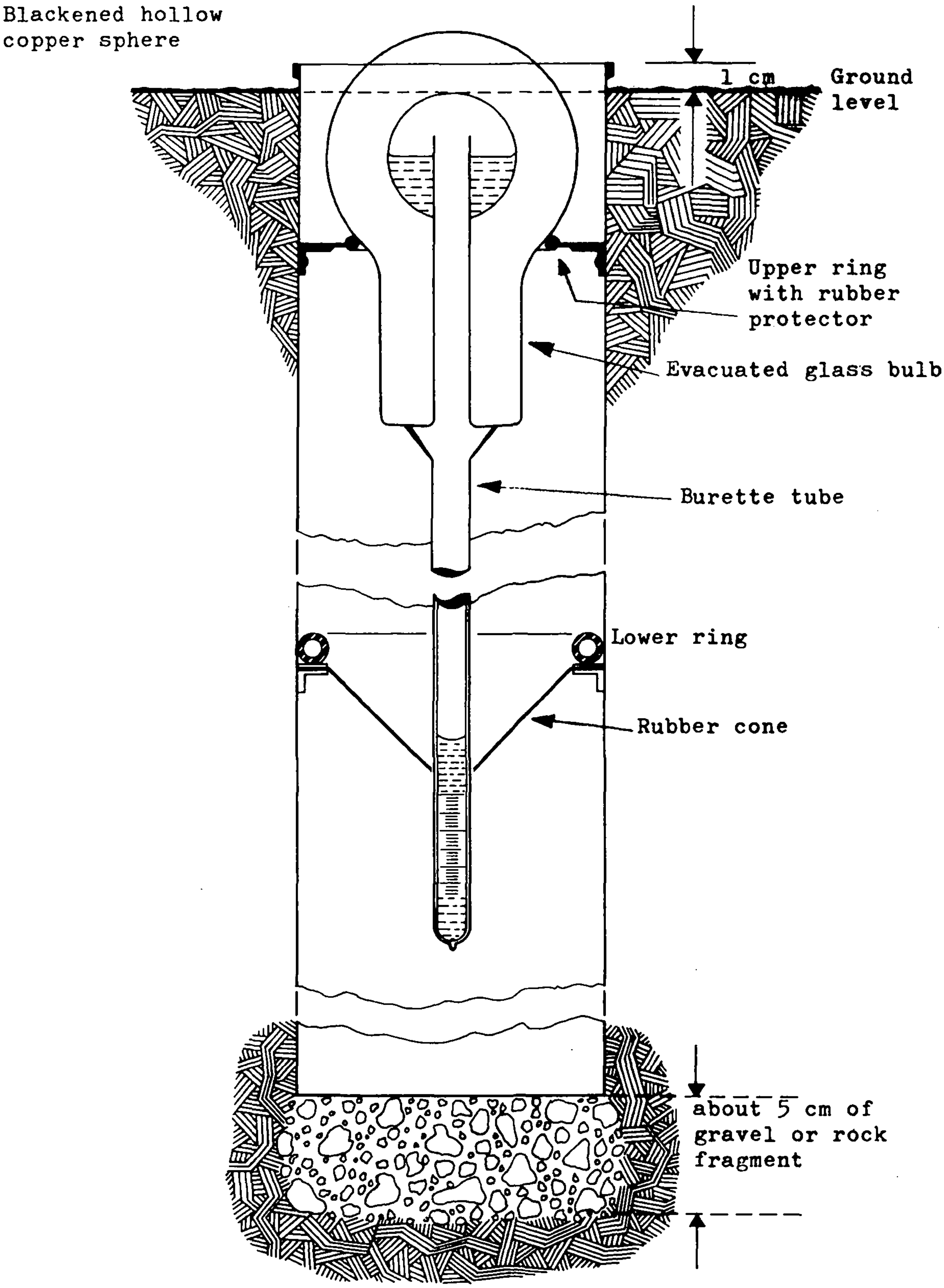


Fig. 19 The Gunn - Bellani radiometer

E. HYDROMETRY

1. Introduction

The term hydrometry as "the science of measurement of water" comprises a network of stations measuring the following elements:

- river and reservoir stages
- discharge of rivers
- sediment transport
- chemical quality
- temperature of water

For hydrological analysis the measurement of water stage and discharge is an extremely important basic step. The design of a hydrometric network depends on many conditions, i.e. economic constraint and practical restrictions. Nevertheless for a stream-gauging network some design principles have been ascertained by WMO.

Table I. Hydrometric network of minimum density

Type of region	Range of norms for minimum network (Area in km^2 for 1 station)	Range of provisional norms tolerated in difficult conditions (Area in km^2 for 1 station)
I Flat regions of temperate, mediterranean and tropical zones	1000 - 2500	3000 - 10000
II Mountainous regions of temperate, mediterranean and tropical zones	300 - 1000	1000 - 5000
Small mountainous islands with very irregular precipitation, very dense stream network	140 - 300	
III Arid and polar zones	5000 - 20000	

2. Measurement of stream stage and discharge

2.1 Selection of site

At a gauging station primarily the stage and discharge are measured. Consideration should be given to the accuracy with which the observation can be made and to the stability of the stream channel. The site for measurement should ensure a stable relation between the stage and the discharge and an easy measurement of the flow. This leads to the following requirements for the stage measurements:

(a) Avoiding backwater effects from tributaries
(b) Immediately upstream from a natural riffle or falls
(c) Channel bed must be stable
(d) Location in a straight reach
(e) Access for all stages

For the discharge measurements a cross-section with the following characteristics will ensure the highest accuracy of streamflow measurement:

(a) The velocities all points are parallel
(b) The velocity profiles are regular
(c) Regular cross-section
(d) Velocity in the section neither too high or too low $(0.1 < v < 5$ m/sec)
(e) No overflow of the banks of the channel
(f) No excessive aquatic growth.

Selections of suitable sites can be facilitated by studying aerial photographs and geological and topographic maps. Tentative sites for gauging stations may first be indicated on maps. The final decision depends on a critical examination of the physical characteristics of the stream channel at the prospective site.

2.2 Gauges for measuring the stage

2.21 Datum of gauge

Stage or gauge height is the height of the water surface referred to a fixed zero level. The datum of the gauge may be a recognized datum such as mean sea level or an arbitrary datum. In order to avoid negative values of gauge height, the datum selected for operating purposes should be below the elevation of the lowest runoff.

Each gauging station requires at least one benchmark which is independent of the gauge structure.

2.22 Manual (non-recording) gauges

The staff gauge is the most common of non-recording gauges. It consists of a graduated plate fixed in the stream or on the bank or on a structure, e.g. bridge abutement or pier. The measuring scale takes the lowest and the highest probable water level in the cross-section. It is graduated by 1 or 2 cm. The graduated staff gauge may be divided in one or more sections (sectional gauge) and may be installed vertically or inclined.

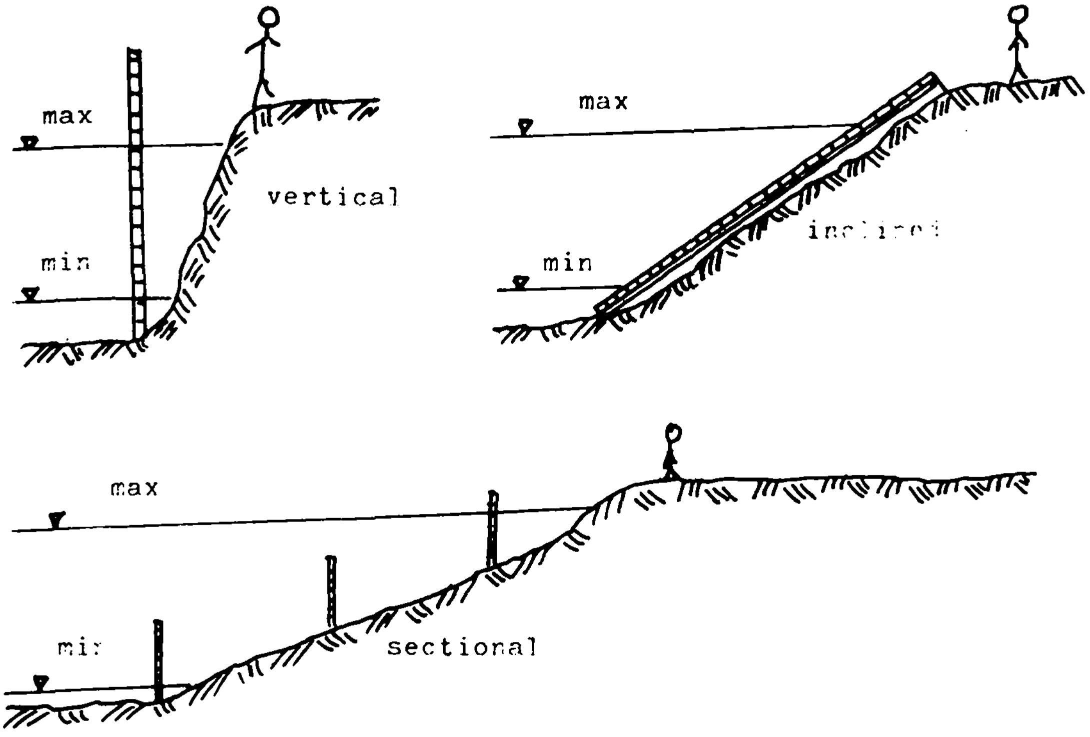


Fig. 1 Types of staff gauges

The wire weight gauge is installed on a bridge above the stream. The wire is wound on a small drum and the distance from a reference bar to the water surface is measured by counting the revolutions of the drum.

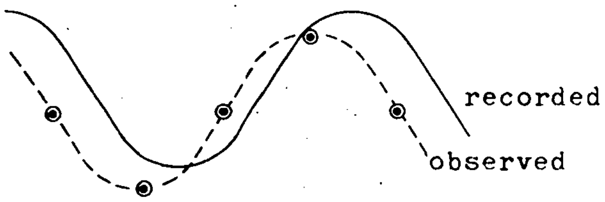
2.23 Recording gauges

Recording gauges can be classified by the method of stage detection and by the mode of recording. The recording mechanism is actuated by float or by hydrostatic pressure. The recording of the stage is obtained either by a continuous line on graph paper (strip chart recorder or continuous type recorder) or by a sequence of punches (digital stage recorder, magnetic tape recorder).

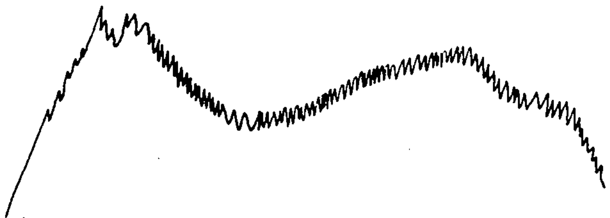
The stage indication of a float actuated recording gauge (float gauge) is given by the vertical movement of the float in a stilling well following the changes of the water level in the stream (see fig. 2). The movement is transmitted into a movement on a reduced scale (1:5, 1:10) of a recorder pen. The movement of the recorder pen is drawn on recording paper fixed on a drum. The time for one revolution of the drum can be varied. The minimum requirement for time scale on the chart is 6 cm per 24 hours. The stage-scale ratio should not be greater than 12:1. The instrument should be capable of recording the stage to the nearest 3 mm. Digital stage recorders are used more extensively because stage data recorded in this form are compatible with the use of computers. One type (Leopold and Stevens) punches holes in a paper tape, e.g. every 15 minutes. Another type (Ott) uses magnetic type which can directly be fed into the computer.

The stage indications of hydrostatic pressure actuated gauges could be measured with a diaphragm or a bubble orifice (pneumatic gauge or bubble gauge). River stages may be recorded to ± 3 mm by this method. The advantage of these types of gauges is that they do not require a stilling well. A transmission of variation of hydrostatic pressure is possible up to 300 m.

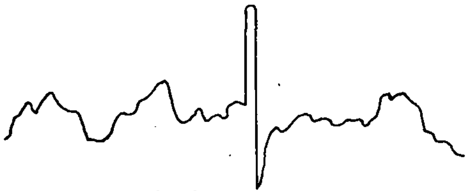
Errors in records of recording gauges are cuased by improper functioning of the mechanism or by siltation of the stilling well (see fig. 2).



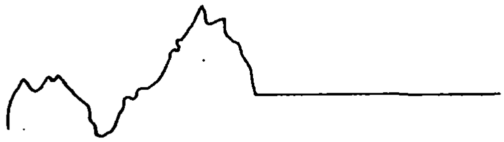
(a) Siltation of well or intake



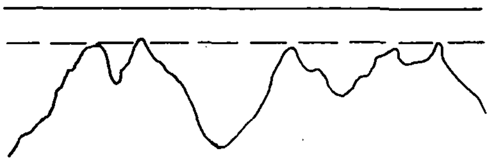
(b) Float well inlet too large, permitting surges inside the well



(c) Clock mechanism stopped for a certain time



(d) Float caught or slipping transmission



(e) Pen reversing or not properly set

Fig. 2 Errors in the records of recording gauges

2.3 Measurement and computation of discharge

2.31 Methods using current meters

The most common method of discharge measurement is the area-velocity method. The discharge Q(m^3/sec) through a cross-section is the wet cross-section area A(m^2) multiplied by the average streamflow velocity v (m/sec). The mean velocity is obtained by measuring the velocity at a point by counting the number of revolutions of a current meter during a short time period, e.g. 60 sec. The relation between rotation n of the propeller and the streamflow velocity v is indicated by a calibration graph of the propeller and is equal to:

$v = \alpha + \beta . n$

v = velocity of current at a point

α β = constants, indicated in a certificate provided by the current meter manufacturer

n = number of revolutions.

The current meter is suspended on a rod. If the water depth is too large, it is suspended on a cable which is fixed to a cableway, boat or bridge. Two types of current meter rotors are produced: (1) a cup current meter with a vertical shaft, and (2) a propeller type with a horizontal shaft. The measurable velocities for current meters range normally between 0.05 and 3 m/sec.

In order to measure the cross-sectional area, a steel tape or a beaded wire is used for measuring the width in decimetres. In a narrow cross-section the depth is measured simultaneously with the velocity. If the section is wide, the depth is measured first with a wading rod, sounding weight or by echo-sounding.

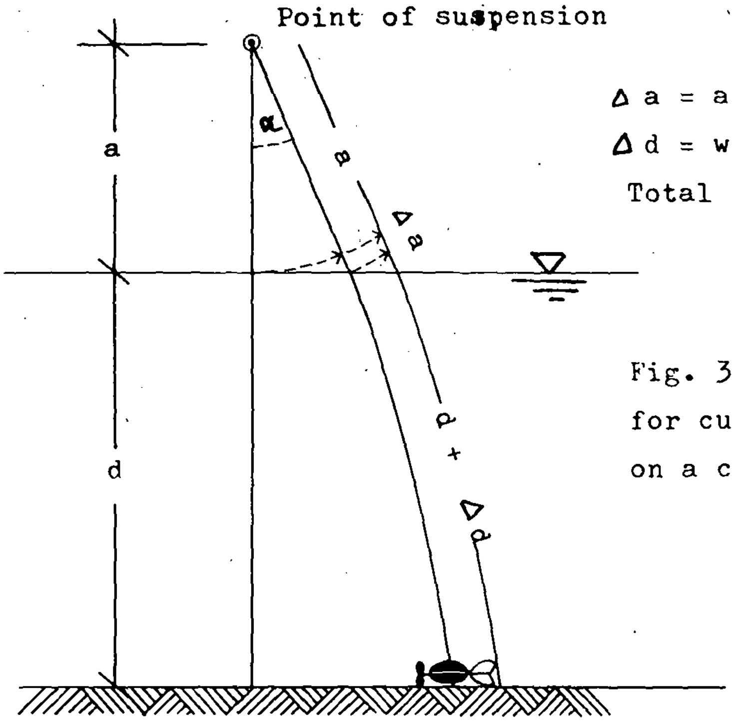
If the wire cable is not normal to the water surface, the angle of departure should be measured with a protractor. A correction should be applied if $7^\circ < \alpha < 30^\circ$ or $\Delta d' : D > 1\%$ (see fig. 3 and Tables II and III). The deviation from the vertical can be avoided by using heavier weights.

Rule of thumb:

$H_{max} \times v_{max} \times 5$ = required weight in kg

where H_{max} = maximum depth (m)

v_{max} = maximum velocity (m/sec)



Δ a = airline correction
Δ d = wetline correction
Total correction = a + d

Fig. 3 Correction of depth for current meter suspended on a cable

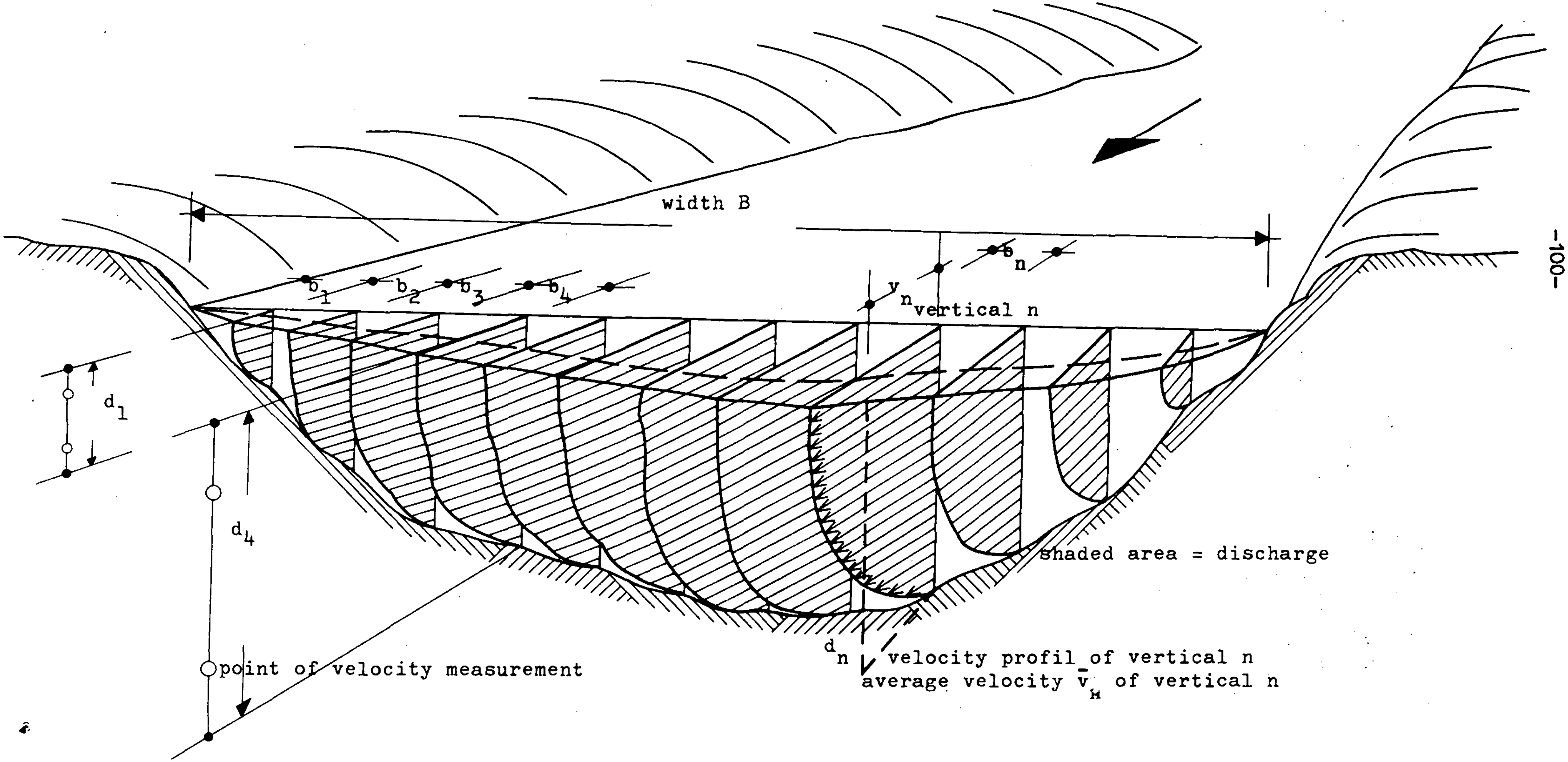
Vertical length Δ a above water surface in meters	Difference Δ a, in cm, between vertical length and slant length of sounding line above water surface for vertical angles between 5° and 30°.																								
	5°	6°	7°	8°	9°	10°	11°	12°	13°	14°	15°	16°	17°	18°	19°	20°	21°	22°	23°	24°	25°	26°	27°	28°	29°	30°
0.5	–	–	–	–	1	1	1	1	1	2	2	2	2	3	3	3	4	4	4	5	5	6	6	7	7	8
1.0	–	–	1	1	1	2	2	2	3	3	4	4	5	5	6	6	7	8	9	9	10	11	12	13	14	15
1.5	1	1	1	1	2	2	3	3	4	5	5	6	7	8	9	10	11	12	13	14	16	17	18	20	21	23
2.0	1	1	2	2	2	3	4	4	5	6	7	8	9	10	12	13	14	16	17	19	21	23	24	27	29	31
2.5	1	1	2	2	3	4	5	6	7	8	9	10	11	13	14	16	18	20	22	24	26	28	31	33	36	39
3.0	1	2	2	3	4	5	6	7	8	9	11	12	14	15	17	19	21	24	26	28	31	34	37	40	43	46
3.5	1	2	3	3	4	5	7	8	9	11	12	14	16	18	20	22	25	27	30	33	36	39	43	46	50	54
4.0	2	2	3	4	5	6	7	9	11	12	14	16	18	21	23	26	28	31	35	38	41	45	49	53	57	62
4.5	2	2	3	4	6	7	8	10	12	14	16	18	21	23	26	29	32	35	39	43	47	51	55	60	64	70
5.0	2	3	4	5	6	8	9	11	13	15	18	20	23	26	29	32	36	39	43	47	52	56	61	66	72	77
5.5	2	3	4	5	7	8	10	12	14	17	19	22	25	28	32	35	39	43	47	52	57	62	67	73	79	85
6.0	2	3	5	6	7	9	11	13	16	18	21	24	27	31	35	39	43	47	52	57	62	68	73	80	86	93
6.5	2	4	5	6	8	10	12	14	17	20	23	26	30	33	37	42	46	51	56	61	67	73	80	86	93	100
7.0	3	4	5	7	9	11	13	16	18	21	25	28	32	36	40	45	50	55	60	66	72	79	86	93	100	108
7.5	3	4	6	7	9	12	14	17	20	23	26	30	34	39	43	48	53	59	65	71	78	84	92	99	107	116
8.0	3	4	6	8	10	12	15	18	21	24	28	32	37	41	46	51	57	63	69	76	83	90	98	106	115	124
8.5	3	5	6	8	11	13	16	19	22	26	30	34	39	44	49	55	60	67	73	80	88	96	104	113	122	131
9.0	3	5	7	9	11	14	17	20	24	28	32	36	41	46	52	58	64	71	78	85	93	102	110	119	129	139
9.5	4	5	7	9	12	15	18	21	25	29	34	38	43	49	55	61	68	75	82	90	98	107	116	126	136	147
10.0	4	6	8	10	12	15	19	22	26	31	35	40	46	51	58	61	71	79	86	95	103	113	122	133	143	155

Table III. Wet-line corrections

Wet-line depth d + Δd in meters	Differences Δd in cm between wet-line length and vertical depth for vertical angles between 5° and 30°.																								
	5°	6°	7°	8°	9°	10°	11°	12°	13°	14°	15°	16°	17°	18°	19°	20°	21°	22°	23°	24°	25°	26°	27°	28°	29°	30°
1	–	–	–	–	–	1	1	1	1	1	1	1	2	2	2	2	2	3	3	3	3	4	4	4	5	5
2	–	–	1	1	1	1	1	1	2	2	2	3	3	3	4	4	5	5	6	6	7	7	8	9	9	10
3	–	1	1	1	1	2	2	2	3	3	4	4	5	5	6	6	7	8	9	9	10	11	12	13	14	15
4	1	1	1	1	2	2	3	3	3	4	5	5	6	7	8	9	9	10	11	13	14	15	16	18	19	20
5	1	1	1	2	2	3	3	4	4	5	6	7	8	9	10	11	12	13	14	16	17	19	20	22	24	26
6	1	1	2	2	2	3	4	4	5	6	7	8	9	10	11	13	14	16	17	19	21	22	24	26	28	31
7	1	1	2	2	3	4	4	5	6	7	8	9	11	12	13	15	17	18	20	22	24	26	28	31	33	36
8	1	1	2	3	3	4	5	6	7	8	9	11	12	14	15	17	19	21	23	25	27	30	32	35	38	41
9	1	2	2	3	4	5	6	7	8	9	11	12	14	15	17	19	21	23	26	28	31	34	36	39	43	46
10	1	2	3	3	4	5	6	7	9	10	12	13	15	17	19	21	24	26	29	31	34	37	40	44	47	51
11	1	2	3	4	5	6	7	8	10	11	13	15	17	19	21	23	26	29	31	34	38	41	44	48	52	56
12	2	2	3	4	5	6	8	9	10	12	14	16	18	21	23	26	28	31	34	38	41	45	49	53	57	61
13	2	2	3	4	5	7	8	10	11	13	15	17	20	22	25	28	31	34	37	41	45	48	53	57	62	66
14	2	3	4	5	6	7	9	10	12	14	16	19	21	24	27	30	33	36	40	41	48	52	57	61	66	71
15	2	3	4	5	6	8	9	11	13	15	18	20	23	26	29	32	35	39	43	47	51	56	61	66	71	77
16	2	3	4	5	7	8	10	12	14	16	19	21	24	27	31	34	38	42	46	50	55	60	65	70	76	82
17	2	3	4	6	7	9	11	13	15	17	20	23	26	29	33	36	40	44	49	53	58	63	69	74	80	87
18	2	3	5	6	7	9	11	13	16	18	21	24	27	31	34	38	43	47	51	56	62	67	73	79	85	92
19	2	3	5	6	8	10	12	14	17	19	22	25	29	32	36	41	45	50	54	60	65	71	77	83	90	97
20	3	4	5	6	8	10	13	15	17	20	23	27	30	34	38	43	47	52	57	63	69	75	81	88	95	102

The velocity measurements are obtained with the current meter at one or more points in each vertical (see fig. 4). In general, the interval between any two verticals should not be greater than $^{1}/20$ of the width. The discharge between any two verticals should not be more than 10% of the total discharge. Fewer verticals are required on small rivers, since the interval between any two verticals must be greater than the diameter of the propeller of the current meter.

Fig. 4 Cross-section of streamflow measurement showing points of observations and velocity



The number of velocity observations taken in the vertical depends on the following methods of computation of the average velocity:

(a) One point method

The mean velocity is approximated by the velocity observation taken at 0.6 of the depth D below the water surface. This method is used for shallower depth ($D < 0.8$ m) and when a quick measurement is necessary (e.g. during a flood).

$$\bar{v} = v_{0.6D}$$

where $\bar{v}$ = mean velocity of the vertical

$v_{0.6D}$ = velocity at 0.6D

(b) Two point method

Velocity measurements are taken at 0.2D and 0.8D.

$$\bar{v} = \frac{v_{0.2D} = v_{0.8D}}{2}$$

This method is used for $D > 0.6$ m and a velocity which is normal.

(c) Three point method

$$\bar{v} = \frac{v_{0.2D} + v_{0.6D} + v_{0.8D}}{3}$$

or in stream channels overgrown by aquatic vegetation

$$\bar{v} = \frac{v_{0.15D} + v_{0.5D} + v_{0.85D}}{3}$$

(d) Five point method

Additional velocity measurements are made just below the water surface v_s and just above the stream bed v_B

$$\bar{v} = \frac{v_s + 3v_{0.2D} + 2v_{0.6D} + 3v_{0.8D} + v_B}{10}$$

It is used where the vertical distribution of velocity is very irregular.

(e) Construction of velocity distribution profile

Velocity measurements are taken at close intervals. The mean velocity $\bar{v}$ is obtained by plotting the measured velocities, planimetering the area below the velocity distribution curve and dividing by the depth.

(f) Integration method

Gives a direct integration of the velocity. The current meter is lowered and moved with a constant velocity ($v < 0.04$ m/sec) through the entire depth. The total number of revolutions are counted. The method is based on a linear relationship between velocity and revolutions per second of the propeller type current meter.

The discharge can be computed much more rapidly by using the following arithmetical methods:

(a) **Mean section method:** The discharge of the section contained by two verticals is computed by

$$q_1 = \left(\frac{\bar{v}_0 + \bar{v}_1}{2}\right) \left(\frac{d_0 + d_1}{2}\right) b$$

$$Q_{total} = \sum q_i$$

(b) **Mid section method:**

$$Q = d_1 \bar{v}_1 \left(\frac{b_1}{2} + \frac{b_2}{2}\right) + d_2 \bar{v}_2 \left(\frac{b_2}{2} + \frac{b_3}{2}\right) + \ldots.$$

(c) **Graphical methods:** Most commonly used is the depth velocity curve. This area represents the product of the mean velocity and the total depth. The value of this product at each vertical should then be plotted over the surface line and a curve drawn up through the points thus obtained. The area below this curve and the water surface line represents the discharge of the cross-section (see fig. 4). Another graphical method uses the velocity contour lines.

Corresponding stage

At the beginning and end of a discharge measurement, the water stage is measured by a staff gauge. Usually the stage at the mid-time of measurement can be used as the stage corresponding to the measured discharge. If the fluctuation is more than 5 cm or if it is a rising and falling stage during the measurement, the average stage H is calculated by the following equation:

$$\bar{H} = \frac{\sum \bar{v}Hb}{\sum \bar{v}b}$$

where v = mean velocity in a vertical

H = water stage read when measuring this vertical

b = half the distance between the measured and adjacent verticals.

2.32 Measurement of discharge by other methods

2.321 Tracer methods

The measurement of discharge is based on the determination of the degree of dilution by the flowing water of an added tracer solution at one (or two) measuring section(s) downstream. The method is recommended for sites with extremely high velocities, high turbulence and shallow depth. In principle, the general requirements of the method are the following:

> The solution of the tracer injected into the stream will be diluted by the streamflow. The accuracy of the methods depends on the complete mixing of the tracer throughout the cross section before the measuring section is reached. The tracer should meet the following requirements:

(a) Dissolves readily in water but does not decompose and not retained by sediments or plants.

(b) Its concentration can be accurately and simply measured.

(c) Not injurious to men and animals.

(d) Not present in the water or in very small quantities.

The most common types of tracers are

(1) Sodium chloride (cheap tracer, concentration 1:20000 - 1:30000, detection by conductivity method)

(2) Sodium dichromate, rhodamine (permits colorimetric analysis of very low concentrations 1:50000 - 1:100000)

(3) Radioactive tracers (e.g. gold 198. Can be determined by a counter up to concentration of 10^{-9}. Health hazard).

The following two tracer methods can be used to measure discharge:

(1) <u>Constant rate injection method</u>

$$C_1 q + C_0 Q = C_2(Q + q)$$

$$Q = q\,\frac{C_1 - C_2}{C_2 - C_0}$$

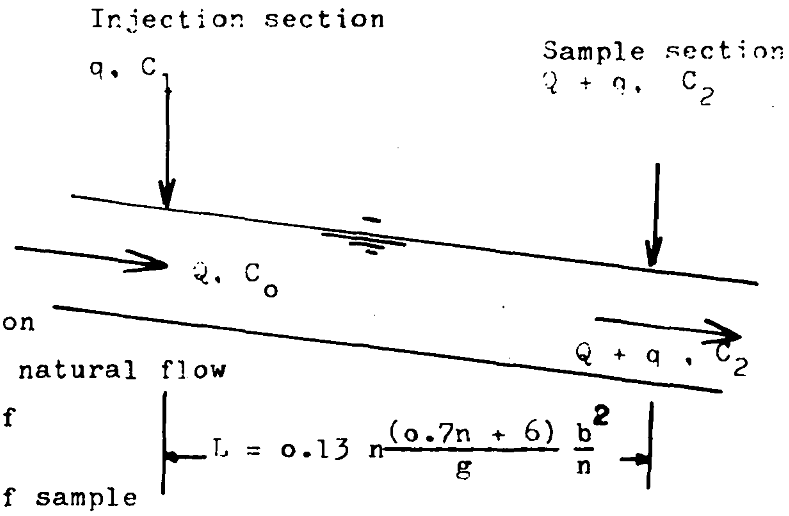


Fig. 5

where

Q = Discharge

q = Rate of injection

C_0 = Concentration of natural flow

C_1 = Concentration of injection

C_2 = Concentration of sample

$15 \quad n < 30 <$; n = Chezy coefficient

b = average width

h = average depth

g = gravitational constant

If $C_1 \gg C_2$, $C_2 \gg C_0$, and $Q \gg q$ then the equation can be written

$$Q = q \quad C_1/C_2$$

(2) <u>Sudden injection method</u>

If the solution is instantaneously injected in the stream, the discharge is determined as follows:

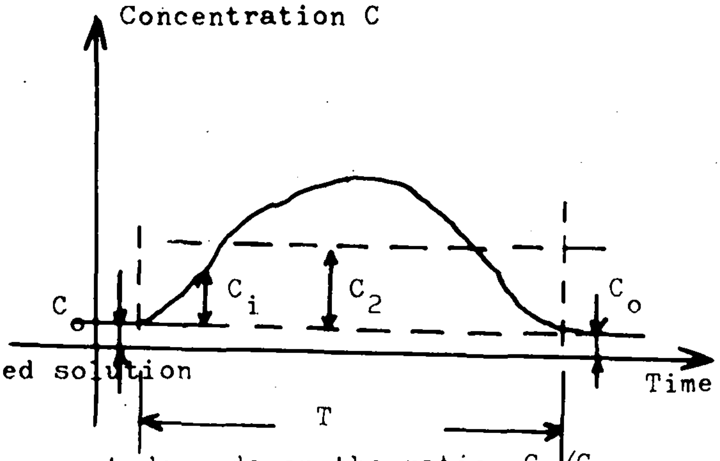
$$C_1 \cdot V = C_2 \cdot T \cdot Q$$

$$Q = \frac{V \cdot C_1}{T \cdot C_2}$$

where

V = Volume of injected solution

T = Time



The accuracy of measurement depends on the ratio C_1/C_2

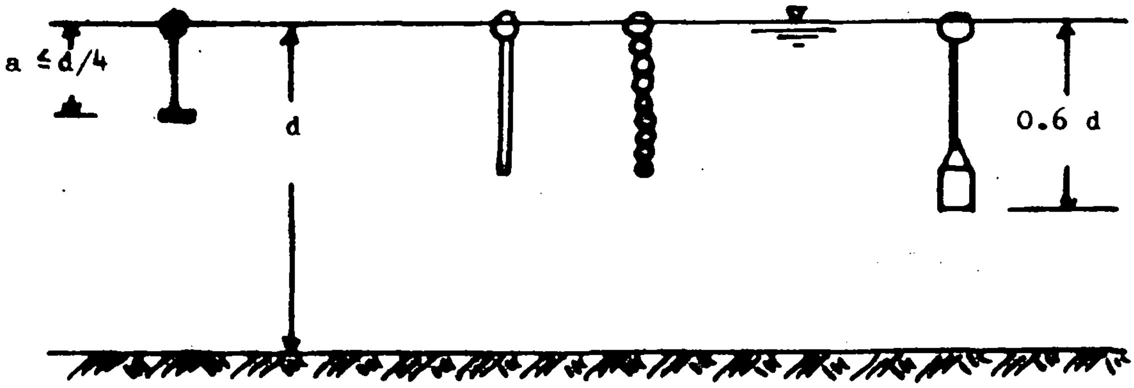
Fig. 6 Sudden injection method

2.322 Float method

The method may be used to determine the approximate velocity if

- current meter measurements become too dangerous (very high velocities)
- very low velocities (lakes).

Three types of floats can be used.



surface float Rod or chain floats Buoyant float

Fig. 7 Different types of floats

The velocity of the float v_F is equal to the distance between two cross sections divided by time of travel. The mean velocity $\bar{v}$ of flow in the panel is given by the following equation

$$\bar{v} = k \cdot v_F$$

with $0.84 < k < 0.90$ for surface float
$0.8 < k < 1.0$ for rod buoy
$k \simeq 1.0$ for buoyant float

The discharge in each panel is computed by multiplying the average area of the cross-section by the mean velocity. The total discharge is the sum of discharge in the panels.

The advantage of the float method is that the measurement of streamflow is cheap and simple. The disadvantages are the influence of wind on the floats and the fairly low accuracy (10 - 25%).

2.323 Discharge measurements by indirect methods

During floods it is often impossible to measure the flood peak directly. By indirect methods a simultaneous solution of the hydraulic equations of continuity and energy is performed considering the following factors:

(i) hydraulic and geometrical characteristic of the channel
(ii) water level at time of peak

The commonly used slope-area method is based on the hydraulic formulas for uniform steady flow:

$$Q = A.v$$
$$v = k.R^{2/3} J^{\frac{1}{2}} \text{ (Manning's equation) or}$$

$$v = n R^{\frac{1}{2}} J^{\frac{1}{2}} \text{ (Chezy's equation)}$$

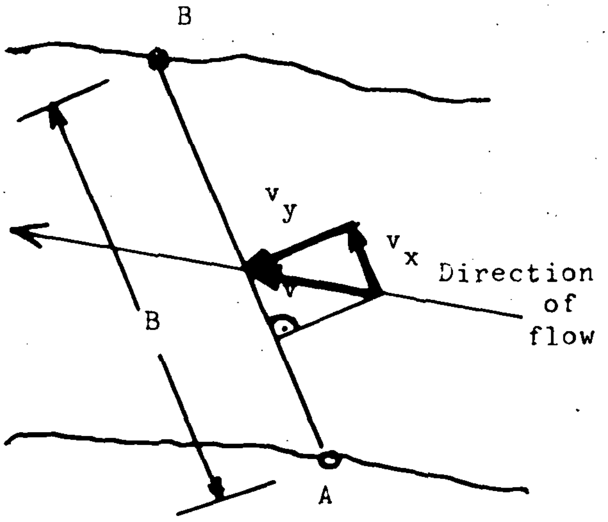
where

Q = total discharge
A = cross section area
v = mean velocity in cross section
k = roughness coefficient
R = hydraulic radius
J = slope of water surface

For the computation, a straight uniform reach of the river is preferred. The estimation of the roughness is normally the weakest point of this method. (See Example 1).

2.324 Acoustic flow measurement

A recently developed method of measuring velocity and flow is the acoustic measuring method by integrating the flow rate (m^3/sec) over the total river width in time intervals between minutes and 24 hours. With one ultrasonic path at a representative water depth, a mean integrated velocity profile is measured. If the velocity profile is very irregular, up to 5 different acoustic paths can be installed. The advantage of this method is that it can be used during high floods. The method is based on the measurement of pulse transit time and of the phase shift of continuous ultrasound waves (see fig. 8).



A,B : Acoustic transducers (emitter, receiver)

B : Distance between transducers

t_1 : Pulse transit time in the direction of flow

t_2 : Pulse transit time against direction of flow

v_x : $v \cdot \cos \alpha$

v_y : $v . \sin \alpha$

Fig. 8 Principle of acoustic flow measurement

2.4 Rating curve (stage-discharge curve)

The relation between the gauge height (stage) and the corresponding discharge computed by means of measured velocity is called the rating curve. The individual discharge measurements may deviate from a mean stage discharge, because of the following:

- flow is unsteady
- aquatic growth varies with season
- influence of variable backwater
- changes in cross section.

If the rating curve is established for a stable cross-section, occasional measurements will suffice to verify whether changes in the section have taken place.

On linear graph paper the stage-discharge curve often has a parabolic shape. Therefore on logarithmic paper it is a straight line and can be used for extrapolation. Theis' regression line may have a break at the stage of bankful discharge (see fig. 9). In analytical form it is written

$$Q = a_o H^{a_1}$$

where

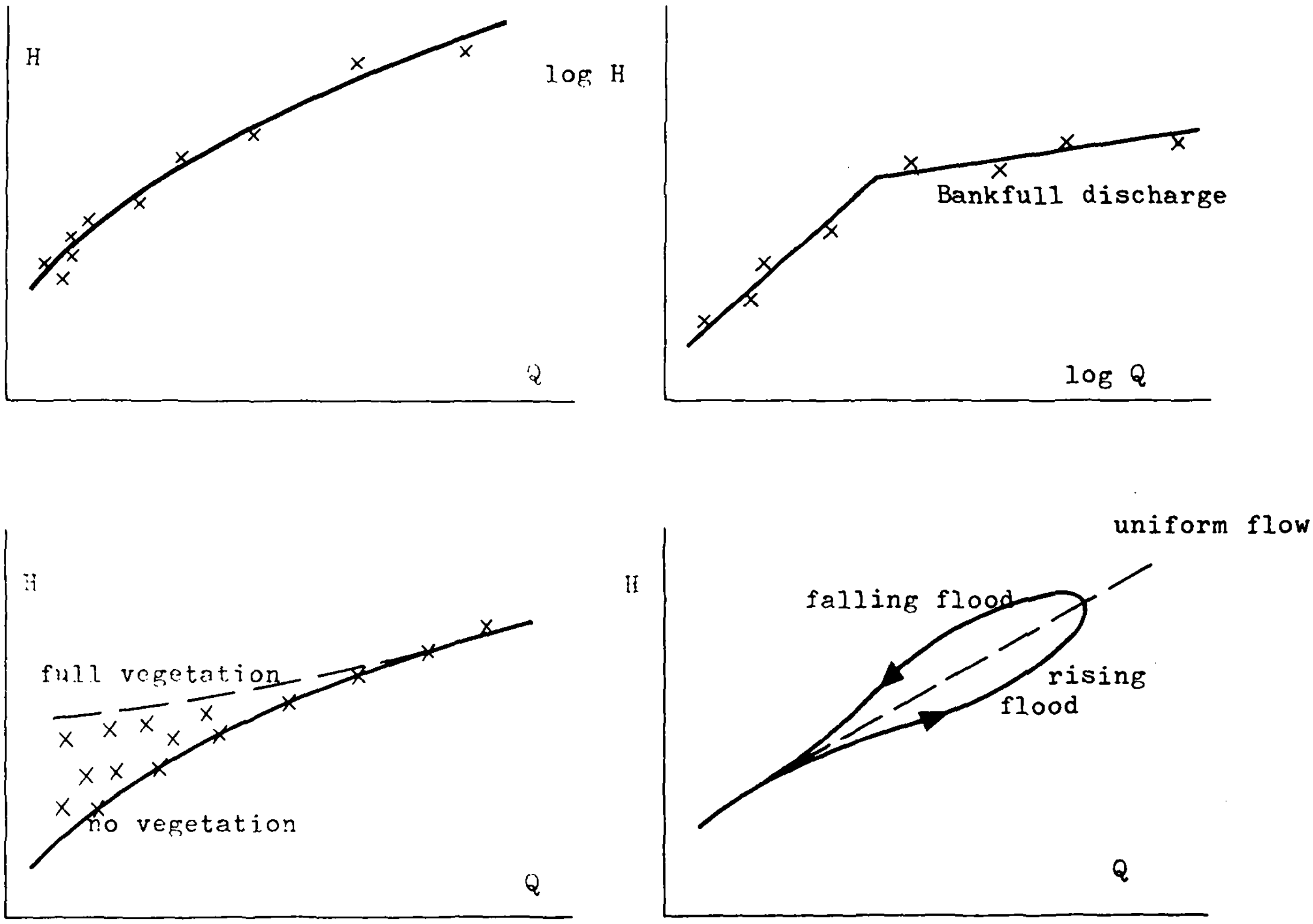
Q = discharge

H = gauge height

a_o, a_1 = constants

For numerical computations, the rating curve is converted into a rating table.

If the rating curve is a straight line on logarithmic paper, the constants in the equation can be found by the method of linear regression (see Example 2):



Aquatic growth

Unsteady flow

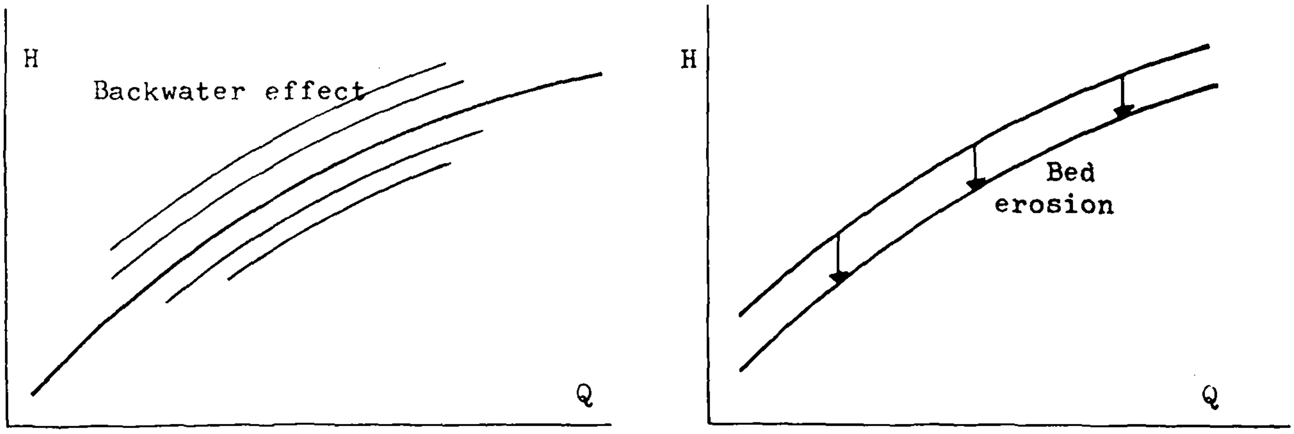


Fig. 9 Different stage-discharge relations

Step 1: Conversion of the arithmetic values H and Q into logarithmic values x and y.

Step 2: Computation of the mean $\bar{x}$ and $\bar{y}$:

$$\bar{x} = \frac{1}{n} \sum x; \quad \bar{y} = \frac{1}{n} \sum y$$

Step 3: Computation of the expressions:

$$(x - \bar{x}); \quad (y - \bar{y}); \quad (x - \bar{x})^2; \quad (y - \bar{y})^2; \quad (x - \bar{x})(y - \bar{y})$$

Step 4: Calculation of standard deviation (σ), variance and covariance:

$$\sigma_x = \sqrt{\frac{1}{n-1} \sum (x - \bar{x})^2}$$

$$\sigma_y = \sqrt{\frac{1}{n-1} \sum (y - \bar{y})^2}$$

Variance: σ_x^2; σ_v^2

Covariance: $\sigma_{xy} = \frac{1}{n-1} \sum (x - \bar{x})(y - \bar{y})$

Step 5: Computation of the regression coeffieient a_1 and correlation coefficient r:

Regression coefficient

$$a_1 = \frac{\text{covariance}}{\text{variance}} = \frac{\sigma_{xy}}{\sigma_x^2}$$

Correlation coefficient

$$r = \frac{\text{covariance}}{\text{standard deviation}} = \frac{\sigma_{xy}}{\sigma_x \cdot \sigma_y}$$

Step 6: Computation of the equation of linear regression (log Q = y; log H = x):

$$y = a_o + a_1 x$$

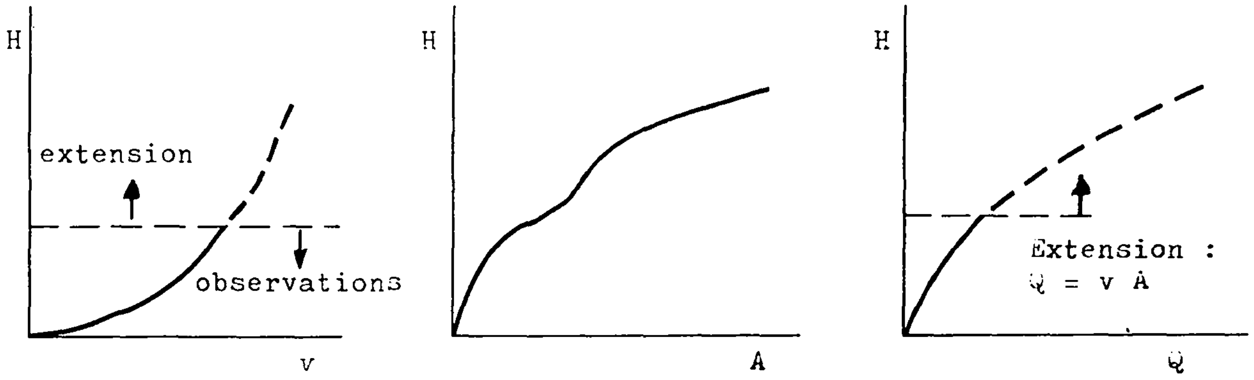
Intercept on the y-axis $a_o = \bar{y} - a_1\bar{x}$

Step 7: Antilog of the variates

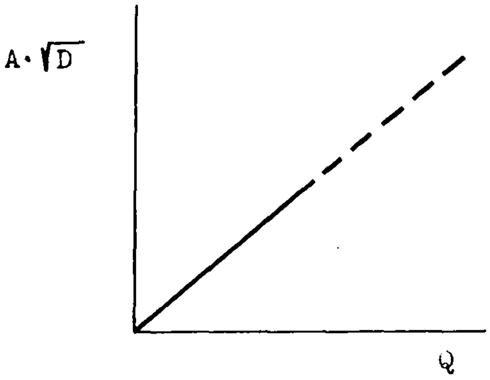
$$Q = a_o \cdot H^{a_1}$$

The graphical solution of a linear regression is given in Example 2.

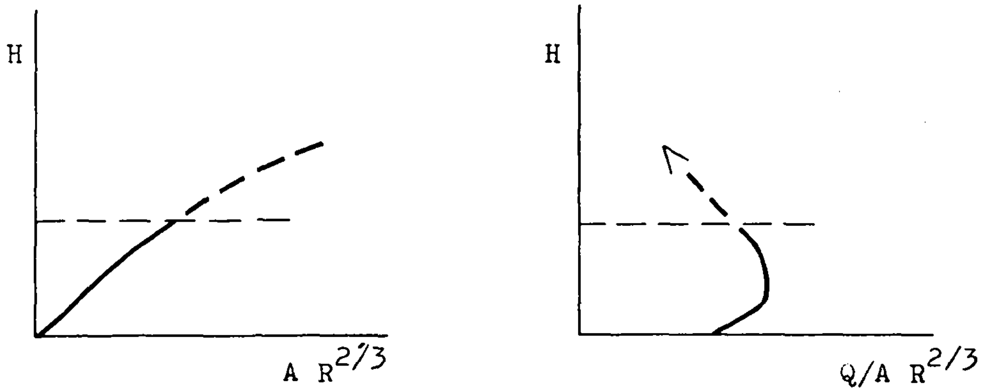
The extension of the rating curve can be performed in different ways. By using the log-log extension method, a straight line is fitted to the observed data if the cross section is regular. In the stage-area, stage-velocity method the stream flow measurements and cross-section areas are plotted against the stage H and extrapolated by using Manning's or Chezy's formula. Instead of plotting v and A, conveyance factors A D or $AR^{2/3}$ can be plotted and extrapolated. (See fig. 10).



(a) Stage- area, stage-velocity method



(b) Steven's method (using Chezy's formula)



(c) Extension with conveyance factor

Fig. 10 Extension of rating curves

2.5 Flow measuring methods utilizing artificial cross-sections

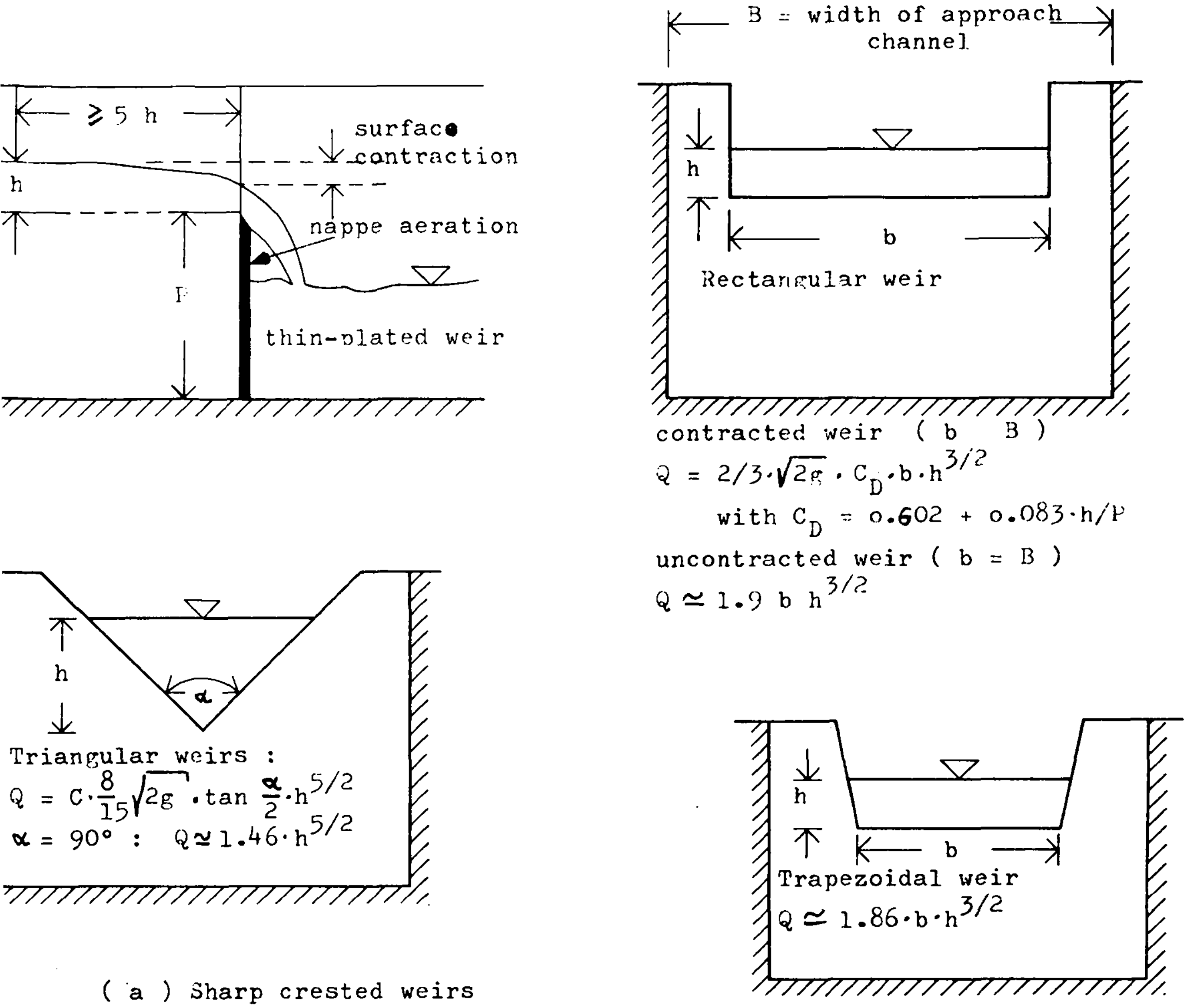
With flow measuring methods utilizing artificial cross-sections, the discharge can be determined by the application of hydraulic formulas. A calibration of the stage-discharge relation is not necessary. Weirs and flumes are used (as artificial cross-sections). They are mostly limited to small stream and laboratory experiments.

2.51 Weirs

Sharp crested (or thin plated) weirs commonly have a rectangular, triangular and trapezoidal opening (see fig. 9). (For details for construction and use see: WMO Techn. Note No. 117). When the range of flow is fairly large compound weirs can be used.

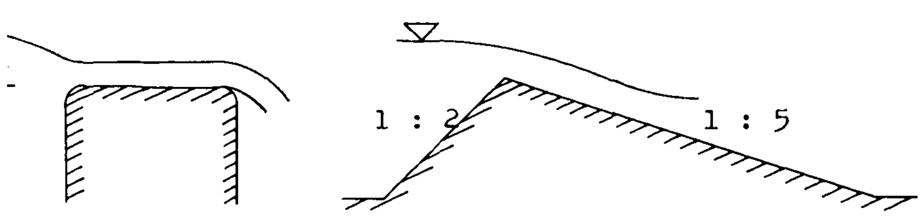
The weirs should be well placed (crest horizontal) and a free fall assured. The weir cannot work under submerged conditions. The weir is sensitive to approach conditions. The stilling pool upstream of the weir should be cleaned of deposited sediments at given time intervals.

The broad-crested weir is used in controls and spillways and is less sensitive to submergence. (For standard types of this weir see WMO Techn. Note No. 118).



(a) Sharp crested weirs

(b) Compound weirs



(c) Broad-crested weirs

Fig. 11 Weirs

2.52 Flumes

A standing wave flume is a streamlined structure in a channel. The shape of the structure (constriction in width and/or in depth) creates a fall in water level in the constricted part and a hydraulic jump. The most commonly used flumes are the Parshall Flume and the Venturi Flume (see fig. 12, Table IV and V).

The geometry and dimensions of the Parshall flume is standardized. The general discharge equations for free-flows are:

$$Q = 4bh_a^{1.522b^{0.026}} \quad \text{(cfs) for } 1 < b < 8 \text{ ft}$$

$$Q = (3.6875 \cdot b + 2.5)\, h_a^{1.6} \quad \text{(cfs) for } 10 < b < 50 \text{ ft.}$$

For Venturi-flumes, a calibration can be necessary.

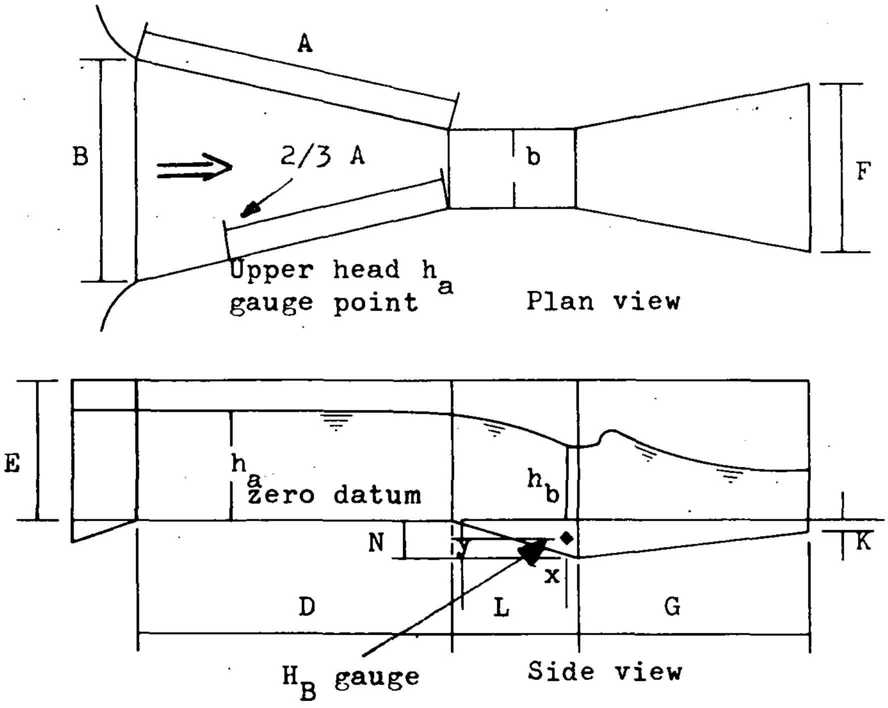


Fig. 12 Configuration and descriptive nomenclature for Parshal flumes

Widths			Axial Lengths			Wall Depth in Converging section	Vertical distance below crest		Converging wall length	Gage points			Free flow capacities	
Size: Throat Width	Upstream end	Down stream end	Converging section	Throat section	Diverging section		Dip at Throat	Lower end of flume		h_a dist. upstream of crest**	h_b		min	max
b	B	F	D	L	G	E	N	K	A*		x	y		
inches	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	cfs	cfs
1	0.549	0.305	1.17	0.250	0.67	0.5–0.75	0.094	0.062	1.19	0.79	0.026	0.042	0.005	0.15
2	.700	.443	1.33	.375	.83	0.50–0.83	.141	.073	1.36	.91	.052	.083	.01	.30
3	.849	.583	1.50	.500	1.00	1.00–2.00	.188	.083	1.53	1.02	.083	.125	.03	1.90
6	1.30	1.29	2.00	1.00	2.00	2.0	.375	.25	2.36	1.36	.167	.25	.05	3.90
9	1.88	1.25	2.83	1.00	1.50	2.5	.375	.25	2.88	1.93	.167	.25	.09	8.90
Feet														
1.0	2.77	2.00	4.41	2.0	3.0	3.0	.75	.25	4.50	3.00	.167	.25	.11	16.1
1.5	3.36	2.50	4.66	2.0	3.0	3.0	.75	.25	4.75	3.17	.167	.25	.15	24.6
2.0	3.96	3.00	4.91	2.0	3.0	3.0	.75	.25	5.00	3.33	.167	.25	.42	33.1
3.0	5.16	4.00	5.40	2.0	3.0	3.0	.75	.25	5.50	3.67	.167	.25	.61	50.4
4.0	6.35	5.00	5.88	2.0	3.0	3.0	.75	.25	5.00	4.00	.167	.25	1.30	67.9
5.0	7.55	6.00	6.38	2.0	3.0	3.0	.75	.25	6.50	4.33	.167	.25	1.60	85.6
6.00	8.75	7.00	6.86	2.0	3.0	3.0	.75	.25	7.0	4.67	.167	.25	2.60	103.5
7.0	9.95	8.00	7.35	2.0	3.0	3.0	.75	.25	7.5	5.0	.167	.25	3.00	121.4
8.0	11.15	9.00	7.84	2.0	3.0	3.0	.75	.25	8.0	5.33	.167	.25	3.50	139.5
10	15.60	12.00	14.0	3.0	6.0	4.0	1.12	.50	9.0	6.00	.75	1.00	6	300
12	18.40	14.67	16.0	3.0	8.0	5.0	1.12	.50	10.0	6.67	.75	1.00	8	520
15	25.0	18.33	25.0	4.0	10.0	6.0	1.50	.75	11.5	7.67	.75	1.00	8	900
20	30.0	24.00	25.0	6.0	12.0	7.0	2.25	1.00	14.0	9.33	.75	1.00	10	1340
25	35.0	29.33	25.0	6.0	13.0	7.0	2.25	1.00	16.5	11.00	.75	1.00	15	1660
30	40.4	34.67	26.0	6.0	14.0	7.0	2.25	1.00	19.0	12.67	.75	1.00	15	1990
40	50.8	45.33	27.0	6.0	16.0	7.0	2.25	1.00	24.0	16.0	.75	1.00	20	2640
50	60.8	56.67	27.0	6.0	20.0	7.0	2.25	1.00	29.0	19.33	.75	1.00	25	3260

* For sizes 1' to 8', A = bl2 + 4

** H_A located 2/3 A distance from crest for all sizes; distance is wall length, not axial.

Note: Flume sizes 3 inches through 8 feet have approach aprons rising at a 1:4 slope and the following entrance roundings: 3 through 9 inches, radius = 1.33 feet; 1 through 3 feet, radius = 1.67 feet; 4 through 8 feet, radius = 2.00 feet.

Table V: Discharge table for Parshall measuring flumes, sizes 1 foot to 8 feet for free-flow conditions.

h_a	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
feet	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
0.10	0.11	0.15							
.15	.20	.30	0.42	0.61					
.20	.35	.51	.66	.97	1.26	1.55			
.25	.49	.71	.93	1.37	1.80	2.22	2.63	3.02	3.46
.30	.64	.94	1.24	1.82	2.30	2.96	3.52	4.08	4.62
.4	.99	1.47	1.93	2.86	3.77	4.68	5.57	6.46	7.34
.5	1.39	2.06	2.73	4.05	5.36	6.66	7.94	9.23	10.5
.6	1.84	2.73	3.62	5.39	7.15	8.89	10.6	12.4	14.1
.7	2.33	3.46	4.60	6.86	9.11	11.4	13.6	15.8	18.0
.8	2.85	4.26	5.66	8.46	11.3	14.0	16.8	19.6	22.4
.9	3.41	5.10	6.80	10.2	13.6	16.9	20.3	23.7	27.0
1.0	4.00	6.00	8.00	12.0	16.0	20.0	24.0	28.0	32.0
1.2	5.28	7.94	10.6	16.0	21.3	26.7	32.1	37.5	42.9
1.4	6.68	10.1	13.5	20.3	27.2	34.1	41.1	48.0	55.0
1.6	8.18	12.4	16.6	25.1	33.6	42.2	50.8	59.4	68.1
1.8	9.79	14.8	19.9	30.1	40.5	50.8	61.3	71.8	82.3
2.0	11.5	17.4	23.4	35.5	47.8	60.1	72.5	84.9	97.5
2.2	13.3	20.2	27.2	41.3	55.5	69.9	84.4	98.9	113.6
2.4	15.2	23.0	31.1	47.3	63.7	80.3	97.0	113.7	130.7

Table VI: Discharge table for Parshall measuring flumes, sizes 10 feet to 50 feet for free-flow conditions

h_a	10 feet	12 feet	15 feet	20 feet	25 feet	30 feet	40 feet	50 feet
feet	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
0.30	5.75	6.75	8.4	11.1	13.8	16.5	21.8	27.3
0.4	9.05	10.85	13.3	17.7	21.8	26.1	34.6	43.2
0.5	13.0	15.4	19.1	25.1	31.2	37.2	49.5	61.8
0.6	17.4	20.6	25.5	33.7	41.8	50.0	66.2	82.6
0.7	22.2	26.2	32.7	43.1	53.4	64.0	84.8	105.5
0.8	27.5	32.7	40.4	53.4	66.3	79.2	105	131
0.9	33.3	39.4	48.9	64.3	80.1	95.5	127	158
1.0	39.4	46.8	57.9	76.3	94.8	113.2	150	187
1.2	52.7	62.6	77.3	102.0	127.0	152	201	250
1.4	67.4	80.1	99.0	130.5	162	194	257	320
1.6	83.5	99.1	122.8	162	201	240	318	396
1.8	100.9	119.8	148.0	195	243	290	384	479
2.0	119.4	141.8	175.3	232	287	343	454	567
2.2	139.0	165.0	204	269	334	400	530	660
2.4	159.9	189.8	235	310	384	459	609	758
2.6	181.7	215.7	267	352	437	522	692	864
3.0	228.4	271.2	335	442	549	656	870	1084
3.5	294	347	429	566	703	840	1113	1387
4.0	363	430	531	700	870	1040	1379	1717
4.5	437	518	641	846	1051	1255	1664	2073
5.0	517	614	759	1002	1244	1486	1970	2453
5.5			885	1166	1448	1730	2295	2860
6.0			1016	1340	1664	1988	2638	3285

Note: Available data indicates that extension of the above ratings to greater heads is reliable.

3. Examples

3.1 Example 1 Computation of discharge by using hydraulic formula and extension of a rating curve by stage-area, stage velocity method.

(i) Calculation of the stage-area relation

For the given cross-section (see fig. 13) and individual gauge heights, the wetted cross-section is calculated separating channel and flood-plain.

a) bankful discharge (= + 68 m msl)

$$A = \left[\frac{(68 - 68) + (68 - 65.0)}{2}\right] \cdot (75 - 70) + \left[\frac{(68.0 - 65.0) + (68.0 - 65.5)}{2}\right]$$

$$\cdot (85 - 75) + \left[\frac{(68 - 65.5) + 68 - 67.5)}{2}\right] (95 - 85) + \frac{68 - 67.5}{2}$$

$$\left[\frac{(68 - 67.5)(100 - 95)}{(69 - 67.5)}\right] = 7.5 + 27.5 + 15 + 0.4 = 50.4 \text{ m}^2$$

b) For given gauge heights, the area A (in m^2) and wetted perimeter P (in m) are given below. The hydraulic radius $R = A/p$ is given according to Manning's formula.

Elevation of water level (msl)	Channel			Flood Plain			Total		
	A (m^2)	P (m)	$R^{2/3}$	A (m^2)	P m	$R^{2/3}$	A (m^2)	P (m)	$R^{2/3}$
+ 68 m	50.4	26.7	1.53	-	-	-	50.4	26.7	1.53
+ 69 m	78.8	30	1.90	71.3	55	1.19	150.1	85	1.46
+ 72 m	176.3	35	2.94	251.3	65	2.46	427.6	100	2.63
+ 74 m	251.3	40	3.40	386.3	70	3.12	637.6	110	3.22

See fig. 14 for the stage-area relation.

(ii) Computation of coefficient of roughness

The bankful discharge (+ 68 m msl) was measured with 43.5 m^3/sec. The highest discharge measurement was 104 m^3/sec at a gauge height of + 69 m msl. The slope of water surface was 0.35‰ .

a) Roughness coefficient k of channel (bankful discharge)

$Q = v.A; \quad v = k.R^{2/3}.J^{\frac{1}{2}}$ (Formula of Gauckler-Manning-Strickler)

$$v = \frac{Q}{A}; \quad k = \frac{v}{R^{2/3}.J^{\frac{1}{2}}}$$

$$v = \frac{43.5}{50.4} = 0.86 \text{ m/sec}; \quad k = \frac{0.86}{1.53 \cdot 0.01874} = 30$$

b) Roughness coefficient of flood plain ($Q = 103$ m^3/sec; + 69 m msl)

Channel flow Q_c :

$v = 30 \cdot 1.90 \cdot 0.01874 = 1.07$ m/s; $Q_c = 1.07 \cdot 78.8 = 84.3$ m^3/sec

Flood plains:

$Q_F = 104 - 84 = 19.7$ m^3/sec $\quad v = \frac{Q}{A} = \frac{19.7}{71.3} = 0.28$ m/sec;

$$k = \frac{0.28}{1.19 \cdot 0.01874} = 12.5$$

(iii) Extension of the rating curve

The highest observed flood stage was at + 72 m msl. Calculate the approximate peak discharge and extend the rating curve.

Channel flow:

$Q_c = 30 \cdot 2.94 \cdot 0.01874 \cdot 176.3 = 291$ m^3/sec

Flood plains

$Q_F = 12.5 \cdot 2.46 \cdot 0.01874 \cdot 251.3 = 145$ m^3/sec

$Q_{Total} = 291 + 145 = 436$ m^3/sec.

See fig. 15 for the extension of the rating curve.

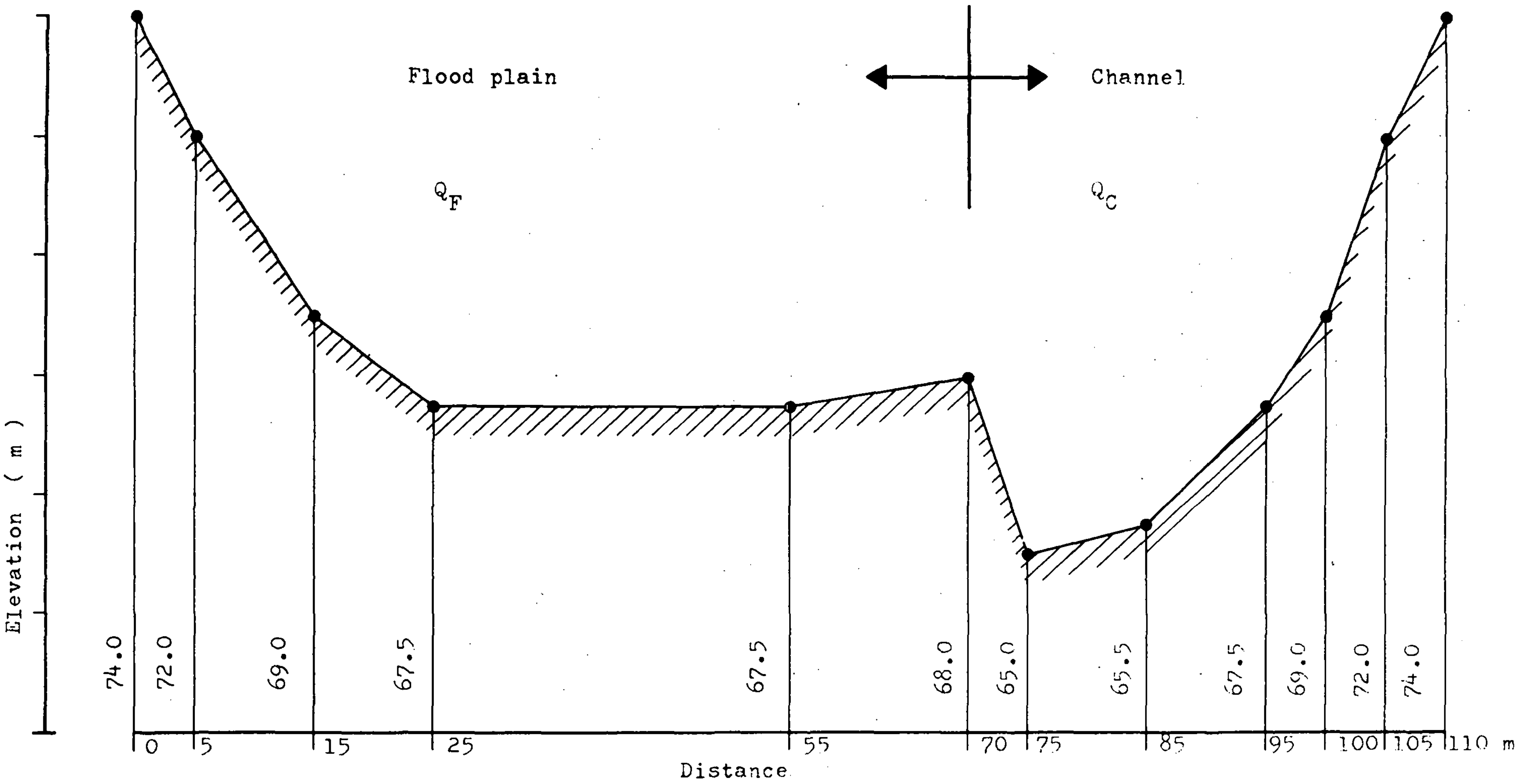


Fig. 13 Cross section of channel and flood plain near the gauging station (example)

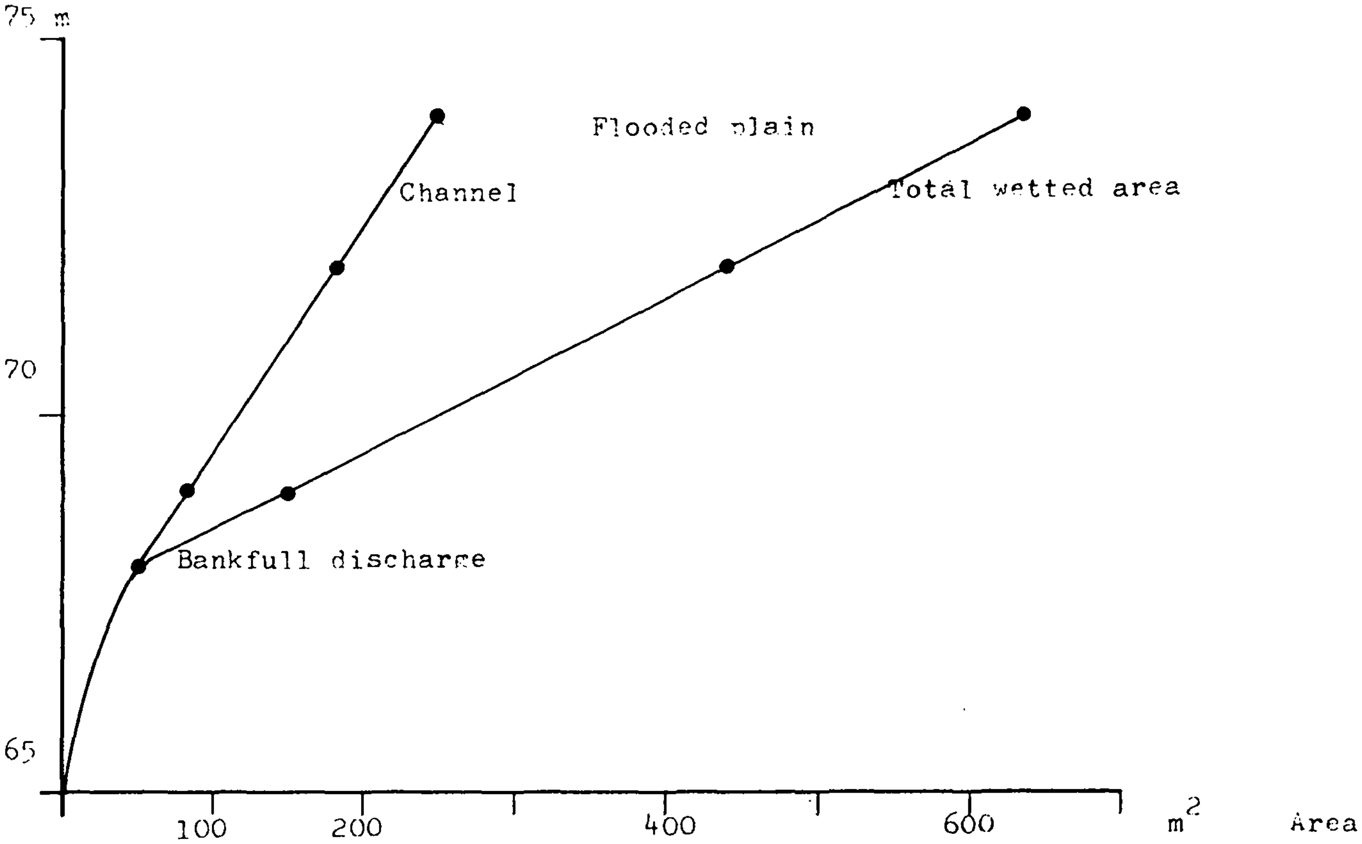
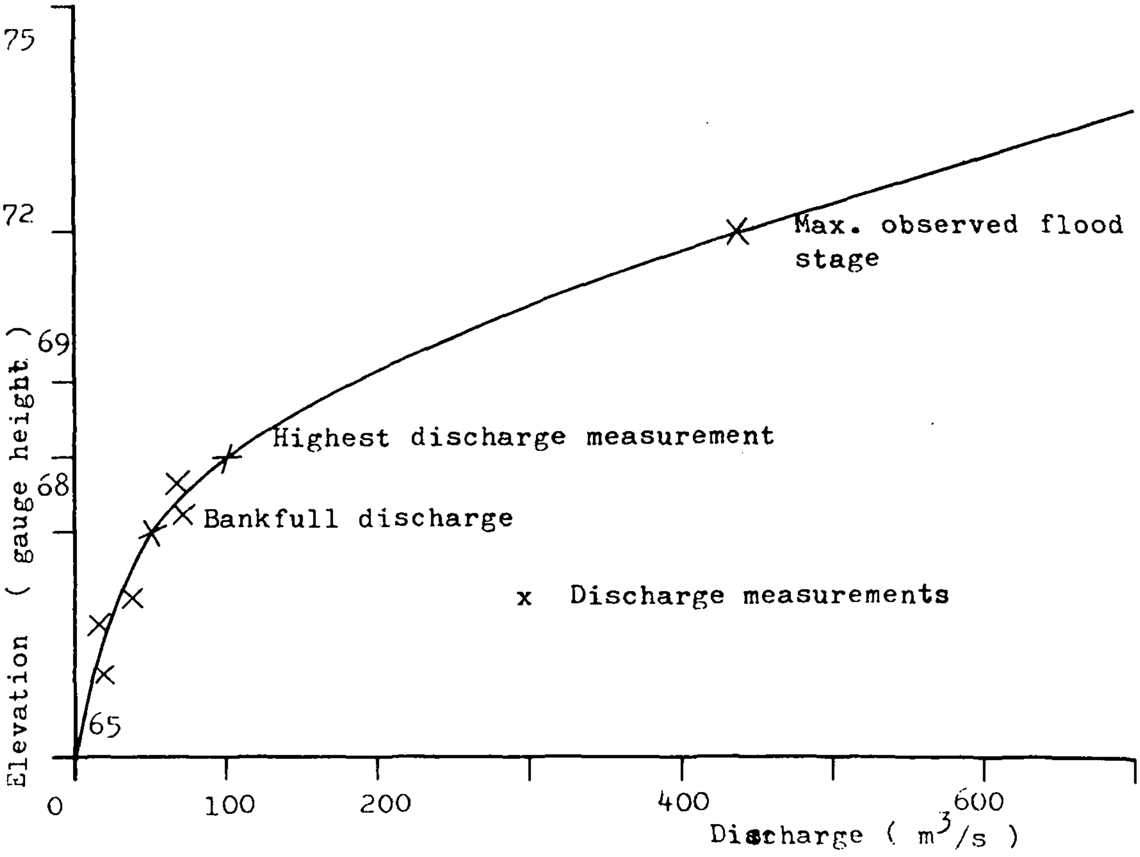


Fig. 14 Stage-area relation (example)

Fig. 15
Rating curve
(example)



3.2 <u>Example 2</u> Computation of a rating curve by using a linear regression of the logarithmic values.

<u>Given</u> Measurements of discharges:

No. of discharge measurement	Time (sec)	Gauge Height H (cm)	Discharge Q (m^3/sec)	Remarks
1	8.02.57	465	122	
2	29.03.58	211	20.0	
3	6.05.58	180	13.0	
4	27.07.58	144	8.05	
5	19.09.59	217	20.5	
6	8.12.60	312	53.0	water rising 8 cm/hour.
7	16.05.61	228	23.5	
8	27.08.62	166	10.7	
9	10.06.62	113	4.33	
10	24.08.63	163	10.2	
11	11.09.63	129	5.08	
12	16.02.64	148	8.25	
13	29.07.64	300	45.0	
14	27.11.64	291	38.4	water falling 10 cm/hour

The numerical solution is given in Table VII; the graphical solution is shown in fig. 14. a_1 is equal to tan α , the tangent between the line and the y-axis, using any rectangular triangle formed by the line with the coordinates. The ratio of real distances used for calculating the tangent measured in centimetres must be transformed according to the scales. Parameter a_o will be the distance from the origin of intersection of the line with the y-axis (H = 1 cm, $a_o \doteq {}^1/21507$).

Table VII: Numerical solution of Example 2

Gauge Height H (cm)	log H = x	Discharge Q (m³/sec)	log Q = y	(log H − log $\bar{H}$) = (x − $\bar{x}$)	(log H − log $\bar{H}$)² = (x − $\bar{x}$)²	(log Q − log $\bar{Q}$) = (y − $\bar{y}$)	(log Q − log $\bar{Q}$)² = (y − $\bar{y}$)²	(log H − log $\bar{H}$) x (log Q − log $\bar{Q}$) = (x − $\bar{x}$)(y − $\bar{y}$)
113	2.05308	4.33	0.63649	− 0.25415	0.06459	− 0.60235	0.36283	0.15309
129	2.11060	5.08	0.69897	− 0.19663	0.03866	− 0.53987	0.29146	0.10615
144	2.15836	8.05	0.90580	− 0.14887	0.02216	− 0.33304	0.11092	0.04958
148	2.17026	8.25	0.91645	− 0.13697	0.01876	− 0.32239	0.10394	0.04416
163	2.21219	10.2	1.00860	− 0.09504	0.00903	− 0.23024	0.05301	0.02188
166	2.22011	10.7	1.02938	− 0.08712	0.00759	− 0.20946	0.04387	0.01825
180	2.25527	13.0	1.11394	− 0.05196	0.00270	− 0.12490	0.01560	0.00649
211	2.32428	20.0	1.30103	0.01705	0.00029	0.06219	0.00387	0.00106
217	2.33646	20.5	1.31175	0.02923	0.00085	0.07291	0.00532	0.00213
228	2.35793	23.5	1.37107	0.05070	0.00257	0.13223	0.01748	0.00670
291	2.46389	38.4	1.58433	0.15666	0.02454	0.34549	0.11936	0.05412
300	2.47712	45.0	1.65532	0.16989	0.02886	0.41648	0.17346	0.07076
312	2.49415	53.0	1.72428	0.18692	0.03494	0.48544	0.23565	0.9074
465	2.66745	122	2.08636	0.36022	0.12976	0.84752	0.71829	0.30529
Total	32.30115		17.34377		0.38530		2.25506	0.93040

Mean:

$\log \bar{H} = \bar{x} = \frac{32.30115}{14} = 2.30726$

$\bar{H} = 10^{2.30726} = \underline{203 \text{ cm}}$

$\log \bar{Q} = \bar{y} = \frac{17.34377}{14} = 1.23884$

$\bar{Q} = 10^{1.23844} = \underline{17.3 \text{ m}^3/\text{sec}}$

Variances:

$\sigma x^2 = \frac{1}{n-1} \sum (x - \bar{x})^2 = \frac{0.38530}{14 - 1} = 0.029638$

$\sigma y^2 = \frac{1}{n-1} \sum (y - \bar{y})^2 = \frac{2.25506}{14 - 1} = 0.1734662$

Covariance:

$\sigma_{xy} = \frac{1}{n-1} \sum (x - \bar{x})(y - \bar{y}) = \frac{0.93040}{14 - 1} = 0.071569$

Correlation coefficient:

$r = \frac{\sigma_{xy}}{\sigma_x \sigma_y} = \frac{0.071569}{(0.17216)(0.41649)} = 0.998$

Standard deviation:

$\sigma_x = \sqrt{\sigma_x^2} = 0.17216; \quad {}_y = \sqrt{\sigma_y^2} = 0.41649$

Linear regression: $y = a_1 x + a_o$

$a_1 = \frac{\sigma_{xy}}{\sigma_x^2} = \frac{0.071569}{0.029638} = 2.415$

$a_o = \bar{y} - a_1 \bar{x} = 1.2388 - 2.415.2.30723$

$= 4.33259$

$\log Q = \underline{-4.33259 + 2.415 . \log H}$

$Q = \frac{1}{21508} H^{2.415}$

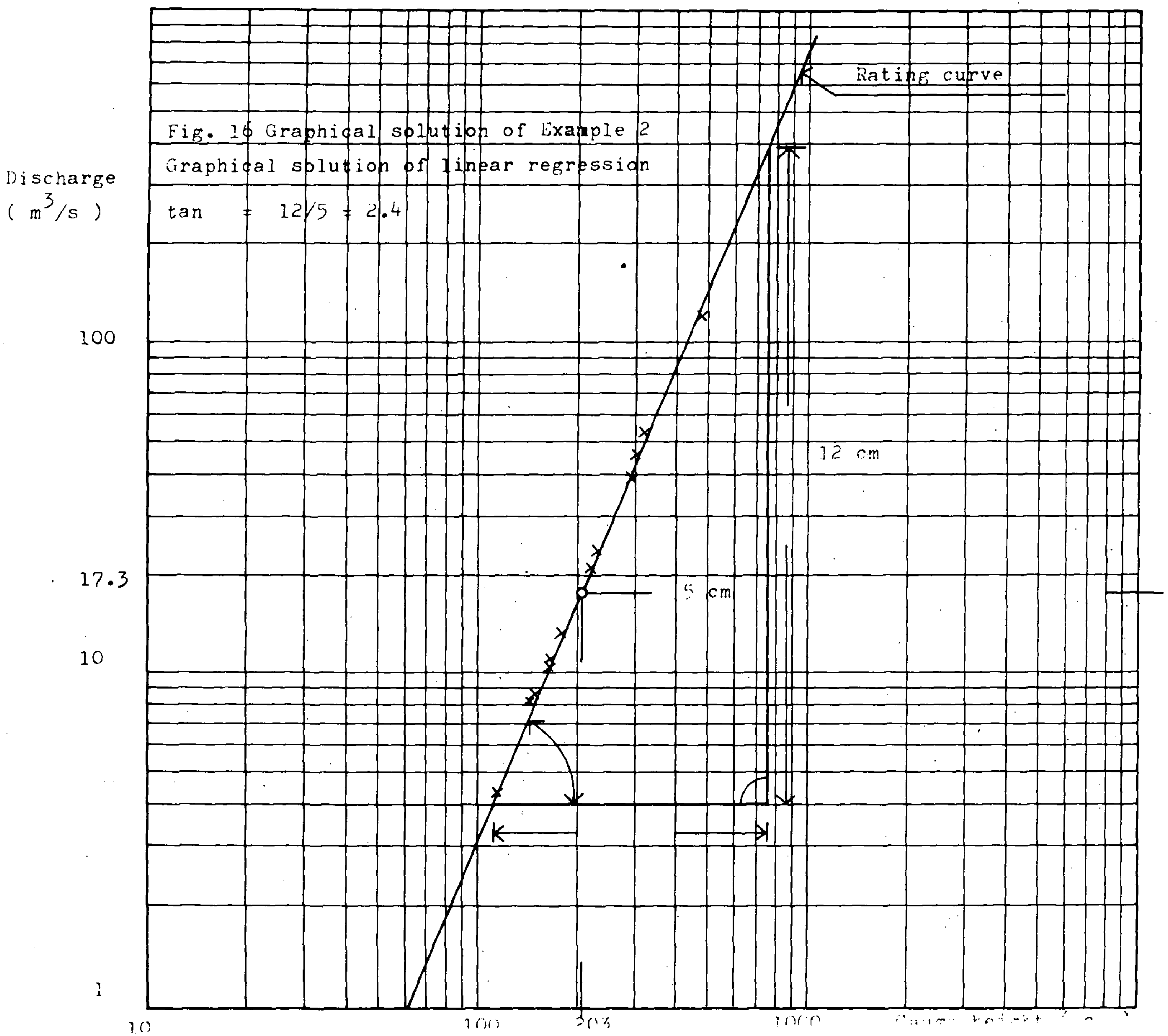


Fig. 16 Graphical solution of Example 2

4. Exercises

4.1 Exercise 1

The streamflow measurements of a gauging station are given. The bankfull discharge is at a gauge height of 340 cm. Find the rating curve by using linear regression of log values (numerical and graphical solution).

No. of observations	Gauge height H (cm)	Discharge Q (m^3/sec)	log H	log Q
1	116	7.00	2.06446	0.84510
2	112	9.00	2.04922	0.95424
3	120	10.0	2.07918	1.00000
4	144	15.2	2.15836	1.18184
5	157	16.9	2.19590	1.22789
6	205	27.2	2.31175	1.42457
7	244	38.4	2.38739	1.58433
8	279	49.6	2.44560	1.69548
9	308	62.7	2.48855	1.79727
10	336	72.9	2.52634	1.86273
11	348	76.4	2.54158	1.88309
12	348	80.2	2.54158	1.90417
13	380	104.0	2.57978	2.01703
14	388	105.0	2.58883	2.02119
15	413	118.0	2.61595	2.07188
16	420	121.0	2.62325	2.08279
17	408	128.0	2.61066	2.10721
18	468	186.0	2.67025	2.26951
19	488	241.0	2.68842	2.38202

Solution: (a) Rating curve 1 below bankfull discharge

$\log Q_1 = - 3.15438 + 1.98740 \log H_1$

Parameters : $\log \bar{H}_1 = 2.27067;\ \sigma_{\log H_1} = 0.18417$

$\log \bar{Q}_1 = 1.35834;\ \sigma_{\log Q_1} = 0.36832$

Correlation coefficient $r_1 = 0.99376$

Rating curve 2 above the bankfull discharge

$\log Q_2 = 6.28533 + 3.20996 \log H_2$

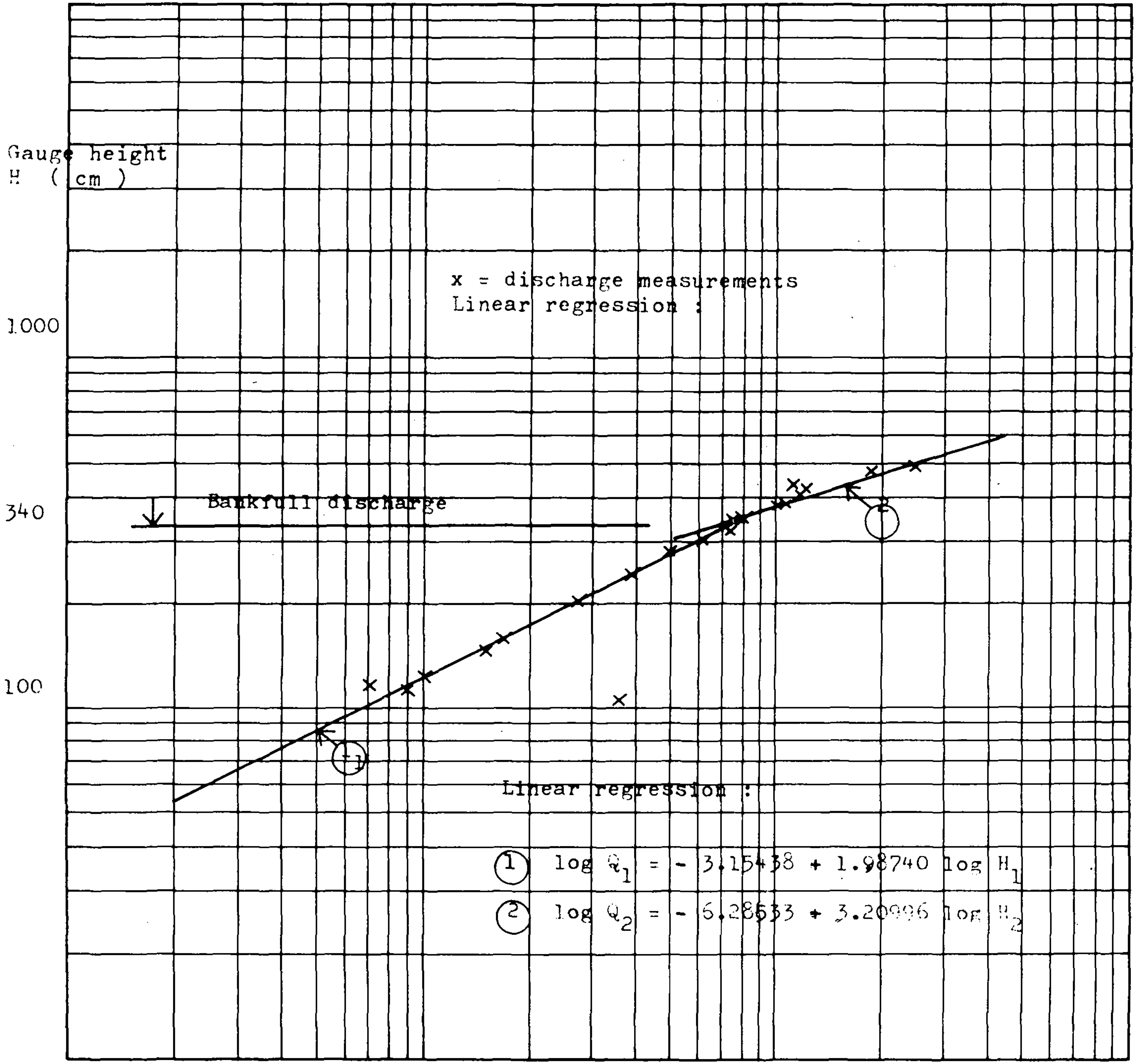
$\log \bar{H}_2 = 2.60670;\ \sigma_{\log H_2} = 0.05075$

$\log \bar{Q}_2 = 2.08209;\ \sigma_{\log Q_2} = 0.16022$

Correlation coefficient $r_2 = 0.98358$

Find point of intersection.

(b) See fig. 17 for graphical solution.



Linear regression :

① $\log Q_1 = -3.15438 + 1.98740 \log H_1$

② $\log Q_2 = -6.28533 + 3.20996 \log H_2$

Fig. 17
Graphical
solution of
exercise I

4.2 Exercise 2

Find the rating curve by using linear regression of log values of the following streamflow observations:

No. of observations	Gauge height H (ft)	Discharge Q (cfs)
1	9.9	7300
2	7.7	2700
3	7.3	2300
4	6.9	2100
5	6.7	1900
6	6.1	1550
7	5.8	1450
8	4.7	1050
9	3.3	500
10	2.8	410
11	2.7	340
12	2.5	210
13	2.3	100
14	2.1	60
15	1.95	40
16	1.90	30

Calculate the rating table.

4.3 Exercise 3

Given the head, $h = 0.30$ m of a sharp crested weir. Find the discharge of:

a) triangular weir ($= 90^{o}$)
b) contracted rectangular weir ($b = 0.50$ m, $h/p = 0.5$)
c) uncontracted rectangular weir, ($b = 0.5$ m)
d) trapezoidal weir ($b = 0.5$ m)

Solutions:

a) 0.072 m^3/sec b) 0.156 m^3/sec
c) 0.156 m^3/sec d) 0.153 m^3/sec

4.4 Exercise 4

In a stream of discharge Q (litre/sec), a tracer of $q = 1$ litre/sec is injected constantly with a concentration of $C_1 = 1000$ g sodium chloride per litre. The natural content of sodium chloride is $c_o = 0$ g/litre. At the sample section the concentration is $C_2 = 10$ g/litre. Find the discharge.

Solution:

$$Q = 1 \cdot \frac{1000 - 10}{10 - 0} = 99 \text{ litres/sec}$$

F. HYDROGEOLOGY

1. Basic concepts in physical geology

1.1 Matter, minerals and rocks

The smallest unit of matter is called an atom. All matter is therefore composed of atoms, each of which is extremely minute. They range in size from 2 to 5 angstroms (Å) in diameter (1Å = 1×10^{-8} cm). All elements, for example hydrogen (H), oxygen (O), carbon (C), etc. consists of atoms.

Elements combine to form compounds; for example, hydrogen combines with oxygen to form water (H_2O). Compounds are therefore combinations of atoms of different elements. The chemical designation H_2O is simply a combination of symbols for the elements H and O in the proportions in which they are present (2 atoms of hydrogen plus 1 atom of oxygen). Each such unit (H_2O) is called a molecule of water. A molecule is thus the smallest unit of a compound. Elements and/or compounds constitute matter of which the basic components are atoms.

We can now define minerals. A mineral is a naturally occurring solid phase possessing a characteristic internal structure determined by a regular arrangement of the atoms. It is a natural object with a definite chemical composition. Some minerals like copper (cu), diamond (C) and sulphur (S) are composed of only one element; others are compounds, being made up of two or more elements, for example water (H_2O), halite (common salt - NaCl), calcite ($CaCO_3$), etc. The orderly pattern of atoms of elements in a mineral is referred to as its crystalline structure.

Although there are more than 2000 minerals known, only a few of these are rock-forming minerals. Perhaps the commonest rock-forming minerals are silicates: quartz, feldspars, micas, olivine, hornblende, augite, and calcite. A rock, therefore, is an aggregate of minerals (a mineral assemblage). Mineral assemblages help us to distinguish one rock from another. To geologists, a rock may be hard or firmly cemented, or it may be 'soft' or loosely coherent. A rock may hold water, either in its pore spaces or in fractures. A water-bearing rock is usually referred to as an aquifer.

Three broad groups of rocks are universally recognized:

(i) Igneous rocks - those which are formed by the solidification of molten rock materials (magma)

(ii) Sedimentary rocks - those which are produced by the deposition and accumulation of debris transported by rivers, ice, wind, etc.

(iii) Metamorphic rocks - those which are formed from the alteration of igneous and sedimentary rocks by heat and pressure.

1.2 Igneous rocks

These rocks are of two types:

(i) Extrusive igneous rocks - those which are produced by volcanic eruptions - basalt, phonolite, trachyte, volcanic ashes, etc.

(ii) Intrusive igneous rocks - those which do not reach the surface but merely intrude into the rocks of the earth's crust - granite, gabbro, dolerite, etc.

1.3 Sedimentary rocks

This group of rocks is divided into detrital (fragmental or clastic) and chemical/biochemical (organic or non-clastic) rocks.

Detrital rocks are further sub-divided into three categories according to the size of constituent particles:

(i) Pebbly (rudaceous) sediments (> 2 mm) - pebbles, gravels and conglomerates.

(ii) Sandy (arenaceous) " (0.06-2mm) - sands and sandstones.

(iii) Muddy (argillaceous) " (< 0.06mm) - clays, silts and shales.

The chemical/biochemical rocks are sub-divided into:

(i) Chemical - e.g., evaporites like gypsum, anhydrite, rock salt (halite)

(ii) Organic - e.g., limestone, dolomite, chalk, diatomite.

1.4 Metamorphic rocks

These are derived from pre-existing rocks which re-crystallize under high pressure and temperature, e.g., granite → gneiss, shale → slate, limestone → marble.

1.5 Weathering, erosion and deposition

Weathering is the disintegration of rocks either through physical processes (physical/mechanical weathering) or through chemical processes (chemical weathering). Weathering products may be eroded away by water, ice, or wind. Water is by far the most widespread agent of erosion.

Water, ice, and wind (as well as gravity) are also the media of transport of eroded materials and other debris. The transported debris is subsequently deposited in marine, fluvial or other depositional environments. Once in the depositional environment, the sediments may subsequently become lithified to form hard sedimentary rocks. The lithification of detrital (clastic) sediments usually takes place by cementation.

2. Hydraulic properties of rocks

2.1 Porosity

Porosity and permeability are fundamental properties of rocks upon which the study of groundwater is based. Porosity is the ratio of total pore volume (total volume of openings in a rock) to the volume of the rock. Pores provide the space for both storage and movement of subsurface water.

Porosity is usually expressed as a percentage, i.e.,

$$a = \frac{w}{V} .100$$

where a is the porosity (in % by volume), w the volume of pore space and V the bulk volume of the sample. The simplest method of measuring the porosity of granular materials in the laboratory is by saturating a completely dry sample with water. The volume of pore space, in this case, is that of water which is added to the dry sample.

Porosity values differ from one rock to another. Porosities of most rocks range between 12 and 45% but in certain clastic rocks, they may be as much as 80 - 90%. The porosity value depends upon the shape, sorting and packing of constituent grains; it also depends upon the degree of cementation.

While the capacity of a rock to hold water (water-holding or water-retention capacity of a rock) is determined by its porosity, the capacity to yield it to pumping (water-yield capacity of a rock) depends upon pore size and intercommunication rather than on porosity alone. While sands and sandstones yield relatively substantial amounts of water, clays and shales yield little water to a well even though they may have high porosity values.

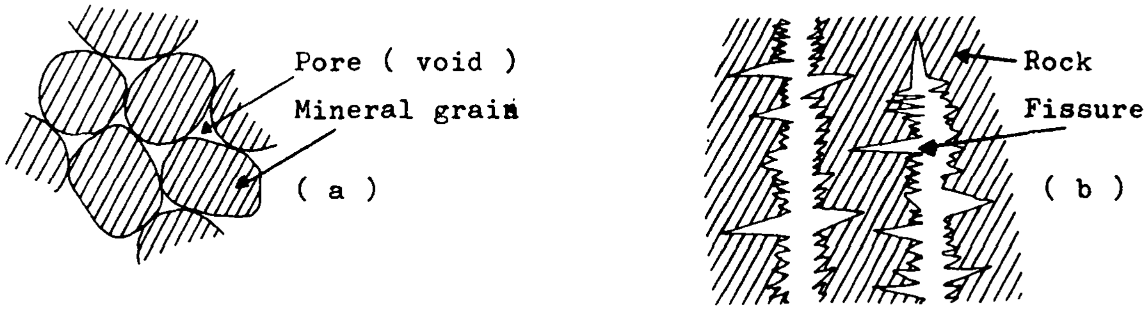


Fig. 1 Porosity in rocks : (a) Porosity due to inter-grain voids, (b) Porosity due to fissures (cracks, joints or faults)

2.2 Permeability

The permeability of a material (rock) is its capacity to transmit a fluid (water). Movement of the fluid through the material is possible due to the presence of openings (pore spaces) in it. In a large rock body of relatively uniform character, permeability tends to increase as the porosity increases. Pore size is again a crucial factor movement or flow of water through the rock depends upon it.

Therefore permeability, also called fluid or hydraulic conductivity, is the property which determines the field of a water-bearing rock. A sand with 20% pore space is much more permeable than a clay with 35% because the size and inter-connection of openings in the sand are much more facilitating to the movement of water. Thus in studying permeability, we find that the character of openings, rather than the total volume of pore space, has greater influence on the flow of water through the rock. The expression of this property in quantitative terms has been attempted and is called the coefficient of permeability, usually designated as K. We shall see below how this is done.

Fig. 2 Permeability in rocks : Impermeable rock-grains are so small a tightly packed that any water there may be is tightly held to the gra by adhesive forces, (b) permeable rock - water can move through larger spaces although some of it is held

2.3 Darcy's Law

A French engineer, Henri Darcy (1856), proposed an equation to express the rate of water movement (flow) through a rock. What is today referred to as Darcy's Law can be stated as follows:

$$v = K \cdot \frac{h}{l} \quad \ldots\ldots\ldots (1)$$

Where v is the velocity of flow, K the coefficient of permeability, h the head (the vertical distance between the point of intake and the point of discharge of the water), and l the length of flow from intake to discharge. The quantity $\frac{h}{l}$ is sometimes referred to as the hydraulic gradient, which is the slope of the water table. Thus if h is 20m and l is 200m, the hydraulic gradient is 0.1 or 10%.

The energy which causes groundwater to flow is derived from gravity. From the point of intake, the water flows through the ground or rock to a point of discharge in a stream, lake or spring. Just as surface water needs a slope to flow on under the influence of gravity, groundwater likewise needs a slope to flow on, hence the hydrualic gradient. In a rock of constant permeability, the velocity of the water increases as the hydraulic gradient increases. Since the hydraulic gradient and the slope of the groundwater table are the same thing, we may also say that the velocity of groundwater varies with the slope of the water table. Thus the steeper the slope, the more rapid the flow. The rate of flow in ordinary water-bearing rocks has been estimated as not faster than 1.5m per day and not less than 1.5m per year. However, rates up to and over 120m per day and as little as a few centimetres per year have been recorded.

Of more practical importance than velocity is the discharge, i.e. the quantity of groundwater that percolates through a given cross-sectional area (of the water-bearing rock) in a given unit of time. Discharge is more directly important to us because it states the amount of water that is available for use. It can be expressed thus:

$$Q = vA \quad \dots\dots\dots\dots(2)$$

where Q is the discharge, v the velocity, and A the cross-sectional area of the rock material. According to Darcy's Law (equation 1),

$$v = K\cdot\frac{h}{l}$$

Therefore, substituting $K\cdot\frac{h}{l}$ for v in equation 2, we get

$$Q = K\cdot\frac{h}{l}\cdot A \quad \dots\dots\dots(3)$$

Sometimes the hydraulic gradient $\frac{h}{l}$ is designated by the symbol i so that:

$$Q = K\cdot i\cdot A$$

It is with this formula that we can estimate the amount of water that a well can be expected to deliver.

3. Rock types and relation to groundwater

3.1 Introduction

We have already observed that a rock may have both the capacity to contain water (the "reservoir" function) and the capacity to transmit it (the "pipeline" function). In studying the occurrence and availability of groundwater, we are concerned with the hydraulic characteristics of natural rock units or groups of them. These may be regarded as natural hydrological units if they are recognized or grouped on the basis of permeability which, as we have seen, is directly related to shape, size and type of openings.

3.2 Aquifer and aquicludes

Most natural hydrological units fall into one of two groups, namely aquifers and aquicludes. An aquifer is a geological formation which is sufficiently permeable to yield water in appreciable (usable) quantity to a spring, well or borehole. An aquiclude, on the other hand, is a formation with such low permeability that it will neither yield water nor transmit it readily. However, there are some rocks between these two extremes which are neither aquifers nor aquicludes, and have been called aquitards, a term which has not been widely used.

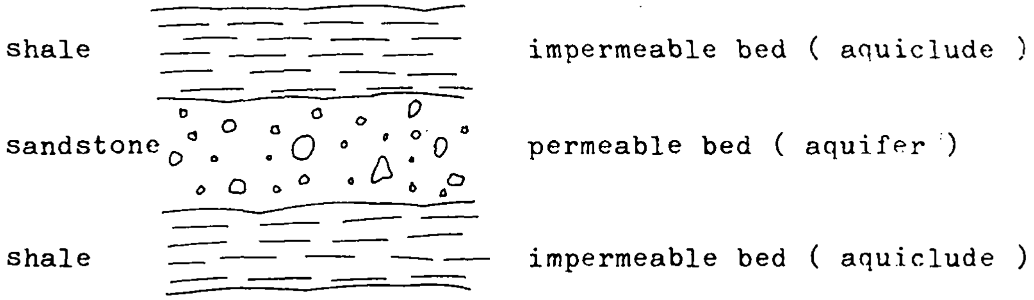


Fig. 3 A hypothetical section showing a permeable sedimentary bed (sandstone) bounded on the upper and lower sides by impermeable beds of shale

3.3 Confined (artesian) aquifers

A confined aquifer is one in which groundwater is confined under pressure by overlying and underlying aquicludes. The water in such aquifers is said to be confined, or to be under artesian conditions. When a well is constructed in an artesian aquifer, the water rises in the well to a level higher than the aquifer. The height to which it rises above the top of the aquifer depends upon the artesian pressure. An artesian pressure surface is frequently referred to as the piezometric surface.

Under unconfined conditions, on the other hand, the water only partly fills a permeable bed, and the water level is free to rise and fall.

Maps showing the position of the water table and artesian pressure surfaces are essential in the quantitative appraisals of groundwater. These maps show the points or areas of recharge and discharge, and the direction of movement of the groundwater. Where such a map shows piezometric surfaces to be higher than the land, the wells which penetrate the artesian aquifer will flow up to the land surface. The map will therefore show areas of artesian flow.

3.4 Perched groundwater

A small water-bearing body of permeable material (e.g. sand or sandstone) may be situated in a basin of impermeable material (e.g. clay or chalk). The aquiclude underlying the aquifer as such retards percolation to the main groundwater level and thus creates a perched groundwater body.

A well constructed in a perched aquifer may yield appreciable quantities of water at a shallow depth. A situation in which discontinuous bodies of permeable and impermeable materials are juxtaposed may result in very different yields to wells.

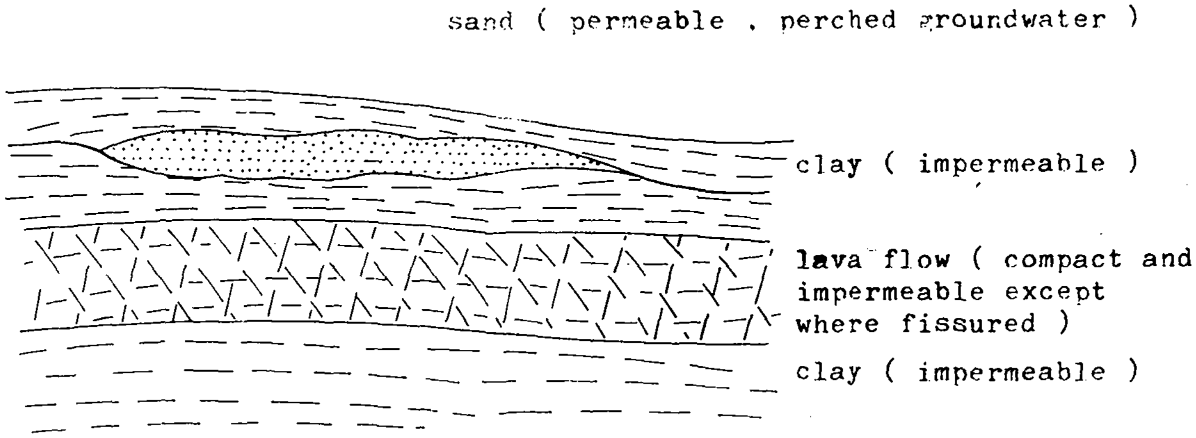


Fig. 4 Diagram showing perched groundwater - the aquifer is a small body of sand in a much larger body of impermeable clay

3.5 Specific yields of water-bearing rocks

We have previously recognized the fact that water stored in saturated porous rocks (expressed as a % of the total volume of the rock) is equivalent to the porosity of the rock. If, on the other hand, we think of water which can be recovered from the rock, we must realize that the recoverable volume of water must be smaller than the porosity because some of the water in the pores is not free to move under the influence of gravity. The remainder of the water is held against the gravity force as thin films around the grains.

Specific yield is thus a measure of water-yield capacity. It may be stated as a per centage of the total volume of rock occupied by the water which is ultimately released from the rock, thus:

$$\text{Specific yield} = \frac{\text{Volume of water retrieved}}{\text{Bulk volume of rock from which it was retrieved}} \times 100$$

The essential data can be obtained, for instance, from a pump test.

Specific yield is very much a function of the permeability of an aquifer. Therefore, a coarse sandstone, for example, should show much higher values than a shale because the former is far more permeable than the latter.

4. Groundwater in the run-off cycle

The run-off cycle is a descriptive term which is applied to that part of the hydrological cycle (general water circulation in its various states from the seas to the atmosphere, to the ground, and back to the seas) occurring between precipitation (rainfall) over land areas and subsequent discharge through streams or through evapotranspiration. Therefore, to ideally explain the run-off cycle, we should focus our attention on the sequence of events associated with precipitation (from the atmosphere), surface run-off, infiltration, percolation, groundwater flow, evaporation and evapotranspiration (back to the atmosphere).

Assume a situation of prolonged drought during which most of the surface water from the previous rainy season is depleted (many streams may even cease to flow) and groundwater levels are so lowered that discharge into streams has become minimal or even ceased. This situation is common in many parts of Africa. Think of the events which follow the advent of abundant rainfall after such a long dry period.

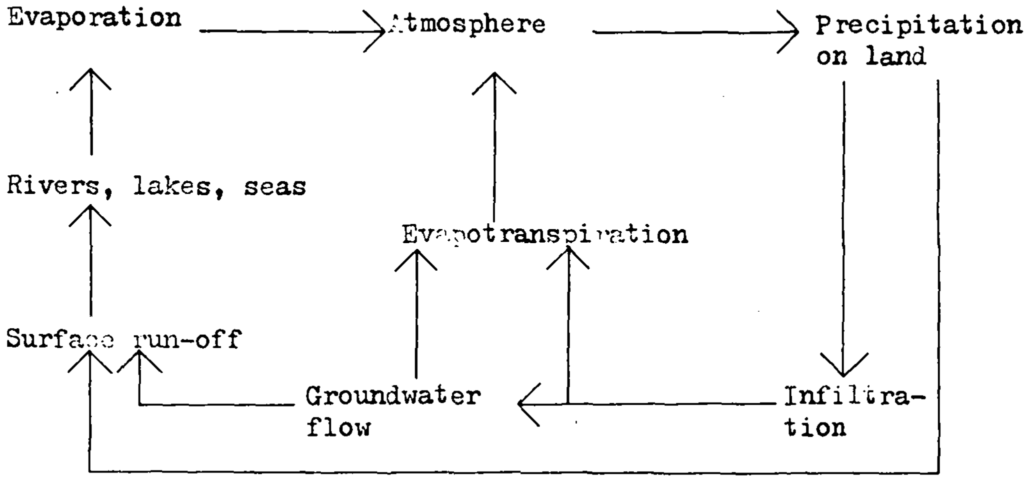
After the beginning of the rains, part of the rain falling on the land flows away on the surface (overland flow) while part of it enters the ground (infiltration). Some of the water which infiltrates into the ground percolates to the much lowered groundwater body (groundwater recharge). Infiltration is the movement of surface water into the ground and is partially controlled by the transmissive properties of surface materials (soils). Percolation is the movement of water toward a water table, essentially in the vertical direction and under the influence of gravity. The amount of water available for percolation in the sub-surface depends upon the infiltration capacity (maximum rate at which water enters the ground) of the surface material.

As pointed out earlier, groundwater movement may occur depending upon the permeability of the aquifer and may subsequently reach discharge points such as streams. This constitutes the groundwater component surface run-off.

Losses through evaporation of surface water and discharge (including evapotranspiration) from groundwater reservoirs are minimized by the fact that, during the rainy season, recharge exceeds discharge. This is particularly true if rainfall intensity is high. The result is increased stream flow and rise in groundwater levels.

To complete this idealized and somewhat oversimplified picture of the run-off cycle, let us now assume that the rains have ceased and much water is once more being lost to the atmosphere. Gradually a situation is reached when discharge exceeds recharge and remains so until a new cycle is once more initiated at the next rainy season.

Fig. 5 The hydrological cycle (the run-off cycle is part of the hydrological cycle)



5. Measuring groundwater levels

5.1 Groundwater table

Percolating water has to initially pass through the zone of aeration before it reaches the zone of saturation. The former is an unsaturated zone where the pores are partly filled with air. There is also water adhering to the grains (vadose water) and is commonly called soil moisture if it is in the soil layer near the surface. The latter zone is where all inter-connected voids are completely filled with water (groundwater) and where, in an observation well, water will begin to pond. Between these two zones is the groundwater table. However, immediately above the water table proper is a narrow zone of saturation (normally no more than 1.5-2.5m above the water table) where water does not begin to pond in an observation well, and is referred to as the capillary fringe.

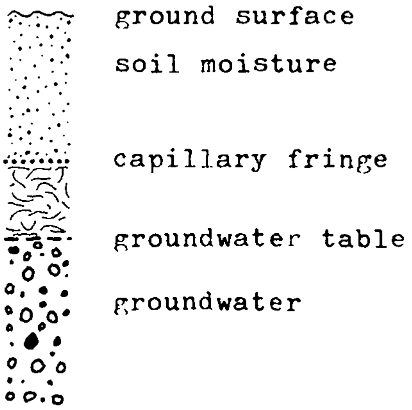


Fig. 6 Capillary fringe and the groundwater table

The shape of the groundwater table usually reflects the topography of the area. It reaches its highest elevation where the ground is highest. The intersection of the ground surface and the water table is an important hydrological boundary, called the spring line, because it represents the elevation below which water may discharge from the ground in spring.

Fig. 7 Groundwater table and the piezometric surface

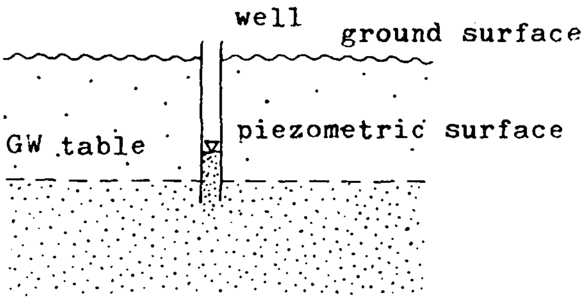
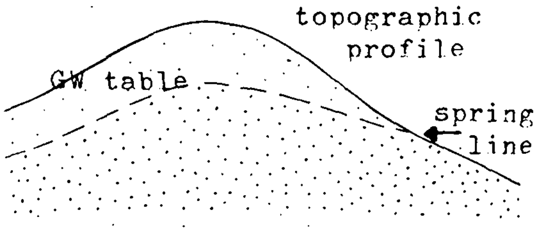


Fig. 8 The groundwater table conforms to topography



5.2 Shallow and deep groundwater table

Shallow groundwater is that occurring at a shallow level where it is directly influenced by meteorological and hydrological events influenced by meteorological and hydrological events occurring at the surface. It is recharged by precipitation and surface waters, and is recharged only through the shallow groundwater.

It is obviously easier and cheaper to exploit shallow groundwater, which is why consumption from this source is much greater than from deeper sources. The major advantage of exploiting deep groundwater is that it is less likely to be polluted than shallow groundwater.

5.3 Investigation of groundwater levels

Investigations of groundwater levels can be classified as surface and subsurface. Surface investigation can be accomplished by observing the level of discharge points such as springs or seepage zones, or by geophysical exploration techniques.

For groundwater prospecting, the most satisfactory geophysical techniques are perhaps the seismic and electrical methods. Under favourable conditions, seismic surveys may be interpreted directly as to the presence or absence of groundwater, as use is made of the principle that the velocity of an elastic wave is greater in moist sediments than in dry ones.

Electrical resistivity methods have been most frequently used in prospecting for water. The resisitivity of porous uncemented sediments is largely dependent upon the amount of water they contain, and the resistances show variations with depth, wherever water-bearing formations are present. In principle, porous uncemented sediments with a high water content show much lower resistance than those solid (consolidated) ones with low water content.

Subsurface methods have involved the use of observation wells or test boreholes. Piezometric levels are recorded and, with such data, water table and piezometric surface maps can be made.

5.4 Data from hydrogeological field investigations of groundwater levels

The data needed for eventual construction of a water-table map should consist of a tabulated list (water table data) of:

(i) well numbers (1, 2, 3 ...) (column 1)

(ii) altitude of land surface (metres above sea level) (column 2)

(iii) depth to water (metres) (column 3)

(iv) altitude of water table (metres above sea level) (column 4).

The data required for eventual construction of a piezometric surface map should consist of a tabulated list (records of wells) of:

(i) well numbers (1, 2, 3 ...) (column 1)

(ii) type of well (whether drilled or dug) (column 2)

(iii) altitude of land surface (metres above sea level) (column 3)

(iv) depth of well (metres) (column 4)

(v) depth of bedrock (metres) (column 5)

(vi) water level (metres) above or below land surface (column 6).

Hydrogeological field data on water levels also usually include well logs. These consist of descriptions of the character of geological materials (lithology, level at which water occurs, etc.) penetrated by digging or drilling. The log of each well (identified by number) is usually presented in tabular form and should at least consist of:

(i) a description of the well (column 1)

(ii) thickness of the statigraphical/lithological unit penetrated (metres) (column 2)

(iii) total depth of well (metres) (column 3).

6. Groundwater level fluctuations and hydrographs

6.1 Introduction

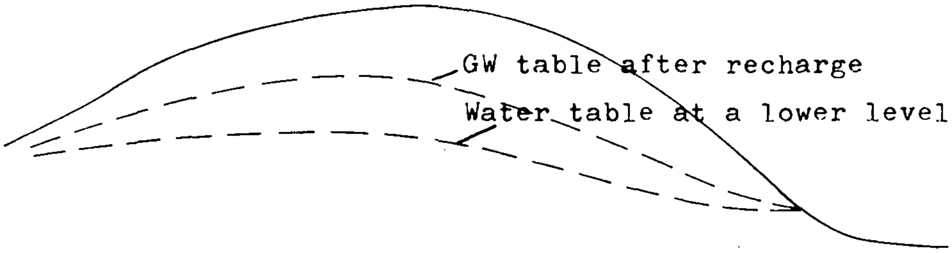
It is necessary to know (as in the case of a surface water reservoir) how the amount of water stored in a groundwater reservoir varies with time. The amount in storage varies in response to one or more natural or man-induced factors.

Changes in groundwater storage are determined by regular measurement of groundwater level (depth to water) in observation wells below a permanent reference point. The frequency of measurement depends upon the intended use of the data and the nature of the observed fluctuations. If a continuous record of the data is required, recording gauges should be employed; if it is not required, the measurements can be made (at regular intervals, say once a day or once a month for several years) manually with a graduated steel tape.

6.2 Groundwater recharge and discharge

The amount of groundwater in storage in an aquifer responds to both recharge and discharge from the aquifer. Storage increases and the water level rises when recharge exceeds discharge. But the water level declines, as water is drawn from storage, when discharge exceeds recharge.

Fig. 9 Groundwater fluctuations

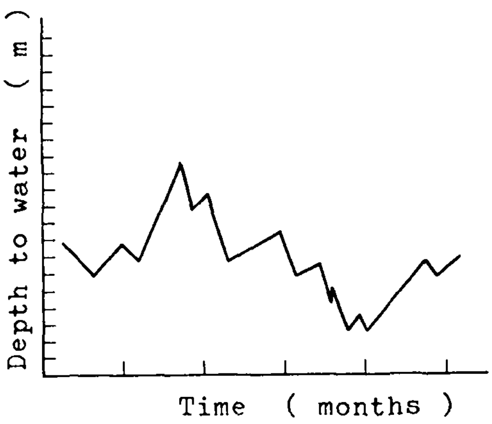


Recharge is intermittent, varying from year to year, season to season, and day to day since precipitation (rain or snow) is the only primary source of recharge under natural conditions. In contrast, discharge is continuous so long as the groundwater level in an aquifer is higher than the lowest level at which the aquifer discharges. Therefore, a graph of depth to water against time (hydrograph) shows short period rises (recharge) superimposed on a continuous recession curve.

6.3 Groundwater hydrograph

A groundwater hydrograph is a plot of recharge or discharge (shown by depth to groundwater level) against time. It shows the fluctuations of groundwater level in an aquifer. Usually, phases of recharge are shown to be short-term events.

Fig. 10 Groundwater hydrograph



The apparent shape of the graph depends upon the scales selected for both time and depth to water level. If these scales are carefully selected, a great deal of useful information can be derived from such a graph.

Although most hydrographs have been constructed on arithmetical graduated paper (e.g., discharge hydrographs of surface streams), groundwater hydrographs are best constructed on semi-logarithmic paper. The major advantage of log scales is that they tend to suppress extremes of recharge, thus making it possible to plot very high recharge or discharge rates within the space provided on a single sheet of paper. However, in a case where the actual magnitude of change in groundwater level is desired, arithmetical paper may be used.

6.4 Recession curve

The decline of groundwater levels in the absence of recharge depends upon the transmissibility of an aquifer (the capacity of an aquifer to transmit water, i.e., the product of the coefficient of permeability and the saturated thickness of the aquifer, or $T = K \times b$ where T is the transmissibility, K the permeability, and b the saturated thickness of the aquifer), the storage coefficient for an aquifer (water released from storage in an aquifer, or $V_W = V_R \times S$ where V_W is the volume of water, V_R the volume through which the change in water level occurred, and S the storage coefficient) and the hydraulic gradient.

After a period of recharge, the water level declines at first, then more gradually as time passes. The graphical representation of the decline is referred to as the natural recession curve. The recession curve therefore represents the withdrawal of water from storage after all recharge has ceased. The curve is usually a straight line. The identification of recession curves is important in studies of recharge and in predicting future groundwater levels, since every aquifer has a characteristic recession curve.

6.5 Long-term and short-term fluctuations

Long-term variations in storage extend over a period of several days to many years. It may be due to long-term variations in climate. Short-term variations, on the other hand, may last only a few hours, and may be due to one or more of several factors including pumping, earthquakes, etc. These short-term fluctuations in groundwater levels either reflect changes in the quantity of water in storage or changes in hydrostatic pressures.

7. Wells and well completion

7.1 Introduction

A well is a hole which is excavated in the earth in order to bring groundwater to the surface. A shallow well may be dug or bored but a deep one is drilled. After it has been drilled, a deep well must be completed and developed for optimum yield, and it must be tested before a pump is installed. It should be sealed in order to prevent contamination, and it should also be given occasional maintenance.

Wells are usually vertical but non-vertical ones may be constructed where special groundwater situations exist. A collector well, for instance, may be horizontal. Wells vary in size and range of complexity from very shallow ones dug with simple hand tools and yielding a few litres per minute to deep large-diameter drilled ones yielding several hundred litres per minute.

7.2 Test holes and well logs

It is common practice to put down a small-diameter test hole (commonly less than 20-25 cm in diameter) before a large-sized deep well is drilled in a little-known area. This is done in order to determine the depth to groundwater, the chemical quality of the water, and the thickness and other characteristics of the aquifer before the locality is selected as a site for a finished well. If the test hole appears suitable as a site for such a well, it can be reamed with hydraulic rotary equipment so as to convert it into a larger permanent well.

While a test hole is being drilled, a careful record or log of the various formations which are penetrated should be kept. The depths at which they are encountered should be recorded. Samples collected during drilling (with records of depths from which they are obtained) are always useful for laboratory study and analysis (for example, grain size distribution analysis).

7.3 Construction of shallow and deep wells

Shallow wells are generally less than 15m in depth (depth depends upon the depth of the water table), and are constructed by digging, boring, driving or jetting. Diameters range from less than one to several metres. Simple tools, for example pick and shovel, are required for digging wells. Water in dug wells is usually encountered at relatively shallow depths in unconsolidated deposits such as alluvium. To prevent caving, or for safety in general, wood lining or sheet piling should be placed in the hole to brace the walls. The wells should be permanently lined with wood, brick, rock, concrete or metal casing. The casing should be perforated to allow the entry of water into the well. A well-constructed dug well should be a few metres (say 3-7m) deeper than the water table.

Bored wells in unconsolidated aquifers can be fairly easily constructed with hand-operated or power-driven augers. These tools are available in several shapes and sizes and all operate with cutting blades which bore into the earth with rotary motion. Hand-bored wells rarely exceed 15-20cm in diameter and 15m in depth. Power-driven augers can bore holes up to 90cm in diameter and to depths exceeding 30m. Reaming can often enlarge diameters of bored wells to more than auger size. Augers perform best in materials which do not cave (e.g., clay); otherwise a casing (of concrete, metal or tile) should be put into the hole so that the boring may contimue inside.

A driven well consists of a series of connected lengths of pipe driven by repeated impacts into the ground to below the water table. During driving the bottom end is protected by a steel cone. Diameters of driven wells are small, most falling in the 3-10cm range, and depths are normally less than 15m. The best results are realized when the water table is within 3.5m of the ground surface - this provides adequate drawdown without exceeding the suction limit (suction-type pumps are used for extracting water from driven wells). Yields are generally small but sufficient for domestic purposes. Driven wells are limited to unconsolidated formations without coarse gravel or rocks which might damage the drive point. Driving can be done with a sledge hammer or any other tool that can be used to administer heavy blows which provide the required impact. The main advantage of driven wells is associated with their low construction costs.

Jetted wells are also suited to unconsolidated materials and are constructed by the cutting action of high-velocity streams of water. The down-directed jet-like stream washes the earth away while the casing conducts the water as it is lowered down into the deeping hole. Small-diameter holes (about 4-8cm) only can usually be produced in this way. They yield small supplies and are largely suited to exploratory purposes.

Most deep high yielding wells are constructed by drilling. Three basic methods are used, namely cable tool (percussion), hydraulic rotary, and reverse rotary. The choice of method depends upon the kind of rock formation into which the drilling is to be performed. It is advantageous to be able to switch methods, when necessary, as the drilling proceeds through stratigraphical units of varying character. These drilling methods will be dealt with in greater detail in a later lecture.

7.4 Well completion

After drilling, a deep well must be completed in order to facilitate the entry of water into the well without much resistance in and around the casing.

7.41 Perforations and screens

In unconsolidated formations, a casing is essential; it must allow water to flow into the well and also support the unstable material surrounding the well. The casing must either contain perforations or be replaced by a well screen. Machine perforated corrosion-resistant casings are available but, if necessary, the perforating can be done in the field before the casing is placed in the well. If screens are used instead, they should also be of corrosion-resistant metal; otherwise a plastic screen should be used.

Screens are particularly suited to sandy aquifers because a mesh screen can be selected to filter a specified fraction, thus leaving around the well a highly permeable coarse sand. A preliminary grain-size analysis of aquifer material would thus be useful.

7.42 Gravel packing

Gravel packing involves the installation of a screen or envelope of gravel around the perforated sections of well-casing. Besides increasing the well diameter, the gravel prevents fine material from entering the well and also protects the casing from caving of the surrounding materials.

A gravel pack is essential in most aquifers with a preponderance of the fine sand fraction (to avoid a sand-pumping well). Although the thickness of a gravel pack will depend upon the formation and method of drilling, a minimum of about 15cm is recommended. Hydraulic rotary or reverse rotary drilling is recommended for deep wells in sandy aquifers (where gravel packing is most essential). The gravel must be washed before it is packed into the well.

7.5 Maintenance and repair of wells

A well which is properly drilled, cased and developed can give long periods of satisfactory service with little or no attention. Many wells subsequently fail (yield decreasing quantities of water) because the pump deteriorates with time (therefore it should be checked before any repairs to the well are carried out) or the groundwater supply is depleted. To deal with this, the pump can be replaced or the well is deepened.

Another cause of well failure results from faulty well construction - poor casing, improper perforations or screens, incomplete gravel casing, poorly-seated wells, and so on. The well can usually be repaired in most cases but in case of collapse of casing or sudden failure due to sand entrance, replacement of the entire well is necessary.

Probably the most prevalent cause of well failure is corrosion. This is best dealt with by initially choosing screens of corrosion-resistant metal (e.g., copper, nickel or stainless steel).

7.6 Well development

After completion a new well should be developed to increase its specific capacity (a measure of the effectiveness of a well, obtained by dividing the discharge by the drawdown of the well), to prevent the accumulation of too much sand in the well and to prolong the life of the well. These objectives are achieved by removing finer materials from formations surrounding the well.

Well development may be carried out by pumping, which requires a pump with a suction pipe. After the water is pumped out, the well should be allowed to fill again to its normal piezometric level before the water is pumped out again. This non-continuous pumping procedure agitates the fine material surrounding the well so that it may be carried into the well and pumped out.

Surging, which is created by rapid up-and-down motion with a plunger, is probably a more effective way of developing a well in an unconsolidated aquifer. As the plunger moves up, it draws water into the well but when it moves down it forces the water back into the aquifer. This flow reversal procedure prevents sand "bridging" and brings fine material into the well. Surging may be performed with a circular surge block which is attached to a pipe through which water is pumped out of the well as it moves up and down. In this way mud is continuously removed from, instead of moving back into, the aquifer at each downward movement of the surge block. The surging should go on until there is no longer any apparent entrance of sediment (sand and mud) to the well.

Another method of developing wells is by using compressed air. To do this an air compressor is connected to an air pipe which is projected into the well. A discharge pipe is fitted around the air pipe. When the air pipe is closed, the air pressure is allowed to build up and is then suddenly released into the well by means of a quick-opening valve. The sudden impact of so much air creates a powerful surge in the well and results in water being forced up the discharge pipe. The process loosens up the fine material which may then be brought into the well through the perforations. The operation is repeated until most of the accumulating sand has been removed.

A fairly new method of well development is based on the surge produced when carbon dioxide (dry ice) is added to a well. The clay which may produce a "choking" effect of the aquifer (and thus reduce flow from the aquifer) must first be loosened; this is done by setting up pressure which forces a reagent (in this case hydrochloric acid which is poured into the well) into the aquifer. Blocks of dry ice are then dropped into the well. The resulting sublimation of the dry ice accumulates gaseous carbon dioxide which builds up pressure within the well and, upon release, causes a powerful thrust of muddy water from the well.

8. Pump tests and pump equipment

8.1 Pump tests for yield

A new well should be tested to determine its yield and drawdown (the drawdown of a well is the distance that the water level, or piezometric surface, is lowered after pumping). This should be done immediately after the new well has been developed. Such a test furnishes information on the quantity of water which can be drawn from the well, the type of pump which can best be installed, and the approximate cost of the pumping.

The test is accomplished by measuring the static water level and then pumping at a maximum rate until the water level in the well re-stabilizes (at a lower depth). The depth to water at the lower level is then measured. Depths to water level can be measured with a steel tape. Alternative methods involve the use of an electric sounding wire or an air line and pressure gauge.

8.2 Pump test data

The difference in depths to the water level (the static water level and the lower water level after pumping) represents the drawdown. An estimate of the specific capacity of the well, as noted before, can then be computed from the discharge/drawdown ratio. The discharge can be determined by an appropriate measuring device connected to the discharge pipe.

8.3 Pumping equipment

Where small discharges only are concerned, and this is usually the case in shallow wells, hand-operated pitcher pumps, turbine pumps or centrifugal pumps may be installed. Such discharges may range from 30 to 450 litres per minute; the discharge rate depends upon the type of pump, the amount of intake and the discharge pipe in use. For efficient and continuous operation, the suction lift should not be in excess of about 6-7.5m.

Large-capacity pumps like those serving industrial, urban and large agricultural (irrigation) requirements should be installed for pumping from deep wells requiring lifts which are greater than about 7.5m. For continued satisfactory yields from high-capacity deep wells, the selection of a proper pump is important. The factors which should be borne in mind include:

(i) diameter and depth of well,
(ii) depth to water level,
(iii) duration of pumping
(iv) well capacity,
(v) initial and maintenance costs,
(vi) required power and
(vii) the quality of the water.

The pump types which are suitable for deep-well operation include the plunger, deep-well turbine, displacement, air lift and jet types. The deep-well turbine pump has probably been the most widely adopted type for large deep wells yielding well flows of several hundred or thousand litres per minute. Manufacturers can usually supply advice as to the pump best suited for a particular well. This depends to a large extent upon the pump test data available.

8.4 Pump test records

The following list contains the sort of information which should be recorded either before, or during, or after a pump test in order to analyse the data:

(i) Construction features of the well - diameter, total depth, depths at which the well is open to the aquifer, and whether it is completed with a gravel pack.

(ii) Geological features of the aquifer - character and thickness of the aquifer, the character and thickness of beds overlying and underlying the aquifer, and the position and nature of hydrological boundaries.

(iii) Regional water-level trend before the test - both direction (up or down) of trend of water level in the aquifer and the rate of rise and decline (a test should not be started if it is expected that the trend will change during the test).

(iv) Amount of decline of water levels (drawdown) resulting from the pumping - including time of observation (the observer should use a stop watch) from the moment the pump picks up suction; the time of each water-level observation is thereafter noted with respect to the number of minutes and seconds since pumping started. The maximum interval between observations is generally about 1/10th the time elapsed since pumping began, e.g. as follows:

Elapsed time (minutes)	Interval between measurements
0	Pumping starts
1-10	30 seconds
10-20	1 minute
20-30	2 minutes
100-200	10 minutes
300-400	30 minutes
600 (end of test)	1 hour

(v) Field plot of data - drawdown versus time (in minutes) on logarithmic paper (to show how the test is going). If anamolous changes in water start near the planned end of the test, the test should be continued until the possible cause of the change can be determined.

(vi) Recovery of water level - normally observed only if the pumping (discharge) rate was irregular at the beginning of the test or if the natural trend of water level changed during the test. As with drawdown measurements, the observer should note the time of each observation accurately with respect to the time of pump cut-off. If complete recovery data are required, measurements should continue until water levels in the aquifer have returned to their natural regional trend.

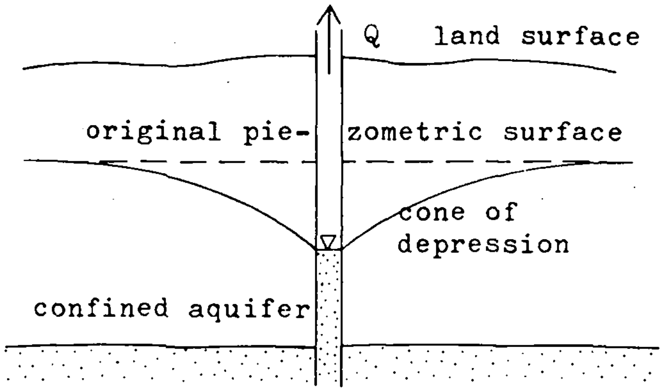
9. Measuring flow of wells

As we are now acquainted with Darcy's Law and some of the fundamental equations for groundwater movement, we can now apply them to particular situations, for example the flow of water to or from wells.

9.1 Steady radial flow to a well

Pumping from a well removes water from the aquifer surrounding it. As a result, the water table or piezometric surface in the vicinity of the well is lowered. We have already termed the distance by which the water level is lowered, at a given point, the drawdown (amount of decline in water level after pumping). A drawdown curve shows the variation of drawdown with distance away from the well. The 3-dimensional form of the drawdown shows an inverted conical shape (the cone of depression) whose apex is at the well. The outer limit of the cone of depression defines the area which is under the influence of the well.

Fig. 11 Cone of depression and the draw-down curve



9.2 Well flow data

The following list includes examples of the sort of data which should be recorded during field tests for subsequent use in well flow analysis:

(i) Topographic location of the well (since ground surface form may reflect the form or shape of the groundwater table).

(ii) Diameter and total depth of the well; also depth at which the well enters the aquifer.

(iii) Thickness, transmissibility, lithological and structural characteristics of the aquifer; also the nature of overlying and underlying aquicludes (in the case of confined aquifers).

(iv) Depth, trend and shape of the groundwater table prior to pumping; also the piezometric surface in the wall.

(v) Water levels after pumping; the drawdown and the radius of the radius of influence; the time of observation should also be recorded.

(vi) Pumping (discharge) rate including time when withdrawal of water from the aquifer started.

(vii) Field plot of drawdown versus time, possibly on semi-log paper (to show the progress of pumpage).

10. Measuring water levels for pump tests

Some of the techniques of water level measurement may have been mentioned in a preceding lecture on pump tests and equipment. The term water level is generally used to refer to groundwater tables and to piezometric surfaces observed in wells.

Changes in groundwater storage can be monitored by repeated measurement of the depth to water in observation wells. This should be done below a permanent reference point. The frequency of measurement depends upon:

(a) the use to which the data will be put
(b) the nature of fluctuations observed in water levels

Where continuous records are required, recording gauges should be used. Manual measurements are made where continuous records are not required; these may be made at regular intervals, for example one per day, one per month, one per year.

Manual measurements of depth to water level can be made in an observation well with a graduated steel tape. The data which should be recorded in the field at the test site should basically consist of the date (1st column) and the depth to water (2nd column) as demonstrated in the table below.

Date (1977)	Depth to water (m)
30 August	4.28
6 September	4.16
13 September	4.08
20 September	4.05
27 September	3.93

In the above example, the measurements would have been made at regular intervals on a weekly basis. A groundwater hydrograph can be constructed when such data is available. This could be done before a pumping test is carried out so that knowledge can be obtained as to whether the amount of water in storage will be increasing or decreasing (when storage increases, the water table or piezometric surface rises; when it decreases, the water level declines) at the intended time of the pump test. Such data is also useful for determining whether any rapid fluctuations occur at the well site due to natural causes. The hydrograph should clearly indicate the response of groundwater level to changes in storage. It is useful to have a reasonable amount of data on natural changes in storage (as reflected in changes in water level), over a period of time, before a major pumping test is carried out.

11. Hydrogeology of East Africa - a survey

East Africa is a region of considerable geological contrasts with rocks varying in age from Precambrian (500-600 million years ago) to Recent. These rocks also vary in chemistry from carbonates (limestones) and basic igneous volcanics to highly acid intrusives and metamorphics; hydrogeology, therefore, varies as well.

As in many other regions of Africa, igneous and metamorphic rocks are widespread in East Africa. These rocks are compact and impervious, which is why they are generally poor aquifers. They become pervious where they are deeply weathered, jointed or faulted, and may in such situations yield substantial amounts of water.

Sedimentary formations of East Africa are relatively extensive in the coastal zone and in northern Kenya. Elsewhere, for example in the Rift valleys, there are largely young unconsolidated or semi-consolidated sediments or extremely restricted occurrences. These sedimentary rocks, like those in the other regions of Africa and overseas, are potentially good aquifers because they are permeable and porous. However, there are local occurrences of impermeable clays or shales.

Groundwater in East Africa also varies in chemistry, and hence in quality, which is attributed to the variable geochemical character of containing rocks. The water is suitable for most purposes but excessively high values of salinity may occur in some of the sedimentary formations of coastal areas (of both Kenya and Tanzania) and of northern Kenya. This is due to the presence of intra-formational salts (for example, the chlorides, sulphates and carbonates which are freely disseminated throughout the Lower-Middle Duruma Sandstone aquifers of coastal Kenya) or to proximity to the sea where salt water may find its way into aquifers where it encroaches upon the fresh water at lower levels (in this case, as in the highly permeable and porous fossil coral reef formation of coastal Kenya, potable water can only be obtained from shallow wells).

One of the most significant hydrogeological discoveries of the last few years is that, in Kenya, the most productive aquifers of non-sedimentary terrains are buried river channels of proto-streams (ancient streams) which may by now be completely masked by considerable thicknesses of younger geological materials. Being confined and permeable, these linear sedimentary formations are potentially excellent aquifers. In certain localities of central Kenya, these subsurface channels have been found to be overlain by substantial thicknesses of ancient lava flows. Being restricted in lateral extent, these aquifers can be illusive during exploration, especially where they are covered by great thicknesses of other rocks. However, extensive use of appropriate geophysical equipment has proved to be the best answer to this problem and is recommended for trial in other regions of similar terrain.

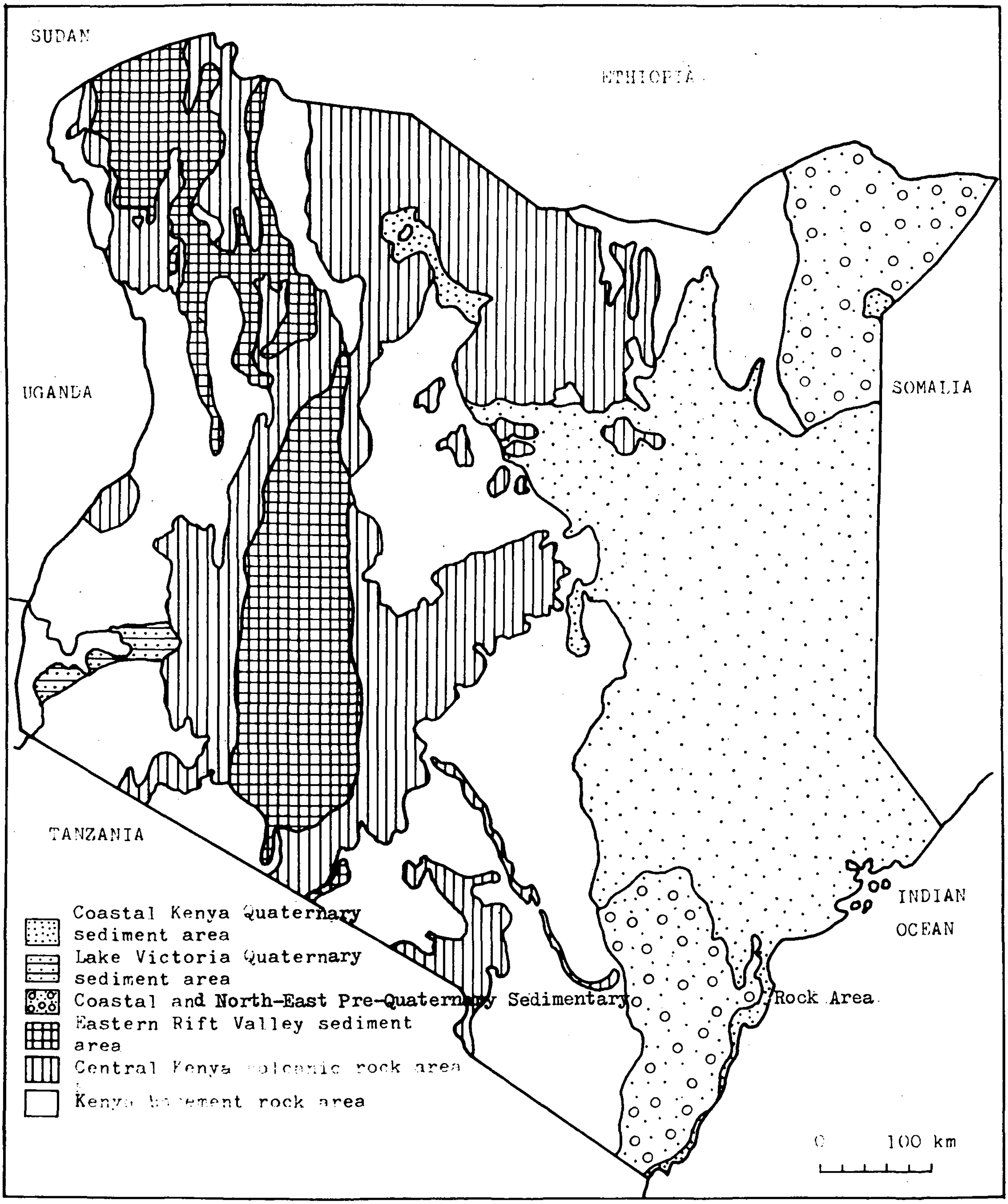
No attempt has yet been made to recognize an elaborate array of what may be regarded as the groundwater "provinces" of East Africa. This could be done on a geological, hence hydrogeological, basis. Applicable geological criteria should include geochemistry structure, mineralogy and lithology of the rocks. Hydrogeological criteria should include permeability and transmissibility and quality, all of which derive from the lithological, structural, geochemical and other characters of the rocks.

A suitable starting point would be to group the rocks of this region (on the basis of the above-mentioned criteria) as follows:

(a) The coastal Kenya-Tanzania Quaternary sediment area
(b) The coastal Kenya-Tanzania and northeast Kenya pre-Quaternary sedimentary rock area.
(c) The Eastern and Western Rift Valley sediment area.
(d) The Lake Victoria Basin sediment area.
(e) The Central Kenya-northern Tanzania volcanic rock area.
(f) The Kenya-Tanzania-Uganda Basement rock area.

The recognition of these 6 broad groups of rocks (which are of differing permeability and perviousness characteristics) should form a basis for the hydrogeological map of East Africa (fig. 12).

Fig. 14 Map of the rock groups of Kenya



G. DATA STORAGE AND ANALYSIS

1. General requirements

Hydrological data is collected to provide information for developing and managing the water resources of a catchment. It is also used for operating purposes: forecasting floods and low flows, prediction of runoff for operation of reservoirs and hydropower plants.

In order to achieve this, an adequate network, particularly the rainfall and streamflow networks, has to be established and observed over a long period. There is a close connection between network density and the principal purposes of the network.

The basis for the solution of water resources problems are readily accessible and reliable observations of the elements of the hydrological cycle. Hydrological elements of major interest are:

(a) precipitation
(b) water levels of streams and lakes
(c) streamflow
(d) evaporation and evapotranspiration
(e) groundwater and soil moisture

As an example the data storage and analysis of runoff is given below.

2. Recording of runoff

2.1 Recording of stage

A record of stage can be obtained by systematic observations on non-recording (manual recording) or recording gauges.

The non-recording gauge most commonly used is the vertical graduated staff gauge. The water levels are read and recorded by an observer, usually once or twice a day and during floods even more. The advantages of this type of gauge are the low initial cost and the easy installation. The disadvantages are the need for a reliable observer and the lack of accuracy of the estimated continuous stage graph sketched through the points of observation.

With a recording gauge the stage is automatically sensed and recorded by a float or a gas purge system (bubble gauge). The accuracy of recording is higher than the readings of a manual recording stage ($\pm$0.2 cm). A continuous recording is obtained either by a continuous line on graph paper (strip chart or continuous paper) or by a sequence of punches (digital-tape-water-level recorder). Instead of paper tape, magnetic tape is coming into operational use.

2.2 Recording of streamflow

Rating curves. If flow measuring methods utilize artificial control sections of such shape that head-discharge relationships can be determined from measured water levels (weirs, flumes), then discharge measurement are not necessary.

In natural rivers stream flow measurements can only be carried out occasionally. The relation between discharge and stage cannot be calculated by discharge formulas. The continuous record of discharge must be derived by converting the continuous record of stages by means of a rating curve. As this relationship between discharge and stage may vary with time and stage, the effect on the conversion from water level to discharge has to be considered. The end product of the computation is a tabulation of the mean discharge for each day, month and year. The average discharge for each day or shorter time intervals is computed from the average stage and the rating curve (see fig. 1 and Table I).

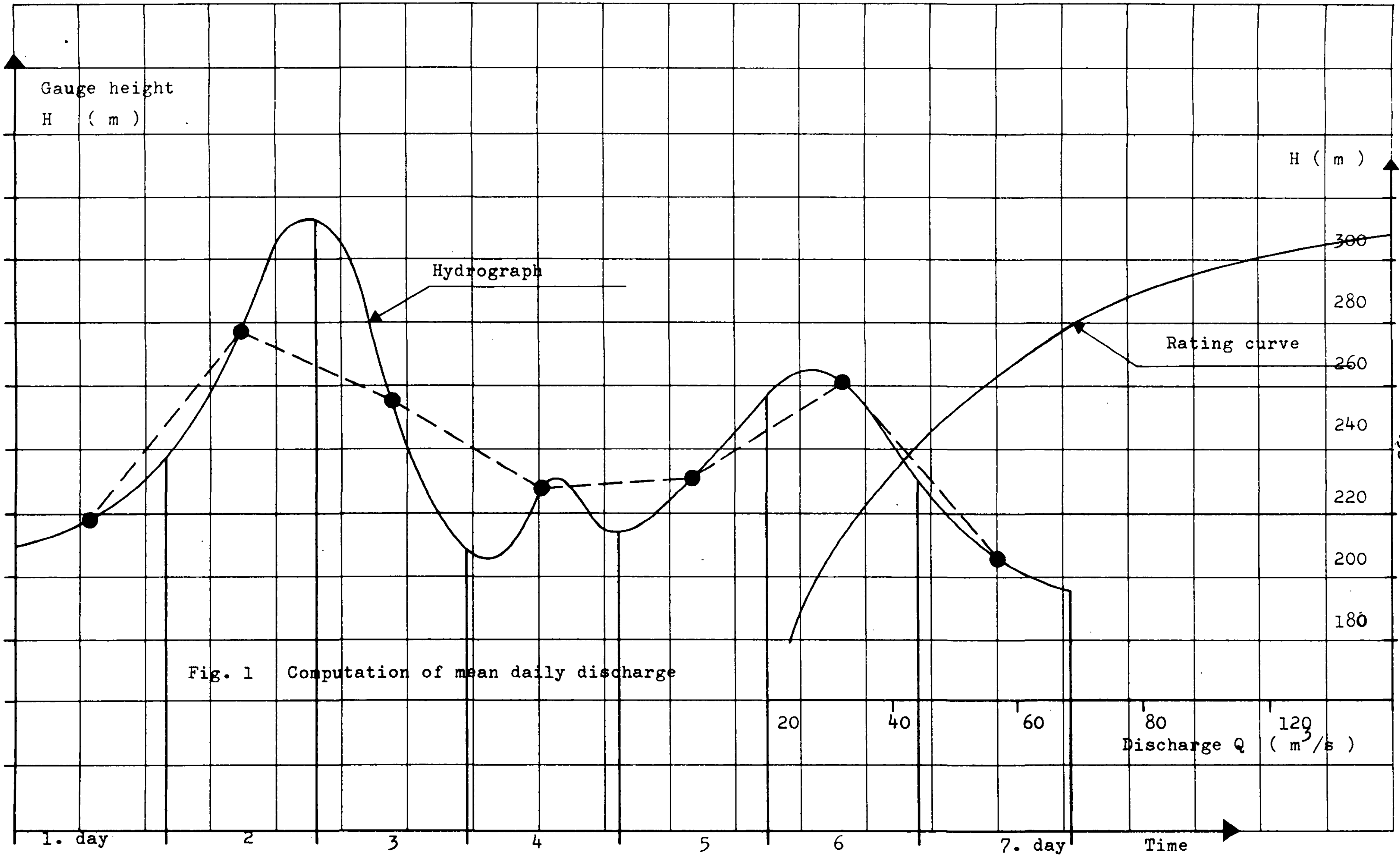


Fig. 1 Computation of mean daily discharge

Table I: Computation of mean daily discharge and cumulative runoff

1st day		2nd day		3rd day		4th day		5th day		6th day		7th day	
time	discharge	time	discharge	time	discharge	time	discharge	time	discharge	time	discharge	time	discharge
0^{00}	31 m^3/s	0	43	0	124	0	30	0	32	0	52	0	39
6^{00}	32 "	2	52	2	120	4	29	6	35	4	58	6	35
12^{00}	35 "	12	65	4	108	8	32	12	40	8	59	12	30
18^{00}	36 "	18	108	6	82	12	38	18	45	10	59	18	28
24^{00}	43 "	20	120	12	54	14	39	24	52	14	54	24	27
		22	128	18	37	16	38			20	45		
		24	124	24	30	18	37			24	39		
						22	33						
						24	32						

Computation of mean daily discharge:

1st day : $\bar{Q}_1 = \frac{1}{24}\left[\left(\frac{31+32}{2}\right)\cdot 6 + \left(\frac{32+35}{2}\right)\cdot 6 + \left(\frac{35+36}{2}\right)\cdot 6 + \left(\frac{36+43}{2}\right)\cdot 6\right] = \underline{35.4\ m^3/s}$

2nd day: $\bar{Q}_2 = \frac{1}{24}\left[\left(\frac{43+52}{2}\right)\cdot 6 + \left(\frac{52+65}{2}\right)\cdot 6 + \left(\frac{65+108}{2}\right)\cdot 6 + \left(\frac{108+120}{2}\right)\cdot 2 + \left(\frac{120+128}{2}\right)\cdot 2 + \left(\frac{128+124}{2}\right)\cdot 2\right] = \underline{78.5\ m^3/s}$

3rd day: $\bar{Q}_3 = \frac{1}{24}\left[\left(\frac{124+120}{2}\right)\cdot 2 + \left(\frac{120+108}{2}\right)\cdot 2 + \left(\frac{108+82}{2}\right)\cdot 2 + \left(\frac{82+54}{2}\right)\cdot 6 + \left(\frac{54+37}{2}\right)\cdot 6 + \left(\frac{37+30}{2}\right)\cdot 6\right] = \underline{64.3\ m^3/s}$

4th day: $\bar{Q}_4 = \frac{1}{24}\left[\left(\frac{30+29}{2}\right)\cdot 4 + \left(\frac{29+32}{2}\right)\cdot 4 + \left(\frac{32+38}{2}\right)\cdot 4 + \left(\frac{38+39}{2}\right)\cdot 2 + \left(\frac{39+38}{2}\right)\cdot 2\right.$
$\left. + \left(\frac{38+37}{2}\right)\cdot 2 + \left(\frac{37+33}{2}\right)\cdot 4 + \left(\frac{33+32}{2}\right)\cdot 2\right] = \underline{34.3\ m^3/s}$

5th day: $\bar{Q}_5 = 40.8\ m^3/s$; 6th day: $\bar{Q}_6 = 52.8\ m^3/s$; 7th day: $\bar{Q}_7 = 31.8\ m^3/s$.

Computation of cumulative weekly runoff:

1. Using mean daily discharges: $\sum Q.\Delta t = (35.4 + 78.5 + 64.3 + 34.3 + 40.8 + 52.8 + 31.8).86400$
$= 29.195$ Mio m^3 (= 100%)
2. Using 12^{00} observations: $\sum Q.\Delta t = (34 + 68 + 50 + 38 + 40 + 55 + 30).86400 = 27.216$ Mio m^3 (= 93%)
3. Using mean daily gauge height: $\sum Q.\Delta t = (35 + 64 + 52 + 34 + 41 + 52 + 30).86400 = 26.611$ Mio m^3 (= 91%)

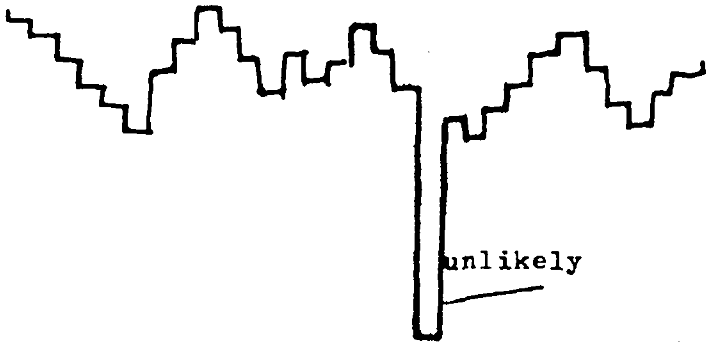
3. Processing of field data and storage

3.1 Quality control

It is desirable to subject the original observational data to a simple analysis in order to put the data in a form most useful to the consumers. Before this step can be taken, it has to be ensured that good quality data emerge from the observations. The main steps of quality control of the data include inspections of the stations within certain time intervals, preliminary checking of data and detection of errors by internal consistency checks.

An inspection of the station should be combined with every streamflow measurement. A preliminary check of the data should ensure its completeness and accuracy. By checks of internal consistency it can be proved if there are errors due to instrumentation (see fig. 2). By comparing daily observations of gauges along a river or of adjacent catchments an error also might be detected. The correction should be made legibly on the original form.

(a) Manual gauge



(b) Recording gauge

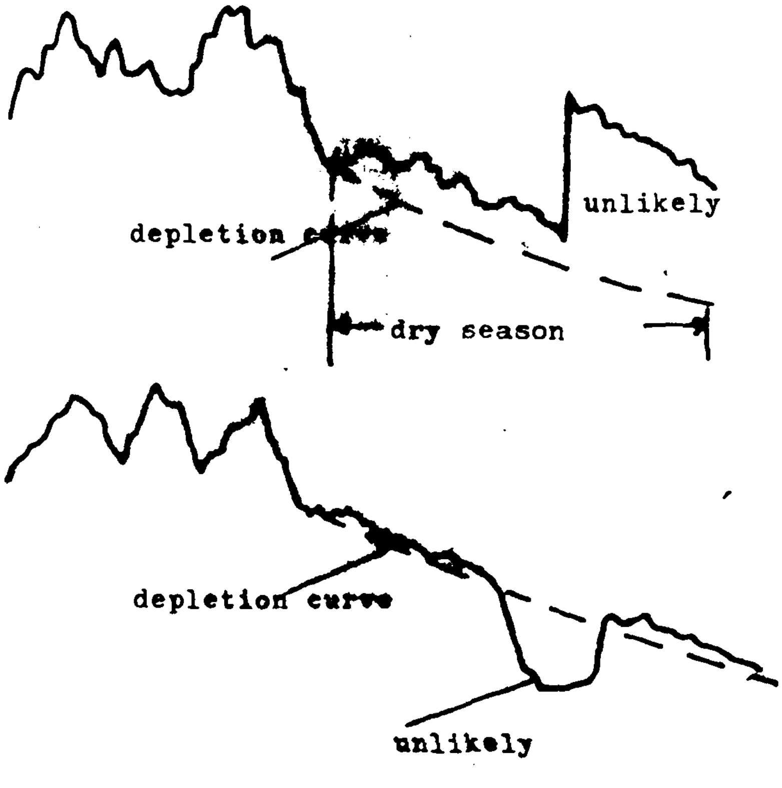


Fig. 2 Possible errors in gauge records

3.2 Processing and storage

The data must be processed in such a way that publication according to international standards is possible. The original records are stored in lists (or microcopies). The data should be readily available for various studies (e.g. flood peaks). For water balance calculations the volume of monthly and annual runoff is needed. For routine publication of streamflow it may be desirable to maintain mean values of streamflow and of selected floods for determining trends.

4. Elementary analysis of hydrological data

4.1 Accuracy of computation

The accuracy of computations must correspond to the accuracy of observations. In the following table the accuracy of computation is indicated:

	Element	Accuracy
	Time	1 sec
Depending on stream size	Water depth	1, 10, 20 cm
	Water stage	1 cm
	River width	0.1; 0.5; 1 m
	Stream length	0.1; 1 km
	Slope	0.1 per cent
	Velocity of flow	0.01 cm/sec
	Long term average runoff runoff	Two- or three-figure numerals
	Runoff depth	1 mm
	Precipitation depth	1 mm
	Runoff coefficient	Two-figure numeral
	Evaporation	1 mm
	Temperature	0.1°C

For computations the individual hydrological observations have to be converted into the same dimensions. The following table gives some conversion factors:

Table II: Conversion factors for time and runoff

Time

1 day (d)			=	24 h	=	86400 sec
1 month	=	30 d	=	720 h	=	2592000 sec
1 month	=	31 d	=	744 h	=	2678000 sec
February	=	28 d	=	672 h	=	2419000 sec
"	=	29 d	=	696 h	=	2506000 sec
1 year (a)	=	365 d	=	8760 h	=	31536000 sec
1 leap year	=	366 d	=	8784 h	=	31622000 sec

Runoff

Unit	litre/sec	m^3/sec	cfs	acre-feet per day	Imperial gallons per hour
litre/sec	1	0.001	0.0353175		791.9
m^3/sec	1000	1	35.3157	70.045	791900
cfs	28.317	0.0283	1	1.9835	22423.8
acre-feet per day	14.276	0.014276	0.00417	1	11305
Imperial gallons per hour	0.0012				1

4.2 Elementary analysis of runoff data

4.21 Mean and flow-mass-curve

The hydrological data is normally arranged in a chronological order (time series). A time series can be characterized by statistical parameters; the simplest parameter is the arithmetic mean.

The variation of the runoff during a month or a year is best expressed by the deviation of long-term monthly or yearly runoff (mean runoff) from the actual runoff. This can be shown by a flow-mass-curve, a graph of the cumulative values of the daily or monthly discharge, generally as ordinate plotted against time as abscissa (see Table III and fig. 3). The flow-mass-curve is also known as the Ripple method. The flow-mass-curve has many useful applications in the design of a storage reservoir (e.g., the determination of storage capacity).

Table III: Computation of average discharge, cumulative runoff, and runoff coefficient

Month	Mean monthly discharge (m^3/sec)	Monthly runoff (Mio m^3)	Cumulative runoff (Mio m^3)	Cumulative mean runoff (Mio m^3)	Difference of cumulative runoff (Mio m^3)	Remarks
O	40	107	107	546	- 439	
N	35	91	198	1075	- 877	
D	60	161	359	1621	- 1262	
J	114	305	664	2167	- 1503	
F	197	477	1141	2660	- $\boxed{1519}$	Deficit
M	342	915	2056	3205	- 1149	
A	367	951	3007	3733	- 726	
M	405	1085	4092	4280	- 188	
J	393	1019	5111	4808	+ 303	
J	302	808	5919	5354	+ $\boxed{565}$	Surplus
A	140	375	6294	5900	+ 394	
S	52	135	6429	6429	0	
Total		6429				

Average annual discharge: $\frac{6429 \times 10^6}{365.86400} = 203.9\ m^3/s$

Depth of runoff: $\frac{6429 \times 10^6 . 10^3}{150965} = 42.6 mm$

Runoff coefficient: $\frac{42.6}{949} = 0.045$

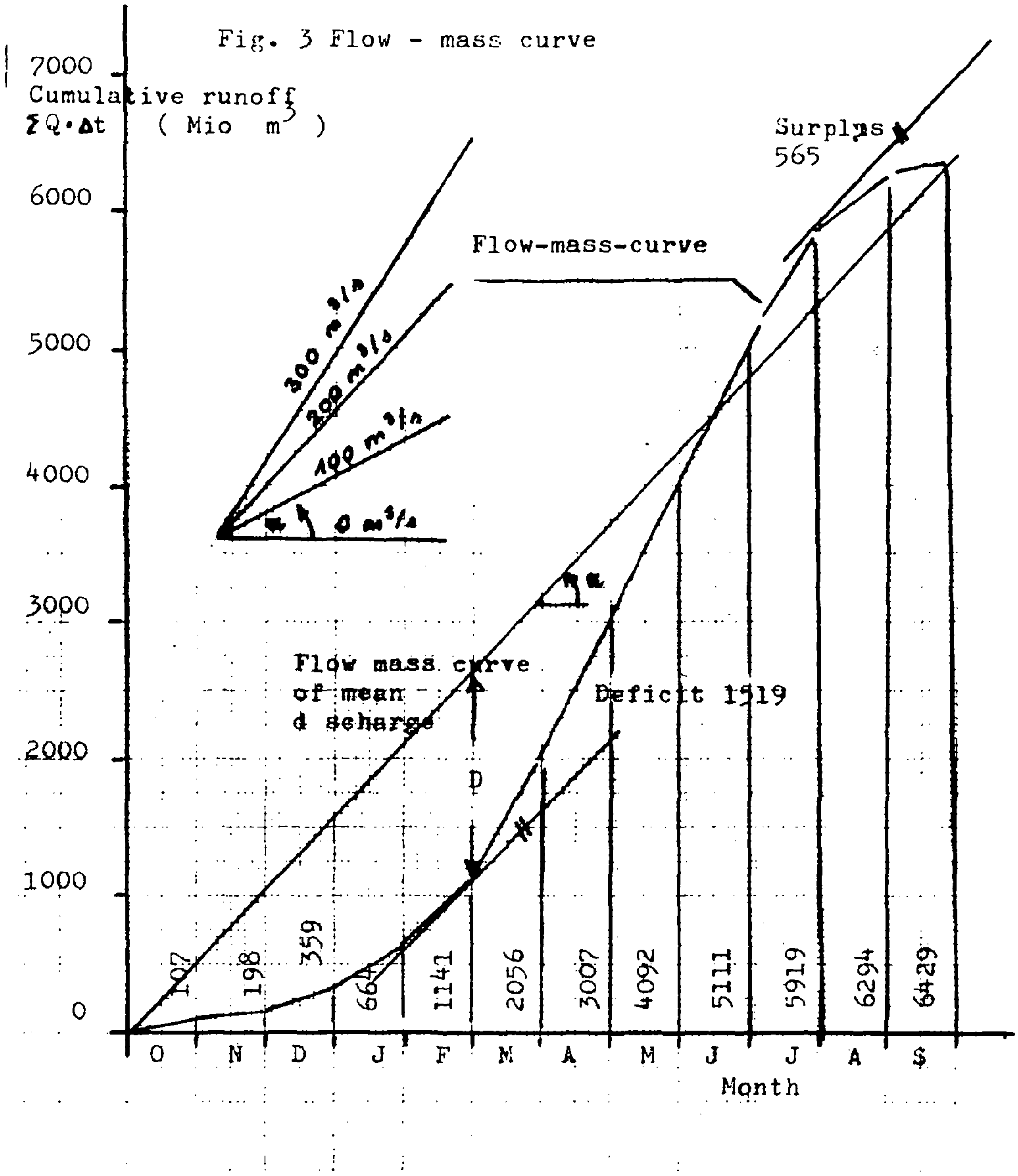


Fig. 3 Flow - mass curve

The difference between the ordinate of any two points on the mass-curve is the summation of the flows during the intervening period of time. Thus, if the two points are connected by a straight line, the slope of the line will be equal the average flow during that period (e.g., the connection of the beginning and end of the mass curve for a hydrological year gives the annual average discharge). If a tangent is drawn to the mass-curve at the lowest tangent point and the highest tangent point, the vertical intercept between these two straight lines represents the storage volume required to permit a continuous release of water for the entire period (see fig. 4). The intercept between the average discharge and the lowest tangent point is known as deficit, the intercept between the average discharge and the highest tangent point as surplus.

4.22 Duration curve

The data of a time series represent the observed sample of the entire data of the continuous runoff process (population). While the period of time of the runoff may be the same (daily or monthly discharges), each observation is characterized by a certain variable, which varies with time. If the value of one variable, the variate, is independent of any other, the variable is a random variable.

For frequency calculations the daily values are arranged according to the magnitude of the variables. The amplitude of this array is the difference between the maximum and minimum variable. This amplitude may be divided into intervals and the values of the variables distributed in these intervals. The number of occurrences in each of the intervals is called frequency n. When divided by the total number of the variates n in the series, it becomes the relative frequency f in per cent. If plotted in arithmetic coordinates a frequency distribution bar hydrograph (histogram) is obtained.

It is usually necessary to ascertain how many occurrences of the variate have a higher (or lower) value than a certain given value. This information may be derived from the cumulative frequency, which is obtained by summation of the occurrences in each time interval. When plotted in a graph, a duration curve is obtained if the period of the observation is one month or one year (see fig. 3(b) and fig. 8).

Flow duration curves of daily discharge show the percentage of time that the river flow exceeds a given value. These curves are commonly used in computing hydro-electric power potential.

Example (Computation of a monthly flow duration curve)

Fig. 4 Example : Computation of a monthly flow duration curve

Given : mean daily discharges
for one month

days	discharge	days	discharge
1	102	19	58
2	105	20	56
3	107	21	58
4	103	22	52
5	79	23	54
6	78	24	55
7	78	25	54
8	74	26	52
9	70	27	53
10	65	28	58
11	65	29	56
12	[illegible]	30	[illegible]
13	[illegible]		
14	62		
15	[illegible]		
16	[illegible]		
17	[illegible]		
18	[illegible]		

Total : 2027

Mean : 2027/30

$\bar{x}$ = 68 m^3/s ; Median : 62 m^3/s

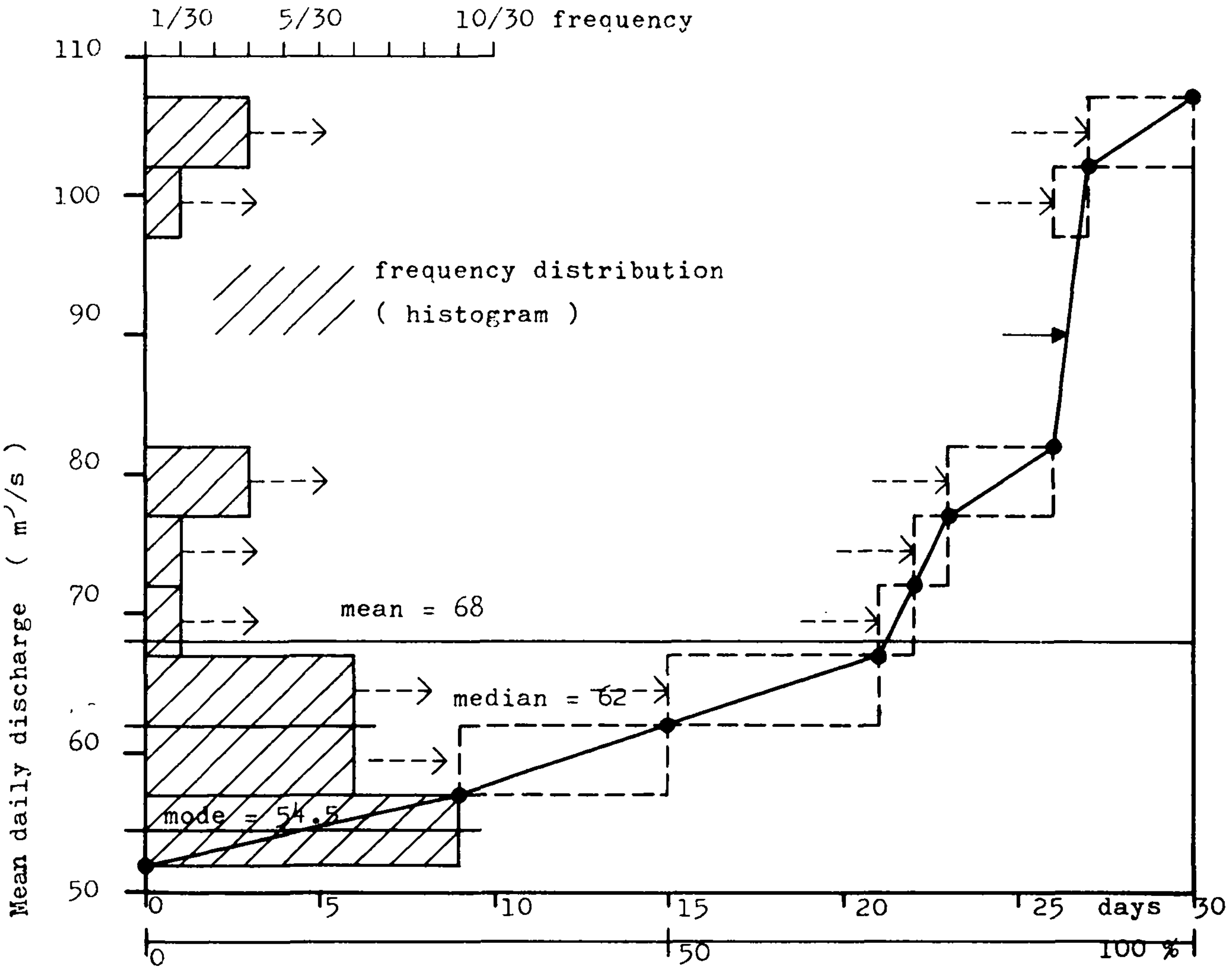
Mode : $\frac{57 + 52}{2}$ = 54.5 m^3/s

Computation of cumulative frequency

Amplitude : 107 - 52 = 55 m^3/s

Selected 11 intervals 5 m^3/s each

Interval	Frequency		cum. Frequency
107 - 102		3/30	3/30
102 - 97		1/30	4/30
97 - 92			4/30
92 - 87			4/30
87 - 82			4/30
82 - 77		3/30	7/30
77 - 72		1/30	8/30
72 - 67		1/30	9/30
67 - 62		6/30	15/30
62 - 57		6/30	21/30
57 - 52		9/30	30/30



4.23 Flood frequencies

4.231 Theoretical frequency curves

The probability of occurrence of rare hydrological events which have not been observed can be predicted by methods of frequency analysis. For flood peaks and flood volumes special methods of computing are used. Generally the frequency distributions ascertained by the use of histograms or probability plotting cover a prediction period not much longer than the period of observation in the past. However, by graphical extrapolation on special graph paper (probability paper) or by substituting the observed frequency distribution by a theoretical frequency distribution, a prediction of a flood with a given recurrence interval is possible. By computation the statistical parameters from the observed series, i.e. flood with a recurrence interval of T_R years, is calculated.

$$x_{T_R} = \bar{x} + k \cdot \sigma$$

where x_{T_R} = Floodpeak (m^3/s) with a recurrence interval of T years

T_R = Recurrence interval (years) $T = \frac{100 - p}{p}$

with p = probability (%)

$\bar{x}$ = Arithmetic mean (m^3/s) $= \frac{1}{n} \Sigma x$

n = size of sample

x = observed floodpeaks

σ = Standard deviation (m^3/s)

$$\sigma = \sqrt{\frac{1}{n-1} (x - \bar{x})^2} \quad \text{or}$$

$$\sigma = \sqrt{\frac{1}{n-1} (\Sigma (x)^2 - \bar{x} \Sigma x)}$$

k = Frequency factor, depending on the frequency distribution used.

C_s = Coefficient of skewness $= \frac{n \Sigma (x - \bar{x})^3}{(n-1)(n-2)\sigma^3}$

Frequency distributions which are often used are

(a) log-Normal distribution.
(b) Gumbel extreme value distribution
(c) Pearson Type III distribution.

For each of these distributions, a probability paper exists. The object of using this paper is to linearize the frequency distribution, so that a theoretical probability curve appears on the paper as a straight line. The plotting positions are commonly determined by the formula

$$T_R = \frac{n + 1}{R}$$

where T_R = recurrence interval
n = number of years of records
R = rank of the peak discharge ($R = 1$ for the highest and $R = n$ for the lowest value)

4.232 Examples of computation of flood frequencies

For the computation of flood frequencies several frequency distributions were proposed. The theoretical distribution function, which fits best to the plotted empirical distribution, can be selected by a statistical test or by inspection.

Distribution functions with two statistical parameters (mean and standard deviation) as Gumbel's distribution and log Normal distributions are less adaptable than functions using three parameters (mean, standard deviation, coefficient of skewness) as the Pearson Type III distribution.

For the plotting of the empirical frequency special graph paper is used. In order to display the theoretical distribution function as a straight line, the following probability papers are used:

Distribution function	Scale of abscissa	Scale of ordinate
Normal	Normal distribution	arithmetic
log Normal	Normal "	logarithmic
Gumbel	Gumbel's "	arithmetic
Pearson Type III	Normal "	log or arithmetic

For the computation of the theoretical frequency distribution, x_{T_R} for several given values of T_R must be determined by substituting the values in the equation $x_{T_R} = \bar{x} + k \cdot \sigma$. The parameters $\bar{x}$ and σ are calculated by means of the sample. Theoretical and empirical frequency distribution are plotted on the respective graph paper.

Using Gumbel's method, k is defined as

$$k = \frac{y_{T_R} - \bar{y}_n}{\sigma_n}$$

where $\bar{y}_n$, σ_n : values depending on sample size (Table V)

y_{T_R} : reduced variate depending on return period (Table IV)

The value k of log Normal distribution is only depending on the recurrence interval. It is identical with the value k of Pearson Type III distribution for $C_s = 0$ (see Table VII).

Example 1 (Application of Gumbel and log Normal distributions for computations of flood frequencies)

For a river (Kafue River at Kasaka, Zambia) the flood peaks of 24 years (1942-1966) are observed. The 100-years flood is calculated using Gumbel's distribution and compared with log Normal distribution (see Table VI, fig. 5 and fig. 6).

Example 2 (Application of Pearson Type III distribution for computation of flood frequencies)

Pearson Type III is most commonly used for flood frequency studies. Pearson Type III distribution has three statistical parameters: mean, standard deviation and coefficient of skewness. If the coefficient of skewness is zero, the Pearson Type III distribution and the Normal distribution are identical. Arithmetic or logarithmic values can be used, depending on the coefficient of skewness. A recommendation of selecting the proper kind of Pearson distribution is given by the flow chart in fig. 8.

On the bases of an observation period of 10 years, a flood of a recurrence interval of 50 years should be calculated according to Pearson Type III distribution. As the observation period is relatively short, a partial series should be used instead of annual series (see Table VIII and fig. 7).

A limitation of annual series data is that each year is represented by only one event. The second highest event (flood) in a particular year may be higher than the highest in some other year, yet would not be counted. A series that lists all events above a particular base is known as partial series. For return periods exceeding 10 years, the distribution between the annual and partial series is inconsequental. Annual series can be computed to partial series using the following table:

Corresponding return periods for annual and partial series

Partial series	Annual series
0.50	1.16
1.00	1.58
1.45	2.00
2.00	2.54
5.00	5.52
10.00	10.50

The recurrence interval of partial series T_p is related to the recurrence interval of annual series T_R by the equation

$$T_p = \frac{1}{\ln T_R - \ln (T_R - 1)}$$

Table IV: Recurrence interval as function of the reduced variate $Y_{T_R} = -\left(0.834 + 2.303 \log \log \frac{T_R}{T_R - 1}\right)$

Return period T_R years	Probability of exceedance (%)	Reduced Y_{T_R}
1.01	99	- 1.73
1.05	95	- 1.12
1.11	90	- 0.83
1.25	80	- 0.48
2	50	0.37
5	20	1.50
10	10	2.25
25	4	3.20
50	2	3.90
100	1	4.60
200	0.5	5.34
500	0.2	6.22
1000	0.1	6.92

Table V: Mean and standard deviation of the reduced variate

N	$\bar{y}_N$	S_N	N	$\bar{y}_N$	S_N
8	0.4843	0.9043	49	0.5481	1.1590
9	0.4902	0.9288	50	0.54854	1.16066
10	0.4952	0.9497	51	0.5489	1.1623
11	0.4996	0.9676	52	0.5493	1.1638
12	0.5035	0.9833	53	0.5497	1.1653
13	0.5070	0.9972	54	0.5501	1.1667
14	0.5100	1.0095	55	0.5504	1.1681
15	0.5128	1.02057	56	0.5508	1.1696
16	0.5157	1.0316	57	0.5511	1.1708
17	0.5181	1.0411	58	0.5515	1.1721
18	0.5202	1.0493	59	0.5518	1.1734
19	0.5220	1.0566	60	0.55208	1.17467
20	0.52355	1.06283	62	0.5527	1.1770
21	0.5252	1.0696	64	0.5533	1.1793
22	0.5268	1.0754	66	0.5538	1.1814
23	0.5283	1.0811	68	0.5543	1.1834
24	0.5296	1.0864	70	0.55477	1.18536
25	0.53086	1.09145	72	0.5552	1.1873
26	0.5320	1.0961	74	0.5557	1.1890
27	0.5332	1.1004	76	0.5561	1.1906
28	0.5343	1.1047	78	0.5565	1.1923
29	0.5353	1.1086	80	0.55688	1.19382
30	0.53622	1.11238	82	0.5572	1.1953
31	0.5371	1.1159	84	0.5576	1.1967
32	0.5380	1.1193	86	0.5580	1.1980
33	0.5388	1.1226	88	0.5583	1.1994
34	0.5396	1.1255	90	0.55860	1.20073
35	0.54034	1.12847	92	0.5589	1.2020
36	0.5410	1.1313	94	0.5592	1.2032
37	0.5418	1.1339	96	0.5595	1.2044
38	0.5424	1.1363	98	0.5598	1.2055
39	0.5430	1.1388	100	0.56002	1.20649
40	0.54362	1.14132	150	0.56461	1.22534
41	0.5442	1.1436	200	0.56715	1.23598
42	0.5448	1.1458	250	0.56878	1.24294
43	0.5453	1.1480	300	0.56993	1.24786
44	0.5458	1.1499	400	0.57144	1.25450
45	0.54630	1.15185	500	0.57240	1.25880
46	0.5468	1.1538	750	0.57377	1.26506
47	0.5473	1.1557	1000	0.57450	1.26851
48	0.5477	1.1574	•	•	•
			•	•	•
			∞	0.57722	1.28255

Period	Annual Flood x (m^3/s)	log x	rank 2	Probability $1 - \frac{r}{n+1}$	Recurrence Interval T_r (years)	$(x - \bar{x})^2$	$(\log x - \log \bar{x})^2$
1962/63	1971	3.29469	24	96.0%	25	1669264	0.26427
1951/52	1729	3.23779	23	92.0	12.5	1102500	0.20900
1957/58	1338	3.12646	22	88.0	8.3	434281	0.11961
1947/48	1311	3.11760	21	84.0	6.3	399424	0.11356
1961/62	865	2.93702	20	80.0	5.0	34596	0.02446
1952/53	742	2.87040	19	76.0	4.2	3969	0.00806
1943/44	695	2.84198	18	72.0	3.6	256	0.00377
1955/56	684	2.83506	17	68.0	3.1	25	0.00296
1942/43	656	2.81690	16	64.0	2.8	529	0.00132
1956/57	653	2.81491	15	60.0	2.5	676	0.00118
1960/61	653	2.81491	14	56.0	2.3	676	0.00118
1949/50	617	2.79029	13	52.0	2.1	3844	0.00009
1955/56	580	2.76343	12	48.0	1.9	9801	0.00030
1946/47	547	2.73799	11	44.0	1.9	17424	0.00182
1944/45	534	2.72754	10	40.0	1.7	21025	0.00217
1945/46	476	2.67761	9	36.0	1.6	41209	0.01061
1963/64	452	2.65514	8	32.0	1.5	51529	0.01574
1958/59	423	2.62634	7	28.0	1.4	65536	0.02380
1965/66	416	2.61909	6	24.0	1.3	69169	0.02601
1953/54	413	2.61595	5	20.0	1.25	70756	0.02712
1964/65	351	2.54530	4	16.0	1.19	107584	0.05537
1950/51	337	2.52762	3	12.0	1.14	116984	0.06401
1959/60	334	2.52375	2	8.0	1.09	119025	0.06598
1948/49	166	2.22011	1	4.0	1.04	263169	0.31418
Total	16943	66.73483				4603231	1.35659

arithmetic mean $\bar{x} = \frac{16943}{24} = 706$ m^3/sec

standard deviation $\sigma_x = \sqrt{\frac{1}{24-1}(4603231)} = 447$ m^3/sec

100 year's flood after Gumbel:

$y_n = 0.5296$; $_n = 1.0864$; $y_{Tr} = 4.60$

$$x_{100} = 706 + \frac{460 - 0.5296}{1.0864} \cdot 447 = \underline{\underline{2370\ m^3/sec}}$$

mean of log x = $\bar{z} = \frac{66.73483}{24} = 2.78062$ $\bar{x}_{geom} = 603$ m^3/sec
(geometric mean)

standard deviation of log x $z = \sqrt{\frac{1}{24-1}(1.35659)} = 0.24286$

100 year's flood after log normal distribution

$\log x_{100} = 2.78062 + 2.326 \cdot 0.24286 = 3.34552$

$$x_{100} = 10^{3.34552} = \underline{\underline{2216\ m^3/sec}}$$

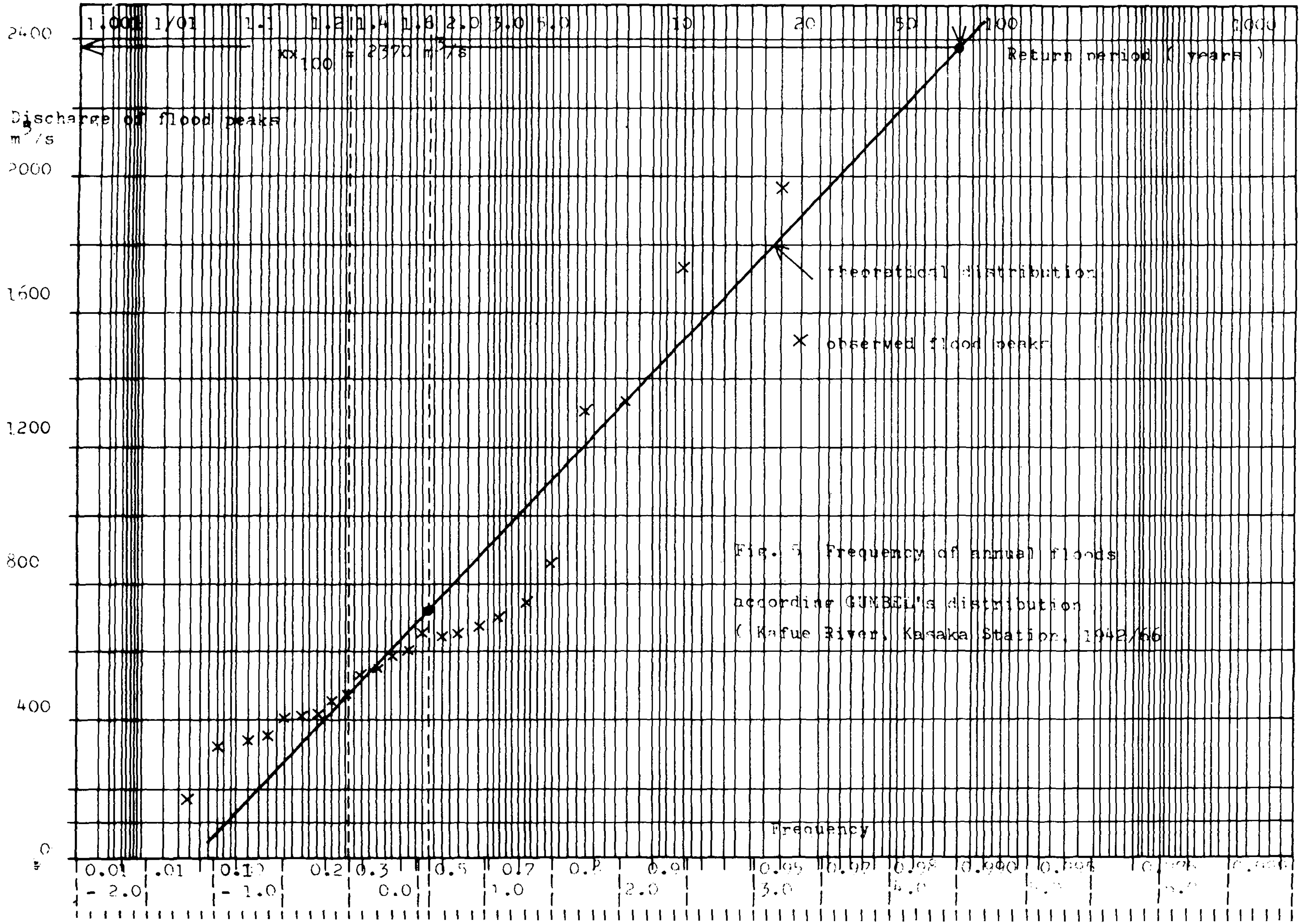


Fig. 5 Frequency of annual floods according GUMBEL's distribution (Kafue River, Kasaka Station, 1942/66)

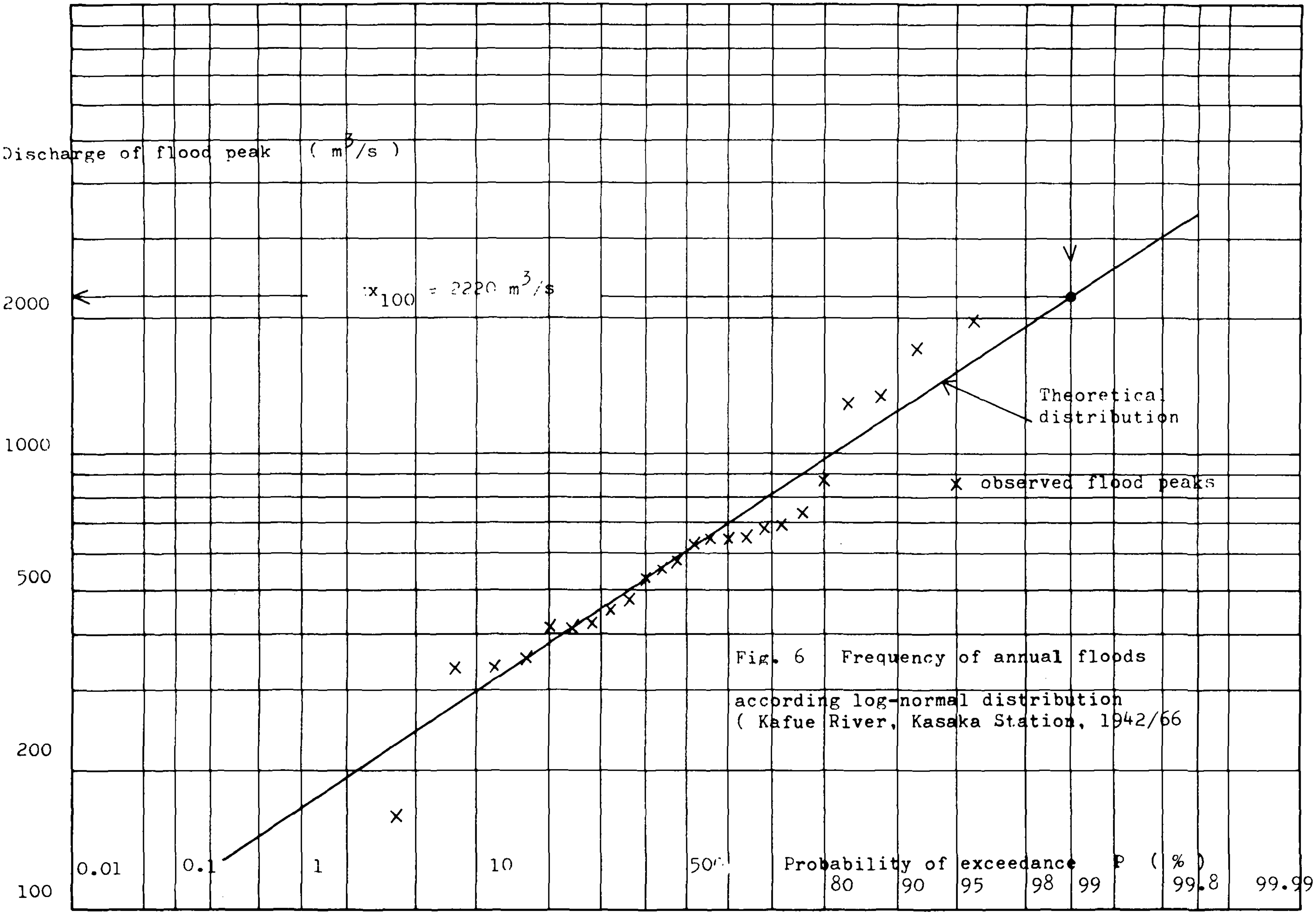


Fig. 6 Frequency of annual floods according log-normal distribution (Kafue River, Kasaka Station, 1942/66

Table VII: k-values of Pearson Type III distribution

Coefficient of skewness C_{sx}	Recurrence Interval, T_R (years)								
	2	5	10	20	25	50	100	200	500
0 = Normal	0.000	0.842	1.282	1.645	1.751	2.054	2.326	2.576	2.878
0.1	- 0.017	0.836	1.292	1.673	1.785	2.107	2.400	2.670	3.004
0.2	- 0.033	0.830	1.301	1.700	1.818	2.159	2.473	2.763	3.118
0.3	- 0.050	0.824	1.309	1.726	1.849	2.211	2.544	2.856	3.244
0.4	- 0.066	0.816	1.317	1.750	1.830	2.261	2.615	2.949	3.366
0.5	- 0.083	0.808	1.323	1.774	1.910	2.311	2.686	2.041	3.488
0.6	- 0.099	0.800	1.328	1.797	1.939	2.359	2.755	3.132	3.609
0.7	- 0.116	0.790	1.333	1.819	1.967	2.407	2.824	3.224	3.730
0.8	- 0.132	0.780	1.336	1.839	1.993	2.543	2.891	3.312	3.850
0.9	- 0.148	0.769	1.339	1.859	2.018	2.498	2.957	3.401	3.969
1.0	- 0.164	0.758	1.340	1.877	2.043	2.542	3.022	3.489	4.088
1.1	- 0.180	0.745	1.341	1.894	2.066	2.585	3.087	3.575	4.206
1.2	- 0.195	0.732	1.340	1.910	2.087	2.626	3.149	3.661	4.323
1.3	- 0.210	0.719	1.339	1.925	2.108	2.666	3.122	3.745	4.438
1.4	- 0.225	0.705	1.337	1.938	2.128	2.706	3.271	3.828	4.553
1.5	- 0.240	0.690	1.333	1.951	2.146	2.743	3.330	3.910	4.667
1.6	- 0.254	0.675	1.329	1.962	2.163	2.780	3.388	3.990	4.779
1.7	- 0.268	0.660	1.324	1.972	2.179	2.815	3.444	4.069	4.890
1.8	- 0.282	0.643	1.318	1.981	2.193	2.848	3.499	4.147	5.000
1.9	- 0.294	0.627	1.310	1.989	2.207	2.881	3.553	4.223	5.108
2.0	- 0.307	0.609	1.302	1.996	2.219	2.912	3.605	4.298	5.215
2.1	- 0.319	0.592	1.293	2.001	2.230	2.942	3.656	4.372	5.320
2.2	- 0.330	0.574	1.284	2.006	2.240	2.970	3.705	4.444	5.424
2.3	- 0.341	0.555	1.273	2.009	2.248	2.997	3.753	4.515	5.527
2.4	- 0.351	0.537	1.262	2.011	2.256	3.023	3.800	4.584	5.628
2.5	- 0.360	0.518	1.250	2.012	2.262	3.048	3.845	4.652	5.728
2.6	- 0.369	0.499	1.238	2.013	2.267	3.071	3.889	4.718	5.827
2.7	- 0.377	0.480	1.224	2.012	2.272	3.093	3.832	4.783	5.923
2.8	- 0.384	0.460	1.210	2.010	2.275	3.114	3.973	4.847	6.019
2.9	- 0.390	0.440	1.195	2.007	2.277	3.134	4.013	4.909	6.113
3.0	- 0.396	0.420	1.180	2.003	2.278	3.152	4.051	4.970	6.205

Table VIII: Frequency distribution of flood peaks on the Thiba River (partial series) according to Pearson Type III distribution.

Flood peak x (m³/sec)	rank r	Probability $p = \frac{r}{n+1}$ (%)	$(x - \bar{x})$	$(x - \bar{x})^2$	$(x - \bar{x})^3$	$\log x$	$(\log x - \log \bar{x})$	$(\log x - \log \bar{x})^2$	$(\log x - \log \bar{x})^3$
89.1	10	90.9	19.6	384.16	7529.54	1.94988	0.112.6	0.0125790	0.00141080
78.5	9	81.8	9.0	81.00	729.00	1.89487	0.05715	0.0032661	0.00018665
77.0	8	72.7	7.5	56.25	421.88	1.88649	0.04877	0.0023785	0.00011599
72.6	7	63.6	3.0	9.00	27.00	1.86106	0.02334	0.0005448	0.00001271
70.1	6	54.5	0.6	0.36	0.22	1.84572	0.00800	0.0000640	0.00000051
68.2	5	45.4	- 1.3	1.69	- 21.97	1.83378	-0.00394	0.0000155	-0.00000006
63.9	4	36.4	- 5.6	31.36	- 175.62	1.80550	-0.03222	0.0010381	-0.00003345
61.1	3	27.2	- 8.4	70.56	- 592.70	1.78604	-0.05168	0.0026708	-0.00013802
58.4	2	18.2	-11.1	123.21	-1367.63	1.76645	-0.07127	0.0050794	-0.00036200
55.9	1	9.1	-13.6	184.96	-2515.46	1.74741	-0.09031	0.0081558	-0.00073660
Total 694.8	10			942.55	+4034.26	18.37716		0.0357921	

arithmetic mean: $\bar{x} = \frac{694.8}{10} = 69.5$ m³/sec

standard deviation: $\sigma_x = \sqrt{\frac{1}{10-1}(942.55)} = 10.2$ m³/sec

coefficient of skewness: $C_{sx} = \frac{10(4034.26)}{9(8)(1061.21)} = 0.52$

variate of test: $d = 69.5\left[1 - \frac{2(10.2)}{69.5(0.52)}\right] = 30.3$

k(1; 0.52): $x_{1.01} = 69.5 - 1.916(10.2) = 49.6$ m³/sec

k(2; 0.52): $x_2 = 69.5 - 0.086(10.2) = 68.6$ m³/sec

k(10; 0.52): $x_{10} = 69.5 + 1.324(10.2) = 83.0$ m³/sec

k(25; 0.52): $x_{25} = 69.5 + 1.916(10.2) = 89.0$ m³/sec

k(50; 0.52): $x_{50} = 69.5 + 2.320(10.2) = 93.2$ m³/sec

k(100; 0.52): $x_{100} = 69.5 + 2.700(10.2) = 97.0$ m³/sec

geometric mean: $\bar{z} = \log \bar{x} = \frac{18.37716}{10} = 1.83772$

standard deviation: $\sigma_z = \sqrt{\frac{1}{(10-1)}(0.0357921)}$

$= 0.063062$

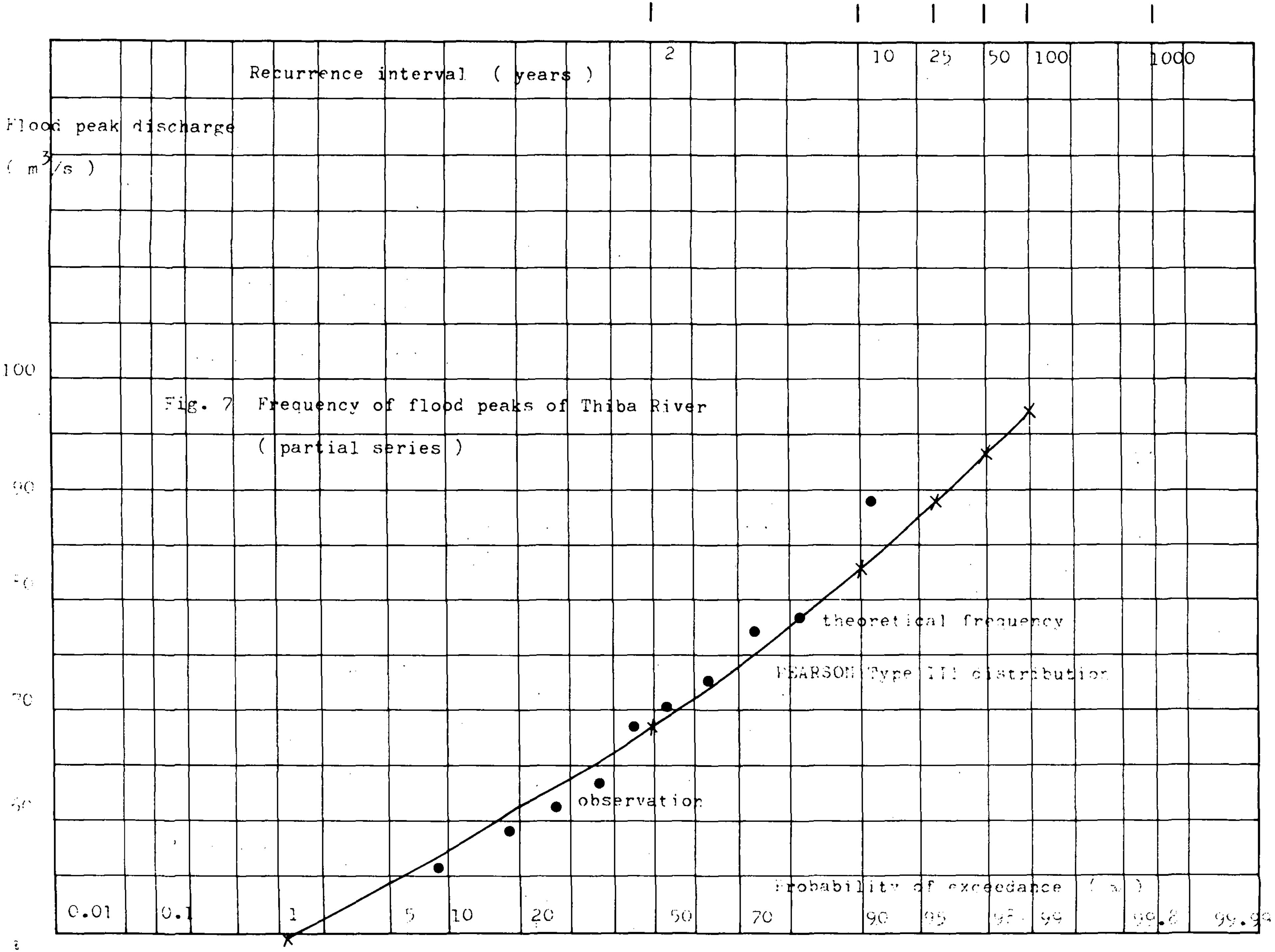
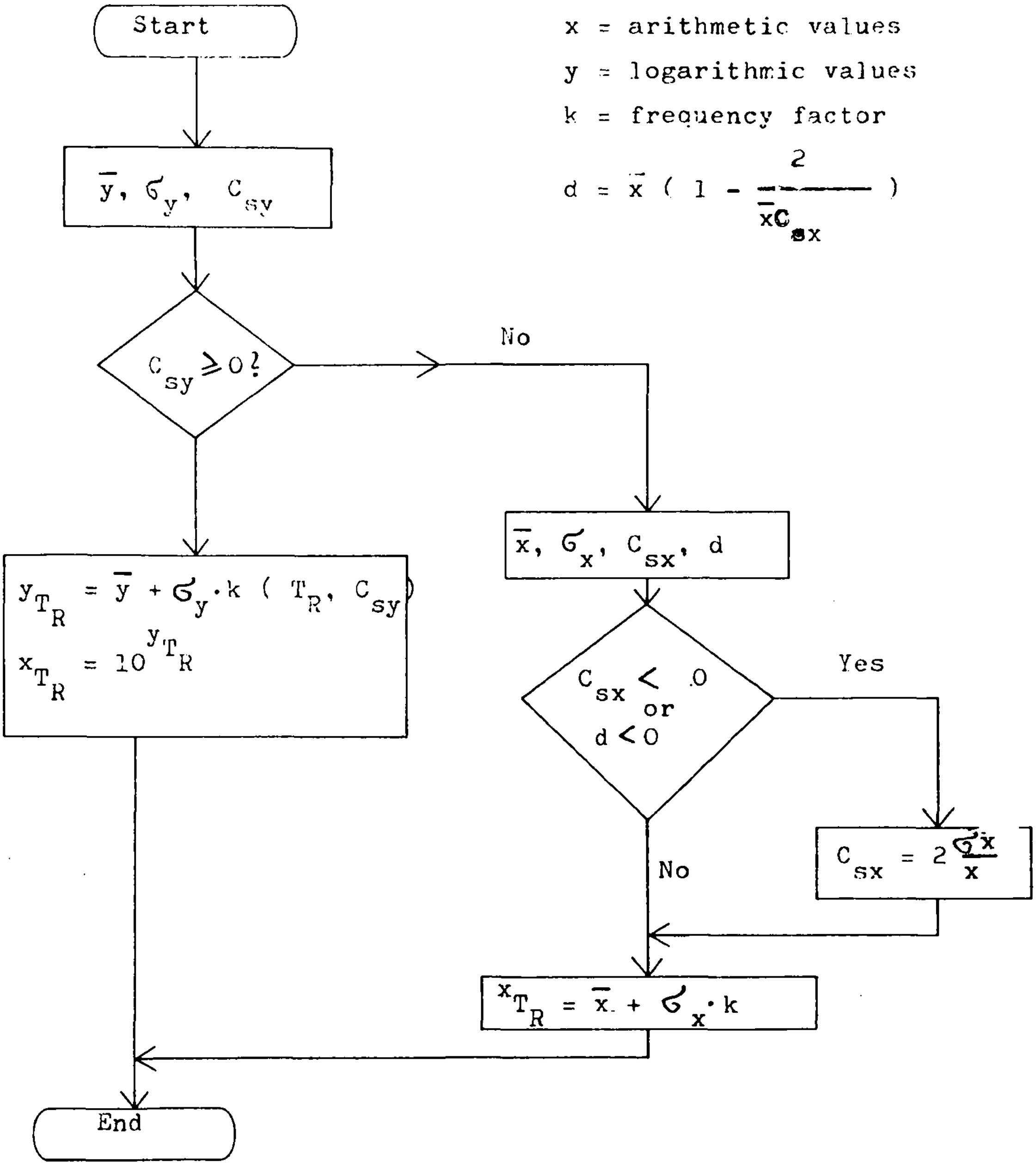


Fig. 7 Frequency of flood peaks of Thiba River (partial series)



x = arithmetic values

y = logarithmic values

k = frequency factor

$$d = \bar{x}\ (\ 1 - \frac{2}{\bar{x}C_{sx}}\ )$$

Fig. 8, Flow chart for computation of the PEARSON Type III distribution

H. WATER QUALITY

1. Water quality control in Kenya (an overview of administrative aspects with special reference to point-source industrial-based pollution)

1.1 Introduction

Pollution control is not contrary to development objectives but should be seen as a positive function in furthering the process; but ironically there is often a mentality that in a less developed country pollution control is not important. In fact, it is probably more important to control water pollution in a developing country than in a developed one since in the latter case the entire population normally has access to a safe treated water supply. In rural Kenya about 90% of the rural population still obtain water from traditional sources and until such time as everyone has a safe source there could be serious consequences in the interim period if pollution control is not enforced.

During the 1960s the country did not have serious industrial-based river pollution problems although the Nairobi River had reached a stage of gross pollution even in the 1950s, i.e. prior to the construction of the Kariobangi Sewage Works. The siting of industries within the densely populated rural areas and the policy of distributing manufacturing industries to municipalities and other urban centres outside the capital during the past decade has been the major cause of concern, especially following three incidents in 1970. 1970, in retrospect, saw a positive change in pollution control policy in Kenya and like the *Torrey Canyon* it needed a disaster to stir the political will to recognize that existing enforcement was inadequate. During 1970 there was excessive pollution from two, relatively new, sugar factories discharging waste-waters consisting mainly of molasses into rivers used directly for drinking water and bathing by thousands of people down-stream. In one case the fish population of the Nyando River, which is an important protein source for the local communities, was destroyed along a 24 km reach. In the same year oil pollution of Lake Nakuru in the Rift Valley occurred. Lake Nakuru, at times, forms the feeding ground for perhaps up to two million lesser flamingoes, *Phoeniconaias minor* Geoffrey, and because of this concentration of birds it is the second most visited national park in Kenya. Tourism is Kenya's largest foreign exchange earner after coffee. During the same year plans for a pulp and paper mill with an annual production of 54,000 tonnes of bleached and unbleached paper to be situated in the middle of a densely populated rural area in the Nzoia River catchment were finalized. This latter development was considered a particularly serious threat to the Kenya environment, especially as at that time, it was being considered to use mercury in the electrolytic cells of the chlorine-production plant.

In December 1970, an agreement was, therefore, signed with the World Health Organization to undertake a secterial study of community water supplies plus sewerage and water pollution. Part of the study was to survey all the major rivers and compile an inventory of sewage works and industrial discharges. This was carried out during 1971-2 and in line with a Cabinet directive and recommendations of the WHO, a Water Quality and Pollution Control Section was established within the Water Resources Branch of the Water Department during the second half of 1972.

This lecture gives a brief description of the organization of the Water Quality and Pollution Control Section in the Ministry of Water Development, Kenya and its operation with particular reference to the implementation of existing legal statutes for the control of point-source pollution, plus an outline of some current water quality problems. A short review of future pollution control management over the next Development Plan period is then discussed in the light of operational experience obtained during the past five years of enforcement.

1.2 The organization and legal background

The employment of consultants (in this case, WHO) to undertake an inventory of river and effluent qualities, river flow depth and bed characteristics and to evaluate the existing legislation and water resources administration in the country before recruiting Pollution Control Scientists and establishing an enforcement organization is to be recommended. In our case my immediate deployment as Pollution Control Officer to work out standards and make routine inspections has been feasible and I feel that my Section would have been less successful in being able to create an immediate impact than if the collection of basic background information had not been done.

The initial professional staff joining in 1972 was one Pollution Control Officer (a biologist and Section Head), a Chemist and a Sanitary Engineer; at the time of writing (September 1977), the professional complement now consists of 16 University graduates, which is a multi-disciplinary team. There is also a support staff of 14 technical officers who undergo a 3-year training course in water engineering and science organized jointly by the Ministry's Training School and Kenya Polytechnic.

The Section has been organized into 2 units and officers entering the section are either attached to the Water Quality Laboratory, which includes a service analytical sub-unit and an advisory/research sub-unit, or the Pollution Control Inspectorate and Network Monitoring Unit. The laboratory is equipped with modern instrumentation suitable for water analysis and has been established with technical and financial assistance from several aid countries, but the main donor has been NORAD.

After a period of in-service training, the work for a new graduate will include the following duties:

(a) - Carrying out pilot and laboratory scale studies for water treatment, including audit sampling where the Director of Water Department is the gazetted water undertaker.

- Analysing ground and surface waters, drinking water from treatment plants and a wide range of industrial effluents.

- Algal growth potential studies.

- Bacterial tests on water supplies.

- Fish and invertebrate toxicity testing.

- Water quality research.

(b) - Biological and chemical monitoring of all the major water resources in order to provide background information for the suitability of water for domestic, industrial and agricultural use; this data is then used by development planners and for the revision of effluent standards. It is in this field in particular that hydrology technicians are often engaged in collecting samples for the Unit's network.

- Inspecting sewage works and on-site industrial waste-water treatment plants in order to ensure that they are being correctly maintained.

- Tracing all reports of pollution to their source and collecting scientific (usually biological and chemical) data for use in any subsequent legal action (Hydrology Technicians should report any observed pollution to the Section during the course of their river gauging duties).

The Water Department is responsible for all technical aspects of water development, water conservation and management. It exercises control of development planning, design and supervision of major construction works for water supplies and sewerage. The Director is the gazetted water undertaker for most of the public drinking water supplies except for the City of Nairobi and certain municipalities.

The Department operates on a functional basis rather than a project-oriented basis. There are three technical branches: Development, Operations and Water Resources. Each Branch Head is responsible to the Director of the Water Department. The Water Resources Branch contains scientists from several disciplines and is divided into five sections - Apportionment, Drilling, Geology, Hydrology and Water Quality and Pollution Control.

The current water legislation which WHO legal experts consider is basically sound is the Water Act. All prospective water users must apply for a water permit before an abstraction or discharge, which is either given or refused by an independent body known as the Water Apportionment Board. The Board consists of members from other ministries and the general public, and the interests of commerce, industry and agriculture are represented; the Director of the Water Department is the Technical Adviser.

The most relevant anti-pollution section of the Water Act is Section 182 which has given the Water Apportionment Board (WAB) the power to make several rules to deal with anti-pollution by way of subsidiary legislation. It should also be noted that all local authorities must apply to WAB for a surface water permit or an authorisation to abstract ground water and conditions on final effluent quality from the sewage works can be laid down. Under the Municipalities and Local Government Acts, local authorities have been given the obligatory duties for sewage disposal. Local authorities can enter into agreements for the discharge of wastewaters into sewers by manufacturers or allow the latter to discharge into the sewers subject to by-laws made in accordance with the Public Health Act.

The Water Resources Branch is now, in effect, carrying out the function of a "Water Resources Authority". The Apportionment Section keeps records of water permits, maintains Water Bailiff personnel to check on abstraction rates, and provides the secretariat for WAB but it is the Hydrology and Geology Sections which maintain gauging station and borehole observations, respectively, and provide the Apportionment Section with relevant information. The first gauging station was installed in 1921 and there are 355 stations now in operation. Using information on river flows from the Hydrology Section and its own build up following the establishment of a national network of 44 monitoring stations during the past five years, the Water Quality and Pollution Control Section is establishing stream standards and from these effluent standards. When a new permit is issued, an agreement on the chemical composition of any effluent is now also incorporated.

1.3 Existing point source pollution

1.31 General

In 1972 the majority of industries were based on production of agricultural products but during the past two years there has been an upsurge in the number of chemical and metal-based industries entering Kenya.

1.32 Organic effluents

There is a wide range of organic effluents containing soluble substrates, and apart from crude sewage, coffee wastes are the most widespread (3). There are now over 1,000 factories situated within the rural areas and during the processing of coffee beans the composited effluent discharge can have a B.O.D.$_5$ of up to 4,000 mg/l. During the relatively short processing period the pollution load in terms of B.O.D.$_5$ population equivalent has been estimated (4) at 120 million people! Coffee-pollution is still the most serious pollutant in Kenya but following the publication of Legal Notice 827 dated 9th March 1976 in the Kenya Gazette new water-permit conditions which will restrict the volume of water abstracted are now being issued. Fuller details on coffee pollution can be obtained from the references cited.

1.33 Manufacturer of fertilizers

A new factory is at present under construction within the Municipality of Mombasa.

1.34 Heavy metals

Copper, zinc and lead contamination was discovered in 1973 at two sites but pollution from mining is not widespread. Chromium discharged from tanneries, which are expanding rapidly at present because of restrictions on the export of rawhides is a potential hazard, but the majority of new factories are being encouraged to recycle chrome since it has been shown that this is also economically favourable to the tanneries also. The increase in the use of dyes containing heavy metals in the textile industry are also under intensive evaluation, especially their effect on algal-pond performance. Two textile mills have been built since 1975, several existing mills have been expanded and two new synthetic-fibre mills are under construction. A lead smelter for battery manufacture is currently being evaluated.

1.35 Pesticides

Three pesticides manufacturing or compounding factories have been established at three towns. In the field organechlorine compounds are used extensively, but levels are still at low levels compared with developed countries.

1.4 Standards

A national standard was considered too inflexible for practical use despite its administrative advantages.

Prior to 1972 the only standard for purely biodegradable organic effluents recommended by the Water Apportionment Board was a standard of 20 mg/l for the 5-days biological oxygen demand test and 30 mg/l for suspended solids irrespective of the degree of dilution in the receiving water. Effluents from factories located near head waters, where the dilution is virtually nil cannot fail to cause pollution and, conversely, there are cases in the country of plants designed for a 20:30 standard where the dilution ratio is as high as 1:500. It is considered necessary to offer to new industry a choice of standards depending on the self-purification characteristics and subsequent down stream use of proposed receiver so that, other considerations being equal, the most economic site can be made available. The latter, of course, applies to organic effluents where there is no public health risk. Standards for sanitary sewage, discharging to inland waters are normally set at $B.O.D._5$ of 20 mg/l at 20°C plus a stream faecal coliform standard of not less than 5,000 organisms per 100 ml; on oxidation ponds over-flows at the inlet are not now permitted.

The applicability of the Streeter-Phelps model of stream purification (5) has been applied to streams in the humid areas and appears suitable. The streams have only sparse macrophyte populations - so that photosynthesis is not important and stream velocities are generally too high to allow organic solids to settle. The two principal factors which mitigate against the model (6) can therefore be eliminated. In practice, this has meant that standards given by W.A.B. have been generous, yet at the same time the river ecology has been preserved. This is considered important to a developing country that does not want to lose industry because of high treatment costs, but or the other hand is determined to prevent water pollution.

For practical inspection purposes for on-site treatment plants and because rivers flow in a single direction, concentration standards are given; these are based on the critical dry weather flow, which is normally taken as the average of the driest 10 days (in sequence) in a year, with a 5 years return period. Over the past 2½ years less stringent standards varying with higher flows have been granted where high operational costs have justified two sets of standards based on high and low flows, respectively.

The argument for establishing effluent standards based on receiver classification has already been published elsewhere (7) and the current policy has not changed.

However, before effluent standards are presented to a manufacturer and other anti-pollution measures laid down, some basic information is obtained prior to construction work. This information can be tabulated:

(i) A map showing the proposed siting (or sitings) of the factory and the surrounding area on an appropriate scale.

(ii) A drawing showing the general layout of the factory, the water supply and sewerage system.

(iii) A description of the different industrial processes involved with an indication of initial and future production capacities, consumption of raw materials, chemicals (and their composition), oil and grease. An indication of water consumption for different purposes.

(iv) The number of employees, the number of daily shifts and number of working days per year. An indication of whether there will be a staff canteen or laundry service within the factory compound. A statement on the period of any proposed routine shut down for annual maintenance, etc.

(v) A flows scheme indicating the different streams of waste-water and their proposed treatment, storage, recycling and discharge. An outline of the physical characteristics and degree of purity of the different waste-water flows with daily volumes and fluctuations. A plan showing the proposed outlet points either to a surface water stream or channels or to a foul sewer.

(vi) A description of effluent discharged into the air and the planned anti-pollution measures.

(vii) A description of the proposed methods for handling any solid waste and/or sludge from industrial processes and effluent treatment.

(viii) A description of the proposed safety precautions for storage of oil and chemicals such as bund-walls, etc.

1.5 Future management

The goals set out for the Section in the 1974-8 Development Plan (part II, p. 342) have now been reached; a network of monitoring stations has been established and the Water Quality laboratory expanded considerably plus an audit sampling organization for drinking water supplies

Legislation has been found to be sound for the control of water pollution in Kenya and successful legal action has been taken by the cancellation of permits on two occasions where operators had wilfully contravened the conditions laid down by W.A.B. and two standstill orders on further development have been imposed under Section 182 Rule 75 when it was known that there was an intention to discharge undesirable effluents into a water course. Where, however, water pollution control has been adequately studied from the commencement of a project and where other Ministries have given the Department prior knowledge and sufficient time to obtain suitable base line data (8) and the Pan-African Pulp Paper Mill is a good example - problems of industry refusing to install adequate wastewater facilities do not arise (9) and it must be stated that co-operation between the Water Quality and Pollution Control Section and industry is normally exceptionally good. Attempts by industrialists at non-co-operation are the exception rather than the rule.

Staff have been recruited, albeit a considerable amount of personnel training in the water quality control field is still required; during the next plan period this will be in the form of academic training (e.g., post graduate studies at the University of Nairobi), specialized courses in particular fields under the auspices of international organizations such as UNEP, FAO, UNESCO, IDRC and in-service training programmes, which will probably be of most significance, aimed at improving the professional capabilities, knowledge and motivation of the staff so that the section achieves high respect in its sampling integrity, analysis and knowledge of local problems.

Care must be taken to ensure that over the next few years the Section does not commit itself to too many new projects so spreading its resources too thinly to be effective, especially in the field of compliance monitoring and legal enforcement. Some new subject areas are, however, being considered for forward planning evaluation in the light of current development: extension service to local authorities in the field of trade effluent control and monitoring in compliance with donor -loan agreements.

This is considered of highest priority because, as mentioned in the current pollution control problems section, the trend of new industrial development is an increase in the number of chemical industries and these are normally being established within municipalities rather than directly on rivers in rural areas where individual permits can be issued by W.A.B. Advanced wastewater treatment for selected flows will be required in many cases before discharge of such flows into public sewers and regular sampling will be required to establish a realistic pricing policy. Many of the newer industrial wastewaters could foul the receiving waters indirectly by affecting the treatment performance of existing sewage-treatment plants, particularly as it is the current Government policy to use algal waste stabilization ponds, wherever possible which appears to be very sensitive to slug-flows of industrial wastewaters in the Kenyan environment. Local authorities will also need central Government assistance to comply with loan agreements with donor countries - a case in point is the German Aid Loan to Nakuru Municipality which insists that local authority undertake compliance monitoring of all trade-effluent discharges to the reticulation system including receiver monitoring. The tendency for donors to attach such conditions to grants or soft loans is increasing.

It is not planned to increase the national network at present but during the 1979-1983 Development Plan Period, the Hydrology Section will establish a silt-monitoring unit and a study of pollutants transport on sediments (e.g., pesticides, nutrients, heavy metals) will be carried out in liaison with this programme. It is also expected to install automatic sampling equipment on the Nairobi River during 1981-2. Data obtained from the network is being published in a series of technical reports (see Appendix). There will, of course, be various intensive survey type monitoring, in addition to network sampling and effluent compliance monitoring. There should be a repeat marine investigation of the tidal waters around Mombasa Island (10) in view of several new factories to be established in the future and the fact that following a recent declaration under Section 31 of the Water Act the W.A.B. can now control the abstraction of water from certain tidal waters. Also, there is some concern regarding eutrophication and this is supported by a recent synoptic study carried out by the Section of increasing eutrophication of Winam Gulf as compared to the other parts of Lake Victoria (11) and a study is needed to determine the siting of new waters intakes and effluent discharge points around the municipality of Kisumu.

A project on the land-treatment of effluents arising from the wet processing of coffee and funded through the Master Plan Study will terminate in 1978 but a current ongoing project to determine appropriate design criteria for algal stabilization ponds, which is part of a wider global study being financed partly by IDRC is not expected to be completed until 1980.

2. Basic chemical definitions

Chemical characteristics are more specific in nature than most physical parameters and as a consequence are more immediately useful in assessing the properties of a sample. Some basic chemical definitions should be clearly understood at the outset:

Atom The smallest quantity of an element that enters into chemical combination. It has a very small, dense nucleus, positively charged. Negative particles called electrons rotate around the nucleus.

Atomic weight Weight of an atom of an element referred to a standard based on the carbon isotope C^{12}.

Compound A pure substance consisting of chemically combined elements.

Molecule Smallest particle of compound capable of having the properties of the compound.

Molecular weight Total atomic weight of all atoms in a molecule.

Mole A quantity of a substance equal in weight to its molecular weight.

Molar solution Solution containing the molecular weight of the substance in one litre.

Acid A compound which yields hydrogen ions in solution.

Ion An atom which has acquired an electrical charge as the result of the gain or loss of electrons. Ion possessing a positive charge is a cation; ions have negative charges.

Metal Element which readily loses electrons to form positive ions; conductor of electricity.

Heavy metal A metal with an atomic weight of 50 or higher.

Non-metal Element which readily gains electrons to form negative ions; non-conductor of electricity.

Valence The valency of an element is measured by the number of atoms of hydrogen which will combine with or replace one atom of the element. For example, chlorine combines with one atom of hydrogen and is therefore univalent, calcium replaces two atoms of hydrogen and is therefore divalent and so on. Several of the elements have different valencies in different compounds; thus iron is divalent in the compound FeO, or trivalent in the compound Fe_2O_3.

Equivalent weight This can be expressed

$$\text{Equivalent weight} = \frac{\text{Molecular weight}}{Z}$$

where Z = for acids, the number of H^+ obtainable from 1 mole of acid

Z = for bases, the number of H^+ which 1 mole of base will react.

(Mole is the molecular weight in grams).

Normal solution Solution containing the g-equivalent weight of the substance in 1 litre.

Avogadro's number Equal volumes of gases contain equal numbers of molecules. Since, at standard conditions, one mole of any gas occupies the same gram-molecular volume, one mole of every molecular substance must contain the same number of molecules. This number called Avogardro's Number is defined as the number of atoms in 12.000 grams of carbon 12 which is 6.023×10^{23}. From it we can find the actual weight of a molecule of a substance.

Oxides Compounds of oxygen with another element are called oxides.

Oxides of non-metals The oxides of non-metals are acidic, and most of them dissolve in water to form acids. All acids are compounds of hydrogen, which is capable of being replaced by a metal; the group of atoms combined with the hydrogen is termed the acid radicle. Thus sulphur, carbon and nitrogen give rise respectively to sulphuric, carbonic and nitric acids. The oxide of silicon, silica (SiO_2) is acidic but is not readily soluble in water and does not give rise to silicic acid.

Salts By the combination of an acid and a base the hydrogen of the acid is replaced by the metal of the base and the result is the formation of a salt. Thus the action of hydrochloric acid (HCl) on the base caustic soda (Na OH) gives the salt sodium chloride (Na Cl) together with water H_2O as shown in equation below:

HCl	+	$NaOH$	=	$NaCl$	+	H_2O
acid		base		salt		water

Many minerals are salts, eg.

Name of acid	Name of salt	Example of salt (mineral)
Hydrofluoric (HF)	Fluoride	Fluorspar (CaF_2)
Carbonite (H_2CO_3)	Carbonate	Calcite ($CaCo_3$)
Phosphoric (H_3PO_4)	Phosphate	Apatite ($Ca_3(PO_4)_2$)

Oxidation and reduction A chemical change by which oxygen is added to an element or compound is called oxidation. The term reduction is applied to a change in which the oxygen or other non-metal is taken away from the compound.

Examples When metallic copper is heated in contact with air it is changed into an oxide of copper:

$$2\ Cu + O_2 = 2\ CuO$$

Here oxidation of the copper has taken place. The reverse process may be seen by heating the copper oxide in a current of hydrogen, with the result that metallic copper and water are formed. This is a case of reduction, and the changes may be represented as:

$$CuO + H_2 = Cu + H_2O$$

Another example of oxidation is the change from ferrous to ferric acids, thus $2\ FeO + O = Fe_2O_3$.

Carbon compounds (organic) vis-à-vis inorganic chemistry

Substances obtained from plants and animals, like sugar, alcohols and oils, have certain properties that distinguish them from the mineral world.

For a long time it was considered that such substances could not be synthetically prepared but following the synthesis of urea in the last century the distinction was disproved. It is now also clear that organic substances are subject to the same chemical laws as inorganic substances. Organic chemistry is, however, still used to designate the study of carbon compounds and there are justifications for the preferential treatment accorded to carbon.

Carbon forms an extremely large number of compounds, unparalleled by any other element. Over 700,000 carbon compounds are currently known and the figure is ever increasing; about 30,000 new compounds being made every year. The reason for the prolific nature of carbon is that the carbon atom possesses the ability, to a very marked degree, to form carbon chains of varying length and rings of different sizes.

Most organic substances are extremely sensitive to heat and melt or decompose below 300°C - a temperature which leaves most inorganic substances unaffected. Generally speaking, organic substances are only slightly soluble in water but are readily soluble in organic solvents like ether, alcohol, chloroform. One other property of organic compounds is their non-ionic nature. This property arises from the position of carbon in the periodic table because it is half way between the electropositive alkali metals and the electronegative halogens. Because of the neutral character of carbon its compounds display a marked reluctance to ionize.

3. Physical characteristics of water

3.1 Temperature

Basically important for its effect on other properties, eg, speeding up chemical and biological reaction, reduction in solubility of gases (especially O_2), amplification of tastes and odours.

3.2 Taste

Due to dissolved impurities, often organic in nature (e.g. phenol). These are subjective properties which are difficult to measure.

3.3 Colour

Even pure water is not colourless; it has a pale green-blue tint in large volumes. It is necessary to differentiate between true colour in solution and "apparent" colour due to suspended matter. Natural colour is often due to organic compounds, especially in streams derived from upland swamp catchments.

Even if the colour is harmless many consumers object to a highly coloured water when they have a piped water scheme on aesthetic grounds and coloured water may be unacceptable for certain industrial purposes, such as in the production of high-quality paper.

3.4 Turbidity

The presence of colloidal solids gives liquids a cloudy appearance which again is aesthetically unattractive but may also be harmful. Turbidity in water may be due to clay particles, presence of large number of micro-organisms (e.g. algae).

3.5 Solids

These may be present in suspension and/or in solution and they may be divided into organic and inorganic matter. Total dissolved solids (TDS) are due to soluble materials but suspended solids (SS) are discrete particles which can be measured by filtering a sample through a fine paper. Settleable solids are those solids settling out in a graduated cone after standing for a fixed period, normally 2 hours and this is a useful way of measuring the performance of sedimentation capabilities.

As a guide, particle sizes are approximately:

(a) Settleable solids or suspensions (i.e. relatively coarse particles which settle rapidly) > 500 nm

(b) Colloidal solutions (very fine particles which do not under normal conditions settle): 1 - 500 nm.

(c) True solutions (ions and molecules which never settle) < 1 nm.

(Note: "nm" or namometre = 10^{-9}m).

No sharp demarcation line, however, exists between these groups and this must be stressed, e.g. there is a borderline "pseudo-colloidal" region with a particle size around 500nm with intermediate settling characteristics between colloidal solutions and suspensions.

3.6 Electrical conductivity

The conductivity of a solution depends on the quantity of dissolved salts present and for dilute solutions it is approximately proportional to the T.D.S. content:

$$K = \frac{\text{conductivity (in units of resistance)}}{\text{T.D.S. (mg/l)}}$$

Knowing the appropriate value of K for a particular water, the measurement of conductivity provides a rapid indication of T.D.S. content and, in addition, it has been used in studies of pollution from sisal wastes in Tanzania to obtain an estimate of colloidal material.

3.7 Viscosity

"Stickiness" power of resisting a change in the arrangement of the molecules.

3.8 Density

This is the mass of a unit volume of substance. The density of fresh water is 1000 kg/m^3. Sea water is 1025 kg/m^3.

3.9 Radioactivity

4. Important chemical characteristics in water quality control

4.1 Dissolved Oxygen (D.O.)

D.O. is a most important element in water quality control. Its presence is essential to maintain the higher forms of biological life and the effect of a waste discharge on a river is largely determined by the oxygen balance of the system. It is, however, only slightly soluble in water:

Temp. (°C)	0	10	20	30
D.O. (mg/l)	14.6	11.3	9.1	7.6

It is also less soluble at higher altitudes. Dissolved oxygen will be discussed in much more detail in the lecture on effluent standards.

4.2 pH

The intensity of acidity or alkalinity of a sample is measured on the pH scale which actually measures the concentration of hydrogen ions present.

Water is only weakly ionized:

$$H_2O \rightleftarrows H^+ + OH^-$$

Since only about 10^{-7} molar concentrations of H^+ and OH^- are present at equilibrium, H_2O may be taken as unity. Thus:

$$(H^+)\ (OH^-) = K = 1 \times 10^{-14}$$

mole/l at 25°C

Since this relationship must be satisfied for all dilute aqueous solutions, the acidic or basic nature of the solution can be specified by one parameter - the concentration of hydrogen ions. This is expressed by the function of pH:

$$pH = -\log_{10} (H^+) = \log_{10} \frac{1}{(H^+)}$$

resulting in a scale from 0 to 14 with 7 as neutral, below 7 being acid and above 7 being alkaline.

Many chemical reactions are controlled by pH; biological action is usually restricted - albeit there are many important exceptions - to a narrow pH range of 6 - 8. Highly acidic or highly alkaline waters are undesirable because of corrosion hazards and possible difficulties in treatment.

4.3 Alaklinity

Due to the presence of bicarbonate HCO_3^-, carbonate $CO_3^=$ or hydroxide OH^-. Most of the natural alkalinity in waters is due to HCO_3^- produced by the action of ground water on lime bearing strata:

$$\underset{\substack{\uparrow\\ \text{insoluble}}}{CaCO_3} + H_2O + \underset{\substack{\uparrow\\ \text{from soil}\\ \text{bacteria and}\\ \text{volcanic}\\ \text{emissions}}}{CO_2} \longrightarrow \underset{\substack{\uparrow\\ \text{soluble}}}{Ca(HCO_3)_2}$$

Alkalinity is useful in waters and wastes in that it provides buffering to resist changes in pH. It is normally divided into caustic alkalinity above pH = 8.2 and total alkalinity above pH = 4.5. Alkalinity can exist down to pH = 4.5 because of the fact that HCO_3^- is not completely neutralized until this pH is reached. Use of alkalinity of waters can give a good indication of sediment sources.

4.4 Acidity

Most natural waters and domestic sewage are buffered by a CO_2 - HCO_3^- system. Carbonic acid H_2CO_3 is not fully neutralized until pH = 8.2 and will not depress the pH below 4.5. Thus CO_2 acidity is in the range 4.5-8.2, mineral acidity (almost always due to industrial wastes and/or mine-drainage) occurs below pH = 4.5.

4.5 Oxygen demand

Organic compounds are generally unstable and may be oxidized biologically or chemically to stable, relatively inert end products such as CO_2, NO_3, H_2O. An indication of the organic content of a waste can be obtained by measuring the amount of oxygen required for its stablization:

(a) Biological Oxygen Demand (BOD) - a measure of the oxygen required by micro-organisms while breaking down organic matter. It is usually measured over 5 days at 20°C.

(b) Permanganate value (P.V.) - chemical oxidation using potassium permanganate. An indication of easily oxidizable material.

(c) Chemical Oxygen Demand (COD) - chemical oxidation using boiling potassium dichromate and concentrated sulphuric acid.

The magnitude of the results obtained is usually $PV < BOD < COD$.

4.6 Hardness

This is the property of a water which prevents lather formation with soap and produces scale in hot water systems. It is normally due to the metallic ions Ca^{++} and Mg^{++} although others (eg. Fe^{++}) are also responsible. The metals are associated with HCO_3^-, SO_4^-, Cl^- and NO_3^-. There is no health hazard and, in fact, from the toxicity point of view, animals generally survive longer in polluted water which is hard as compared to soft water, but there are economic disadvantages expressed in increased soap usage, fuel costs, etc.

Hardness is divided into 2 forms:

(a) Carbonate hardness - metals associated with HCO_3^-,

(b) Non-carbonate hardness - metals associated with sodium, chloride and nitrate.

4.7 Nitrogen

This is an important element in biological systems; biological treatment can only proceed in the presence of nitrogen. Nitrogen exists in four main forms as far as public health engineering is concerned:

(a) Organic nitrogen - nitrogen in the form of proteins, amino acids and urea. Albuminoid nitrogen is an analysis which measures part of the organic nitrogen.

(b) Ammonia nitrogen - nitrogen as free ammonia or as ammonium salts.

(c) Nitrite nitrogen - an intermediate oxidation stage not present normally in large amounts.

(d) Nitrate nitrogen - final oxidation product of nitrogen.

Oxidation of nitrogen compounds is termed nitrification.

4.8 Chloride

Responsible for brackish taste in water and can be an indicator of sewage pollution because of the chlorine content of urine. Threshold level for Cl^- taste is 250-500 mg/l, although up to 1500 mg/l is unlikely to be harmful to healthy people and goats and beef; cattle can tolerate much higher levels.

Cl^- is the major constituent of sea water:

	g/kg	%
Cl^-	18.9	55.04
$SO_4^=$	2.6	7.69
Mg^{++}	1.2	3.69
Ca^{++}	0.4	1.16
K^+	0.3	1.10
Na^+	10.5	30.61
Other	0.2	0.72
	34.4	100

4.9 Specialized characteristics

Many other parameters have to be monitored on occasions, e.g. toxic metals, oils, insecticides, etc. Because of the possible mode of sediment transport in distributing the latter, a special lecture will be given on pesticide residues in East Africa.

5. Chemical classification of waters

To classify waters solely on the basis of chemistry is rather difficult because of the great number of ions which may occur in natural waters. Fortunately, however, only a few of these are sufficiently common and widespread to be of practical use for the construction of a general classification scheme. The 4 potential anions are:

bicarbonate
carbonate
chloride
sulphate

The corresponding cations are:

sodium
potassium
calcium
magnesium

Anions are assumed to be the independent ingredients while the cations are the dependent variables. The major geochemical groups of water are therefore those determined by the occurrence and prevalence of anions. On this basis one may distinguish between three major categories of natural waters:

(1) Bicarbonate waters
(2) Chloride waters
(3) Sulphate waters

If two of the anions are far in excess of the third, one may speak, for example, of bicarbonate sulphate water or sulphate chloride water, always putting the common anion in front.

The terms "fresh water" and "salt water" are frequently employed. The upper boundary for fresh water is 1% salinity. This is about the maximum salt concentration tolerated by natural fresh water fauna and flora. The salt limit for human consumption is 0.5% although a few instances are known where waters up to 0.8% salinity are used without noticeable damage to health. Waters having a salt content between 1 and 3.5% are called salinized water; the upper value corresponds to the salinity of the ocean. Waters beyond this concentration are known as brines.

6. Water chemistry basic analysis

6.1 Introduction

Quantitative analysis may be carried out by gravimetric, volumetric or colorimetric methods. Some parameters can now be measured using special electrodes. Because only small amounts of impurities are present in samples, much of the work tends to be of a microanalytical nature. In order to be able to compare results obtained by different laboratories, it is important that the determinations should be carried out by a standard method (in Kenya we use Standard Methods for Examination of Water and Waste Water, American Public Health Association, New York).

6.2 Gravimetric analysis

This form of analysis depends on weighing solids obtained by evaporation, filtration or precipitation. The procedures tend to be time-consuming because of the need for careful drying of moisture before weighing, both from the solids and the dish or container in which they are placed. Because of the small weights involved, an accurate analytical balance, which must be constantly checked, and capable of reading to 0.0001 g is essential.

The main uses of gravimetric analysis are as follows:

(1) Total and volatile solids

A known volume of the sample in a pro-weighted dish (nickel or similar) is evaporated to dryness on a water bath, dried (this is 180°C for potable waters) and weighed.

The volatile solids (organic + carbonates) are burnt off at 600°C in a furnace and the residue weighed.

Because of the large weight of the container compared with the weight of the solids, it is vital that all moisture be removed from the dish before its weighing otherwise negative results will appear.

(2) Suspended solids

Suspended solids are measured by filtering the sample through a preweighed glass fibre paper weighing about 0.2 g. Careful drying to constant weight is again essential. (This test normally also includes some colloidal portion).

(3) Sulphate

In concentrations above 10 mg/l it is possible to measure sulphates gravimetrically with great accuracy. Barium chloride is added to the sample to produce insoluble barium sulphate:

$$BaCl_2 + H_2SO_4 \longrightarrow \underline{BaSO_4} + 2HCl$$

The barium sulphate is then removed by filter paper, dried and weighed.

All gravimetric procedures take a considerable amount of time.

6.3 Volumetric Analysis

Many determinations can be rapidly and accurately carried out by volumetric analysis which depends on the measurement of volumes of liquid reagents of known strength.

The requirements for volumetric analysis are relatively simple:

(1) Equipment to measure the sample accurately (e.g. pipette).

(2) A standard solution of suitable strength (a standard solution is a solution of known strength).

(3) An indicator to show when the end point has been reached.

(4) A graduated burette for accurate measurement of the volume of standard solution necessary to reach the end point.

Standard solutions: To simplify volumetric analysis, standard solutions of such strength that they are equivalent to one another are usually employed. This involves knowledge of the equivalent weights of the reacting materials.

Indicators: Volumetric analysis requires some way of indicating the end point of the reaction as accurately as possible; various types of indicator are used.

Typical uses of volumetric analysis are as follows:

(1) Acidity and Alkalinity

Only with strong acid and strong base does neutrality occur at pH 7; with all other combinations the neutralization points occur at pH 8.2 and pH 4.5.

Several organic compounds have well-defined colour changes over limited pH ranges and it is possible to prepare a wide range indicator with a continuous change of colour over a wide pH range. The indicators normally adopted for acidity and alkalinity are phenolphalein (pink above pH 8.2, colourless below 8.2), and methyl orange (red below pH 4.5, yellow-orange above pH 4.5).

Volumetric analysis can be most useful in the measurement of different forms of alkalinity. Neutralization of OH^- is complete at pH 8.2, whereas neutralization of $CO_3^=$ is only half completed at pH 8.2 and not fully completed until a pH value of 4.5 is reached:

$$CO_3^= + H^+ \longrightarrow HCO_3^- + H^+ \longrightarrow H_2CO_3$$

The following rules should be remembered:

(a) HCO_3^- and OH^- cannot exist together.

(b) OH^- alone gives initial pH of about 10 and in this case OH^- alkalinity = caustic alkalinity = total alkalinity.

(c) $CO_3^=$ alone gives initial pH of about 9.5.
$CO_3^=$ alkalinity = 2 x caustic alkalinity = total alkalinity.

(d) OH^- and $CO_3^=$ together give initial pH of about 10.
$CO_3^=$ alkalinity = 2 x titration from pH 8.2 - 4.5.
OH^- alkalinity = total alkalinity - $CO_3^=$ alkalinity.

(e) $CO_3^=$ and HCO^- together have initial pH $> 8.2 < 11$.
CO_3 alkalinity = 2 x caustic alkalinity.
HCO_3^- alkalinity = total alkalinity - $CO_3^=$ alkalinity.

(f) HCO_3^- alone gives initial pH <8.2.
HCO_3^- alkalinity = total alkalinity.

(2) Chloride

The determination of chloride is usually carried out using silver nitrate with potassium chromate as a precipitation indicator.

$$AgNO_3 + NaCl \longrightarrow \underline{AgCl} + NaNO_3$$

(white precipitate)

also $2AgNO_3 + K_2CrO_4 \longrightarrow \underline{Ag_2CrO_4} + 2KNO_3$

(red precipitate)

When all the chloride has reacted, the red precipitate of silver chromate indicates the end point of the titration.

(3) Dissolved Oxygen (DO)

In the Winkler method for DO the oxygen is converted in a series of steps, using manganous sulphate, alkaline iodide and sulphuric acid, to free iodine which is then titrated with sodium thiosulphate:

$$2Na_2S_2O_3 + I_2 \longrightarrow Na_2S_4O_6 + 2NaI$$

Starch is used as an adsorption indicator to indicate the end point. Iodine is adsorbed on the surface of colloidal starch, giving a blue colour which is extinguished when all the iodine has reacted.

(4) Chemical Oxygen Demand (COD)

In this determination the excess potassium dichromate remaining after the reaction is neutralized with ferrous ammonium sulphate solution:

$$6Fe^{++} + Cr_2O_7^{=} + 14\ H^+ \longrightarrow 6Fe^{3+} + 2Cr^{3+} + 7H_2O$$

The end point is obtained using ferroin which changes from blue-green to red-blue when all the dichromate has been reduced. The colour change is easily visible even in the presence of the green colour of trivalent chromium.

6.4 Colorimetric Analysis

Colorimetric analyses are particularly useful when dealing with dilute solutions and there are many determinations in water quality control which can be quickly carried out by this method.

To be of quantitative use a colorimetric method must be based on the formation of a compound or complex with definite colour characteristics and the density of the colour must be proportional to the concentration of the substance under determination.

The coloured solutions must obey:

(i) Beer's Law: Light absorbed is proportional to concentration of substance.

(ii) Lambert's Law: Light intensity transmitted is inversely proportional to the length of the light path in solution.

6.41 Visual Methods

Comparison tubes (Nessler tubes): Standard range of concentrations prepared in tall glass tubes with optical flat bases. The unknown is placed in a similar tube and matched to standards by looking down through solution on to white base. The procedure is tedious and time-consuming and not suited to accurate work since standard colours fade.

Colour comparator (Nesslerizer or Tintometer): Based on same principle as comparison tubes but in this case the standard is in the form of a series of coloured glass filters through which a tube of distilled water or the sample without colour-forming reagents is viewed. The sample in a similar tube is compared with the colour discs and the best visual match selected. This method is used to determine colour in water and as a qualitative test for nitrate.

Both the two methods above are subject to personal judgement and individual operators may get quite different results for the same sample. The colour disc method is, however, very suitable for field tests and discs are available for many analyses.

6.42 Instrumental Methods

Absorptionmeter or colorimeter: This consists basically of a photoelectric cell coupled to a millivolt meter calibrated in per cent transmission usually on an expanded range. Light from a standard source, often a low voltage bulb, passes through the sample contained in a glass cell - various sizes of cell extend the range of the instrument. To increase sensitivity, provision is usually made for the insertion in the light path of a coloured filter selected to give the maximum light absorption by the sample, i.e. the filter is the complementary colour to the solution colour. Provision of a large number of colour filters is costly and the wavelength of light passing a filter has a band width of about 60 nm, so that great sensitivity is not possible.

Spectrophotometer: This is an accurate instrument for laboratory determinations. The basic principle is similar to an absorptionmeter, but in place of filters a prism is used to provide monochromatic light of the desired wavelength. In the better instruments a null meter is used for adjustment and per cent transmission or optical density appear on a long scale. With both types of instrument, a blank of the sample without the last colour-forming reagent is used to set the aero optical density (100% transmission) position and is then replaced by the sample. After rebalancing, the optical density of the sample may be read off.

The procedure for colorimetric analysis on a spectrophotometer is first to determine the optimum wavelength for the sample colour, i.e. the wavelength which gives greatest absorption of light. By using the optimum wavelength a series of standard samples of known concentration are used to prepare a calibration chart. Future sample readings can then be read off the calibration chart. In colorimetric analysis it is important to ensure that full colour development has taken place before the sample is examined.

The following determinations are commonly carried out by colorimetric methods at the wavelengths noted:

Parameter	Wavelength
Amm.n	400 - 425 nm
NO_2-N	520
NO_3-N	410
"Detergents" (Methylene blue)	650
Cr	540
PO_4	690

Before carrying out colorimetric analyses, any turbidity in the sample must be removed by centrifugation or filtration and care must be taken to maintain cleanliness of the sample cells.

6.5 Electrodes

The measurement of such parameters as pH and oxygen reduction potential (ORP) by electrodes has been commonplace for many years. pH is normally measured by the potential produced from a glass electrode - an electrode with a special sensitive glass area and an acid electrolyte - used in conjunction with a standard reference electrode. The output from a glass electrode is very small and requires amplification. It is possible to obtain combined glass and reference electrodes so that only a single probe is handled. This allows small volumes of water to be used. The measurement of ORP is achieved using a redox probe with a platinum or gold electrode again in conjunction with a reference electrode. Recent developments in electrode technology have produced the oxygen probe for dissolved oxygen using a lead/silver cell in the Mackereth type or a carbon/silver system in various American types. Such dissolved oxygen probes which may be operated from battery or mains sources are most useful for field studies and for rapid laboratory determinations but their accuracy must always be checked against the Winkler method.

Specific ion electrodes are now available for NO_3, Ca, Na, Cl, Br and F and the availability of such electrodes is proving useful in work on remote water quality monitoring. There are drawbacks, however. For example, hydroxyl ions may interfere when the fluoride ion electrode is used, but their interference can be eliminated by working between pH 5 and 9 where the hydroxyl ion concentration is low (below pH.5 the fluoride ion complexes with the hydrogen ion).

6.6 Significance of results

It is important to realize that there is a random variation in all results which must be allowed for when examining results for significant changes. For important work it is desirable to carry out at least three replicate determinations on each sample and to express the results as a mean concentration, the reliability of the mean being indicated by the standard deviation. The standard deviation (σ) is a measure of the spread of the data and for a series of n observations is given by

$$\sigma = \sqrt{\frac{\Sigma(x - \bar{x})^2}{n - 1}}$$

where x is any single observation and $\bar{x}$ is the mean value of n observations. For small numbers of observations (up to 10) a simplified expression for standard deviation may be used

$$\sigma = \frac{(\text{Difference between highest and lowest results})}{\sqrt{\text{number of results}}}$$

When the standard deviation is relatively large compared with the mean, it follows that the mean value has little significance.

It is the aim of all forms of analysis to produce an accurate and precise answer. Accuracy means the nearness of the results obtained to the true mean; precision indicates the repeatability of the determination. It is quite possible to carry out an analysis which gives precise answers with poor accuracy because of the use of non-analytical grade reagents, etc.

7. Chemical hydrology

7.1 Background information

The chemical content of running water varies enormously from region to region and is a reflection of the local geography and climate. It should also be noted that rainwater is not pure water. It contains appreciable amounts of sodium, potassium, calcium, magnesium, chloride and sulphate plus the dissolved gases, carbon dioxide, oxygen and nitrogen from the atmosphere. Ions originate from: (a) sea spray carried high into the atmosphere, (b) from dust and (c) from air pollution, which contributed sulphate in particular.

The proportions vary with the origin of the air masses contributing the rain. Sodium, chloride and much of the magnesium are apparently derived from sea spray and are carried far inland on to the continents. Calcium, some magnesium and potassium are derived primarily from dust. Dust also contributes organic matter and probably most of the nitrate and phosphate. For example, during an 18 month study in Ontario, rain contributed 6.5 km/ha per year of nitrogen and 61% of this fell on only 25% of the days on which it rained which would seem to indicate that much of it came from dust in the atmosphere.

Rain normally has a fairly low pH because of its high content of carbon dioxide, and because sulphate is often present as free acid. Usually this is neutralized as the water percolates through the soil and picks up calcium and magnesium ions but in boggy areas and in some rain forests, the water enters streams without much contact with mineral soils. Streams in such areas have very low mineral content, not many times that of rainwater, and they tend to be even more acid than rainwater because of base-exchange with the organic soil and the loss of such as were originally present. They are also often brown because of dissolved organic matter.

More usually, however, much water enters streams as subsurface run-off in mineral soil and only when there is very heavy precipitation does a large proportion enter as direct surface run-off. As a result, there is a fairly clear inverse relationship between the discharge and the concentration of dissolved salts in the water. Up to a point the total amount of dissolved material carried off from the landscape as opposed to concentration increases with arrival precipitation. Not everywhere, however, does concentration decline with increasing discharge. Studies, for example, in some small spring-fed woodland streams have shown that chemical conditions can be remarkably constant over long periods of time despite considerable variations in discharge caused by rainfall. Possibly in small spring-fed streams which have not been much altered by man normally present exceptionally stable conditions and do not follow the general rule.

One further point, which is related to the proportion of discharge which is direct run-off, is that in areas of damp climate where the landscape is well covered with vegetation, the proportion of the total erosional load which is carried in the dissolved state, as opposed to particles in suspension, is higher than it is in drier areas. Vegetation decreases run-off and hence the particulate load. It thus allows more time for the water to infiltrate into and through the soil and so to acquire more dissolved matter. It has some importance in relation to man's tendency to clear the land for agriculture and leave it bare for long periods. We are not only making streams more turbid, we may be softening their water to some extent.

7.2 Distribution of dissolved matter in rivers

In the majority of rivers turbulent mixing normally ensures a uniform distribution of dissolved substances although small temperature discontinuities and small differences in chemistry may occur in pools and deep places. The small differences in chemical content in such cases are usually due to biological activity. Often the bacterial count increases with depth and this is negatively correlated with dissolved oxygen content due to respiration; slightly polluted water containing organic matter tends to flow downwards from the banks where it enters and to occupy the lower layers. Another factor which may cause local chemical differences is the activity of plants. Beds of aquatic plants impede water circulation and the flow rate. This can lead to super saturation with oxygen in the day time and to de-oxygenation caused by respiration at night. The effects of the stream flow on the oxygen regime of rivers is a very important factor to be considered in the assessment of establishing chemical standards for sluggish rivers. In some very large rivers lateral differences caused by the entry of tributaries sometimes persist over fairly long distances as the inflowing water tends to follow the bank from which it entered.

7.3 Composition

7.31 Dissolved gases

Apart from differences due to biological factors as noted above, the gases dissolved in running water are usually more or less in equilibrium with the atmosphere. The three most important are oxygen, nitrogen and carbon dioxide; of these nitrogen is not much affected by the biota nor does it enter into complicated relationships with other dissolved substances. It is normally present at about 100% saturation. Oxygen and carbon dioxide levels may fluctuate considerably; they are usually inversely related to one another because of the photosynthetic and respiratory activities of the biota. When the oxygen content of running water is high, the carbon dioxide content is low and vice versa.

Even in torrential streams, oxygen content varies seasonally from source to mouth. Studies in Western Europe show that in autumn there is more oxygen at high altitudes than at low ones and this situation is reversed in winter. This reflects an oxygen demand created by fallen leaves in the autumn and greater photosynthesis at lower altitudes during the colder winter months. As the temperature increases in spring photosynthesis increases and causes supersaturation and this occurs first at lower altitudes and then further and further upstream and then during the summer the percentage saturation declines. Even in the tropics there is evidence from work on Mt. Elgon stream of at least striking biotic changes and presumably differences in oxygen content with change in altitude.

In the larger rivers similar but rather greater variations occur and very often the over-riding factor here is normally simply due to discharge. In the Mississippi and the Amazon for example, high water is accompanied by lowered oxygen concentrations and these are brought about by wash-in of organic matter and the decrease of photosynthesis caused by turbidity. Preliminary analysis of the results of the chemical sampling programme started jointly by the Hydrology and Water Quality Sections on the Nzoia river in Kenya during 1973 indicates a similar trend.

In many streams there is also a diurnal variation in oxygen content. The solubility of the gas varies inversely with the temperature and this would in theory lead to low values (in terms of concentration but not of course in percentage saturation) in the day time and higher values at night; but the daily variation is more, usually in the other direction - high in the day time, usually highest in the afternoon, and lowest at night, shortly before dawn! It is therefore clearly related to photosynthesis and the respiration of organisms. To maintain a clean water fauna in a stream the dissolved oxygen should never fall below about 4.5 mg/l but pollution, which encourages algal growth for example, will increase the diurnal variation range and at dawn the dissolved oxygen can fall to 1.0 mg/l or less in such circumstances. Only where vegetation is extremely sparse and there is no pollution is it likely that a downward trend in the daytime dissolved oxygen level due to temperature will be recorded.

Very often underground water is almost or totally devoid of oxygen and contains large amounts of carbon dioxide. This is because of its exposure to organic matter and bacterial respiration in the soil. Spring water is therefore often totally deoxygenated and downstream of the spring very active exchange of gases with the atmosphere occurs. Frequently water from springs has zero oxygen. On the other hand where underground water flows through large solution-channels, such as in limestone, the oxygen content of the emerging water may be quite high.

Carbon dioxide in high concentration is fairly rapidly lost both to the atmosphere (i.e. why all samples for CO_2 determination must be filled completely to the lid) and by interactions with calcium carbonate. It continues to be produced by respiration within river channels; dams usually reduce the oxygen and increase the carbon dioxide of the water as it passes through them. Even large rivers may display such features; the great swampy area of the Sudd reduces the oxygen and raises the carbon content of the waters of the White Nile and the relatively high content of carbonic acid also markedly lowers the pH.

The rate at which such anomolies are restored to a state of equilibrium between the water and the atmosphere depends upon local factors affecting turbulence and the ruffling of the water surface. Falls and weirs which cause entrainment of bubbles restore lowered oxygen content, although the rate will be slower for polluted waters containing detergents (such as in the Athi River near Thika) by as much as 50%, and also eliminate supersaturation more rapidly than does turbulent flow through channels with the same loss of head. In addition the rougher the stream bed and the larger the surface to volume ratio, the quicker the equilibrium is restored with respect to dissolved gases.

7.32 Major dissolved salts

As stated in the introduction the amount of total dissolved solids in rivers varies with local geography and several studies have been able to relate at least the bicarbonate content of water to the geological structure of their drainage basin.

Most running waters are bicarbonate waters in the limnological sense since they show relationships between pH, CO_2, H_2CO_3, H^+, $CO_3^=$, HCO_3^-, Ca^{++} and Mg^{++}. For example:

(i) Rain water reaching watercourses as run-off from bogs, dense forest and litter has a low pH because of (a) hydrogen ions produced by dissociation of carbonic acid and (b) loss of cations by base exchange with organic matter.

(ii) Water which has percolated through the soil is also rich in carbon dioxide and similarly tends to be rich in hydrogen ions.

$$H_2O + CO_2 \longrightarrow H_2CO_3 \rightleftarrows H^+ + HCO_3^-$$

(iii) Calcium carbonate which is a common constituent of many rocks is almost insoluble in water but it dissolves fairly readily, as bicarbonate, in carbonic acid, and it neutralizes the soil water where it occurs.

$$CaCO_3 + H_2CO_3 \rightarrow Ca(HCO_3)_2 \rightleftarrows Ca^{++} + 2HCO_3^-$$

(iv) Calcium bicarbonate in solution is a good buffer system and thus resists changes in pH and it remains in solution only in the presence of a certain amount of free carbon dioxide, the so-called equilibrium CO_2. Any process which removes CO_2 as does photosynthesis, tends to cause precipitation of calcium carbonate from solution, especially where the bicarbonate is abundant.

It can be understood, therefore, that spring water in limestone regions is very often rich in calcium bicarbonate where it emerges to the surface. As it flows along it loses carbon dioxide to the atmosphere and by photosynthesis.

To sum up then, if the source is from a very calcareous rock, the water will be heavily charged with calcium bicarbonate. It will lose CO_2 rapidly and its pH will rise. After some distance the loss of CO_2 to atmosphere and by plant photosynthesis will lead to deposition of $CaCO_3$, but the process will decline as equilibrium is attained. In this well-buffered hard water, the pH will not rise above about 8.3 even at times of very active photosynthesis.

Water from a non-calcareous spring will also lose CO_2, increase its pH and acquire oxygen in a short distance. If the source is a swampy woodland, swamp or mine drainage, the water may contain ferrous bicarbonate. As the pH rises and oxygen is acquired this will be deposited as ferric hydroxide and this part will be coated with masses of iron bacteria. Iron will only remain in solution if the water remains acid. If as probable, the water is soft, as are most rivers in Kenya, its pH will fluctuate markedly because of photosynthesis and it may exceed 8.3 in the day time. The Chania River at the intake to Nairobi's mid-Chania supply has a marked diurnal variation and during late afternoon the pH can rise to 9. Close supervision at the water treatment plant is necessary. Many soft-water streams are brownish because of dissolved organic matter and they remain so until the stream acquires a richer supply of inorganic irons.

7.33 Plant nutrient salts

The three important ions needed for plant growth are potassium, nitrate and phosphate and in addition trace amounts of a long list of other elements are also necessary. Only exceptionally are the latter ever in short supply.

Potassium is a common constituent of many minerals and is always present in considerable amounts in all but the most unmineralized waters. At no time has it been supposed that it is a factor limiting plant growth in natural waters. This leaves nitrogen and phosphorous as the two plant-nutrient elements which are most likely to be important.

Nitrogen can exist in saline form as ammonium, nitrite or nitrate, and of these the last is most available for plant growth. It is also the fully oxidized form (NO_3), and so except under conditions of pollution, it is the normal one which occurs in rivers. 66 samples collected in 1973 from unpolluted streams in Central and Western Kenya over a wide range of flow conditions give the following figures:

Table I

Test	Mean concentration (mg/l)
NH_3	NIL
NO_2	NIL
NO_3	1.0

Phosphorous is more complex since it occurs both as simple ionic orthophosphate and as bound phosphate in soluble and particulate form. In lakes only 10% or less is present as simple ions and these are rapidly taken up by plants. However, bound phosphate is continuously released by bacterial action so in turbulent water fertility is measureable in terms of total phosphate.

The main sources of nitrate and phosphate in rivers are rainfall and the land surface. Rain contributes fairly large amounts of nitrate and drainage from agricultural land also produces very large quantities of nitrate and phosphate. The steeper the slope the greater the contribution (data from arable land in Wisconsin):

Table II

Slope	Nitrogen (kg/ha)	Phosphorous (kg/ha)
20°	34	1.6
8°	16	0.45

The amounts contributed by non-agricultural land are generally smaller than the above figures obtained from cultivated land but nevertheless the quantities involved are not inconsiderable. Most enters in particulate form and especially in cultivated areas through soil erosion. A new source of phosphate entering rivers and lakes is that source derived from the breakdown of synthetic detergents. In developed countries this is frequently now the major source.

Some blue-green algae can also fix molecular nitrogen and they are especially common in eutrophic (i.e. nutrient enriched waters). Some of the Rift Valley lakes, especially Lake Nakuru, are very autrophic. From a water supply point of view many algae which occur in eutrophic water are a nuisance since they produce unpleasant tastes in the water and some can produce toxins.

7.34 Organic matter

All natural surface waters contain dissolved and particulate organic matter. Normally the quantity is assessed from the Biological Oxygen Demand test or the 4 hour permanganate value. In the humid areas of Kenya where the self-purification rate is high, the oxygen demand from potassium permanganate is 2 mg/l but this contrasts with much higher figures from sluggish rivers in the more arid regions, particularly those receiving swamp drainage where the oxygen demand can reach 30 mg/l (e.g. River Trukwell). It seems fairly clear that the oxygen demand of the natural dissolved and particulate organic matter accounts in large measure for oxygen concentrations of less than 100% saturation now recorded from several unpolluted rivers by the Water Quality Section here in East Africa.

8. Collection of samples for subsequent chemical analysis

Hydrological Assistants may in the future be increasingly asked to collect water samples especially from rivers for a Water Quality Control Section. These notes are intended only as a guide to sampling. It is anticipated that any officer involved in a river chemical survey as also those involved in ground water sampling will be briefed on the objects of the survey before hand and given detailed instructions regarding sampling and analysis, any on-the-spot preservation required, etc. Water Bailiffs will probably be more often involved in being asked to occasionally take a sample from a factory or a spot river sample; they should pay particular attention to the labelling instructions outlined.

Samples should be taken in clean, colourless glass bottles provided with ground-in glass stoppers. At least 2 litres is required for a routine chemical analysis. 10 litres are required for Jar Test. Rinse out at least three times with the water that is to be sampled before it is filled. Polythene bottles may be substituted for glass but should never be used if it is planned to subsequently look for pesticides in the sample.

The mean velocity of a stream on any vertical is usually about 0.6 h from the surface (where h = depth). Try to sample approximately at this depth. In most rivers, since turbulent conditions occur, it should not be necessary to sample at different depths or along the cross-section of the river, but it will depend on intelligent observation. If you are taking regular samples, then periodically take two identical samples so that the laboratory can check the degree of their analytical accuracy. In shallow streams care should be taken to see that a representative sample containing the true proportion of suspended matter to liquid is obtained so that any bottom deposit or sewage fungus growing on the bed is excluded.

The label on the bottle should give a very <u>detailed</u> account of the sampling point, ideally with grid location also. If taking a sample from an outfall, give its internal diameter and side of stream (e.g. looking downstream - either right and left bank) and distance from an obvious landmark (e.g. 100 metres downstream of Uganda Railway Bridge over River Nzoia at Webuye). Record any measurements taken at the time of sampling on the label also (e.g. temperature).

The samples should be sent to a Water Department Water Laboratory with as little delay as possible and the samples kept cool (below $5^{o}C$) during transport (e.g. by storing in an ice-box). Chemical analysis should be started as soon as practicable after the collection of sample. Many tests require that the samples must be completely filled to the brim, and for such determination as B.O.D. and CO_2 the test is invalid if there is an air-gap between the level of the sample and the stopper.

Certain examinations should be carried out at the time of collection: temperature, pH, residual chlorine, dissolved oxygen. For some other determinations, it may be necessary to take a separate sample and "fix" at the time of collection if the sample cannot be taken to the laboratory at once (e.g. for iron nitrite, add 1 ml of concentrated sufuric acid for each litre of water).

9. Eutrophication

The enrichment of water systems with plant nutrients is known as eutrophication. This also generally implies an increase of primary production in water, i.e. the production of photosynthetic higher plants (macrophytes) and algae. As is the case with agricultural production on land, an increased production in rivers and lakes is usually considered advantageous by man, so long as it brings about an increased yield of organisms, such as fish, that he can utilise. Often, however, human activities in the watersheds of rivers and lakes have increased the input of nutrients to the waters to such an extent that eutrophication has become uncontrollable and unfavourable.

The nutrients that are considered usually responsible for the onset of eutrophication are phosphorus (P) and nitrogen (N). These elements are essential for the production of all living material, but are usually present in low concentrations in natural fresh water, while other necessary elements such as carbon, potassium, sulphur and trace elements in general are available in excess.

The undesirable effects of eutrophication, which have become a menace in many countries in Europe and America, include heavy growth of vegetation along the edges of lakes and rivers, increased biomass of planktonic algae, and accumulation of decaying organic material on the bottom of lakes. The latter can cause lowered oxygen concentrations in the water, resulting in complete changes in animal communities. If the water is being used for domestic purposes, the abundance of planktonic algae may cause troubles in the waterworks, clogging the filters and making the water bad tasting or even poisonous. Toxins produced by certain algae found under eutrophic conditions can kill aquatic life and livestock drinking the water. It has also been demonstrated that there is a relationship between eutrophication and an increase in potential fish pathogenic bacteria in an English lake. Furthermore, high concentrations of nitrogen as nitrate are known to cause methaemoglobinaemia in infants who are fed on artificial diets when nitrate/nitrite nitrogen is higher than 10.2 mg/l in the water utilised for preparing such diets. Consequently, several countries have laid down standards on allowable nitrate concentration in drinking water.

Plants are able to take up phosphorus only in the form of phosphate, but organic phosphorus compounds are oxidized to phosphate by micro-organisms in the water, making practically all phosphorus that enters the water a potential plant nutrient. The natural source of phosphorus to lakes and rivers is solution from the surrounding bedrocks and soils, together with organic material washed out of the watershed. Therefore the geology and vegetation type in an area have a determining influence on the rate of phosphorus transport from land to the drainage water.

Nitrogen is assimilated by the water plants mainly in the forms of nitrate or ammonia. The origin of nitrogen compounds entering the surface waters is partly the same as for phosphorus (i.e. soils and vegetation in the watershed), but for nitrogen there are also additional sources. Some atmospheric nitrogen is oxidized to nitrate in connection with lightning. The nitrates formed enter the rivers and lakes dissolved in rainwater. Atmospheric nitrogen is also utilised directly by some nitrogen fixating blue-green algae in the water.

The input of phosphorus and nitrogen to the waters has drastically increased due to human activities. Domestic wastewater is very rich in these nutrients, and the use of rivers and lakes as receiving waters for wastewater is one of the main reasons for the man-caused eutrophication. Furthermore, wastewater from several types of industries contain high concentrations of nutrients which also contribute to the eutrophication of water systems. Also, agricultural activities, including deforestation, use of fertilizers and holding of livestock, are responsible for the increased transport of nutrients to lakes and rivers.

10. Pesticides

Pesticides are chemicals used to control insects and other organisms in order to protect agricultural crop or livestock. The pesticides can be divided into several groups as regards their chemical composition. The predominating groups are: chlorinated hydrocarbons, organo-phosphorus pesticides and carbomates. Of these three groups the chlorinated hydrocarbons, which comprises DDT, Aldrin, Dieldrin, Lindane and several others are generally more persistent than the other types of pesticides. They are also far more soluble and are thus likely to be accumulated in biological systems. This tendency of the pesticides of the chlorinated hydrocarbon type to accumulate in organisms has caused severe problems in countries where pesticides have been used extensively over long periods. High pesticide concentrations have been detected in wildlife and in several cases the residue levels have been high enough to be lethal. However, sublethal concentrations can also have severe consequences as, for example, for the peregrine falcon and other birds of prey in Europe. These birds, being the final consumers in frequently very long food chains are especially susceptible to this kind of chemicals that accumulate in higher concentrations in every step in the food chain. It has been shown that the high levels of pesticide residues and PCB (poly chlorinated biphenyles - an industrial product with environmental side effects resembling those of the persistent pesticides) in the eggs of the peregrine falcons (Falco peregrinus) in Britain reduced the breeding results. As a consequence, the population of peregrine falcons in Britain decreased in the middle of this century until the use of pesticides was restricted in the sixties.

In parts of Kenya there is an extensive and probably increasing use of different kinds of pesticides although the exact figures of the total amount spread annually seem to be difficult to obtain. The experiences of unwanted effects of pesticides in Europe and America have made the scientists worry about the possible contamination of the African environment. There has also been the fear that certain pesticides, which have been banned in other countries might be "dumped" in Kenya. As a result some recordings of pesticide residue levels in the East African environment have already been done.

In Kenya's national report to the United Nations on the human environment (1972), an investigation of pesticide residue levels in adipose tissues of humans and baboons is reported. The total DDT in humans aged 25 - 40 years averaged a mean value of 4.60 ppm and in the baboons the total DDT averaged 0.07 ppm. Also residues of dieldrin and BHC were found in all the samples. The results show that living organisms including man himself is exposed to contamination of pesticides even if, perhaps, the levels are still not alarmingly high, as yet.

Similar surveys of residue levels in adipose tissues of humans have been carried out in several countries. The levels in Kenyan humans are approximately the same as in humans in France, but higher than the levels found in several other developed countries in temperate areas. On the other hand countries like Poland, Hungary, Italy, Spain and Israel have 3 -4 times the concentrations in Kenya. On top of the list is India where the average DDT concentration of human adipose tissues was 26 ppm. The different levels found in different countries reflects partly the amount of pesticides used in different parts of the world.

11. Glossary of wastewater terms

ACTIVATED SLUDGE PROCESS: The activated sludge process is a method of biological treatment which produced a high quality effluent. It is a secondary process, usually following primary treatment. The activated sludge removes the finely divided, suspended and dissolved organic matter remaining in the wastewater. When we talk about activated sludge, we are referring to the biological communities of micro-organisms which are developed in the aeration tank. If supplied with enough dissolved oxygen, they will aerobically decompose organics in the wastewater. Activated sludge is settled from wastewater and returned to the aeration tank for reuse.

ADSORPTION: One of the advanced wastewater treatment methods. It involves the removal of a pollutant, such as organics, by making it stick to the surface of a solid, like carbon. (ADsorption is not to be confused with ABsorption. In absorption, one substance is taken into the body of another, like water into a sponge).

AEROBIC BACTERIA: Bacteria which live and reproduce only in an environment containing dissolved oxygen.

AEROBIC DECOMPOSITION: Biological decomposition or decay of organic material by aerobic and facultative microorganisms in the presence of dissolved oxygen.

AEROBIC DIGESTER: A digester designed to make use of aerobic-process bacteria to decompose and reduce the volume of sludge to be handled. The sludge is continuously aerated without the addition of new food other than the sludge itself. After about 20 days, the material is considered stable enough for ultimate disposal.

AEROBIC PONDS: A very shallow waste stabilization pond that has dissolved oxygen throughout it so that treatment can occur by aerobic decomposition.

AEROBIC-PROCESS BACTERIA: These are the bacteria that cause aerobic decomposition. Conditions are provided so that it is the aerobic and facultative bacteria that do the work.

ANAEROBIC BACTERIA: Bacteria that live and reproduce in an environment containing no dissolved oxygen. Anaerobic bacteria obtain their oxygen supply by breaking down chemical compounds which contain no dissolved oxygen.

ANAEROBIC DECOMPOSITION: Decomposition and decay of organic material in an environment containing no dissolved oxygen.

ANAEROBIC DIGESTER: A digester designed to make use of anaerobic-process bacteria to decompose the sludge and reduce the volume of sludge to be handled. The anaerobic digester is completely sealed so that no air gets into it.

ANAEROBIC PONDS: A deep waste stabilization pond. These ponds usually don't have any dissolved oxygen throughout the liquid, and anaerobic decomposition occurs. These ponds can smell pretty badly, and are thus restricted in use.

ANAEROBIC-PROCESS BACTERIA: These are the bacteria that live and work when there is no dissolved oxygen present. Anaerobic bacteria, and facultative bacteria in the anaerobic state, result in anaerobic decomposition.

BACTERIA: Bacteria are living organisms, microscopic in size. Most bacteria use organic matter for their food. Under proper conditions, they multiply with great rapidity.

BAR SCREEN: A bar screen is usually a set of parallel bars placed at an angle in a channel so that the wastewater flows through the bars. Larger objects entering the plant collect on these bars and can be removed. Screening is usually the first step in pretreatment.

BIOCHEMICAL OXYGEN DEMAND (B.O.D.): A measure of the amount of organics, or food, available in the wastewater.

CHLORINATION: This refers to the addition of chlorine, usually to treated wastewater in a container called a chlorine contact tank or basin. The main purpose of these tanks is to provide sufficient mixing and sufficient contact time for the disinfection to take place.

CHLORINE CONTACT TIME: In the chlorine chamber or tank, the chlorine must have enough time in contact with the bacteria to be able to kill them. The amount of time that we give for this is called the chlorine contact time. Usually the contact time is 20 to 30 minutes, although more time is not unusual.

CHLORINE RESIDUAL: The amount of chlorine to be used for disinfection depends on how much treatment the wastewater has already received. Usually we add enough chlorine so that there will be, at the least, 0.5 parts per million of chlorine in the mixture after 20 minutes of contact time. The actual amount and contact time will be specified by the regulatory agency.

CLARIFIER: Sometimes also called a settling tank or sedimentation basin. It is a tank or basin in which wastewater is held for a period of time, during which the heavier solids settle to the bottom. Lighter materials which float to the water surface are removed by a surface skimming device. If the unit is placed as part of primary treatment, it is called a primary clarifier. Similarly, clarifiers in the secondary treatment phase are called secondary clarifiers.

COAGULATION-SEDIMENTATION: This is a more complex physical-chemical wastewater treatment process, which uses chemicals to help settle out some of the smaller particles in the wastewater.

COLLECTION SYSTEM: The network of sewers collecting wastewater from the community and bringing it to your plant is called the collection system.

COMBINED SEWAGE: Combined sewage is a mixture of municipal (or sanitary) sewage and storm waters when both are collected in the same sewers.

COMMINUTOR: Besides screening and grit removal, pre-treatment also usually involves shredding. There are machines available, like the comminutor, that cut up or shred material while it is still in the wastewater stream. The cutup material is left in the wastewater.

COMPOSITE SAMPLES: When samples are collected at regular intervals (e.g. hourly), and then combined, a composite sample is obtained. This reduces the effect of variation in individual samples. Individual samples may have equal volumes, or may be sized in proportion to the flow at the time of sampling.

CONTACT STABILIZATION: In this variation of the activated sludge process, pre-treated wastewater is mixed with return activated sludge in a contact basin for about 30 minutes to one hour. Then the solids are allowed to settle. The resulting supernatant is the plan effluent. The resulting sludge is then returned to the contact basin.

DETENTION TIME: The theoretical time required for a given flow of wastewater to pass through a tank, or the amount of time that the wastewater is in the tank.

DEWATERING: This is done to reduce the volume of the sludge that requires further handling. For example, digested sludge could be spread on sand drying beds to allow the water to drain from the sludge.

DIGESTER: A tank in which sludge is placed to allow sludge digestion to occur. Digestion may occur under anaerobic or aerobic conditions.

DIGESTER GAS: Both aerobic and anaerobic digesters produce gases. As the anaerobic digester is usually completely sealed, these gases are important, especially since anaerobic digester gas is about 70% methane and 30% carbon dioxide. When mixed with air, digester gas is extremely explosive. The methane gas is often used as the energy source for heating the plant.

The main gas produced in aerobic digestion is carbon dioxide. The gases produced are not offensive, and therefore it is only necessary to cover aerobic digesters for heat preservation.

DIGESTION: Biological decomposition of the organic matter within the sludge without the addition of new food. Digestion reduces sludge volume, and makes the sludge easier to dewater. Properly digested sludge is stable and inoffensive.

DISINFECTION: By disinfection, we mean the killing of microorganisms in the water. The point is to kill the pathogenic organisms or bacteria that are harmful to man. Note that not all of the organisms in the water are killed by disinfection. Any process that produces a harsh environment for the organisms can be used for disinfection. Most plants use chlorine.

DISSOLVED OXYGEN (D.O.): Atmospheric oxygen dissolved in water or wastewater. It is usually abbreviated D.O.

DISSOLVED SOLIDS: Dissolved solids are the ones that are actually in solution in the liquid. In normal domestic wastewater, just about half of the dissolved solids are organic, and the rest of the dissolved solids are inorganic.

DOMESTIC WASTE: These are human and household wastes. Human wastes are the most important in terms of public health because they may contain organisms which produce disease in man. Household wastes include that from laundry, bathing, washing, cooling foods, and dishwashing. Most of these particles contain soap. Kitchen wastes also have particles of food or grease that enter the wastewater system.

DOMESTIC SEWAGE: Domestic sewage is that containing human and household wastes. This is the kind of wastewater coming from residential areas where there is little or no industry.

DOWNSTREAM USER: Treated plant effluent usually goes into a river, lake or stream. Users of this water downstream from your plant are called downstream users.

DRYING BEDS: Sludge can be dewatered by spreading the sludge on specially constructed beds of sand or fine stone. The water drains through this bed, leaving the sludge on top.

EFFLUENT: Wastewater or other liquid - raw, or partially or completely treated - flowing from a tank, treatment process, or treatment plant.

ELECTRODIALYSIS: An advanced physical-chemical wastewater treatment method that works through the use of electricity.

EXTENDED AERATION: This activated sludge treatment variation is usually provided for a small-sized treatment facility. Whereas contact stabilization has a relatively short aeration time, in extended aeration the liquid is aerated longer. This means that the amount of sludge produced in this system is reduced, because more time is allowed for decomposition.

EVAPORATION: When moisture evaporates, it changes into water vapour. This process is called evaporation.

FACULTATIVE BACTERIA: Facultative bacteria can use either dissolved oxygen or oxygen obtained from food materials. In other words, facultative bacteria can survive and function actively in both the aerobic or anaerobic conditions.

FACULTATIVE PONDS: The most common type of pond in current use. The upper portion is aerobic, while the bottom layer is anaerobic. Algae supply most of the oxygen to the upper layer.

FLOC: Larger particles that have formed from the coming together or joining of smaller particles as a result of biological or chemical activity.

FLOCCULATION: An action resulting in the gathering of fine particles to form larger particles that are heavier and more easily settled.

GRAB SAMPLE: When you collect all of your testing or sample liquid at the same time, it is called a grab (or catch) sample). Grab samples can only give you information about the liquid at the location and time of sampling. Grab samples should be collected at the same time and location every day, so that comparing results for each day is more accurate.

GRIT: The heavy inorganic material present in wastewater. Examples are sand and gravel. Settled solids in the grit chamber are called grit.

GRIT CHAMBER: Grit removal is accomplished by providing an enlarged channel or tank, called a grit chamber, which causes the flow velocity to be reduced and allows the heavier grit to settle to the bottom of the channel where it can be removed.

GROUNDWATER INFILTRATION: Sewers are often below groundwater level, and if the joints between sections of the sewers are not tight or if there are cracks in the sewers, groundwater might be entering into the sewers.

INDUSTRIAL WASTES: Industrial wastes are another important part of wastewater. In many areas, industrial or manufacturing wastes are collected with other community wastewater for treatment and disposal. Sometimes the amount and kind of waste will require separate collection and disposal systems.

INFLUENT: Wastewater or other liquid - raw or partially treated - flowing into a reservoir, tank, treatment process, or treatment plant.

INORGANIC SOLIDS: Waste material such as sand, salt, iron, calcium, and other mineral materials which are not converted in large quantities by organic action. Inorganic wastes are substances of animal or vegetable origin and contain carbon and hydrogen along with other elements.

MEDIA (TRICKLING FILTER): The material in a trickling filter over which settled wastewater is sprinkled and then flows over and around during treatment. Slime, consisting of micro-organisms, grows on the surface of the media and treats the wastewater. Rock or plastic media are most commonly used.

MICROORGANISMS: Besides other things, wastewater also contains countless numbers of living organisms, most of them too small to be seen except under a microscope. These are of two general types: bacteria, and other more complex living organisms.

MIXED LIQUOR: The contents of the aeration tank as well as the aeration tank effluent is referred to as the mixed liquor.

MUNICIPAL SEWAGE: Municipal sewage contains all of domestic sewage, as well as some of the industrial wastes.

ORGANIC SOLIDS: Waste material which comes from animal or vegetable sources. Organic waste generally can be consumed by bacteria and other small organisms as food.

OXIDATION DITCH: This is a variation of the extended aeration process. Raw or pre-treated wastewater flows to an oxidation ditch which is usually a race-track affair. Cylindrical brush rotors in the oxidation ditch turn, aerating the liquid and causing it to move within the track. The liquid from the oxidation ditch flows to a final clarifier. Activated sludge from the final clarifier is returned to the oxidation ditch.

PARTS PER MILLION (P.P.M.): Our example in the programme was the number of red dots for every million yellow dots. P.P.M. refers to the number of units of the smaller component for each million units or parts of the larger component. In wastewater terminology, one P.P.M. is roughly equal to one milligram per litre (one mg/l).

PATHOGENS: Microorganisms which are harmful to man. Bacteria or viruses which can cause disease like typhoid, cholera, dysentery and others. (There are many types of bacteria which do not cause disease and which are not called pathogens).

POLLUTION: Any interference with use or reuse of water, or failure to meet quality requirements.

PRE-CHLORINATION: Chlorination at the headworks of the plant; influent chlorination prior to plant treatment. Primarily used to reduce odor problems and prevent septic conditions from upsetting later treatment processes.

PRE-TREATMENT: Use of racks, screens, comminutors, and grit removal devices to remove metal, rocks, sand and similar materials which may hinder operation of a treatment plant.

PRIMARY CLARIFIER: The clarifier or sedimentation tank included as part of the primary treatment process.

PRIMARY TREATMENT: A wastewater treatment process consisting of a rectangular or circular tank which allows those substances in wastewater that readily settle or float to be separated from the water being treated.

PUMPING STATION: At points in the wastewater collection system where it is not possible to provide sufficient slope from the source of the wastewater to the treatment plant, pumping stations are used to pump the wastewater up so that gravity flow can start again. For example, a pumping station would be used to move wastewater over a hill.

RECEIVING WATER: A stream, river, lake, or ocean into which treated or untreated wastewater is discharged.

REPRESENTATIVE SAMPLE: A portion of material or water identical in content to that in the larger body of material or water being sampled.

RESIDUAL CHLORINE: Residual chlorine is the amount of chlorine remaining after a given contact time and under specified conditions.

RETURN ACTIVATED SLUDGE: Sludge collected in the secondary clarifier of the activated sludge process has a large number of hungry or active bacteria in it, and can be used again. The activated sludge collected from the clarifier is returned to the aeration tank, where it is again mixed with incoming wastewater and air. Return activated sludge is an important part of the activated sludge process.

SAMPLE: See representative sample.

SAMPLING DIPPER: A sampling dipper has a long handle to make it easier to collect samples from tanks and channels.

SAMPLING POINTS: These are the places from which you sample. They should be places where the liquid is well mixed.

SCREENINGS: Refer to the materials collected on the bar screen. They are removed from the bar screen either mechanically or by hand, and disposed of either by burning or burial.

SCUM: Refers to that portion of the wastewater that floats to the surface. It is composed mainly of grease and oil, and is removed in the clarifier.

SECONDARY CLARIFIER: The clarifier or sedimentation tank included as part of the secondary treatment process is called the secondary clarifier.

SECONDARY TREATMENT (BIOLOGICAL): Secondary treatment consists of two processes. The first process biologically decomposes or converts dissolved or suspended organic materials into a more settleable form. The second process separates the solids from the liquid. The remaining liquid becomes the effluent. Some systems return the biological solids to the first process for reuse.

SEDIMENTATION: Refers to the settling out of suspended materials from a liquid.

SEPTIC: A condition produced by the absence of dissolved oxygen in an aquatic environment. This promotes the growth of anaerobic process organisms. The wastewater turns black, giving off foul odors.

SEWAGE: see wastewater.

SLUDGE BLANKET: In a properly operated secondary clarifier, the solids that have flocculated settle to the bottom. This produces a layer of light, fluffy solids which extend from the bottom of the clarifier for several feet. This layer is called the sludge blanket.

SLUDGE DIGESTER: see digester.

STERILIZATION: Refers to the killing off of all microorganisms in the liquid. In wastewater treatment, we disinfect rather than sterilize. That is, not all of the microorganisms in the liquid are killed off.

STREAM ASSIMILATION CAPACITY: Refers to the amount of waste that a stream or other receiving water can handle or deal with through natural processes without significant undesirable effects on the receiving water.

STORM WATER: Refers to the rainwater which is not absorbed by the ground. If this rain or storm waters enters the sewer system, and your plant is not designed to handle such flows, you could run into treatment problems.

SUPERNATANT: When solids settle to the bottom of a container, the clear liquid formed in the upper portion is the supernatant. If a scum layer also develops, then the supernatant is the clear liquid between the settled and floating solids.

SUSPENDED SOLIDS: Suspended solids, both organic and inorganic, are the ones that are in suspension in the water. Most of these solids will settle if allowed to stand. Different standing times will allow different amounts of suspended solids to settle.

TERTIARY TREATMENT: Refers to treatment phases or processes that follow secondary treatment. They are often referred to as advanced or physical-chemical wastewater treatment methods.

30-MINUTE SETTLING TEST: Well-mixed liquor is put into a one-litre graduated cylinder and allowed to sit for 30 minutes. After this time, the amount of sludge settled is measured. The top part is the supernatant. This test gives some information on the settleability and nature of the sludge.

TRICKLING FILTER: A treatment process in which the wastewater trickles over media that provides the opportunity for the formation of biological slimes which treat the wastewater.

WASTE STABILIZATION POND: A basin used for wastewater treatment by biological decomposition of organic matter. The kind of treatment activity occurring depends on the depth of the pond. Aerobic ponds are too shallow to be common in Canada. Deep ponds (8-12 feet) usually work by anaerobic decomposition. Because of their smell, they are usually only used in sparsely populated areas. Ponds that have an aerobic top layer and an anaerobic bottom layer are called facultative ponds; they are the most common. Waste stabilization ponds that have oxygen mechanically introduced are called aerated ponds. The effluent from waste stabilization ponds is usually quite low in bacteria, especially if the effluent flows from one pond to another. A long detention time, usually a month or more, is needed, and disinfection may also be required.

WASTEWATER: Refers to all waterborne waste materials. Normal domestic wastewater is a mixture of about 99.9% water and 0.1% solids. Just about half of the dissolved solids are organic, and the other half inorganic. Normal domestic sewage contains inorganic solids, gases, and microorganisms as well. Fresh wastewater is usually gray in colour and has a musty, not unpleasant smell.