LECTURE NOTES

ON

LAND SURVEY-II (TH. 1)FOR

DIPLOMA IN CIVIL ENGINEERING

(6^{th} SEMESTER STUDENTS)

AS PER SCTE&VT SYLLABUS

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TACHEOMETRIC SURVEYING Dt. 23.04.21

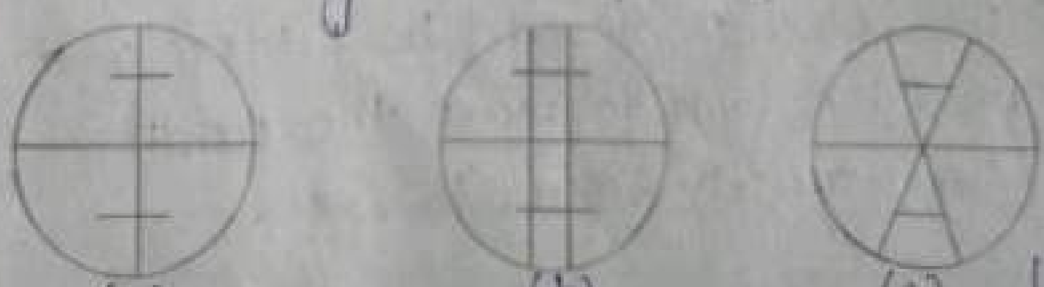
INTRODUCTION

Tacheometry is a branch of surveying in which horizontal and vertical distances are determined by taking angular observations with an instrument known as tacheometer. The chaining operation is completely eliminated in such survey. Tacheometric surveying is adopted in rough and difficult terrain where direct levelling and chaining are either not possible or very tedious. It is also used in location survey for railways, roads, reservoirs, etc. Though not very accurate, tacheometric surveying is very rapid and a reasonable contour map can be prepared for investigation works within a short time on the basis of such survey.

(1) Instruments used in tacheometry :-

(a) The Tacheometer :-

It is nothing but a transit theodolite fitted with a stadia diaphragm and an anallatic lens. figure 11.1 shows the different forms of stadia diaphragm commonly used :



(a) (b) (c) (11.1)

(b) The levelling staff and stadia rod :-

For short distances, ordinary levelling staves are used. The levelling staff is normally 4m long, and can be folded into three parts. The graduations are so marked that a minimum reading of 0.005 or 0.001 m, can be taken.

For long sights a specially designed graduated rod is used, which is known as a stadia rod. It is also 4m long, and may be folded or telescopic. The graduations are comparatively bold and clear and the minimum reading that can be taken as 0.001 m.

(2) Characteristics of tacheometer :-

(a) The value of the multiplying constant $\frac{f}{i}$ is should be 100.

(b) The telescope should be powerful, having a magnification of 20 to 30 diameters.

(c) The aperture of the objective should be of a 35 to 45 mm diameter for there to be a bright image.

(d) The telescope should be fitted with an anallatic lens to make the additive constant $(f+d)$ exactly equal to zero.

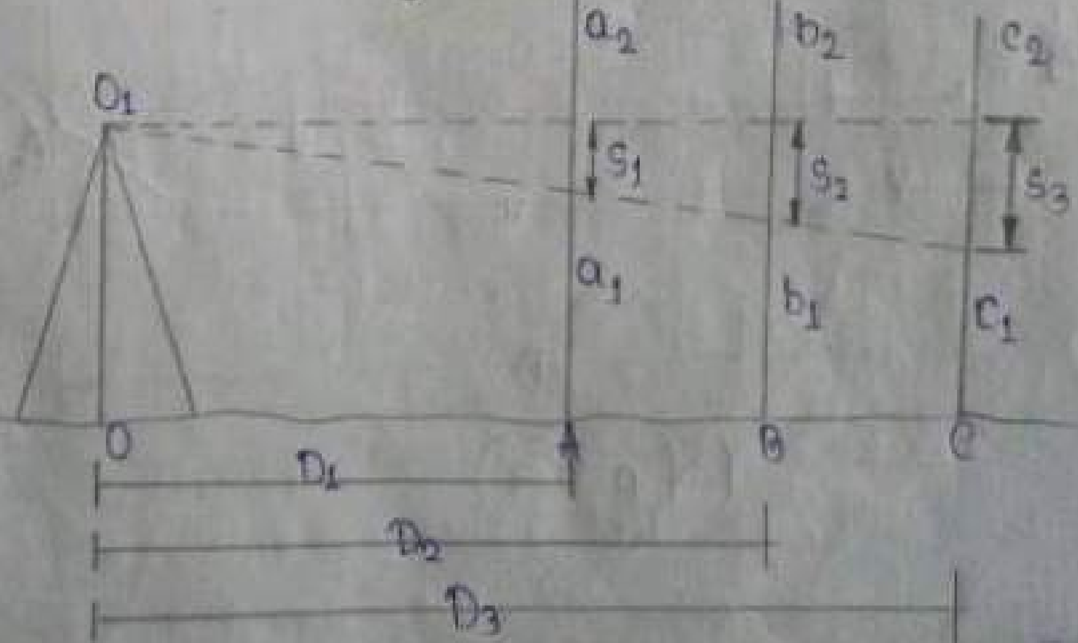
(2) The eye-piece should be of greater magnifying power than usual, so that it is possible to obtain a clear staff reading from a long distance.

(3) principle of Tacheometry :-

The principle of tacheometry is based on the property of isosceles triangles, where the ratio of distance of the base from the apex and the length of the base is always constant.

In fig. 11.2 $O_1a_1a_2$, $O_1b_1b_2$, and $O_1c_1c_2$ are all isosceles triangles where D_1, D_2 and D_3 are the distances of the bases from the apices and s_1, s_2 and s_3 are the lengths of the bases (staff intercepts).

fig 11.2



so, according to the stated principle,

$$\frac{D_1}{S_1} = \frac{D_2}{S_2} = \frac{D_3}{S_3} = \frac{f}{i} \text{ (constant)}$$

The constant $\frac{f}{i}$ is known as the multiplying constant.

where,

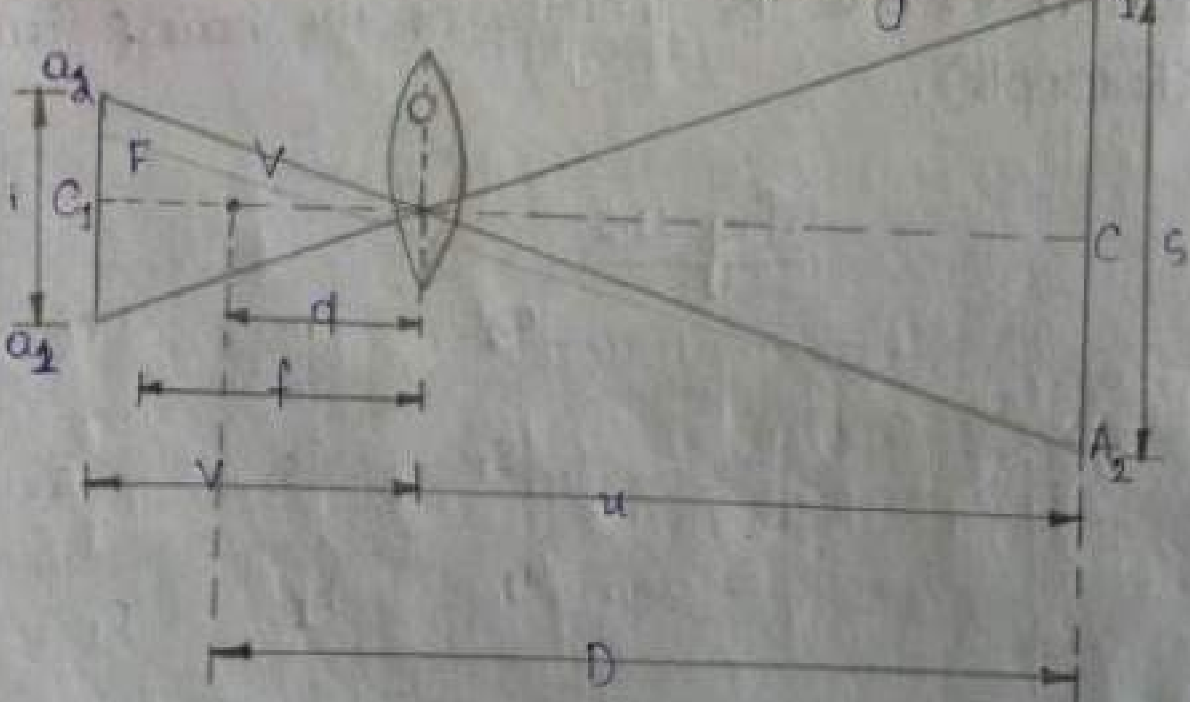
f = focal length of objective

i = stadia intercept

THEORY OF STADIA TACHEOMETRY

The following is the notation used in stadia tacheometry. (fig-11.3)

O = optical centre of object glass



(fig 11.3)

A_1, A_2, C = readings on staff cut by these hairs

a_1, a_2, c = bottom, tops and central hairs of diaphragm.

$a_1 a_2 = i$ = length of image

$A_1 A_2 = S$ = staff intercept

F = Focus

V = vertical axis of instrument

f = focal length of object glass

d = distance between optical square centre and vertical axis of the instrument.

u = distance between optical centre and staff

v = distance between optical centre and image

from similar triangles $a_1 O a_2$ and $A_1 O A_2$,

$$\frac{i}{S} = \frac{v}{u} \quad \text{or} \quad v = \frac{iu}{S}$$

from the properties of lenses,

$$\frac{1}{v} + \frac{1}{u} = \frac{1}{f}$$

Putting the value of v in Eq. (1)

$$\frac{1}{iu/S} + \frac{1}{u} = \frac{1}{f}$$

(OR)

$$\frac{s}{iu} + \frac{1}{u} = \frac{1}{f}$$

(or)

$$\frac{1}{u}\left(\frac{s}{i} + 1\right) = \frac{1}{f}$$

(or)

$$u = \left(\frac{s}{i} + 1\right) f$$

But,

$$D = u + d$$

So,

$$D = \left(\frac{s}{i} + 1\right) f + d$$

$$= \frac{s}{i} \times f + f + d$$

$$= \left(\frac{f}{i}\right) \times s + (f + d)$$

The quantities (f/i) and $(f+d)$ are known as tacheometric constants. $\left(\frac{f}{i}\right)$ is called the multiplying constant, as already stated, and $(f+d)$ the additive constant. By adopting an anallatic lens in the telescope of a tacheometer the multiplying constant is made 100, and the additive constant zero. However, in some of tacheometers the additive constants are not exactly zero, but vary from 30 cm to 60 cm (which are generally mentioned in the catalog supplied by the manufacturer.

DETERMINATION OF TACHEOMETRIC OR STADIA CONSTANT

The constants may be determined by

(1) Laboratory measurement

(2) field measurement

(1) Laboratory Measurement

The focal length f of the lens can be determined by means of an optical bench, according to the equation:

$$\frac{1}{f} = \frac{1}{u} + \frac{1}{v}$$

The stadia intercept i can be measured from the diaphragm with the help of a Vernier Caliper.

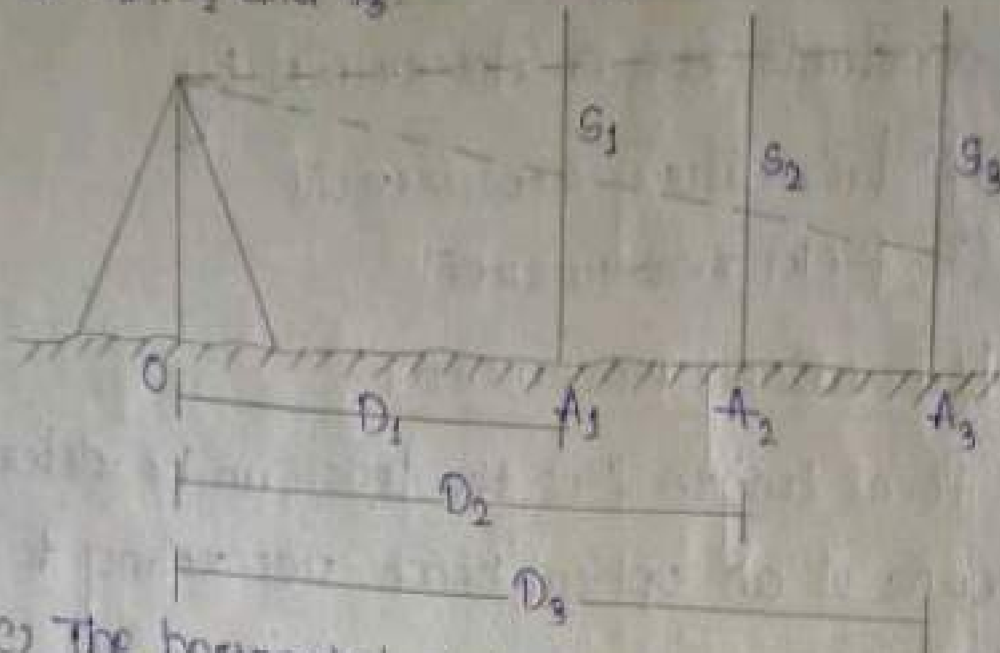
The distance d between the optical centre and the vertical axis of the measurement can also be determined / measured.

In this manner, the multiplying $(\frac{f}{i})$ and additive $(f+d)$ Constants can be calculated.

(2) field Measurement

(a) A fairly level ground is selected. The tacheometer is set up at O and pegs are fixed at A_1, A_2 and A_3 known distances apart. (11·1)

(b) The staff intercepts (stadia hair readings) are noted at each of the pegs. Let these intercepts be s_1, s_2 and s_3.



(c) The horizontal distances of the pegs from O are accurately measured. Let these distances be D_1, D_2 and D_3.

(d) By substituting the values of D_1, D_2 ..... and s_1, s_2 ..... in the general equation

$$D = \left(\frac{f}{i}\right) s + (f+d)$$

We get a number of equations, as follows:

$$D_1 = \left(\frac{f}{i}\right) s_1 + (f+d)$$

$$D_2 = \left(\frac{f}{i}\right) s_2 + (f+d)$$ and so on.

(e) By solving the equations in pairs, several values of $\left(\frac{f}{i}\right)$ and $(f+d)$ are obtained. The mean of these values gives the required Constant.

METHODS OF TACHEOMETRY

Tacheometry involves mainly two methods :

1. The stadia method
2. The tangential method

1. The stadia method

In this method the diaphragm of the tacheometer is provided with two stadia hairs (upper and lower). Looking through the telescope the stadia hair readings are taken. The difference in these readings gives the staff intercept. To determine the distance between the station and the staff, the staff intercept is multiplied by the stadia constant (i.e. multiplying constant, 100). The stadia method may, in turn, be of two kinds.

(a) The fixed Hair Method -

The distance between the stadia hairs is fixed in this method, which is the one commonly used. When the staff is sighted through the telescope, a certain portion of the staff is intercepted by the upper and lower stadia. The value of the staff intercept varies with the distance. However, the distance between the station and the staff can be obtained by multiplying the staff intercept by the stadia constant.

(b) The movable Hair Method :

The stadia hairs are not fixed in this method. They can be moved or adjusted by micrometer screws. The staff is provided with two targets, or vanes a known distance apart. During observation, the distances between stadia hairs is so adjusted that the upper hair bisects the upper target and the lower hair bisects the lower target. The variable stadia intercept is measured and the required distance is then computed.

This method is not generally used.

2. The Tangential Method :

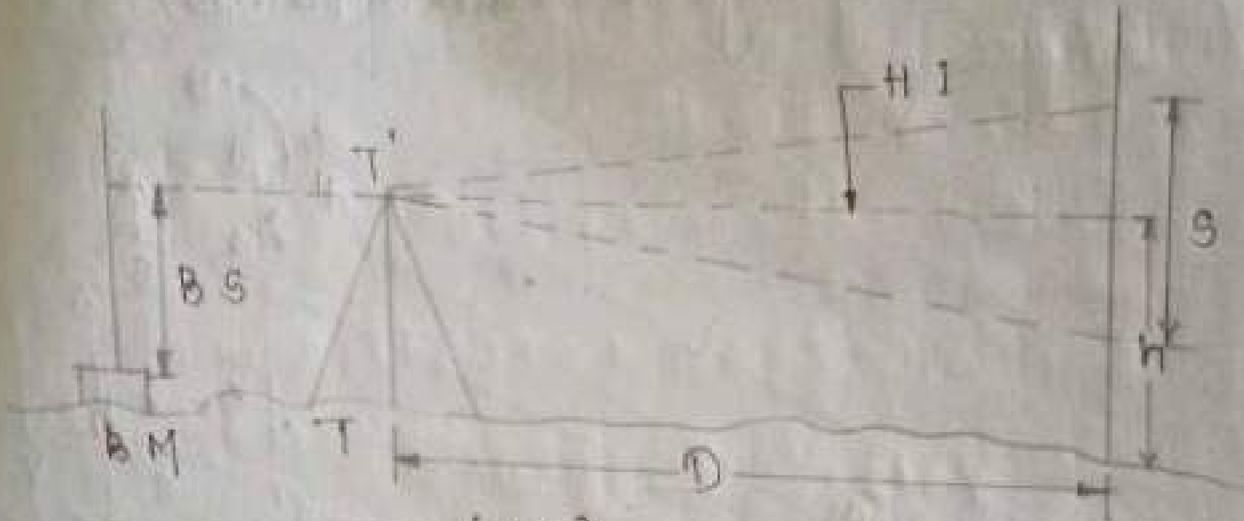
In this method, the diaphragm of the tacheometer is not provided with stadia hair. The readings are taken by the single horizontal hair.

The staff consists of two vanes or targets a known distance apart. To measure the staff intercepts, two pointings are required. The angles of elevation or depression are measured and their tangents are used for finding the horizontal distances and elevation.

This method is also not generally used. The stadia methods requires only one observation, but the tangential method requires two pointings of the telescope.

FIXED HAIR METHOD

Case 1: When line of sight is horizontal and the staff is held vertically.



(11.6)

When the line of sight is horizontal, the general tacheometric equation for distance is given by

$$D = \left(\frac{f}{i}\right) s + (f + d)$$

The multiplying Constant $\left(\frac{f}{i}\right)$ is 100, and additive Constant $(f+d)$ is generally zero.

RL of staff station $P = HI - h$

Where, $HI = RL \text{ of } BM + BS$

h = central hair reading

(HI = height of instrument)

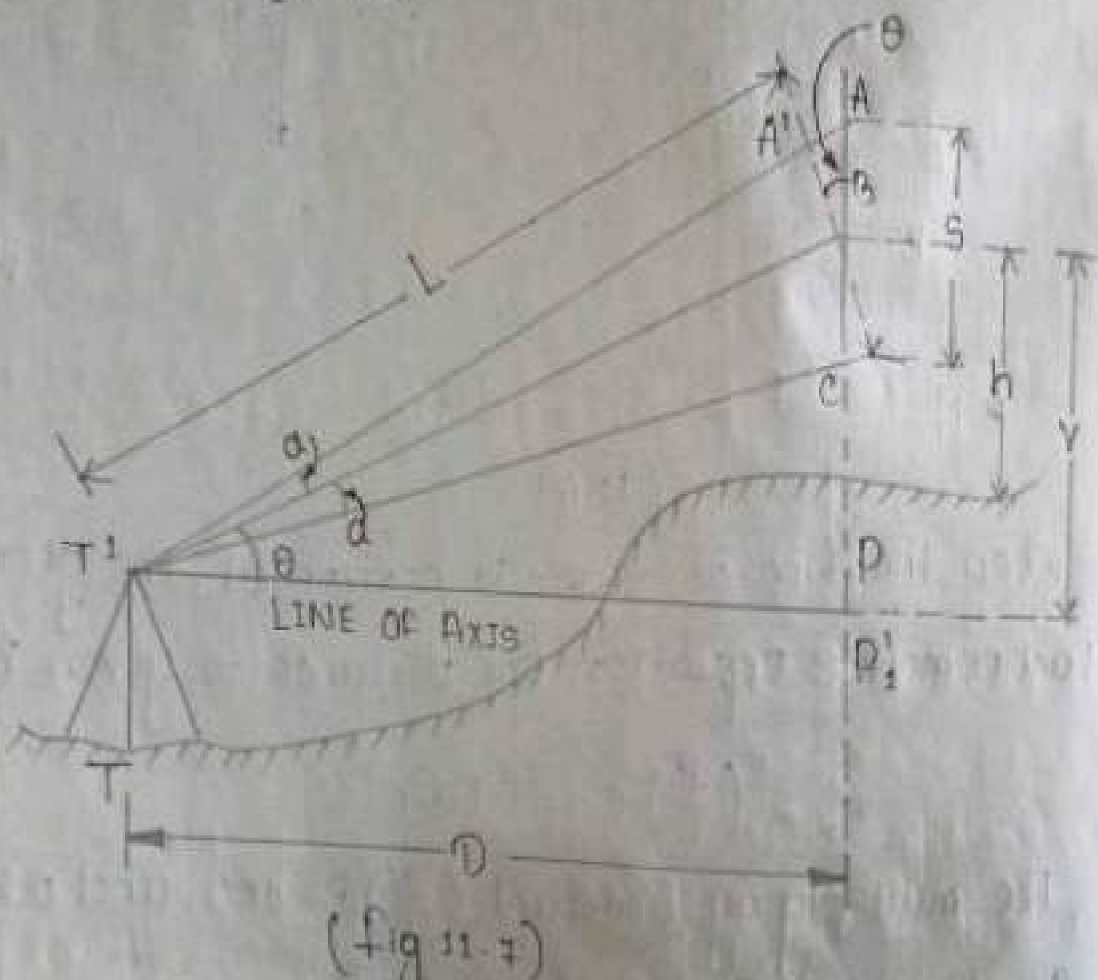
(BS = backsight)

Case - II

When line of sight is inclined, but staff is held vertically.

- Here, the measured angle may be the angle of elevation or that of depression.

(a) Considering Angle of Elevation (positive)



(fig 11.7)

T = instrument station

T_1 = axis of instrument

P = staff station

A, C, B = position of staff cut by hairs

s = AC = staff intercept

h = central hair readings

v = vertical distance between instrument axis and central hair

D = horizontal distance between instrument and staff

L = inclined distance between instrument axis and B

θ = angle of elevation

α = angle made by outer and inner rays with central ray

A'C' is drawn perpendicular to the central ray, T_1B.

Now, inclined distance,

$$L = \frac{f}{i}(A'C') + (f+d)$$

Horizontal distance $D = L\cos\theta$

$$= \frac{f}{i}(A'C')\cos\theta + (f+d)\cos\theta$$

Now A'C' is to be expressed in terms of AC (i.e. s).

In Δs ABA' and CBC',

$$\angle ABA' = \angle CBC' = \theta$$

$$\angle AA'B = 90° + \alpha$$

$$\angle BC'C = 90 - \alpha$$

The angle α is very small.

∠AA'B and ∠BC'C may be taken equal to 90°

so,

$$AC' = AC \cos\theta = s\cos\theta$$

from Eq.(1),

$$D = \frac{f}{i}(s\cos\theta)\cos\theta + (f+d)\cos\theta$$

$$D = \frac{f}{i} \times s\cos^2\theta + (f+d)\cos\theta \qquad (1)$$

Again,

$$V = L\sin\theta$$

$$= \left\{\frac{f}{i} \times s\cos\theta + (f+d)\right\}\sin\theta$$

$$= \frac{f}{i} \times s\cos\theta\sin\theta + (f+d)\sin\theta$$

$$V = \frac{f}{i} \times \frac{s\sin 2\theta}{2} + (f+d)\sin\theta \qquad (2)$$

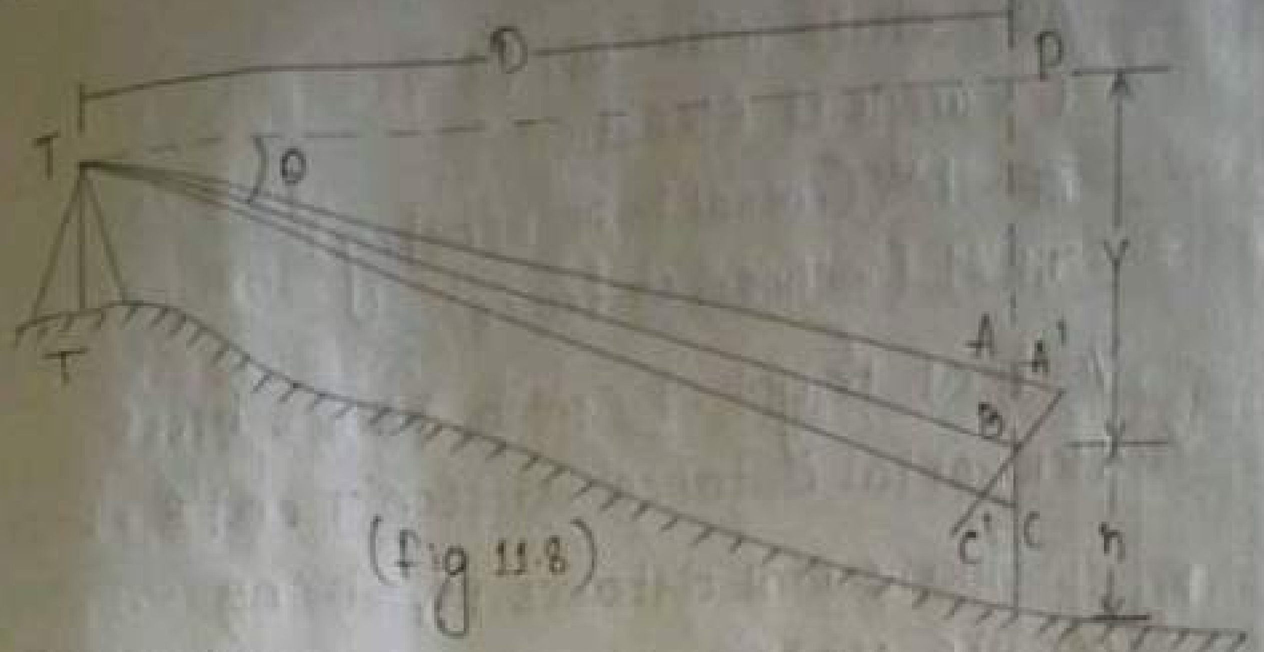
$$V = D\tan\theta$$

RL of staff station P = RL of axis of instrument + V − h (3)

(b) Considering angle of depression (negative). In this case also (fig 11.8), the expressions for D and V are same as in (a). That is,

$$D = \frac{f}{i} \times s\cos^2\theta + (f+d)\cos\theta \qquad (4)$$

$$V = \frac{f}{i} \times \frac{s \times \sin^2\theta}{2} + (f+d)\sin\theta \qquad (5)$$



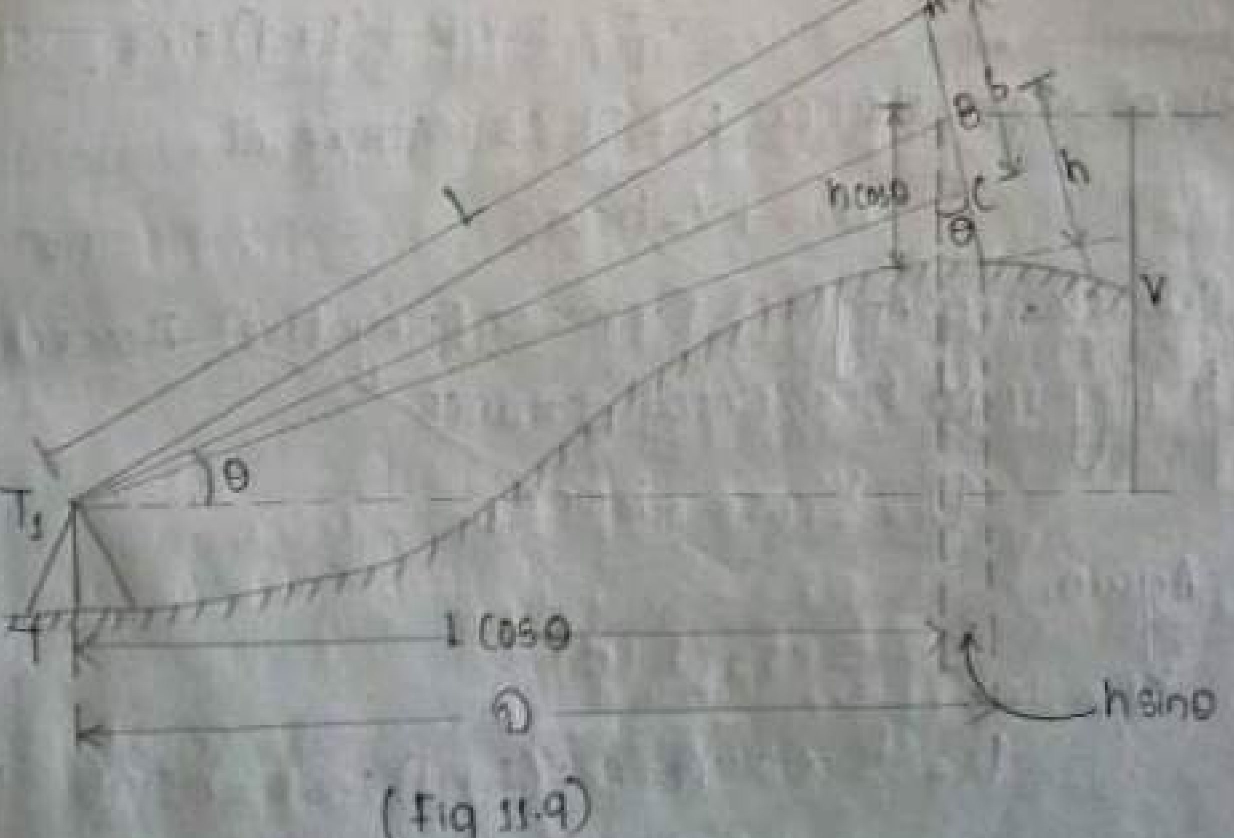
(fig 11.8)

RL of staff station, P = RL of axis of instrument − V − h (3)

<u>Case-III</u> Dt. 03.05.2021

Line of sight inclined, but staff normal to it

(a) Considering Angle of Elevation (positive)



(fig 11.9)

AC = staff intercept (S)

θ = angle of elevation

BP = h (Central hair reading)

T_1B = L (inclined distance)

Vertical height of Central hair = $h\cos\theta$

Horizontal distance between T and B = $L\cos\theta$

Horizontal distance $PP_1 = h\sin\theta$

Since the staff is perpendicular to the line of Collimation,

$$L = \frac{f}{i} \times S + (f+d)$$

Horizontal distance $D = L\cos\theta + h\sin\theta$

$$= \frac{f}{i} \times S\cos\theta + (f+d)\cos\theta + h\sin\theta$$

Vertical distance $V = L\sin\theta$

$$= \frac{f}{i} \times S\sin\theta + (f+d)\sin\theta \quad (8)$$

RL of staff station P = RL of instrument axis + V − $h\cos\theta$ (9)

(b) Considering Angle of Depression (Negative) According to fig 11.10, horizontal distance

$$D = L\cos\theta - h\sin\theta$$

Again,

$$L = \frac{f}{i} \times S + (f+d)$$

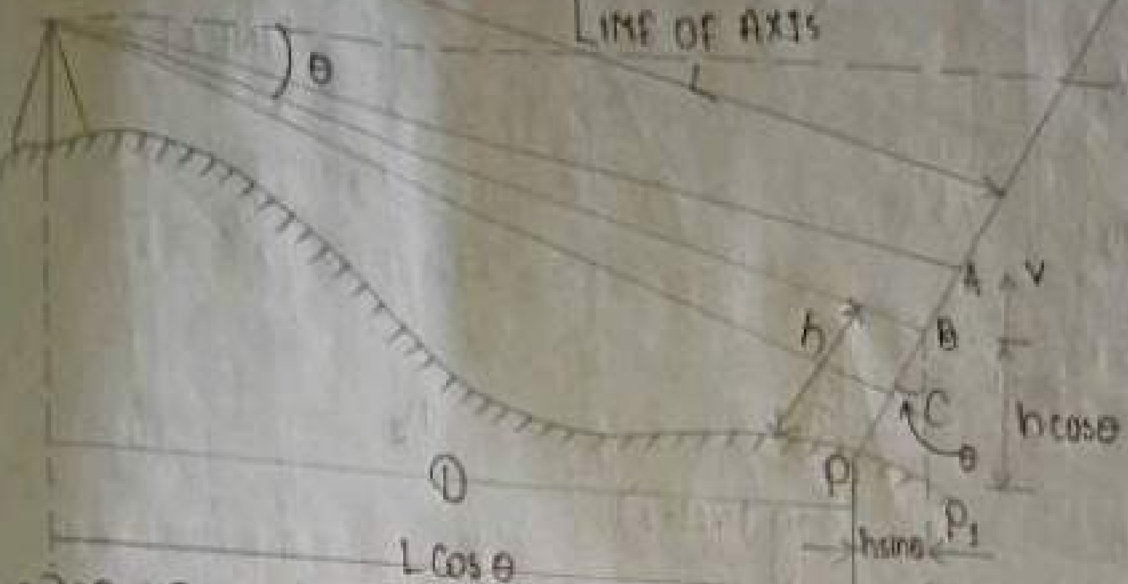
$$\therefore D = \frac{f}{i} \times S \times \cos\theta + (f+d)\cos\theta - h\sin\theta \quad (10)$$

vertical distance,

$$V = L\sin\theta$$

$$\therefore V = \frac{f}{i} \times s\sin\theta + (f+d)\sin\theta \qquad (11)$$

RL of P = RL of instrument axis − V − h cos θ



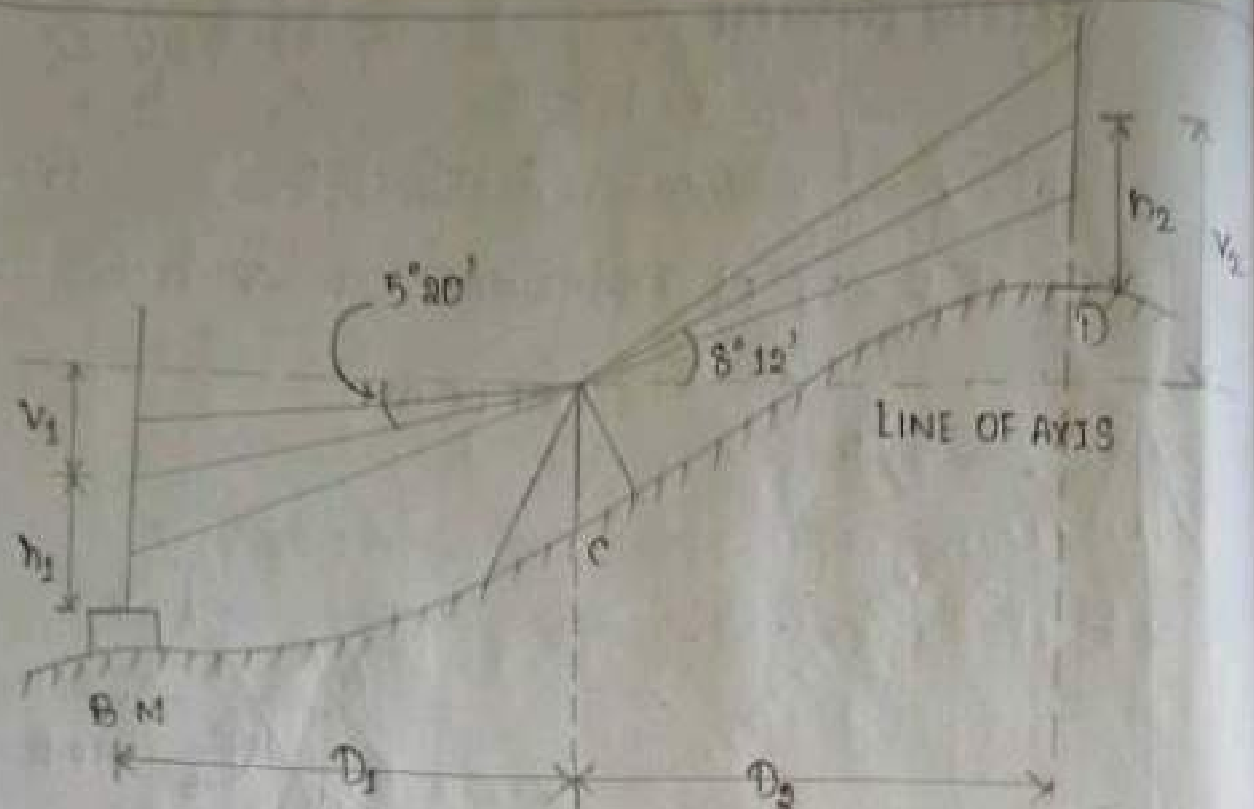
* **WORKED-OUT PROBLEMS ON FIXED HAIR METHOD OF TACHEOMETRY :-**

Problem 1

A tacheometer was set up at a station C and the following readings were obtained on a staff vertically hold.

Inst. station	Staff Station	Vertical angle	Hair readings (m)	Remark
C	BM	−5°20′	1.750, 1.300, 2.450	RL of BM
C	D	+8°12′	1.750, 1.500, 2.250	= 750.50m

Calculate the horizontal distance CD and RL of D, when the constants of instrument are 100 and 0.15.



(fig p.11.1)

Solution

When the staff is held vertically, the horizontal and vertical distances are given by relations

$$D = \frac{f}{i} \times S\cos^2\theta + (f+d)\cos\theta$$

$$V = \frac{f}{i} \times S \times \frac{\sin 2\theta}{2} + (f+d)\sin\theta$$

Here,

$$\frac{f}{i} = 100 \text{ and } (f+d) = 0.15$$

In the first observations,

$$S_1 = 2.450 - 1.150 = 1.300\,m$$

$$\theta_1 = 5^\circ 20' \text{ (depression)}$$

$$V_1 = 100 \times 1.300 \times \frac{\sin 10^\circ 4'}{2} + 0.15 \times \sin 5^\circ 20' = 12.045\,m$$

In the Second observation,

$$S_2 = 2.250 - 0.750$$
$$= 1.500$$
$$\theta_2 = 8°12' \text{ (elevation)}$$

$V_2 = 100 \times 1.500 \times \frac{\sin 16°24'}{2} + 0.15 \times \sin 8°12' = 21.197$ m

$D_2 = 100 \times 1.500 \times \cos^2 8°12' + 0.15 \times \cos 8°12'$

$= 147.097$ m

RL of instrument axis = RL of BM + h_1 + V_1

$= 750.500 + 1.800 + 12.045$

$= 764.345$ m

RL of D = RL of inst. axis + V_2 − h_2

$= 764.345 + 21.197 - 1.500$

$= 784.042$ m

So, the distance CD = 147.097 m and RL of D = 784.042 m

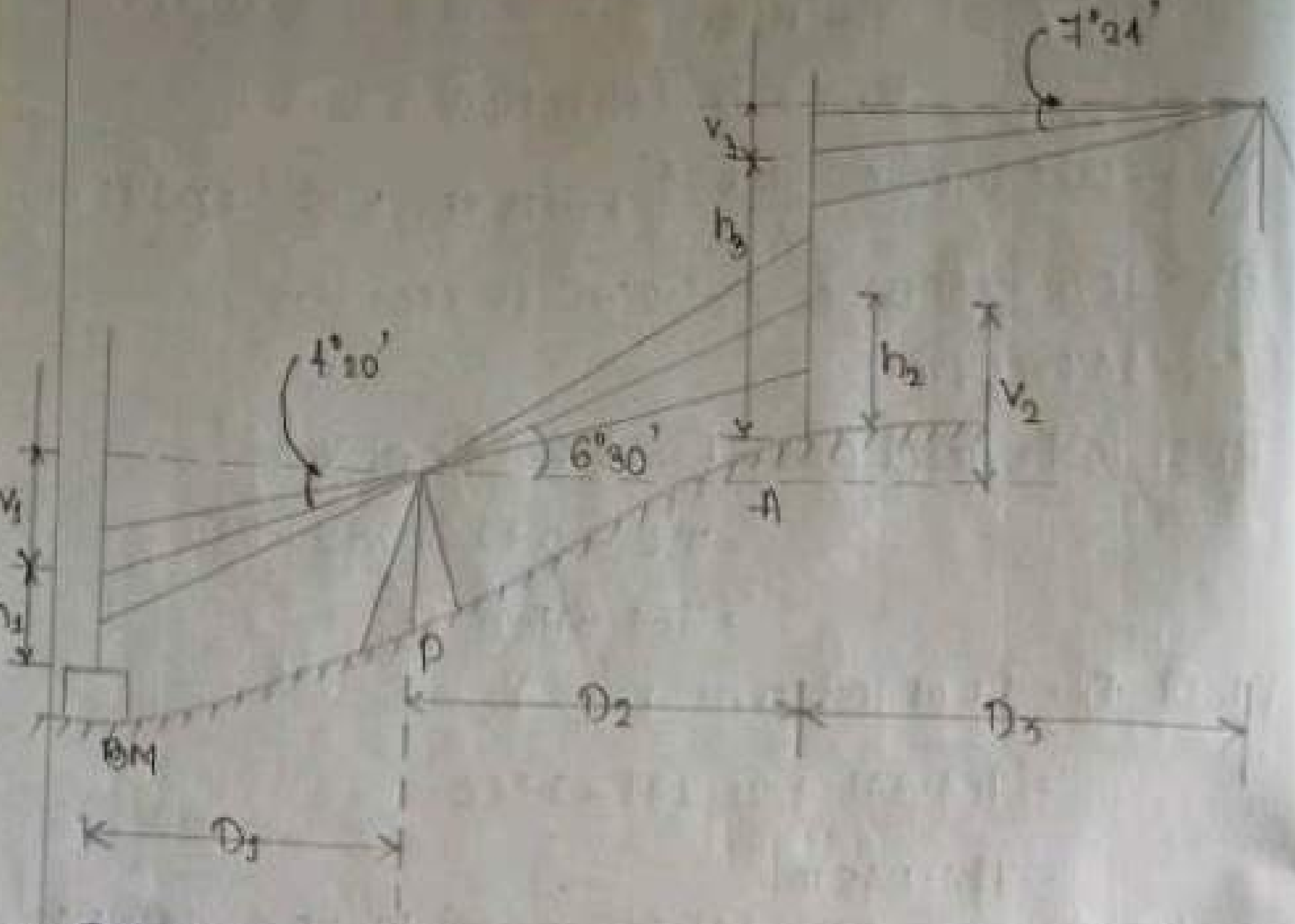
Problem-2

Dt-05.05.2021

The following observations were taken with a tacheometer fitted with an anallatic lens, the staff being held vertically. The Constant of the tacheometer is 100.

Inst. Station	Height of instrument	Staff Station	Vertical angle	Staff reading (m)	Remark
P	1.255	BM	−4°20′	1.325, 1.825, 2.325	RL of BM = 255.750 m
P	1.255	A	+6°30′	0.850, 1.600, 2.350	
B	1.450	A	−7°24′	1.715, 2.315, 2.915	

Calculate the RL of B and the distance between A and B.



Solution :-

Here,

Multiplying Constant, $\frac{f}{i} = 100$ and additive Constant $f + d = 0$

Since,

the staff is held vertically, the vertical distance is given by

$$V = \frac{f}{i} \times S \times \frac{\sin 2\theta}{2}$$

In the observations,

$$V_1 = 100(2.335 - 1.335) \times \frac{\sin 8°40'}{2}$$

$$= 7.534 \text{ m}$$

In the second observation,

$$V_2 = 100(2.350 - 0.850) \times \frac{\sin 13^\circ 0'}{2}$$

$= 16.871\,m$

In the third observation,

$$V_3 = 100(2.915 - 1.715) \times \frac{\sin 14^\circ 18'}{2}$$

$= 15.826\,m$

RL of axis when Inst. at P = RL of BM + $h_1 + V_1$

$= 255.750 + 1.825 + 7.534$

$= 265.109\,m$

RL of A = $265.109 + V_2 - h_2$

$= 265.109 + 16.871 - 600$

$= 280.880\,m$

RL of axis when inst. at B = $280.380 + h_3 + V_3$

$= 280.380 + 2.315 + 15.326$

$= 298.021\,m$

RL of B = $298.021 - h_3$

$= 298.021 - 1.450$

$= 296.571\,m$

Distance between A and B, $D_3 = 100(2.915 - 1.715) \times \cos^2 14^\circ$

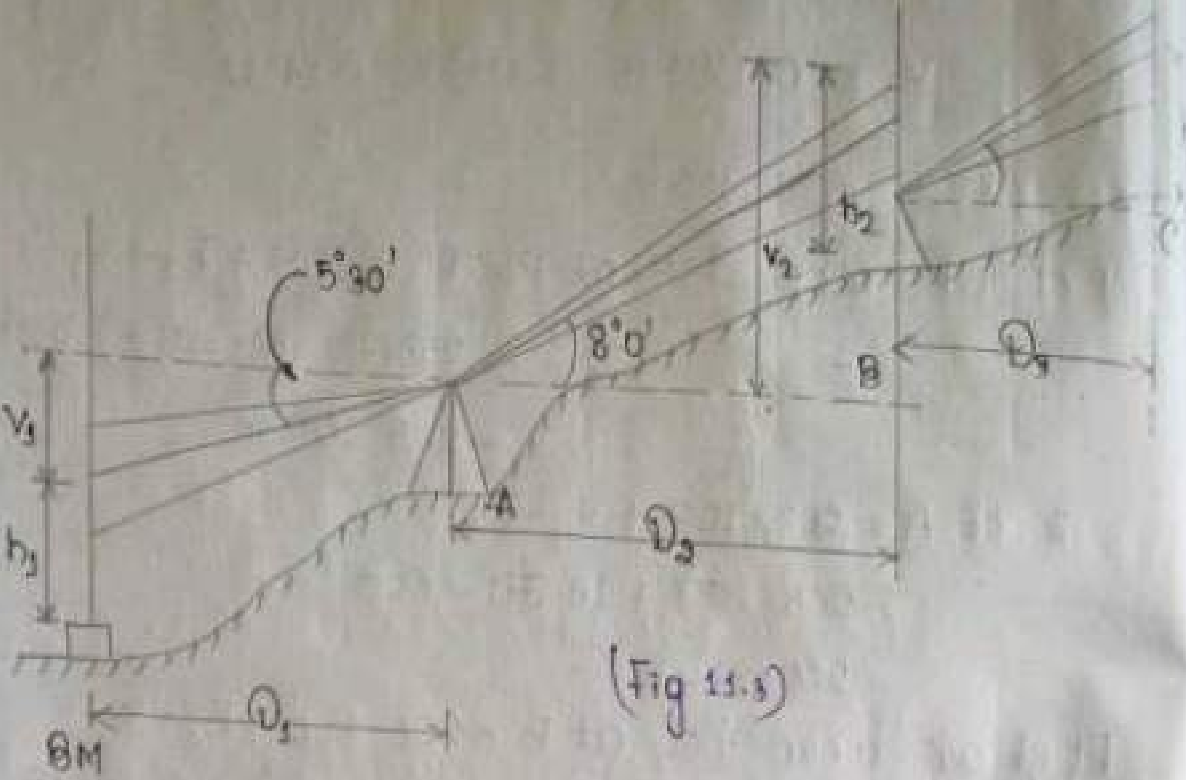
$= 118.009\,m$

Problem 3 :-

The following observation were made in a tacheometric survey.

Inst. station	Height of axis	Staff station	Vertical angle	Hair readings (m)	Remark
A	1.845	BM	$-5^\circ 30'$	0.905, 1.455, 2.005	RL of BM = 450.500m
A	1.845	B	$+6^\circ 0'$	0.755, 1.855, 2.555	
B	1.550	C	$+10^\circ 0'$	1.500, 2.250, 3.000	

Calculate the RLs of A, B and C and the horizontal distances AB and BC. The tacheometer is fitted with an anallatic lens and the multiplying constant is 100.



(Fig 11.3)

Solution

Here,

$$\frac{f}{i} = 100 \text{ and } (f+d) = 0$$

Since the staff is held vertically,

$$\text{Horizontal distance } D = \left(\frac{f}{i}\right) \times s\cos^2\theta$$

$$\text{Vertical distance } V = \left(\frac{f}{i}\right) \times s \times \frac{\sin 2\theta}{2}$$

In the first observation,

$$V_1 = 100 \times (2.005 - 0.905) \times \frac{\sin 11^\circ}{2}$$

$$= 10.494 \text{ m}$$

$$D_1 = 100 \times (2.005 - 0.905) \times \cos^2 5^\circ 30'$$

= 105.989 m

In the second observation,

$$V_2 = 100(2.555 - 0.755) \times \frac{\sin 16^\circ}{2}$$

$$= 24.807 \text{ m}$$

$$D_2 = 100(2.555 - 0.755) \times \cos^2 8^\circ$$

$$= 176.514 \text{ m}$$

In the third observation,

$$V_3 = 100 \times (3.000 - 1.500) \times \frac{\sin 20^\circ}{2}$$

$$= 25.652 \text{ m}$$

$$D_3 = 100 \times (3.000 - 1.500) \times \cos^2 10^\circ$$

$$= 145.477 \text{ m}$$

Distance AB = D_2 = 176.514 m

Distance BC = D_3 = 145.477 m

RL of axis when inst at A = RL of BM + V_1 + h_1

= 450.500 + 10.494 + 1.455

= 462.449 m.

RL of A = 462.449 height of axis

= 462.449 − 1.345.

= 461.104 m.

RL of B = 462.449 + V_2 − h_2

= 462.449 + 24.807 − 1.655

= 485.601 m

RL of axis when inst at B = 485.601 + 1.550

= 487.151 m.

RL of C = 487.151 + V_3 − h_3

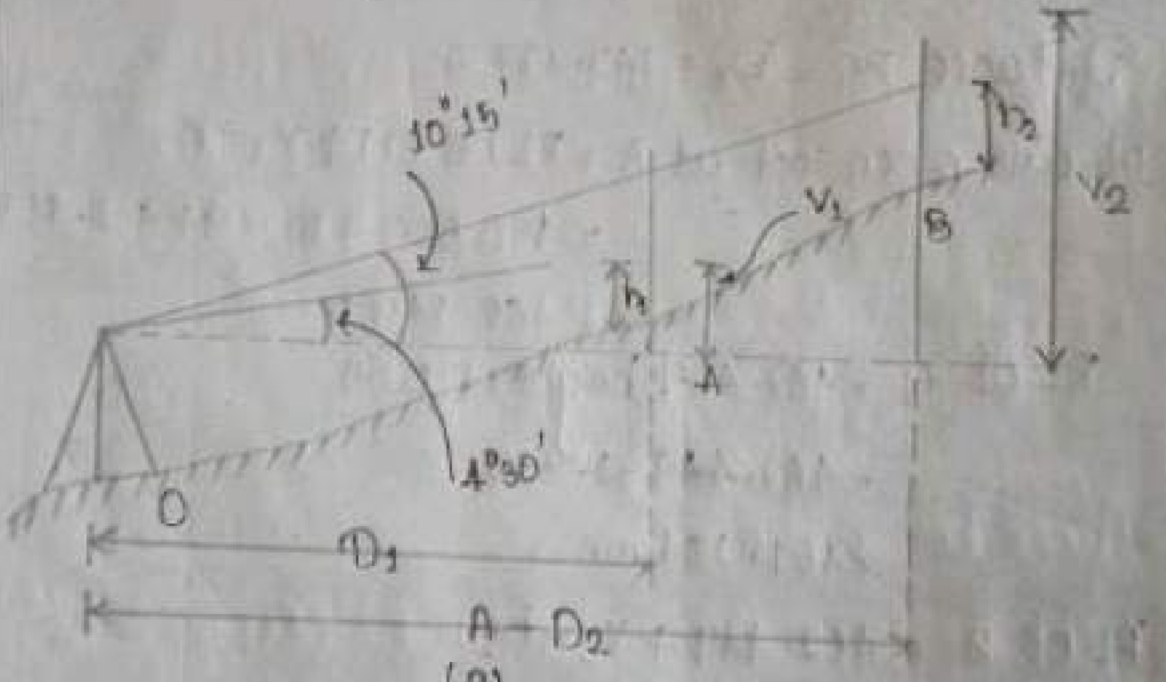
= 487.151 + 25.652 − 2.250 = 510.553 m

Problem-1

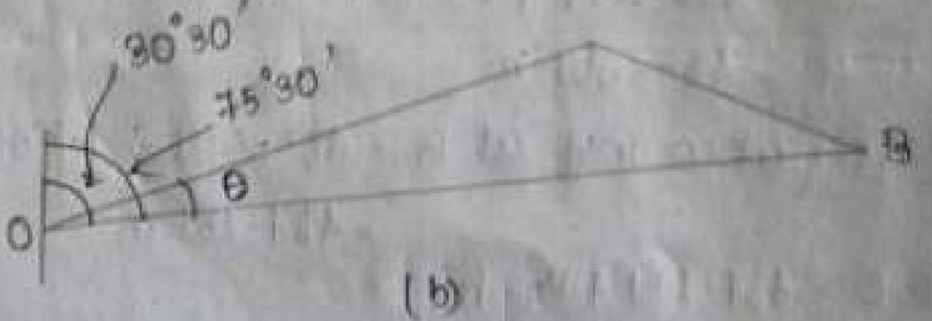
The following observation were made using a tacheometer fitted with an anallatic lens, the multiplying constant being 100.

Inst. station	Height of station	Staff station	WCB	Vertical angle	Hair readings	Remarks
O	1.550	A	30°30'	4°30'	1.155, 1.755, 0.355	RL of O = 150.000
		B	75°30'	10°15'	1.250, 2.000, 2.750	

Calculate the distance AB and the RLs of A and B. Find also the gradient of the line AB.



(a)



(b)

In the first Observation :-

$$V_1 = 100 \times (2.355 - 1.155) \times \frac{\sin 9^\circ}{2} = 9.386 \text{ m}$$

$$D_1 = 100 \times (2.355 - 1.155) \times \cos^2 4^\circ 30' = 119.261 \text{ m}$$

In the Second observation :-

$$V_2 = 100(2.750 - 1.250) \times \frac{\sin 20^\circ 30'}{2} = 26.265 \text{ m}$$

$$D_2 = 100(2.750 - 1.250) \times \cos^2 10^\circ 15' = 145.250 \text{ m}$$

RL of axis = RL of O + height of inst.

= 150.000 + 1.550

= 151.550 m

RL of A = $151.550 + V_1 - h_1$

= 151.550 + 9.386 − 1.755

= 159.181 m.

RL of B = $151.550 + V_2 - h_2$

= 151.550 + 26.265 − 2.000

= 175.815 m.

$OA = D_1 = 119.261$ m.

$OB = D_2 = 145.250$ m

$\theta = 75^\circ 30' - 30^\circ 30' = 45^\circ 0'$

$$AB = \sqrt{OA^2 + OB^2 - 2 \times OA \times OB \times \cos 45^\circ}$$

$$= \sqrt{(119.261)^2 + (145.250)^2 - 2 \times 119.261 \times 145.250 \times 0.707}$$

= 104.05 m

Distance of level between

A and B = 175.815 − 159.181

= 16.634 m (Rise from A to B)

Gradient of AB (rising) = $\frac{16.631}{104.05} = \frac{1}{6.25}$ i.e. 1 in 6.25

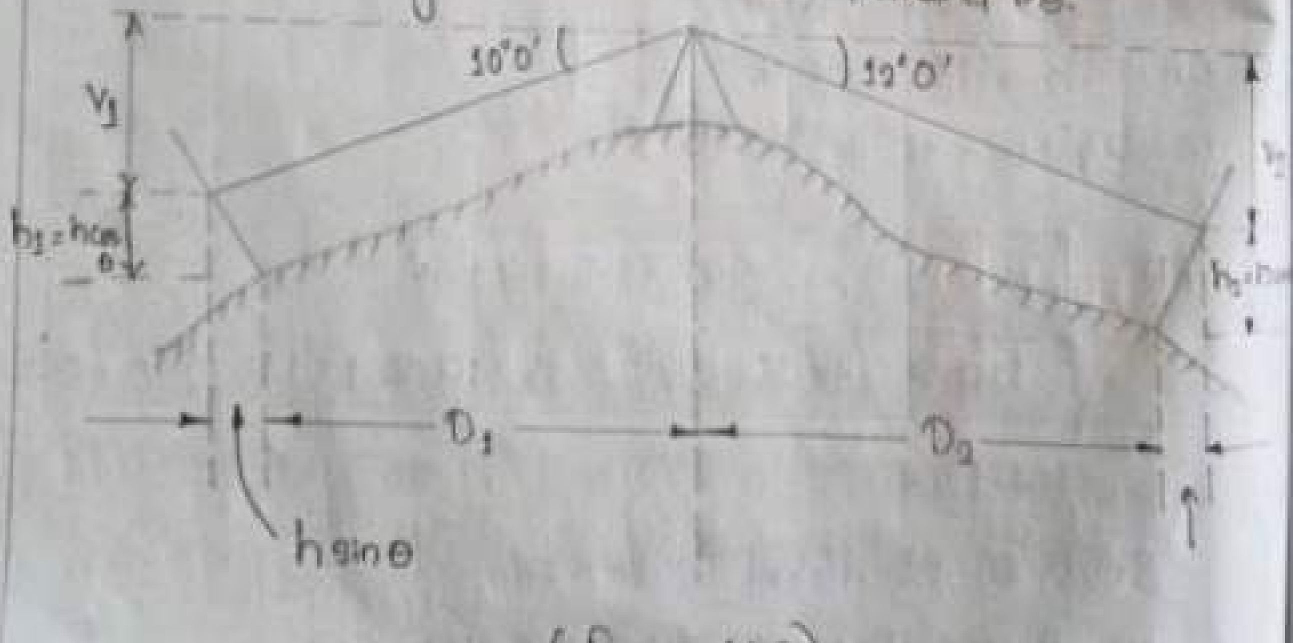
Problem 5 :-

Two points A and B are on opposite sides of a summit. The tacheometer was set up at P on top of the summit and the following readings were taken.

Inst. station	Height of inst	Staff station	Vertical angle	Hair readings	Remarks
P	1.500	A	$-10°0'$	1.150, 2.050, 2.950	RL of P = 450.500 m
P	1.500	B	$-12°0'$	0.855, 1.605, 2.355	

The tacheometer is fitted with an anallatic lens the multiplying constant being 100. The staff was held normal to the line of sight.

find : (a) The distance between A and B, and
(b) The gradients of lines PA and PB.



(fig P. 11.5)

Solution

We know that when the staff is held normal to the line of sight, the vertical distance is given by

$$V = \frac{f}{i} \times S \sin\theta + (f+d)\sin\theta$$

Here, $\frac{f}{i} = 100$ and $(f+d) = 0$

$\theta_1 = 10°$ and $\theta_2 = 12°$

from eq. (11),

$V_1 = \frac{f}{i} \times S \sin\theta_1 = 100 \times (2.950 - 1.150) \times \sin 10°$

$= 31.256\ m$

Similarly,

$V_2 = 100(2.355 - 0.855)\sin 12°$

$= 31.186\ m$

$h_1 = 2.050 \times \cos 10°$

$= 2.018\ m$

$h_2 = 1.605 \times \cos 12°$

$= 1.569\ m.$

RL of A inst. axis $= 450.500 + 1.500$

$= 452.000\ m$

RL of A = RL of inst axis $- V_1 - h_1$

$= 452.000 - 31.256 - 2.018$

$= 418.726\ m$

RL of B $= 452.000 - V_2 - h_2$

$= 452.000 - 31.186 - 1.569$

$= 419.245\ m$

The horizontal distances are given by equation,

$$D = \frac{f}{i} \times S\cos\theta + (f+d)\cos\theta + h\sin\theta$$

Hence, $D_1 = 100 \times (2.950 - 1.150) \cos 10^\circ - 2.050 \sin 10^\circ$

$= 177.265 - 0.355$

$= 176.91$ m

$D_2 = 100(2.355 - 0.855) \cos 12^\circ - 1.605 \sin 12^\circ$

$= 146.722 - 0.333$

$= 146.389$ m

Distance between A and B $= D_1 + D_2$

$= 176.910 + 146.389$

$= 323.299$ m

Gradient of PA $= \dfrac{450.500 - 418.726}{176.910} = \dfrac{1}{5.56}$ (1 in 5.56)

Gradient of PB (Falling) $= \dfrac{450.500 - 419.245}{146.389} = \dfrac{1}{4.68}$ (1 in 4.68)

Problem 6:-

The following are the records of a tacheometric survey:

Inst. station	Staff station	Bearing	Vertical angle	Hair readings
A	B	N 30°30' E	+ 10°0'	1.250, 1.750, 2.250
B	C	S 40°0' E	+ 5°0'	0.950, 1.750, 2.550
C	D	S 45°0' W	+ 8°0'	1.550, 2.150, 2.750

Multiplying constant = 100, and additive constant = 0. The staff is held vertically. Calculate the length and bearing of DA.

Solution:

The distances are Calculated from the formula

$$D = \frac{f}{i} \times S \cos^2\theta$$

$AB = 100(2.250 - 1.250) \times \cos^2 10°$

$= 96.98$ m

$BC = 100(2.500 - 0.950) \times \cos^2 5°$

$= 153.78$ m

$CD = 100(2.750 - 1.550) \times \cos^2 8°$

$= 117.67$ m

Let,

Length of DA = L and Bearing of DA = θ

Latitude

$AB = +96.98 \cos 30°30'$

$= +83.40$ (northing)

$BC = -158.78 \cos 40°0'$

$= -121.63$ (Southing)

$CD = -117.67 \cos 45°0'$

$= -83.20$ (Southing)

$DA = L \cos\theta$

Departure

$AB = +96.98 \sin 30°30'$

$= +49.22$ (easting)

$BC = +158.78 \sin 40°0'$

$= +102.06$ (easting)

$CD = -117.67 \sin 45°0'$

$= -83.20$ (westing)

$DA = L \sin\theta$

For a closed traverse, the algebraic sum of latitude and departures must equal to zero.

So,

$+83.40 - 121.68 - 83.20 + L\cos\theta = 0$

or

$L\cos\theta = 121.43$ (1)

and $+49.22 + 102.06 - 83.20 + L\sin\theta = 0$

$L\sin\theta = -68.08$ (2)

since the latitude is positive and departure is negative, the line DA lies in the NW quadrant

$$\tan\theta = \frac{68.08}{121.43}$$

$$= 0.5605$$

$$\theta = 29^\circ 16' 38''$$

Bearing of DA = N 29° 16' 38" W

$$\text{Length of DA} = \sqrt{(121.46)^2 + (68.08)^2}$$

$$= 139.21\ m.$$

Problem 07 :-

The following observations were taken from traverse stations A and B to points C and D by means of a stadia tacheometer fitted with an anallatic lens, the instrument constant being 100.

Inst. station	Staff Station	Height of inst.	Bearing	Vertical angle	Staff reading
A	C	1.48	126° 30'	+12° 10'	0.77, 1.60, [illegible]
B	D	1.42	184° 45'	−10° 30'	0.86, 1.84, [illegible]

Coordinates of A = 112.8 N, 106.4 W

Coordinates of B = 198.5 N, 292.6 W

Determine the length of the line CD.

Solution:-

$$\text{Distance AC} = 100 \times (2.43 - 0.77) \times \cos^2 12^\circ 10'$$

$$= 158.68\ m$$

Distance BD = $100(2.82 - 0.86) \times \cos^2 10°30'$

= 189.49 m

Reduced bearing of AC = S 53°30' E

Reduced bearing of BD = S 4°45' W

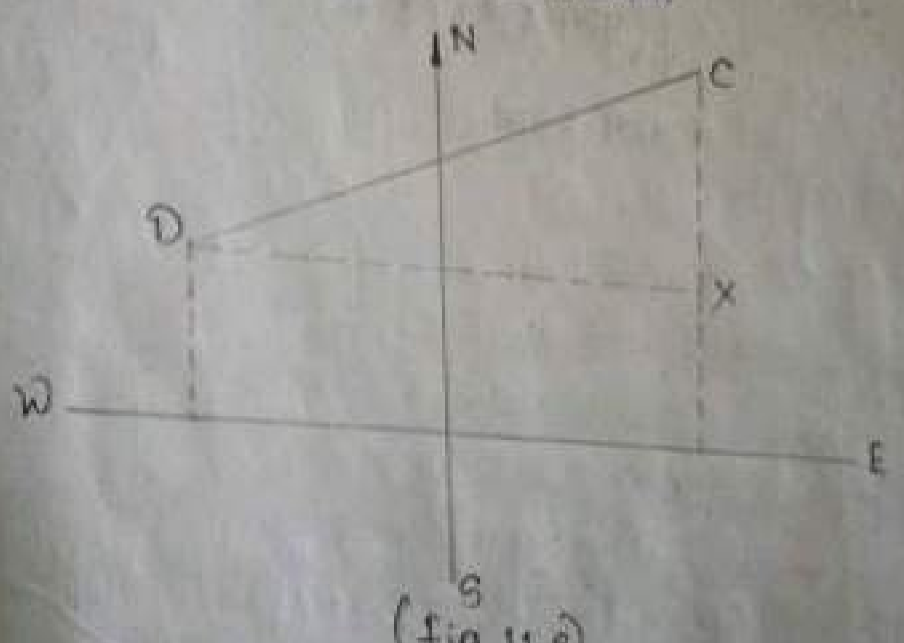
Line	Latitude	Departure
AC	$-158.63 \cos 53°30'$ = −94.36 m	$+158.63 \sin 53°30'$ = +127.52 m
BD	$-189.49 \cos 4°45'$ = −188.84 m	$+189.49 \sin 4°45'$ = −15.69 m

Coordinates of C

Latitude of A = +112.82 m (northing)

Total latitude of C = +112.83 − 94.36

= 18.44 m



(fig 11.6)

Departure of A = −106.4 m (westing)

Total departure of C = −106.40 + 127.52

= +21.12 m

Coordinates of D

Latitude of B = +198.5 m (northing)

Total latitude of D = +198.50 − 188.84

= +9.66 m

Departure of B = −292.6 m (westing)

Total departure of D = −292.6 − 15.69

= 308.29 m

Dx = departure of C + departure of D

= 21.12 + 308.29

= 329.41 m

Cx = Latitude of C − latitude of D

= 18.44 − 9.66

= 8.78 m.

Length CD = $\sqrt{(Dx)^2 + (Cx)^2}$

$= \sqrt{(329.4)^2 + (8.78)^2}$

= 329.52 m.

Chp: 2

CURVES

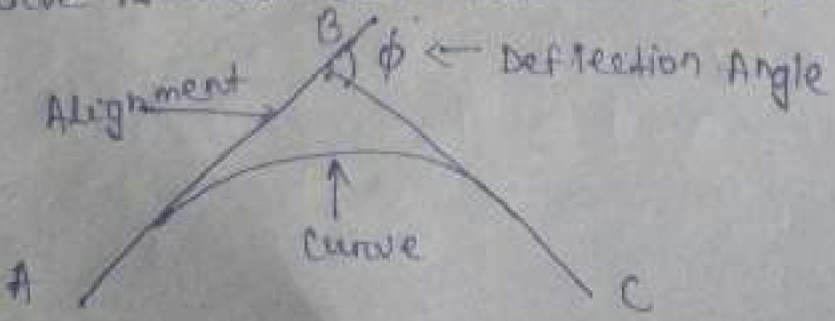
Introduction :- During the survey of the alignment of a project involving roads or railways, the direction of the line may change due to some unavoidable circumstances. The angle of the change in direction is known as the deflection angle.

→ For it to be possible for a vehicle to run easily along the road or railway track, the two straight lines (the original line and the deflected line) are connected by an arc. which is known as the curve of the road or track.

→ When the curve is provided in the horizontal plane, it is known as a horizontal curve.

→ Again, along the alignment of any project the nature of the ground may not be uniform and may consist of different gradients (for instance, rising gradient may be followed by falling gradient and vice versa)

→ In such a case, a parabolic curved path is provided in the vertical plane in order to connect the gradients for easy movement of the vehicles. This curve is known as a vertical curve.

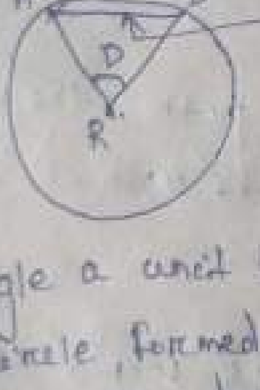


Types of curve

Curve
- Horizontal curve
  - Simple curve
  - Compound curve
  - Reverse curve
  - Transition curve
  - Lemniscate curve
- Vertical curve

Definitions and Explanations of Different Terms

① Degree of curve :-



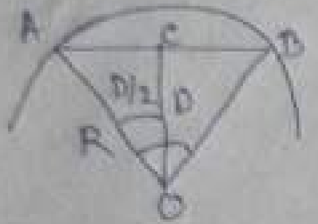
→ The angle a unit chord of length 30m subtends at the centre of the circle, formed by the curve is known as the degree of curve. It is designated as D.

→ A curve may be designated according to either the radius or the degree of the curve.

→ When the unit chord subtends an angle of 1°, it is called a one-degree curve, when the angle is 2°, a two-degree curve, and so on.

→ It may be calculated that the radius of a one-degree curve is 1,719 m

② Relation between radius and degree of curve



Let AB be the unit chord of 30m, O the centre, 'R' the radius and 'D' the degree of the curve.

Here $OA = R$

$AB = 30m$

$AC = 15m$

$\angle AOC = D/2$

From triangle OAC,

$$\sin D/2 = \frac{AC}{OA} = \frac{15}{R}$$

$$R = \frac{15}{\sin D/2}$$

- When 'D' is very small, sin D/2 may be taken as D/2 radians.

$$R = \frac{15}{(D/2) \times (\pi/180)} = \frac{15 \times 360}{\pi D} = \frac{1719}{D}$$

③ Superelevation

→ When a particle moves in a circular path, then a force (known as centrifugal force) acts upon it, and tends to push it away from the centre.

→ Similarly, when a vehicle suddenly moves from a straight to a curved path, the centrifugal force tends to push the vehicle away from the road or track.

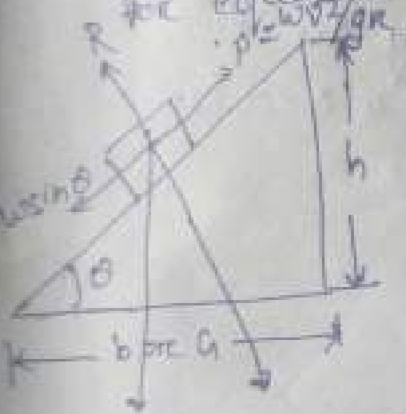
→ This is because there is no component force to counterbalance this centrifugal force.

→ To counterbalance the centrifugal force, the outer edge of the road or rail is raised to some height (with respect to inner edge), so that the sine component of the weight of the vehicle ($w \sin\theta$) may counterbalance the overturning force.

→ The height through which the outer edge of the road or rail is raised is known as super elevation or cant.

→ In fig 'P' is the centrifugal force, $w \sin\theta$ is the component of the weight of the vehicle, and 'h' is the superelevation given to the road or rail

for equilibrium,



$$w \sin\theta = \frac{wv^2}{gR}$$

$$\text{or } w \times \frac{h}{b} = \frac{wv^2}{gR} \quad (\text{when } \theta \text{ is very small}, \sin\theta = \tan\theta = \frac{h}{b})$$

$$\text{or } h = \frac{bv^2}{gR} \text{ for Roads } ①$$

$$\text{or } h = \frac{Gv^2}{gR} \text{ for railways } ②$$

where b = Width of the road in metres

G = distance between centres of rails (gauge) in metres

R = Radius of the curve in metres

g = acceleration due to gravity = 9.8 m/s²

v = Speed of the vehicle in metres per second

h = superelevation in metres

(14) Centrifugal Ratio :-

The ratio between the centrifugal force and the weight of the vehicle is known as centrifugal ratio.

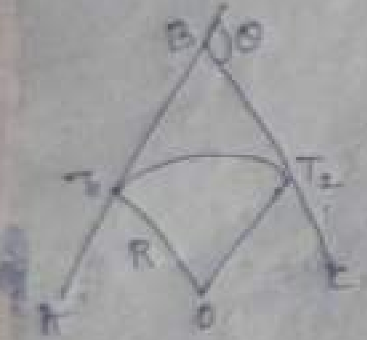
$$\text{Centrifugal Ratio (CR)} = \frac{P}{W} = \frac{Wv^2}{gR \times W} = \frac{v^2}{gR}$$

Allowable value for 'CR' in roads = $\frac{1}{4}$

Allowable value for 'CR' in railways = $\frac{1}{8}$

Types of horizontal curves

(1) Simple circular curve

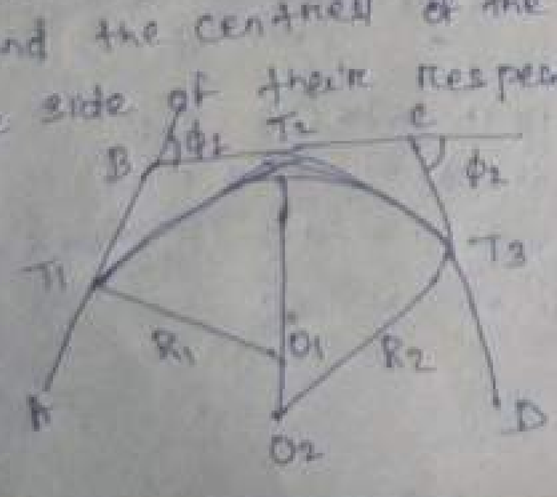


When a curve consists of a single arc with a constant radius connecting the two tangents, it is said to be a circular curve.

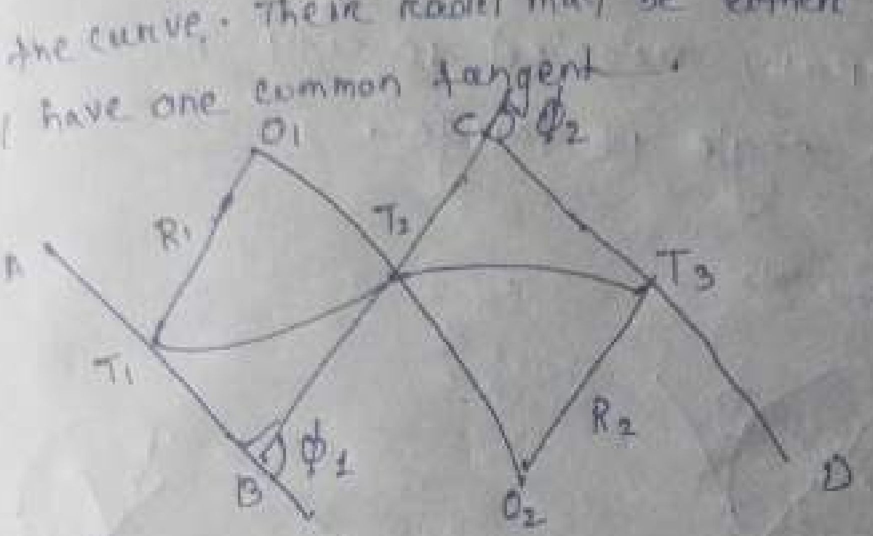
(2) Compound curve

When a curve consists of two or more arcs with different radii, it is called a compound curve.

→ Such a curve lies on the same side of a common tangent and the centres of the different arcs lie on the same side of their respective tangents.



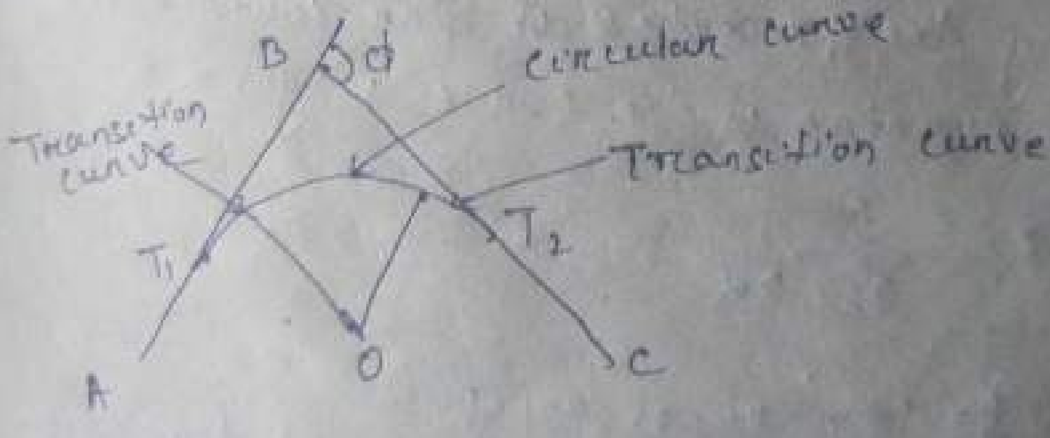
(3) Reverse Curve :- A reverse curve consists of two arc bending in opposite directions. Their centres lie on opposite sides of the curve. Their radii may be either equal or different, and they have one common tangent.



(4) Transition curve :-

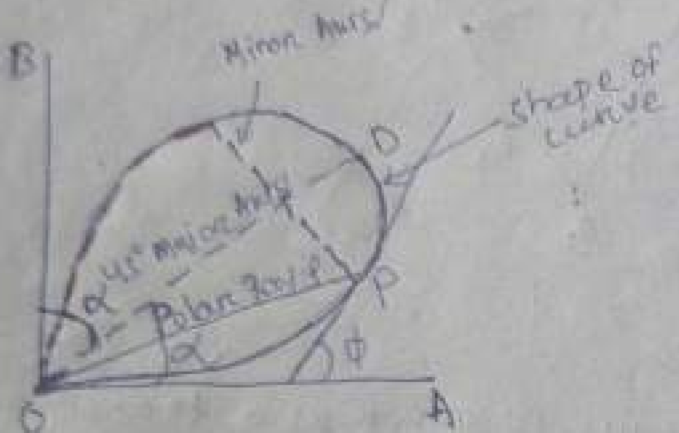
A curve of variable radius is known as a transition curve. It is also called a spiral curve or easement curve. In railways, such a curve is provided on both sides of a circular curve to minimise super elevation.

→ Excessive super elevation may cause wear and tear of the rail section and discomfort to passengers.



⑤ Lemniscate curve :-

→ A lemniscate curve is similar to a transition curve and is generally adopted in city roads where the deflection angle is large.



→ In fig OPD shows the shape of such a curve. The curve is designed by taking a major axis OD, minor axis PP', with origin O, and axis OA and OB. OP (ρ) is known as the polar ray, and α as the polar angle.

→ Considering the properties of polar coordinates the polar equation of the curve is given by :-

$$r = \frac{\rho}{3 \sin 2\alpha}$$

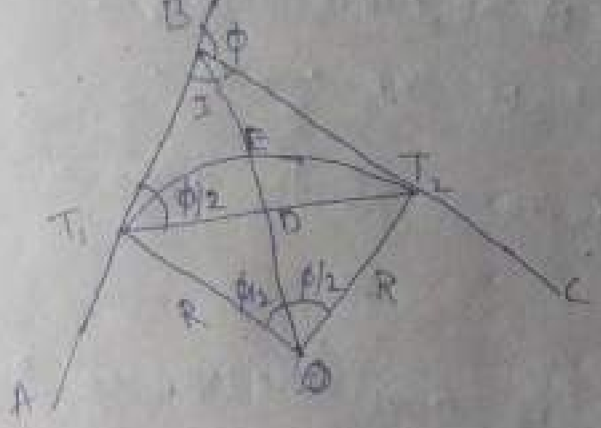
where, ρ = polar ray of any point
r = Radius of curvature at that point
α = polar deflection angle

At the origin, the radius of curvature is infinity. It then gradually decreases and becomes minimum at the apex D.

where, Length of curve OPD = 1·3115 k

$k = 3r\sqrt{\sin 2\alpha}$

NOTATION USED WITH CIRCULAR CURVES



1. AB and BC are known as the tangents to the curve (in fig).
2. B is known as the point of intersection or vertex
3. The angle ϕ is known as the angle of deflection
4. The angle I is called the angle of intersection
5. points T_1 and T_2 are known as tangent points
6. Distances BT_1 and BT_2 are known as tangent lengths.
7. when the curve deflects to the right, it is called a right-hand curve, when it deflects to the left,

It is said to be a left-hand curve.

(8) AB is called the rear tangent and BC the forward tangent.

(9) The straight line T_1DT_2 is known as the long chord.

(10) The curved line T_1ET_2 is said to be the length of the curve.

(11) The mid-point 'E' of the curve T_1ET_2 is known as the apex or summit of the curve.

(12) The distance BE is known as the apex distance or external distance.

(13) The distance DE is called the versed sine of the curve.

(14) 'R' is the radius of the curve

(15) $\angle T_1OT_2$ is equal to the deflection angle ϕ

(16) The point T_1 is known as the beginning of the curve or the point of curve.

(17) The end of the curve (T_2) is known as the point of tangency.

A ... representation of an

Properties of simple circular curve

① If the angle of intersection is given, then

$\phi = 180° - I$ (I = angle of intersection)

② If radius is not given, then

$R = \frac{1719}{D}$ (D = degree of curve)

③ Tangent length BT_1 or $BT_2 = R \tan \phi/2$

④ Length of curve = length of arc T_1ET_2

$= R \times \phi$ radians

$= \frac{\pi R \phi°}{180°}$ m

Again, length of curve $= \frac{30\phi}{D}$ (If degree of curve D is given)

⑤ Length of long chord $= 2T_1D = 2OT_1 \sin \phi/2$

$= 2R \sin \phi/2$ m

⑥ Apex distance $= BE = OB - OE$

$= R \sec \phi/2 - R$

$= R(\sec \phi/2 - 1)$ m

⑦ Versed sine of curve $= DE = OE - OD$

$= R - R\cos \frac{\phi}{2}$

$= R(1 - \cos \phi/2)$ m

⑧ Full chord (peg interval) : pegs are fixed at regular intervals along the curve. Each interval is said to equal the length of a full chord or unit chord. The curve is represented by a series of chords, instead of arcs. Thus, the length of the chord is practically equal to the length of the arc. In usual practice the length of the unit chord should not be more than 1/20th of the radius of the curve.

→ In railway curves, the unit chords (peg intervals) are generally taken between 20 and 30 m.

→ In road curves, the unit chord should be 10m or less.

It should be remembered that the curve will be more accurate if short unit chords are taken.

(9) Initial subchord :— Sometimes the chainage of the first tangent point works out to be a very odd number. To make it a round number, a short chord is introduced at the beginning. This short chord is known as the initial subchord.

(10) Final subchord :— Sometimes it is found that after introducing a number of full chords, some distance still remains to be covered in

order to reach the second tangent point. The short chord introduced for covering this distance is known as the final subchord.

(i) chainage of first tangent point = chainage of intersection point − tangent length

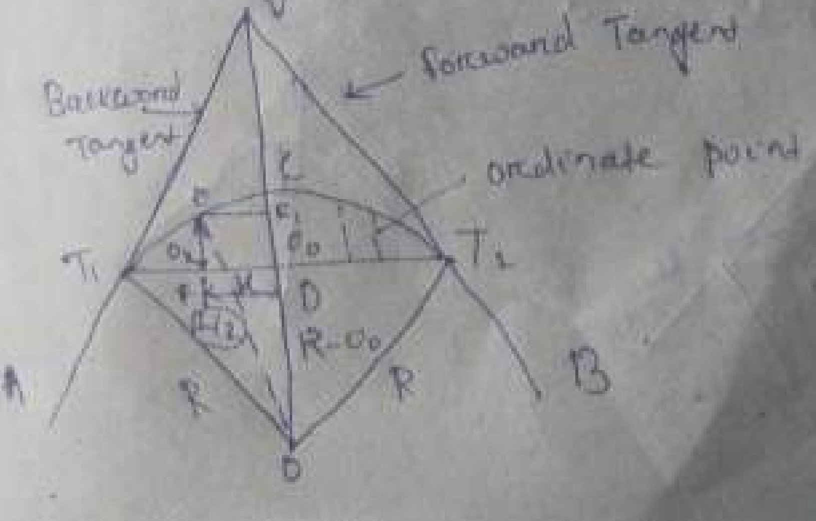
(ii) chainage of second tangent point = chainage of first tangent point + curve length.

Horizontal curve setting By chain and Tape method

The following are the general methods employed for setting out curves by chain and tape :-

1. Taking offsets or ordinates from the long chord
2. Taking offsets from the chord produced
3. successively bisecting the arcs
4. Taking offsets from the tangents

(1) Offsets or ordinates from Long chord



$O_0 = R - \sqrt{R^2 - (L/2)^2}$

$O_x = \sqrt{R^2 - x^2} - (R - O_0)$

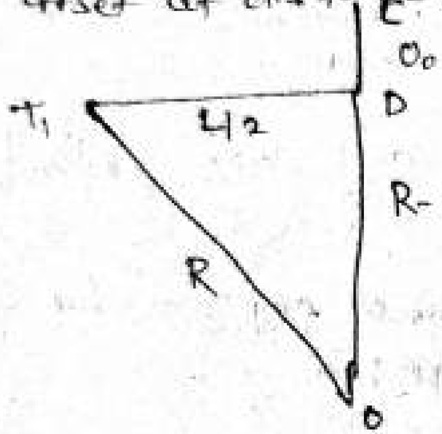
O_0 = center offset distance

R = Radius of circular curve

L = Length of chord

X = offset interval distance

O = offset at distance X from center offset line



In $\Delta T_1OD = R - O_0$

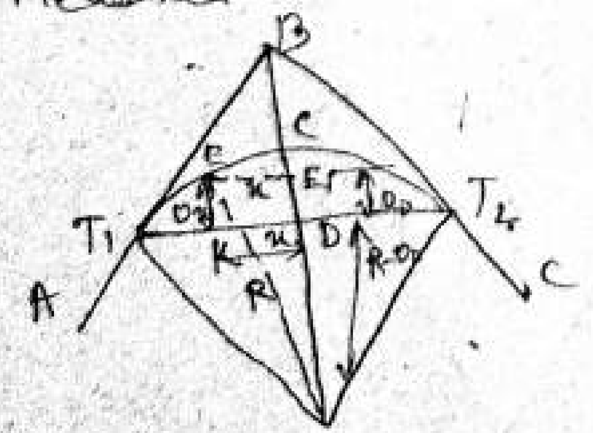
$= \sqrt{R^2 - (L/2)^2}$

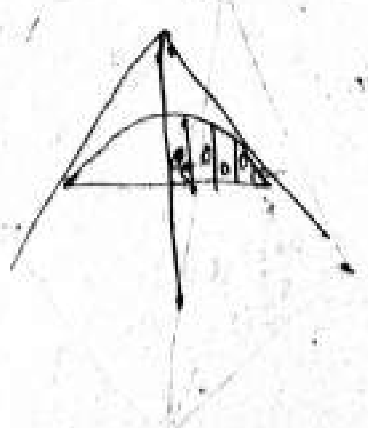
So R (O to C) $= R - O_0$

$\therefore O_0 = R - \sqrt{R^2 - (L/2)^2}$

long chord $T_1T_2 = 2R \sin \phi/2$

for linear method ϕ angle of deflection is not measured.





In Right angle Triangle EOE_1

$E_1O = \sqrt{R^2 - x^2}$

$O_0 = (E_1O) - (R - O_0)$

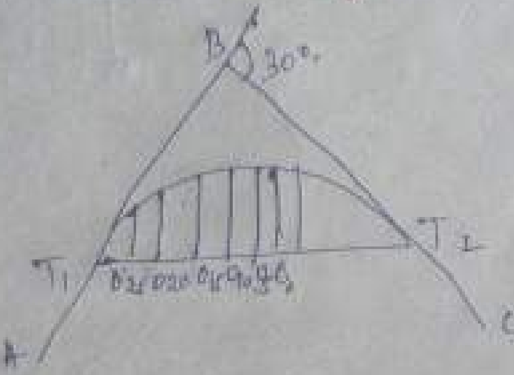
$\therefore O_x = O_x$

So $O_x = \sqrt{R^2 - x^2} - (R - O_0)$

Setting out simple curve

Linear method	Angular method
① only chain and tape is used	① Theodolite + Tape + chain is used
② curve is short	② for long curve
③ High degree of curve is not required.	③ High accuracy is required
④ (A) Offset from long chord	④ (A) Rankine's method of deflection angle
(B) Offset from Tangent	(B) Point of intersection method
(C) By successive bisection of arc	(C) Two theodolite method
(D) Offset from chord produced	(D) Tacheometric method

Pb Two tangents AB and BC intersect at a point 'B' at chainage 150.5 m. Calculate all the necessary data for setting out a circular curve of radius 100 m and deflection angle 30° by the method of offsets from the long chord.



Soln :- ① Tangent length $= R \tan\frac{\phi}{2}$

$= 100 \times \tan 15°$

$= 26.79$ m

② chainage of $T_1 = 150.50 - 26.79$

$= 123.71$ m

③ curve length $= \frac{\pi R \phi°}{180°} = \frac{3.14 \times 100 \times 30°}{180°} = 52.36$ m

④ Chainage of $T_2 = 123.71 + 52.36 = 176.07$ m

⑤ Length of long chord (L) $= 2R \sin\phi/2$

$= 2 \times 100 \times \sin 15°$

$= 51.76$ m

⑥ The long chord is divided into two equal halves

Each half $= \frac{1}{2} \times 51.76$

$= 25.88$ m

(7) Mid-ordinate, $O_0 = R - \sqrt{R^2 - (L/2)^2}$

$= 100 - \sqrt{100^2 - 25.88^2}$

$= 3.41 m$

(8) The ordinates are calculated at 5m intervals starting from the centre towards T_1 for the left half.

$O_5 = \sqrt{R^2 - x^2} \; (R - O_0)$

$= \sqrt{(100)^2 - (5)^2} - (100 - 3.41)$

$= 99.87 - 96.59$

$= 3.28 m$

~~$O_{10} = \sqrt{(100)^2 -}$~~

$O_{10} = \sqrt{(100^2 - 10^2)} - 96.59$

$= 99.50 - 96.59$

$= 2.91 m$

$O_{15} = \sqrt{(100^2 - 15^2)} - 96.59$

$= 98.87 - 96.59$

$= 2.28 m$

$O_{20} = \sqrt{(100^2 - 20^2)} - 96.59$

$= 97.97 - 96.59$

$= 1.38 m$

$O_{25} = \sqrt{(100^2 - 25^2)} - 96.59$

$= 96.82 - 96.59$

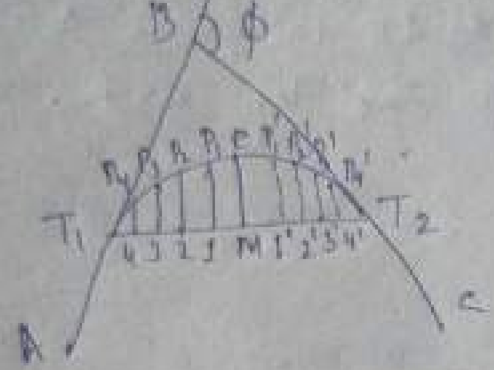
$= 0.23 m$

$O_{25.88} = \sqrt{(100^2 - 25.88^2)} - 96.59$

$= 0$ (checked)

(9) The ordinates for the right half are similar to those for the left half.

Field procedure for measuring ordinates by long chord method :-



① Let AB and BC be two tangents meeting at a point B, with deflection angle ϕ

② The tangent length is calculated from the usual formula, and points T_1 and T_2 are marked on the ground with pegs.

③ The length of the long chord, T_1T_2 is calculated from the usual formula. The long chord is bisected at point M. The curve will be symmetrical on both sides of M.

④ The ordinates are calculated for the left half at some regular intervals. points 1, 2, 3 and 4 are marked with pegs along the long chord as shown in fig.

⑤ Ordinates O_1, O_2, O_3 and O_4 are calculated from the usual formula.

⑥ Perpendiculars are set out at points 1, 2, 3 and 4. The calculated ordinates O_1, O_2, O_3 and O_4 identified along these perpendiculars and points P_1, P_2 and P_3 and P_4 are marked with pegs.

(7) In the right half, points 1', 2', 3' and 4' are marked with pegs and the corresponding ordinates (obtained for the left half) are set out to mark the points P_1', P_2', P_3' & P_4'.

(8) All these points $P_1, P_2 \ldots$ & $P_1', P_2' \ldots$ are on the curve. These points are joined by rope or thread to show the shape of the curve along the alignment (centreline) of the project.

(4) Offsets from Tangents

offsets from tangents may be :-

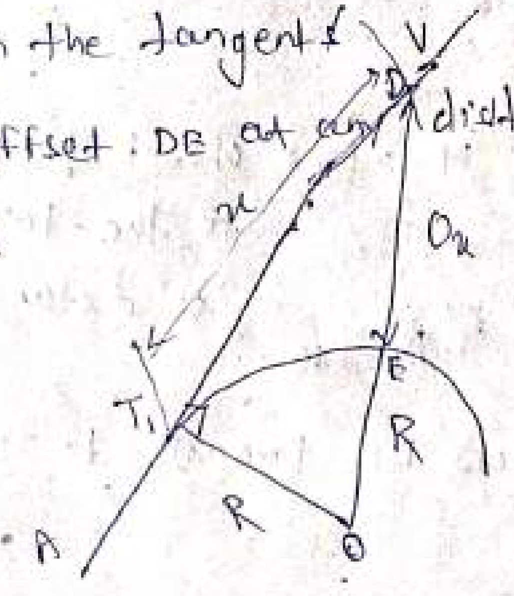
1. Radial
2. Perpendicular

(1) Radial offsets

By offsets from the tangents

Let O_x = Radial offset DE at any distance 'x' along the tangent

$\therefore T_1D = x$



From ΔT_1DO

$$(DO)^2 = (T_1O)^2 + (T_1D)^2$$

$$(DE + EO)^2 = (R)^2 + (x)^2$$

$$(O_x + R)^2 = (R)^2 + (x)^2$$

$$\boxed{O_x = \sqrt{R^2 + x^2} - R}$$

If radial value is very high, then we calculate approximate expanation value :-

Expand :- $\sqrt{R^2 + x^2}$

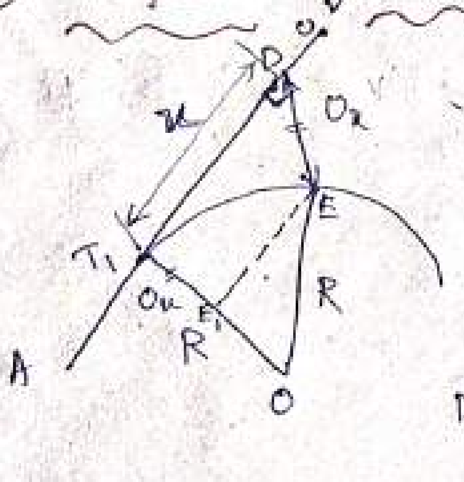
$$O_x = R\left(1 + \frac{x^2}{2R^2} - \frac{x^4}{8R^4} + \dots\right) - R$$

(Binomial Equation)

$$O_x = R + \frac{x^2}{2R} - R$$

$$\boxed{O_x = \frac{x^2}{2R}}$$

(ii) Perpendicular offsets



DE = O_x = offset perpendicular to the tangent at 'x' distance

$T_1D = x$

Draw EE_1 parallel to the tangent

In ΔE_1EO

$$(EO)^2 = (E_1O)^2 + (EE_1)^2$$

$$(R)^2 = (T_1O - T_1E_1)^2 + (x)^2$$

$$(R)^2 = (R - O_x)^2 + (x)^2$$

$$\boxed{O_x = R - \sqrt{R^2 - x^2}}$$

Approx Expression

Expand $\sqrt{R^2 - x^2}$

$$O_x = R - R\left(1 + \frac{x^2}{2R^2} - \frac{x^4}{8R^4} + \cdots\right)$$

$$O_x = R - R + \frac{x^2}{2R}$$

$$\boxed{O_x = \frac{x^2}{2R}}$$

Q: Determine the offsets to be set out at 1/2 chain interval along the tangents to locate a 16-chain curve, the length of each chain is 20m.

Radial offset

$$O_x = \sqrt{R^2 + x^2} - R$$

$$O_{0.5} = \sqrt{(16)^2 - (0.5)^2} - 16$$

$= 0.0078$ chains

$= 0.0078 \times 20m$ (chain length)

$= 0.16\ m$

$$O_1 = \sqrt{(16)^2 + (1)^2} - 16$$

$= 0.031$ chain $\times 20m = 0.62m$

$$O_{1.5} = \sqrt{(16)^2 + (1.5)^2} - 16$$

$= 0.0702$ chain $\times 20 = 1.40m$

(ii) Approx method

$O_x = x^2/2R$ $O_1 = \frac{(1)^2}{32} = 0.0311$ chain × 20 $= 0.62m$

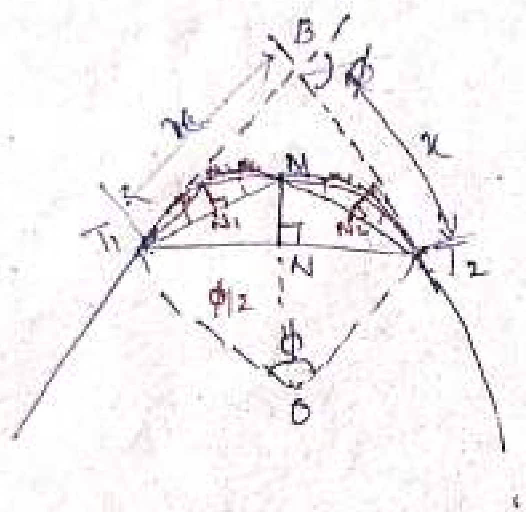
$O_{0.5} = \frac{(0.5)^2}{2 \times 16}$

$= 0.0078 \times 20$

$= 0.16m$

③ Successively bisecting the arcs

Radius of curve = R



Bi chord T_1M & MT_2

Step ① Tangent length $= R\tan\phi/2$

Distance $= (\text{Let } x_{m/n}) = T_1B = T_2B$

② Marked point T_1 & T_2

③ Join chord T_1T_2 & Bisect it.

④ Draw a perpendicular at point 'N' such that length of Perpendicular MN = Mid ordinate.

$MN = R(1 - \cos\phi/2)$

⑤ Draw two new chords T_1M & T_2M & Bisect them at point N_1 & N_2

$M_1N_1 = R(1 - \cos(\phi/2)/2)$

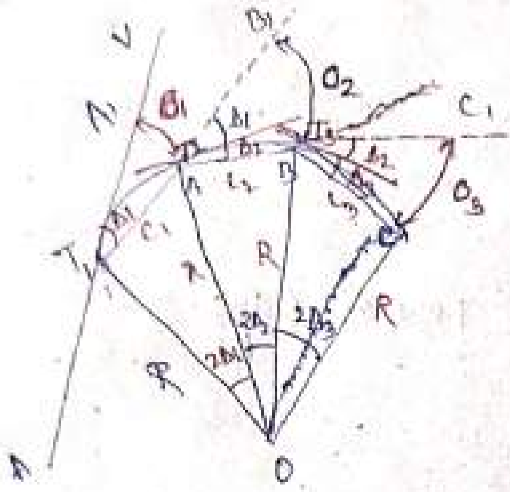
$= R(1 - \cos\phi/4)$

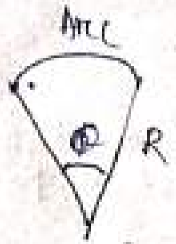
② Taking offsets from the chord produced / or offsets from chord produced : ← (Deflection Distances)

Sub chord - $c_1\ c_2\ c_3$

subchord of Tangent = Δ_1 | ϕ (Deflection angle)

Subtended angle betw A (opposite angle) T_1AO so $2\Delta_1$

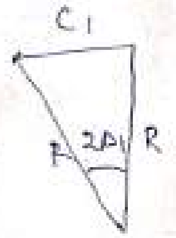




Arc = $R\theta$

$$\theta = \frac{Arc}{R}$$

from $\triangle T_1AO$



$$\theta = \frac{Arc}{R}$$

So $2\Delta_1 = \frac{c_1}{R}$

$$\Delta_1 = \frac{c_1}{2R}$$

Similarly $\Delta_2 = \frac{c_2}{2R}$

$\Delta_3 = \frac{c_3}{2R}$

nth Δ - $\Delta_n = \frac{c_n}{2R}$

$$O_n = \frac{c_n}{2R}(c_{n-1} + c_n)$$

from offsets $A'TA$

Arc = $R\theta$

$O_1 = c_1\Delta_1$

$= c_1 \times \frac{c_1}{2R}$ $\left(\because \Delta_1 = \frac{c_1}{2R}\right)$

$O_1 = \frac{c_1^2}{2R}$

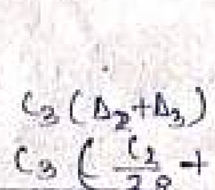
Let c (chords are same) $c_2 = c_3 = c_4 = c_n$ are equal then

$O = \frac{c}{2R}(c+c)$

$= \frac{2c^2}{2R}$

$$O = c^2/R$$

from $B'AB$



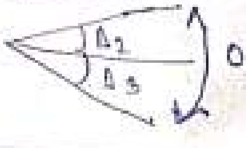
$\theta = \Delta_1 + \Delta_2$

$O_2 = c_2(\Delta_1 + \Delta_2)$

$= c_2\left(\frac{c_1}{2R} + \frac{c_2}{2R}\right)$

$$O_2 = \frac{c_2}{2R}(c_1 + c_2)$$

from $C'BC$



$O_3 = c_3(\Delta_2 + \Delta_3)$

$O_3 = c_3\left(\frac{c_2}{2R} + \frac{c_3}{2R}\right)$

$$O_3 = \frac{c_3}{2R}(c_2 + c_3)$$

$$O_3 = \frac{c_3}{2R}(c_2 + c_3)$$

Pb :- Two tangents intersect at a chainage of 1,000 m, the deflection angle being 30°. calculate all the necessary data for setting out a circular curve of radius 200 m by the method of offsets from the chord produced, taking a peg interval of 20m.

Sol Given data

$\phi = 30°$

$R = 200m$

chainage of intersection point = 1·000 m (Intersection point A & B of tangent length T_1 & T_2)

full chord = 20m (C_2)

① Tangent length $= R \tan \phi/2$

$= 200 \times \tan 15°$

$= 53.58m$

② Curve length $= \frac{\pi R \phi°}{180°} = \frac{\pi \times 200 \times 30}{180} = 104.72m$

③ chainage of first tangent point (T_1) $= 1000 - 53.58$

$= 946.42m$

④ chainage of second tangent point (T_2) $= 946.42 + 104.72$

$= 1,051.14 m$

⑤ Initial sub-chord $= 950.00 - 946.42$

$= 3.58m$

⑥ No. of full chords of length 20m (C_2) = 5

chainage covered $= 950.00 + 100.00$

$= 1,050.00 m$

⑦ final sub-chord $= 1,051.14 - 1,050.00$

$= 1.14m$

⑧ First offset for initial sub-chord,

$O_1 = c_1^2 / 2R$

A projection ... on

$$O_1 = \frac{(3.58)^2}{2 \times 200} = 0.03\,m$$

∴ Second offset for full chord,

$$O_2 = \frac{C_2(C_1+C_2)}{2R} = \frac{20\,(3.58+20)}{2 \times 200} = 1.18\,m$$

Third offset for full chord,

$$O_3 = \frac{C_3^2}{R} = \frac{20^2}{200} = 2.0\,m$$

Fourth offset for full chord

$$O_4 = \frac{C_4^2}{R} = \frac{20^2}{200} = 2.0\,m$$

Fifth offset for full chord

$$O_5 = \frac{C_5^2}{R} = \frac{20^2}{200} = 2.0\,m$$

Sixth offset for full chord

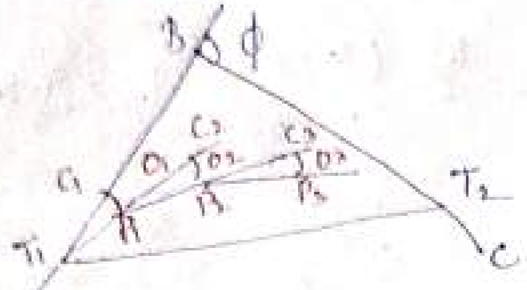
$$O_6 = \frac{C_6^2}{R} = 20^2/200 = 2.0\,m$$

Seventh offset for final sub-chord

$$O_7 = \frac{1.14\,(20+1.14)}{2 \times 200} = 0.06\,m$$

NOTE :- There will be a total of seven offsets, one for the initial sub-chord, five for full chords, and one for the final sub-chord. Here, the third through sixth offsets will be of the same length.

Field procedure for setting out curve by method of offsets from chord produced :—



① Suppose AB and BC are the tangents, and B is the point of intersection

② By calculating the tangent length, points T_1 and T_2 are marked on the ground with pegs.

③ The curve length is calculated and then the chainages of T_1 & T_2 are found out.

④ The lengths of the initial and final sub-chords and the number of full chords are determined.

⑤ The offsets for the initial sub-chord, full chord and final sub-chord are calculated.

⑥ The distance T_1C_1 is marked along the rear tangent AB so that T_1C_1 is equal to the initial sub-chord.

⑦ The zero end of the tape is held at T_1 and an arc of radius T_1C_1 is drawn. from this arc, a distance C_1P_1 is cut off as the first offset (O_1).

(8) The line T_1P_1 is now extended by a distance P_1C_2, which is the second chord (i.e a full chord)

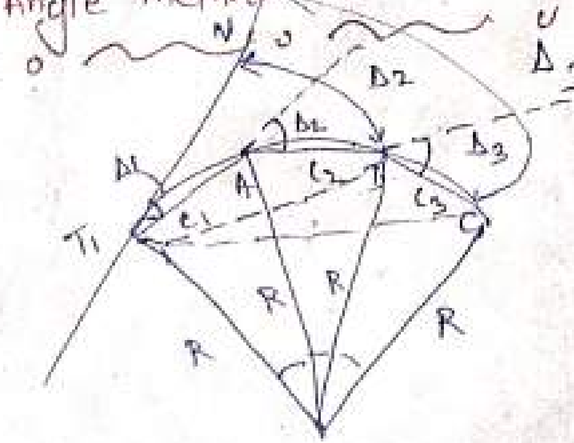
(9) Then the zero end of the tape is held at P_1 and an arc of radius P_1C_2 is drawn. from this arc, a distance C_2P_2 is cut off as the second offset (O_2).

(10) This process is continued until the second tangent point T_2 is reached.

(11) The last point should coincide with T_2. If it does not, the amount of error is found out. If the error is large, the entire operation should be repeated.

If the error is small, all the points are moved sideways by an amount proportional to the square of their distances from T_1. The error is thus distributed among all the points of the curve.

Instrumental Method - Horizontal curve setting By Deflection Angle method or Rankine's method



Chapter - 05

Basics of Aerial Photography, Photogrammetry, DEM and Ortho Image generation.

* Aerial photography
  (Film, focal length, scale)
  (Types of Aerial photographs (oblique, straight))

* Photogrammetry
  (Classification of photogrammetry)
  (Aerial photogrammetry)
  (Terrestrial photogrammetry)

* Photogrammetry process
  (Acquisition of imagery using aerial and satellite platform)
  (Control survey)
  (Geometric distortion in imagery)
  (Application of imagery and its support data)
  (Orientation and triangulation)
  (Stereoscopic Measurement)
  - (x - parallax)
  - (y - parallax)

* DTM / DEM Generation
* Ortho image generation.

Aerial photography

These photographs are taken from camera stations in the air with the axis of the camera vertical or nearly vertical.

According to the direction of the camera axis at the time of exposure, aerial photographs are further divides into the following main class

* vertical photographs.
* oblique photographs

Vertical photographs:

These photographs are taken from the air with the axis of the camera vertical or nearly vertical. a truly vertical photograps closely resembles a map these are utilised for the compilation of topographical and engineering surveys on various scales.

Oblique photographs:

These photographs are taken from air with the axis of the camera intentionally tilted from the vertical. An oblique photograph covers larger area of the ground but clarity of details diminishes towards the far end of the photograph. Depending upon the angle of obliquity.

Oblique photographs may be further ~~tired~~ divided into two categories

* Low oblique photographs
* High oblique photographs.

Low oblique photographs

An low oblique photograph which does not show the horizon is known as low oblique photograph it used to compile reconnaissance maps of the accessible areas.

High oblique photograph

An oblique photograph which is sufficiently tilted to show the horizon is known as high oblique photograph. such photographs are generally used to previously used for the extension of planimetric and height control in areas having scanty ground control.

Aerial photography:-

mapping of large areas from aerial photographs is faster and cheaper than only other method yet developed. with the aerial photographs more complete and ~~height control in areas having scanty ground~~ accurate topographic maps contours can be accurately surveyed upto 50cm vertical interval.

Scales ranging 1:500 to 1000,000

<u>Photogrammetry:</u>

Photogrammetry surveying or photogrammetry is the branch of surveying in which maps are prepared from photo-graphs taken from ground or air stations.

photogrammetry may high speed imaging and remote sensing in order to detect, measure and record complex 2D and 3D motion fields.

<u>Broadly photogrammetry requires</u>.

* Planing and taking the photographs
* processing the photographs
* measuring the photographs and reducing the measurement to produce end results.

<u>field application of photogrammetry</u>.

1. used to conduct topographical survey or engineering surveys.

* suitable for mountains and hilly terrain with little vegetation.

* used for projects demanding higher accuracy, since it provides accurate measurements.

* used for geological mapping which includes identification of land forms, rock type and rock structure.

* used in urban and regional planning application.

advantages of photogrammetry.

* covers large area.
* less time consuming.
* can reach inaccessible and restricted area.
* cost effective for large area and in a long run.
* used to interpret, understand.

Disadvantages of photogrammetry:-

* complex system, highly trained human resource needed.
* costly at the time of installation / initiation.
* Heavy and sophisticated equipments needed.

* lengthy administrative procedure for getting permission to fly.
* weather dependent.

For photogrammetry, photographs may be taken from known positions on the ground or from the camera mounted in an aircraft.

Survey in which the photographs taken from the ground are used is known as Terrestrial Photogrammetry and the survey based on the photos taken from the flying aircraft is known as aerial Photogrammetry.

Photo-theodolite:-

photo theodolite is a combination of a theodolite and a photographic camera fitted on a tripod with its axis horizontal. The camera box is mounted on the axis exactly in the same manner as the vernier plate of the theodolite.

Difference between terrestrial photogrammetry and aerial photogrammetry

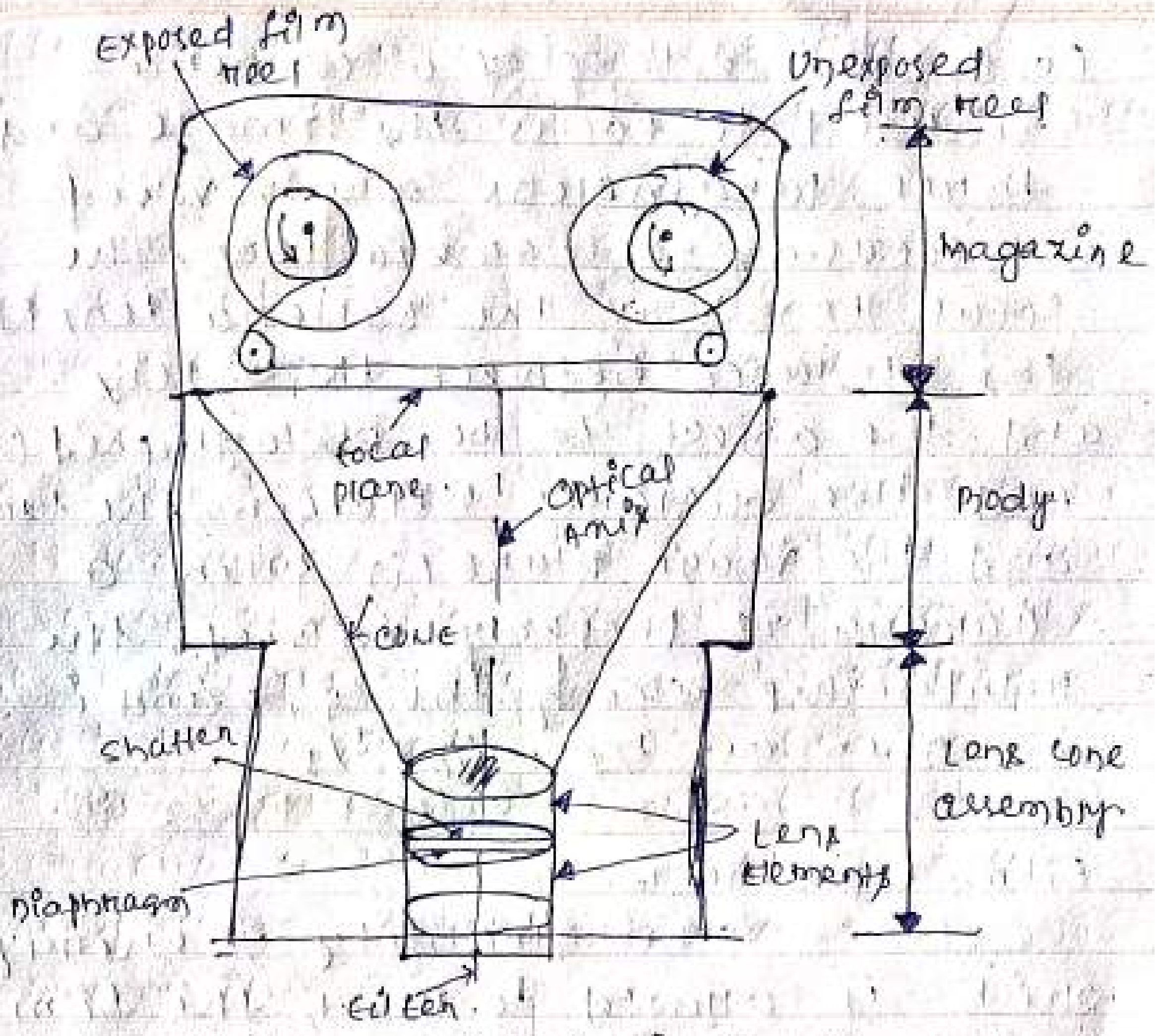
Terrestrial photo-grammetry	Aerial Photogrammetry
* It is the branch of photographic surveying in which the photographs of the area to be surveyed are taken by keeping the camera on the ground with horizontal camera axis	* It is the technique the reliable measurement from the photographs taken by keeping the camera in aeroplane with vertical axis of camera.
* Used for preparation of small scale maps and highly mountains, open country where the conventional methods of survey are not possible	* Used for preparing small scale map of large area and large scale map of small area
* Speed of work and accuracy is less	* Speed of work and degree of precision is higher.
* less accuracy as compared to tape transit and plane table survey	* It is somewhat less accurate.

* Equipment used - photo theodolite, photographic plate.	* Equipment required - aerial camera, aircraft, plotting machine.
* The position and the orientation of photo theodolite are measured directly at the time of exposure.	* A camera with its axis vertical, mounted on an aircraft.
* Useful for architectural and archaeological monuments	* Useful for military and intelligence.

Aerial camera:-

An aerial camera is one which is specifically designed for use in aircraft. A frame aerial camera is capable of producing photographs of high pictorial (image or picture) quality as well as maintains geometric accuracy.

It is mechanical optical instrument with automatic and electronic elements. It is designed for obtaining aerial photograph of the earth surface from an airplane or air craft.



(principal components of a single lens frame aerial camera)

<u>component parts of Aerial camera:-</u>

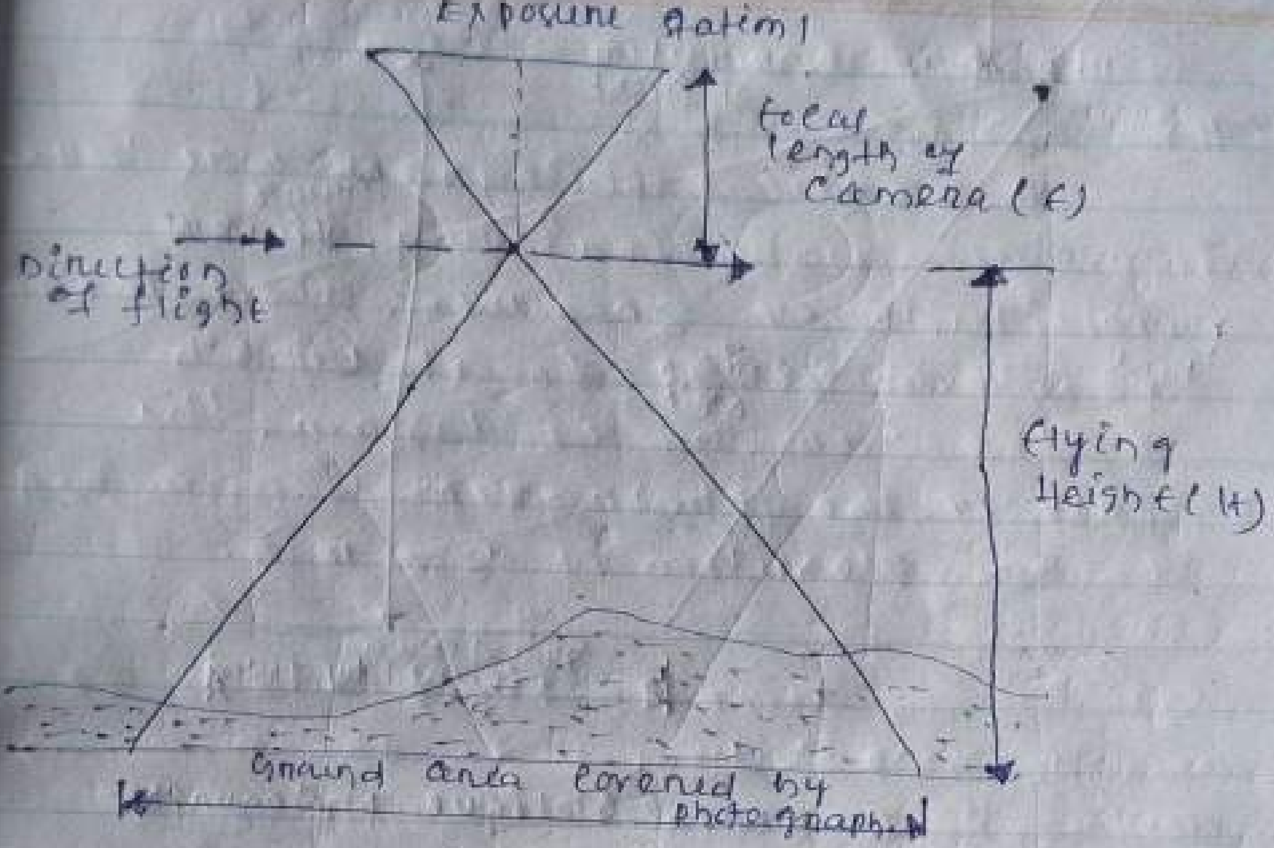
focal plane:-

The plane in which all incident rays are brought to focus. in aerial photograph object distance greater than image distance. thus to have focus fixed for @ infinite object distance. this is done by setting the focal plane as exactly as possible at a distance equal to the focal length behind the rear nodal point of the camera lens.

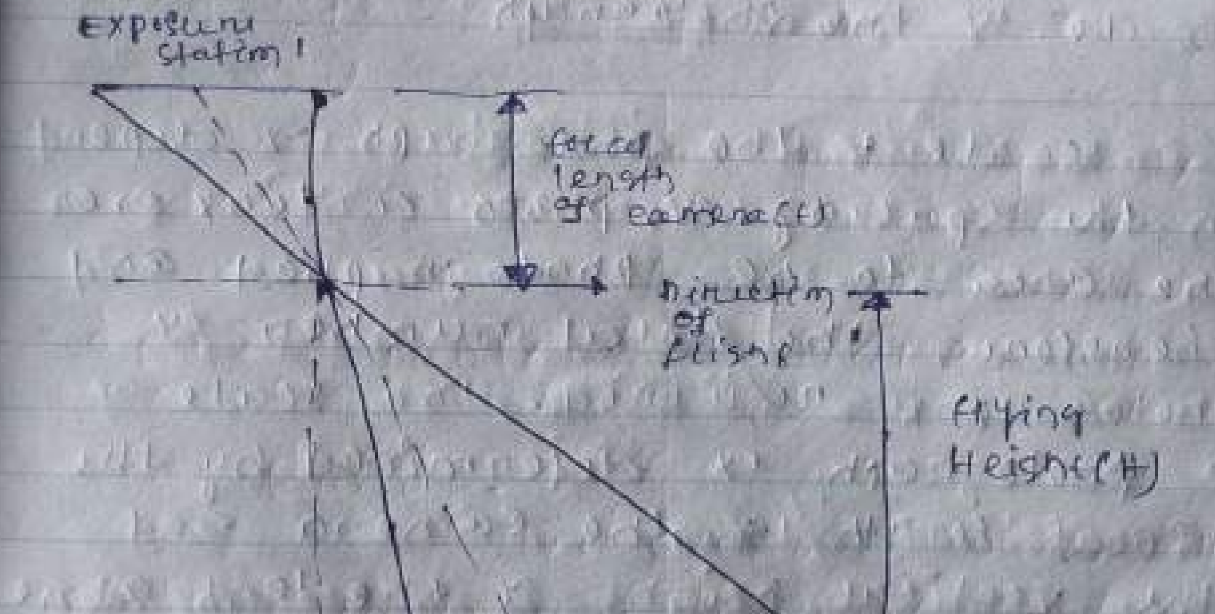
focus :- The lens of a camera is used to focus the image and three parameters are involved in focusing a camera lens, the focal length of the camera lens (f) the distance between the lens and the object to be photographed (o) and the distance between the lens and the image plane (i) when a camera is properly focused, the relationship among the f, o and i is given as $1/f = 1/o + 1/i$.

film exposure :-

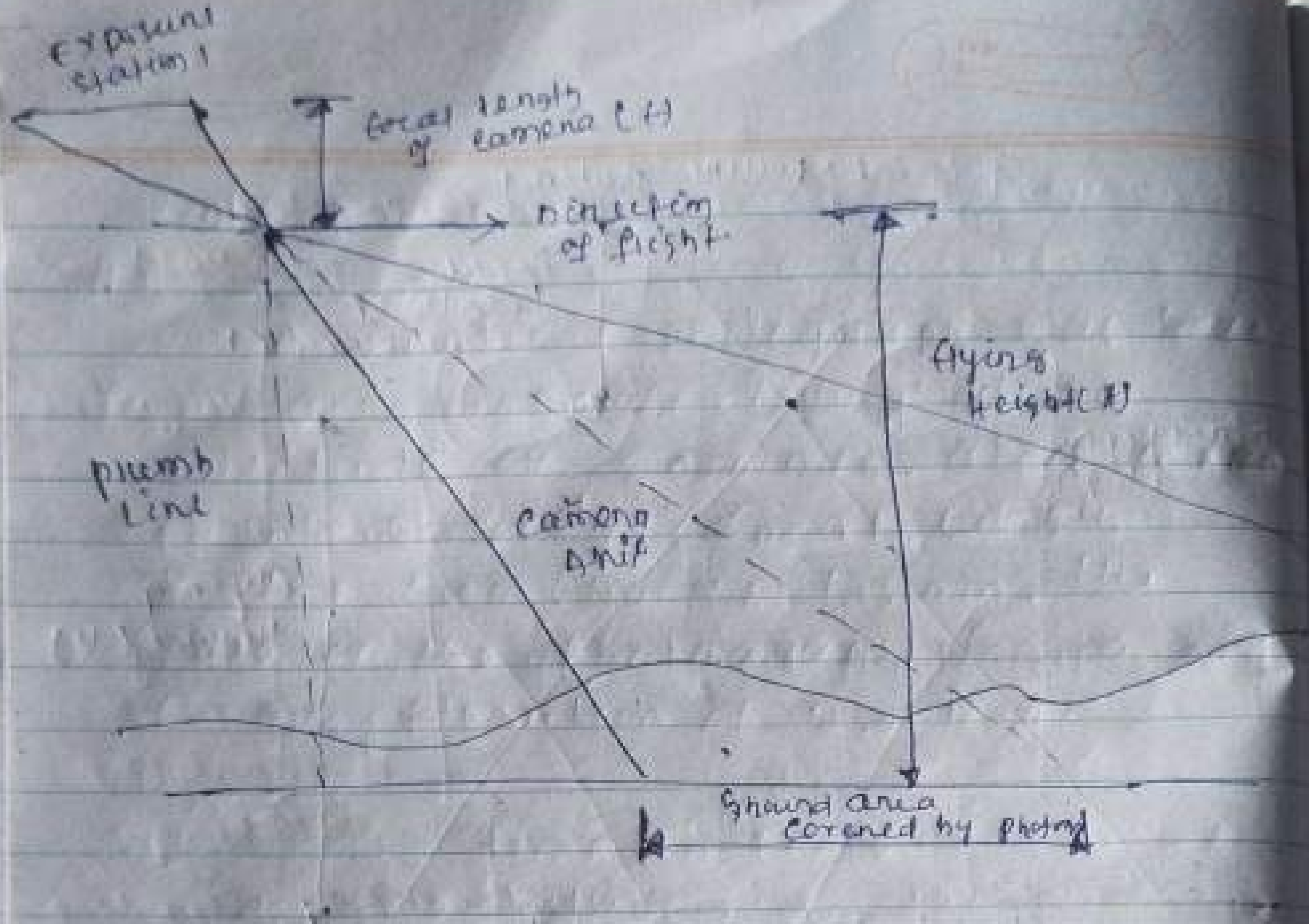
The quantity of energy which is allowed to reach the film is called film exposure. the exposure at any point on a photographic film depends on several factors, including the scene brightness, the diameter of the camera lens opening, the exposure time and the camera lens focal length. the energy is largely controlled by the relative aperture and shutter speed of the camera as well as the intensity of the energy source.



(vertical photograph).



(Tilted photograph).



(Oblique photography)

Scale of the photograph :-

Scale of the photograph is depend on the spacing of photo centers over the area to be photographed and therefore, the total number of photographs required. The scale of a photograph is determined by the focal length of the camera and the vertical height of the lens above the ground.

The focal length (f) of the camera is the distance measured from the centre of the camera lens to the film.

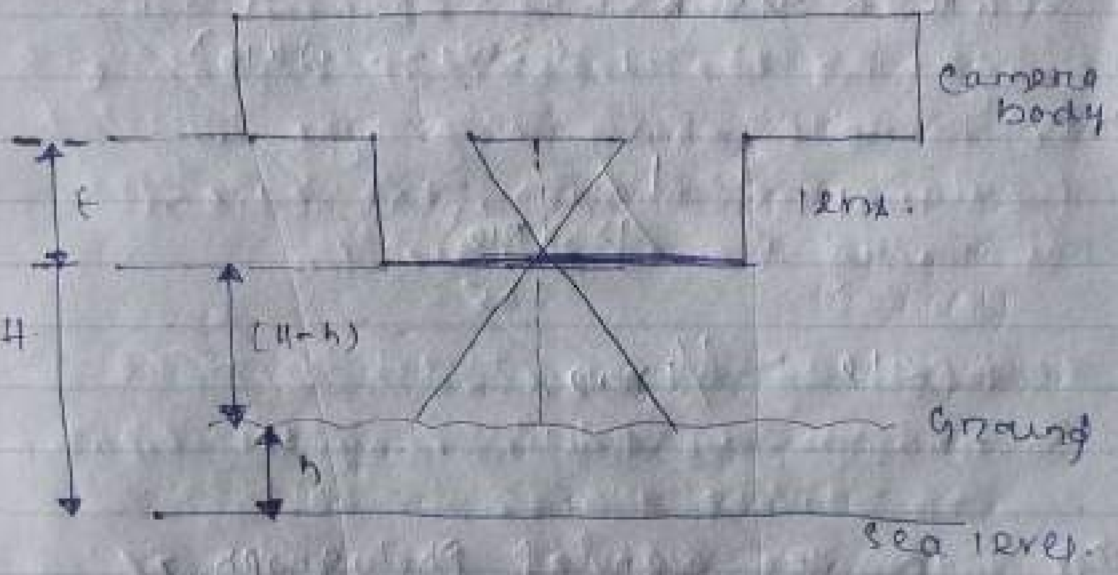
The vertical height of the lens above the ground (H - h) is the height of the lens above sea level (H) minus the height of the ground above sea level (h), when the optical axis is vertical and the ground is flat.

Formula:-

$$S = f/(H-h)$$

Scale may be expressed three ways

* Unit Equivalent.
* Representative fraction.
* Ratio.



Scale problem :-

Q1) 1

A vertical photograph was taken from an aircraft flying at an altitude of 2000 mtr above mean sea level, the focal length of the camera is 175 mm. the scale of the photograph for a hill of an elevation of 250 mtr.

Sol^n

Given data.

H = 2000 mtr.

f = 175 MM.

h = 250 mtr.

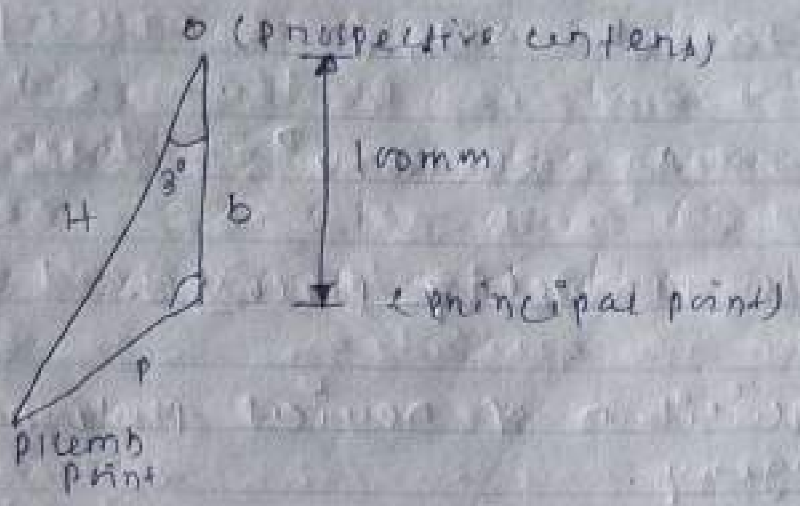
$$\text{Scale} = f/H-h = \frac{175\,MM}{(2000-250)\times 10^3}$$

$$= 1 / \frac{1750 \times 10^3}{175}$$

$$\text{Scale} = 1/10000$$

Q1) in an aerial photograph, if the photograph has a tilt of 3° and the focal length is 100mm. then the distance between the perspective center and the plumb point will be.

Sol^n



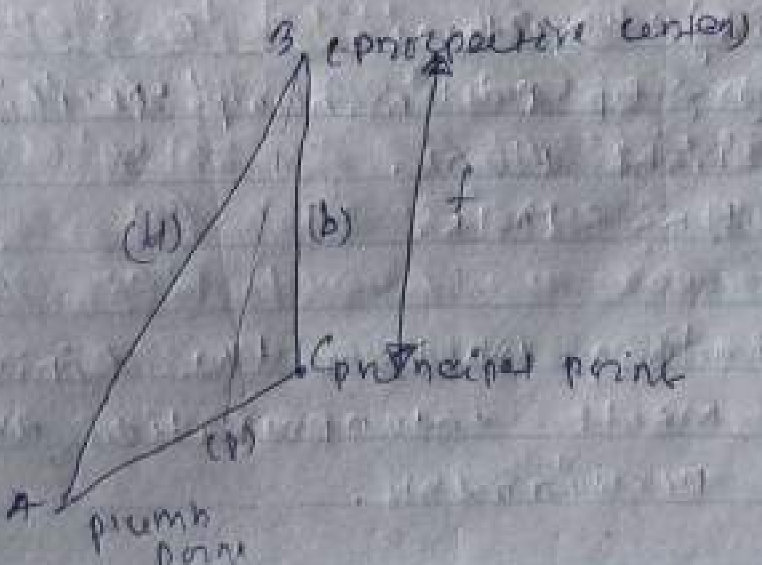
$\cos t = b/H$

$\cos 3° = 100/H$

$H = 100/\cos 3°$

Q11 In aerial photograph, if the photograph has a tilt of 4° and the focal length is 200mm, then the distance between the principal point and plumb point will be.

Sol^n



$\tan \theta = P/B$

$\tan 4^\circ = AB/200$

$AB = 200 \tan 4^\circ$

<u>Photogrammetry process</u>:-

* <u>Acquisition of aerial photographs and imagery</u>.

From time to time there is considerable interest in the purchase of special-purpose photography contracted through commercial survey firms.

Even though commercial firm may have the technical expertise to handle almost any type of photographic mission, the client still may be responsible for

* Defining objectives..
* Drawing up preliminary specification on flight plans.
* Estimating costs.
* Determining whether the finished product meets interpretation and maping requirements.

Ground coverage vs. focal length variations

In practice the range of scale actually employed is usually from 1:1,000 about 1:50,000.

When simultaneous coverage at two different scales is desired, certain aircraft can be fitted with two maping cameras employing different focal length lens.

Control survey -

Ground control consists of establishing a set of well-defined ground points, which are distinctly visible in aerial photographs and determining their relative positions by usual ground surveying. these points are known as control stations or control points. At least three to four control points should appear in each photograph.

Geometric Distortions.

Geometric distortion is an error on an image between the actual image coordinates and the ideal image coordinates which would be projected theoretically with an ideal sensor and under ideal conditions.

All images are susceptible to geometric distortions caused by variations in platform stability including changes in their speed, altitude and angular orientation with respect to the ground during data acquisition.

<u>Application of imagery and it's support data.</u>

Imagery is a data type that is extremely useful for GIS. It comes in many different types and it is able to show both large and small areas in varying levels of detail. This makes it versatile for a wide variety of different GIS projects and as such it is a common type of data for GIS projects.

Image data is used to store remotely sensed imagery i.e satellite scenes or orthophotos or ancillary graphics such as photographs, scanned plan documents.

most of the common image processing functions available in image analysis systems can be categorized into the following four categories

* preprocessing
* Image Enhancement
* Image Transformation
* Image classification and analysis.

Orientation and triangulation:-

Triangulation is the principle used by both photogrammetry and theodolites to produce 3-dimensional point measurement.

The exterior orientation aims to define the position and rotation of the camera at the instant of exposure. in photogrammetry, three fundamental conditions are frequently used to compute the exterior orientation parameter. these conditions are known as collinearity, coplanarity and coangularity conditions.

Stereoscopic measurement :-

An stereoscope is an instrument which enables the surveyor to observe a pair of nearly identical photographs, one with each eye and fuse the photograph into a single three-dimensional picture. thus, the primary function of an stereoscope is to accommodate the wide separation of the individual photographs to the fixed length of the eye base.

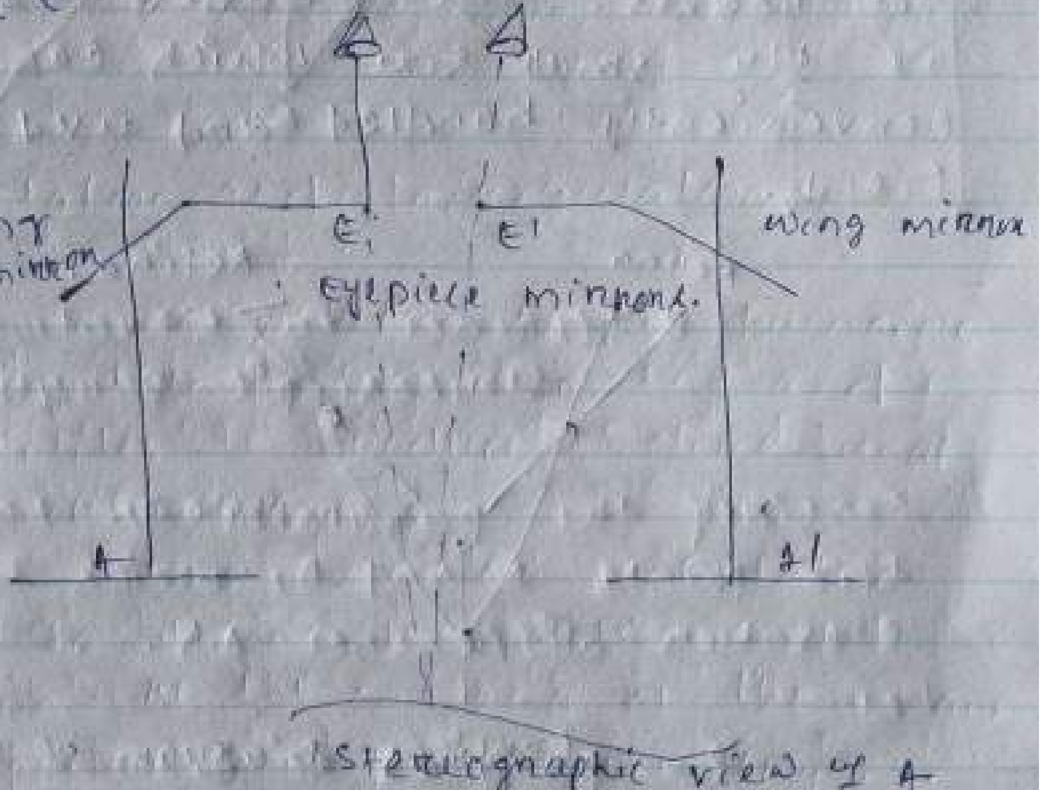
the following two types of stereoscopes are commonly used by surveyors.

* mirror stereoscope.
* lens stereoscope.

Mirror stereoscope :-

It consists of an arrangement of four mirrors each of which is oriented at an angle of 45° with the plane of the photograph. the photographs to be viewed stereoscopically are placed at a certain distance from the wing mirrors with correct orientations.

The image of the object say A, strikes the respective eyes. The brain gets the stereoscopic view of the object. A pair of magnifying lenses are placed at E and E' for getting magnified view. In same types of mirror stereoscopes a set of removable binoculars are placed at the positions E and E'.

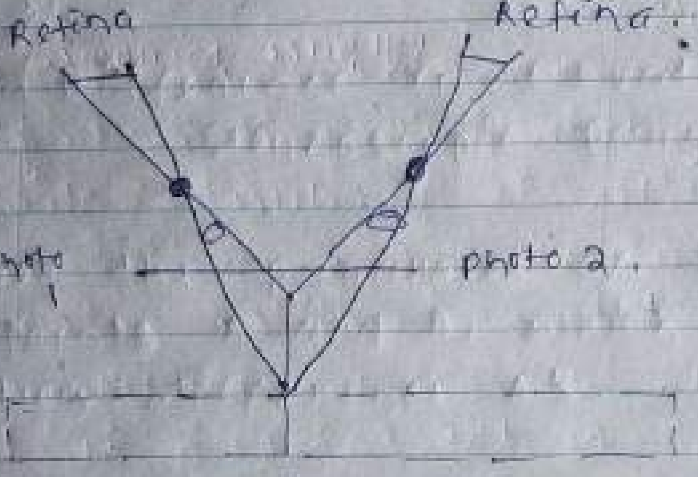


Lens stereoscope :-

A lens stereoscope consists of one magnifying lens for each eye. The lenses are fitted on an assembly, the distance between them being equal to the average eye base. A provision is made to make minor adjustment in eye base to suit the user.

The distance between the photographs and the lenses is decided by the focal length of the lens.

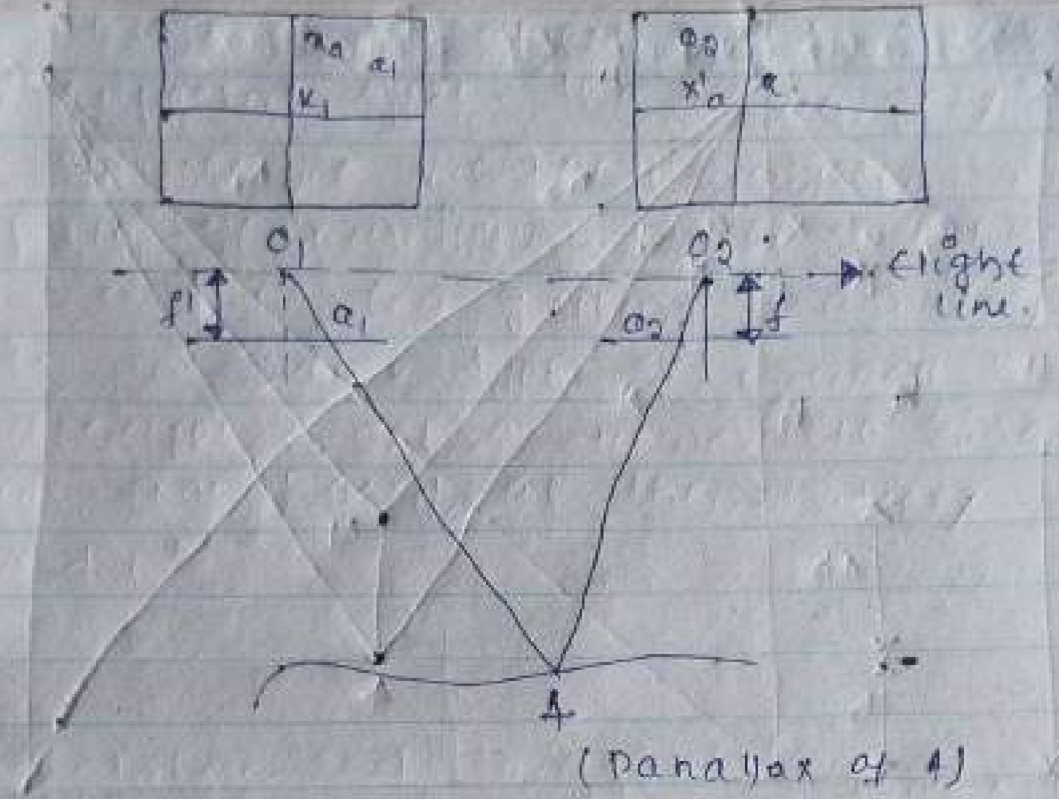
The disadvantage of a lens stereoscope is the strain to the eyes due to mismatch of the correct distance between photographs and leaves. the greatest advantage is the small size which can be conveniently handled and used in the field.



(optical Diagram of lens stereoscope)

Parallax:

parallax of a point is the displacement of the image of the point on two successive exposures.



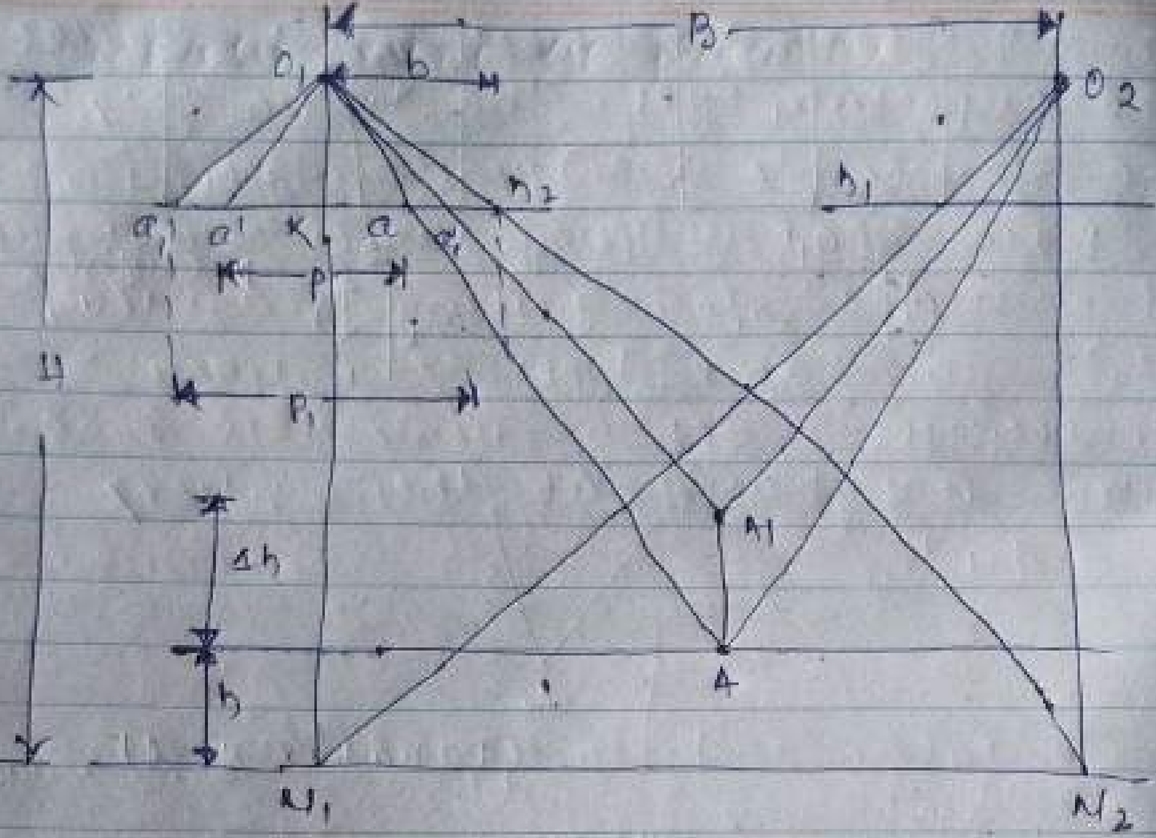
(Parallax of A)

Taking the line of flight as X-coordinate, let x_a be the coordinate of a_1 in photograph at O_1 and x_a' the coordinate of a_2 in photograph at O_2.

Then parallax of A

$P = x_a - x_a'$

Difference in elevation by stereoscopic parallaxes.



(Difference in elevation by stereoscopic parallaxes)

Consider the parallaxes of the two points A and A_1 which have elevation difference Δh.

Let

p = parallax of A

p' = parallax of A_1

their difference of parallaxes

$$\Delta p = p' - p$$

By drawing $O_1a' \parallel O_2A$, a' is located.

$$p = a'a$$

similarly, by drawing $O_1a_1' \parallel O_2A_1$, a_1' is located on the photograph.

$$p' = a_1'a_1$$

From similar triangles $O_1 a' a$ and $O_1 A O_2$.

$$f/(H-h) = D/B$$

$$P = Bf/(H-h) \quad \text{——— (1)}$$

From similar triangles $O_1 K h_2$ and $O_1 N_1 N_2$

$$f/H = b/N_1 N_2$$
$$= b/B$$

$$Bf = bH \quad \text{——— (2)}$$

From equations (1) and (2) we get.

$$P = \frac{bH}{H-h}$$

$$P' = \frac{bH}{H-h-\Delta h}$$

$$\Delta P = P' - P = \frac{bH}{H-h-\Delta h} - \frac{bH}{H-h}.$$

$$= bH\left[\frac{1}{H-h-\Delta h} - \frac{1}{H-h}\right]$$

$$= bH \cdot \frac{\Delta h}{(H-h-\Delta h)(H-h)}$$

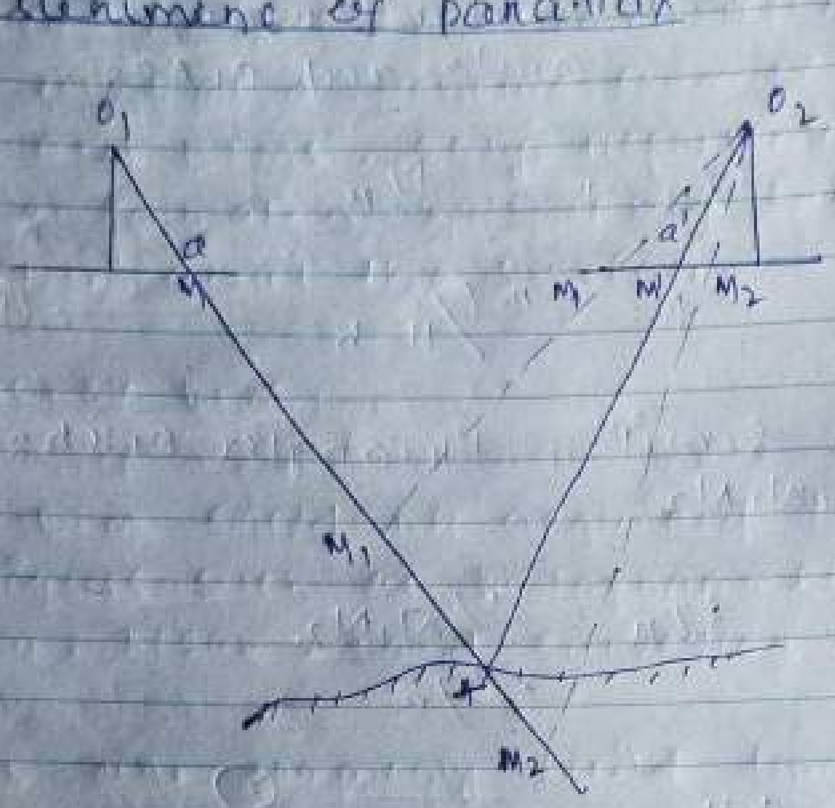
$$\Delta P\,(H-h-\Delta h)(H-h) = bH\Delta h$$

$$\Delta P\,(H-h)(H-h) - \Delta h\,\Delta P\,(H-h) = bH\Delta h$$

$$\Delta h\,[bH + (H-h)\,\Delta P] = (H-h)^2\,\Delta P$$

$$\boxed{\Delta h = \frac{(H-h)^2\,\Delta P}{bH + (H-h)\,\Delta P}}$$

Measurement of parallax



It is seen that if difference in parallax Δp is known, the difference in elevation Δh between the two points can be calculated. To measure the difference of parallax Δp between a pair of overlapping photographs an instrument called a parallax bar is used. The measurement of parallax by parallax bar is based on the principle of floating mark.

On photograph on left side the ground point A appears at m and on right hand side photograph it appears at m'. If the left side photograph is held at fixed position and right hand side photograph is slightly pushed so as to get m' at position m_1, the ground point A shifts to position m_1; similarly with the right hand side photograph it is slightly moved so that m' is shifted to m_2.

For measuring the difference in parallax between two overlapping points, the photographs are on a large sheet of paper such that their flight lines are carefully aligned.

If height h of one point is known the difference of elevation between the two points may be found using

$$\Delta h = \frac{(H-h)^2 \Delta p}{bH + (H-h)\Delta p}$$

Since the value of b may slightly vary for the two points, it should be taken as mean value.

DTM / DEM Generation:

There is no common usage of the terms digital elevation model (DEM), digital terrain model (DTM) and digital surface model (DSM) in scientific literature.

The term digital elevation model is often used as a generic term of DSMs and DTMs, only representing height information without any further definition about the surface.

Types of DEM

* The DEM could be acquired through techniques such as photogrammetry, lidar, IfSAR, land surveying etc.

* DEMs are commonly build using data collected using remote sensing techniques, but they may also be built from land surveying.

* DEMs are used often in geographic information systems, and are the most common basis for digitally produced relief maps.

* While a DSM may be useful for landscape modelling, city modeling and visualization applications, a DTM is often required for flood or drainage modeling, land-use studies, geological application and other applications.

DTM Acquisition :-

* Photogrammetric data capture (passive sensor)

* Aerial photography.

* Digital satellite imagery.

- Image matching used to automatically extract a dense
- point cloud of 3D surface points from stereo image pairs and potentially multi-image coverage. DTM derived via regularization of the point cloud.

RADAR :- Radio Detection and Ranging (Active sensor)

LIDAR :- Light Detection and Ranging (active sensor)

* Digitized contour maps.
* Ground surveying

DTM Application

* Planning and Resource management
* The management of natural resources.
* Site location.
* Support of image classification in remote sensing by DTM derivatives.

* Geometric and radiometric correction of remote sensing image.
* Wind flow and pollution dispersion models.

DTM manipulation:-
→ Nearest neighbour assignment.
→ Linear interpolation.
→ Bilinear interpolation.
→ Cubic convolution.

Ortho image generation:-

An orthophoto or ortho-image is an image that is free of distortion (it has been ortho-rectified) and which is characterized by a uniform scale over its entire surface. The consider by simplifying that it is like if each element shown on the image has been photographed directly from the vertical over it.

Orthophotographs are produced by scanning an aerial photo diapositive, orthorectifying the digital image, and registering it to a coordinate system and map projection.

orthophotographs have the positive attributes of a photograph such as detail and timely coverage, and the positive attributes of a map including uniform scale and true geometry this enables orthophotographs to be used in their primary role as a backdrop on which map features can be over laid.

The most important disadvantages of orthophoto maps are explained below. Each topographic detail/feature can not be seen easily on the orthophoto maps. understanding and interpretability of an orthophoto map is closely connected to the photographic interpretation capabilities of the map users.

An orthophoto map is therefore a vertical aerial photo on a lower level of 1:10,000 with contour lines added. contour lines of 5 metre intervals are used in order to identify detail. Even names of places, railway lines and roads are shown on the photo.

——— End ———

Theodolite:-

Theodolite is a surveying instrument and precision instrument for the measurement of horizontal and vertical angles. it has wide applicability in surveying such as locating points on line, laying grades, determining difference in elevation, setting out curves, aligning tunnels, mining works etc. they are classified into two major categories as * Transit theodolite.
* Non-transit theodolite.

In a transit theodolite, the telescope can be inverted in the vertical plane, whereas the rotation in the same plane is restricted to a semi-circle in a non-transit theodolite, which has become obsolete.
(mugdha)

principle, features and use of micro-optic theodolite and Digital theodolite.

optical theodolite :-

optical theodolite are also known as glass arc or micro optic theodolite. in this the graduated brass or silver circles are replaced by graduated glass circles, the graduations are photographically printed on a glass disc a system of prisms and lenses is used by means of which an image of graduations, near the point at which they are being read, is reflected into the microscope. the eyepiece of the microscope lies near the eyepiece of the telescope. the observer can thus simultaneously see the object as well as the reading without changing his position from the end of the telescope to the side of the instrument

when a theodolite is fitted with a glass circle, ray of light generally pass through the glass of the circle, and are then reflected by prisms or mirrors to the micrometer. Hence, means must be provided for reflecting ordinary sunlight into the interior and then through the circle and micrometers. sometimes, electric illuminates provided by dry batteries usually fitted to supply natural illumination.

The horizontal and vertical circles are both graduated to 20". the microscope scales facilitate the reading of graduations.

The microscope scales are graduated to 2', estimating the 1/10 of a scale division the accuracy of an angular measurement with optical theodolite is ± 9". the important advantage of this theodolite is, it enables all the reading within a single view.

Diagonal eyepiece is fitted to the telescope to read vertical angles upto 80°. this diagonal eyepiece consists of a mirror or a right angles prism attached to the eyepiece. the light rays are turned through 90° and the surveyor can sight objects at quite high elevation, even upto 80°.

The optical theodolite are equipped with optical plummet for accurate centering. it consists of a small eyepiece built into the

tribrach of the theodolite.
A prism, placed at 45°, deviates the ray from the ground station just below the instrument's vertical axis by 90° so as to view it through the eyepiece.

Digital theodolite :-

In today's theodolites, the reading out of the horizontal and vertical circles is usually done electronically with a rotary encoder. Additionally, C.C.D sensors have been added to the focal plane of the telescope allowing both auto-targeting and the automated measurement of residual target offset. An electronic theodolite is similar to an optical theodolite in design. Since, it provides a digital read-out of angles instead of a scale, it is both more accurate and less prone to errors arising from interpolating between marks on the scale or from mis-reading. the read-out is also continuous, so angles can be checked at any time. two measurement systems are used to scan this light pattern and these are known as incremental and absolute.

Digital theodolite :-

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when the horizontal and vertical circle of an electronic theodolite is rotated is an incremental reading system, the amount of incident light passing through to the photodiodes varies in proportion to the angle through which the theodolite has been rotated. this varying light intensity is converted into electrical signals by the photodiodes and these in turn are passed to the microprocessor which converts the signals into an angular output.

Total station:-

Total station is used for computing slant distances, horizontal and vertical angles, elevations in topographic and geodetic works, tacheometric surveys etc. the total station is a pre-eminent contribution to modern surveying and hence the equipment is designed for speed, range, and accuracy. they are a combination of theodolite and electronic Distance measurement (EDM). this enables computing the vertical, horizontal as well as slope measurements.

They acts as a substitute for theodolite, EDM, data collector and a microprocessor, moreover, they are lightweight and compact machines and perform like transit stadia and plane table alidades.

The integration of microprocessors helps in the data collection and measurement computation process. Further to that, the inbuilt software helps to generate the maps instantly.

Applications :-

Apart from taking the measurements the total station helps in computing, interpreting and documenting the data. Here is a list of activities that are computed, interpreted and Analyzed.

* Horizontal Angle.
* Vertical Angle.
* Slope distance
* coordinate of point.
* Missing line measurement.
* Area calculation.
* contour level.

Principle of total station:-

The total station consists of a built-in emitter, capable of emitting microwaves and infrared signals. The wave length of these emitted waves helps in calculating the distance between the points.

Here the distance is calculated by multiplying the time taken to cover a certain distance by the velocity. However, triangulation and trigonometry methods are adopted for computing the angles and determining the coordinates.

Total station is used to measure the following three fundamental quantities in the field.

* Horizontal Angle.
* Vertical Angle.
* Sloping Distance.

* **Measurement of Horizontal Angle.**

For measuring horizontal directions any convenient direction may be taken as reference direction.

usually the direction to a prominent object from first station is taken as reference direction. this direction is called instrument north also. in most of the instruments angular measurements are to the accuracy of 2 to 6 seconds.

<u>vertical Angle</u> :-

The vertical angles are measured taking vertical upward direction as reference. to get vertical angles correctly the instrument should be levelled properly. However the provision has been made in the instrument by providing a sensor that can detect small deviations of the instrument and compensate the verticality. if the deviation is large, the tilt error is indicated which means the instrument should be levelled properly.

If the instrument is not properly levelled, vertical angle display changes with change of direction for horizontal measurement. this can be used as another test for checking the level.

Slope Distance:-
major part of total station is electronic distance measurement (EDM). the instrument always measures the sloping distance from the instrument station to the object. the range of slope distance that can be measured with a total station varies from 2.8 km to 4.2 km. the accuracy of the instrument is quoted in the form.

Accuracy = a mm + b ppm

The constant 'a' is independent of length being measured. it is due to internal sources within the instrument such as unwanted phase shifts in electronic components, errors in phase measurement and difference between the mechanical and electrical centres of the instrument.

The systematic error 'b' is proportional to the distance measured. 1 ppm means one part per million. in metric system it means 1 mm for every kilometer distance measured. it depends upon the atmospheric conditions at the time of measurement.

For long distance measurements, reflectors are used. However in many applications in construction surveying, it is difficult to place a reflector at desired points. For example the point is on cooling tower, on highest point of roofs, the point may be hazardous to go and hold the reflector like in case of waste disposal sites, in such cases a visible red laser beam is used.

Major components:-

The equipment is composed of different parts.

* Handle
* Handle securing screw
* Data input/output terminal
* Instrument height mark
* Battery cover.
* Operation panel
* Tribrach clamp.
* Base plate.
* Levelling foot screw
* Circular level
* Circular level
* Display
* Prism and prism pole
* Objective lens.
* Tubular compass slot
* The optical plummet focusing ring
* Optical plummet reticle
* Optical plummet eyepiece
* Horizontal clamp.
* A horizontal fine motion
* Data input/output
* External power source
* Plate level
* Plate level adjusting
* Vertical clamp.
* A vertical fine motion
* Telescope eyepiece.
* Telescope focusing
* Peep sight.
* Instrument centre mark.

A tripod enables to affix the equipment onto the ground. A handle is available on the top of the equipment for holding it. there are a data input and output terminal below the handle enabling the data transfer to the computer.

The total stations comes with inbuilt software, an operation panel, keyboard, and a screen. the prism and prism poles helps in measuring distances.

The total station consists of an EDM, theodolite and a microprocessor combined into one. they got a memory card for data storage and a battery. A fully charged battery works for about 3 to 5 hrs continuously.

* Coordination determination.
* Distance measurement.
* Angular measurement.
* Data processing.
* Coordination Determination.

The total station determines the coordinate of an unknown point.

The instrument is placed over a unown coordinate, the equipment digitally computes the coordinates.

<u>Distance measurement</u> :-
As mentioned earlier, the total station got an inbuilt miniature emitter. the emitters release the microwave signals. the prism reflector on object under survey at the other end reflects the waves. onboard data interpriters compute the distance by emitting and receiving multiple frequencies

<u>~~Angular measurement~~</u> :- <u>Data processing</u>
The integration of the micro-processor in a total station helps to read and interpret the survey parameters. subsequently, the data stores in the memory card of the instrument and later transferred to a computer, they can compute multiple measurements with high ~~per~~ precision with compensation for temperature, pressure, and humidity corrections.

<u>Angular measurement</u>:-
The total station uses the distance between the point and the time taken by the waves reflected for deciding angular measurement.

Setting up of a total Station :-

* The basic steps involved in setting up are tripod setup, levelling and instrument focusing.

* The tripod legs are placed firmly on the ground with equidistant legs with the head position above the survey point.

* The total station is placed on the tripod. it is fixed, and secured using a centering screw.

* The next step is to sharpen the optical plummet towards the survey point. on sharpening, the optical plummet reticle centers the survey point.
* The bubble is adjusted to the center by levelling foot screws.

* The time taken for levelling the equipment depend on the skill of the operator. However, levelling is the topmost activity to maintain the accuracy of measurements.

* Adjust the plate level by loosening the horizontal clamp. the equipment is parallel to levelling foot screw.

* Turn the instrument at 90 degree and leveled using and levelling screw.

* Subsequently, turn on the instrument.

* Select the tilt function from the opening window followed from the opening window followed by adjusting the foot level screw and center the bubble.

* Rotate the instrument at 90 degree repeat the process.

* Finally, adjust the ring and focus the telescope to the target point.

Advantages of total station:-

* High precision and accuracy.
* Requires limited manpower.
* perform quick field work.
* Reduction in manual errors.
* correction for temperature, pressure etc and digitally rectified.
* Storage and interpretation of data is easy.
* Time consumed is less.
* inbuilt GIS software helps in instant map ~~area~~ creation.

* Assists in local languages.

Disadvantages of total station:-

* The cost of the instrument is high.
* Need high skilled surveyor with autocad knowledge and professional training.
* Checking error during the operation is impossible.
* Low battery life.

Chapter-07:

Basics on GPS & DGPS and ETS:-

7.1 - GPS:- Global positioning

7.1.1 - Working principles of GPS, GPS signal

7.1.2 - Errors of GPS, positioning methods.

7.2 - DGPS - Differential global positioning system.

7.2.1 - Base station setup.

7.2.2 - Rover GPS set up.

7.2.3 - Download, post-process and export GPS data

7.2.4 - Sequence to download GPS data from flashcards.

7.2.5 - Sequence to post-process GPS data.

7.2.6 - Sequence to export post process GPS data.

7.2.7 - Sequence to export GPS time tags to file.

7.3 - ETS:- Electronic total station.

7.3.1 - Distance measurement.

7.3.2 - Angle measurement.

7.3.3 - Levelling.

7.3.4 - Determining position.

7.3.5 - Reference networks.

7.3.6 - Errors and accuracy.

7.1 GPS :- Global positioning

The global positioning system (GPS) is a satellite-based navigation and surveying system for determination of precise position and time, using radio signals from the satellites, in real time or in post-processing mode. GPS is being used all over the world for numerous navigational and positioning applications, including navigation on land, in air and on sea, determining the precise coordinates of important geographical features as an essential input to maping and geographical information system (GIS), along with it's use for precise cadastral surveys, vehicle guidance in cities and on highways using GPS-GIS integrated systems, earthquake and landslide monitoring etc. in india also, GPS is being used for numerous applications in diverse fields like aircraft and ship navigation, surveying, geodetic control networks, crustal deformation studies, cadastral surveys, creation of GIS data base, time service etc. by various organisation

GPS is primarily a navigation system for real time positioning. However, with the transformation from the ground-to-ground survey measurements to ground to space measurement made possibly by gps, this technique overcomes the numerous limitations of terrestrial surveying methods, like the requirement of intervisibility of survey stations, dependability on weather, difficulties in night observations etc. these advantages over the conventional techniques, and the economy of operations make gps the most promising surveying technique of the future, with the well established high accuracy achievable with gps in positioning of points separated by ~~the~~ few hundreds of meters to hundreds of km. this unique surveying technique has found important applications in diverse fields.

The global positioning system basically consists of three segments.

* space segment
* Control segment.
* user segment.

Space segment :-

The space segment ~~cons~~ contains 24 satellites in 12-hour near circular orbits at altitude of about 20000 km, with inclination of orbit 55°.

The constellation ensures at least 4 satellites in view from any point on the earth at any time for 3-D positioning and navigation on world wide basis.

Control segment:-

This has a master control station (MCS) few monitor stations (MSs) and up load station (ULS). The MSs are transportable shelters with receivers and computers.

User segment:

The user equipment consists of an antena, a receiver, a data-processor with software and a control/display unit. the GPS receiver measures the pseudo range, phase and other data using navigation signals from minimum 4 satellite and computes the 3-D position.

Working principle of GPS:-

Principle of operation. GPS is a satellite based navigation system. it uses a digital signal at about 1.5 GHz from each satellite to send data to the receiver. The receiver can then deduce its exact range from the satellite, as well as the geographic position (GP) of the satellite.

The range from the satellite is determined by the time the signal is received. the satellite signal includes the time at which it was sent. By comparing that to the receiver clock, the time delay and hence the range can be determined.

In order for the system to work, there must be at least four satellite visible to the receiver at all times, in fact there are 21 operational satellites, and three spares, in orbit at about 20,000 km, which circle the globe every twelve hours.

The GPS system is accurate to within of about 53ft. in position, 0.3fps in velocity and ~~100~~ 100 n sec in time. However, for the purpose of denying a precise navigational system to possible hostile forces, a random error of anywhere between 130 and 330 feet is intentionally inserted.

For any extended range weapon system. Precise and continuous positioning information is invaluable. GPS using PPS can be expected to be used in all platforms, missiles, guided projectiles, sonobouys etc.

GPS signals:-

Each GPS satellite transmits data on two frequencies; L1 (1575.42 MHz) and L2 (1227.60 MHz).

The atomic clocks aboard the satellite produces the fundamental L-band frequency; 10.23 MHz. the L_1 and L_2 carrier frequencies are generated by multiplying the fundamental frequency by 154 and 120 respectively.

Two pseudorandom noise (PRN) codes, along with satellite ephemerides (Broadcast ephemerides), ionospheric modeling coefficients, status information, system time and satellite clock corrections, are superimposed onto the carrier frequencies, L1 and L2. the measured travel times of the signals from the satellite to the receivers are used to compute the pseudoranges.

Errors of GPS signal

GPS signal propagation is significantly affected by travel through the atmosphere and such errors are one of the main GPS error factors that wide area Augmentation system (WAAS) and other satellite based Augmentation system (SBAS)

As gps signals travel down to the earth from space, the layers of the atmosphere refracts and slightly delays the signals particularly within the ionosphere. this delay interferes with the range solutions from the gps receiver on the ground to the satellite, resulting in positional errors of several meters. WAAS (wide area Augmentation system) corrects for this by determining how the atmosphere is interfering the signal in a region, and then providing real time correction data to WAAS - enabled receivers via it's own satellites.

similarly, local space conditions - especially solar output - can affect the gps signal major space weather events can and do affect WAAS as well, but the FAA (which operates it) has upgraded and hardened WAAS so it is more robust against solar interference.

However, WAAS does not correct for other common sources of gps error, such as gps points collected during a cold start of the receivo, receivers turned on after being off for several days or moved more than 500 miles use outdated satellite.

The major sources of GPS positional error are
* Atmospheric interference.
* calculation and rounding errors.
* ephemeris (orbital path) data errors.
* multi-path effects

satellite positions as a function of time, which are included in the broadcast satellite navigation message, are predicted from previous GPS observations at the ground control station. typically, overlapping 4-hour GPS data spans are used by the operational control system to predict fresh satellite orbital elements for each 1 hour period. As might be expected, modelling the forces acting on the GPS satellites will not be perfect, which causes some errors in the estimated satellite positions.

multipath error is one of the predominant error sources in all GPS applications, particularly the multipath error has to be precisely estimated in the global Navigation satellite system as it is the major error source the limits the GPS receiver's performance.

multipath is a major error source for both the carrier - phase and pseudo range measurement. it occurs when the gps signal arrives at the receivers antenna through different paths. These paths can be the direct line of sight signal and reflected signals from objects surrounding the receiver antenna. multipath distorts the origional singnal through interference with the reflected signals at the GPS antenna.
it affects both the carrier - phase and pseudo range measurements. however, it's size is much larger in the pseudo range measurement.

<u>positioning Method</u>:-
GPS surveying is concerned with the first of these, positioning: in general there are two techniques used in surveying, they are kinematics and static. in static GPS surveying sessions, the receivers are motionless on the earth during the observation. Because static work most often provides higher accuracy and more redundancy than kinematic work, it is usually done to establish control. the results of static GPS surveying are processed after the session is completed.

The data is typically post-processed. the majority of GPS surveying control and geodetic work still relies on static applications.

in kinematic GPS surveying, the receivers are either in periodic or continuous motion. kinematic GPS is done when real-time or near real-time, results are needed. when the singular objective of kinematic work is positioning, the receivers move periodically using the start and stop methodology originated. by

kinematic applications imply movement, one or more GPS receivers actually in motion during their observations. A moving GPS receiver on land, sea or air is characteristic of kinematics GPS. other characteristics of the application include results in real time and little redundancy. Hydrography, aerial mapping, gravimetric and more and more land surveying projects are done using kinematic GPS.

Chapter - 08

Basics of GIS and map preparation using GIS.

8.1 - Components of GIS, integration of spatial and attribute information.

8.2 - Three views of information system

8.2.1 - Database or table view map view and model view

8.3 - Spatial Data model.

8.4 - Attribute Data management and meta data concept.

8.5 - Prepare data and adding to Arc map.

8.6 - Organizing data as layers.

8.7 - Editing the layers.

8.8 - Switching to layout view.

8.9 - Change page orientation.

8.10 - Removing Borders.

8.11 - Adding and editing map information.

8.12 - Finalize the map.

Components of GIS, integration of spatial and attribute information.

A geographic information system (GIS) is a system designed to capture, store, manipulate, analyze, manage and present all types of geographical data. Geography means that some portion of the data is spatial. In other words, data that is in some way referenced to locations on the earth.

Coupled with this data is usually tabular data known as attribute data. Attribute data can be generally defined as additional information about each of the spatial features. An example of this would be schools. The actual location of the schools is the spatial data. Additional data such as the school name, level of education taught, student capacity would make up the attribute data.

It is the partnership of these two data types that enables GIS to be such an effective problem solving tool through spatial analysis.

GIS is more just software, people and methods are combined with geospatial software and tools, to enable spatial analysis, manage large datasets, and display information in a map/ graphical form.

✓ What can we do with GIS

GIS can be used as tool in both problem solving and decision making processes, as well as for visualization of data in a spatial environment. Geospatial data can be analyzed to determine (1) the location of features and relationships to other features (2) where the most or least of some feature exists. (3) the density of features in a given space, (4) what is happening inside an area of interest (AOI) (5) what is happening nearby some feature or phenomenon (6) how a specific area has changed over time

Mapping where things are:-

We can map the spatial location of real world features and visualize the spatial relationships among them. Example:- below we see a map of frac and mine locations and sandstone areas in Wisconsin.

we can see visual patterns in the data by determining that frac sand mining activity occurs in a region with a specific type of geology.

<u>Mapping quantities:-</u>

people map quantities, such as where the most and least are, to find places that meet their criteria or to see the relationships between places.

Example:- Below is a map of cemetery locations in wisconsin. the map shows the cemetery locations as dots (dot density) and each county is color coded to show where the most and least are

<u>Mapping densities:-</u>

Sometimes, it is more important to map concentrations, or a quantity normalized by area or total number.

example - Below we have mapped the population density of Manhattan (total population counts normalized by the area in sq. miles of census tracts)

Finding what is inside:-

We can use GIS to determine what is happening or what features are located inside a specific area/region. We can determine the characteristics of 'inside' by creating specific criteria to define an area of interest (AOI)

Example - Below is a map showing a flood event and the tax parcels and building in the floodway. We can use tools like clip to determine which parcels fall inside the flood event. Further, we can use attributes of the parcels to determine potential costs of property damage.

Finding what is nearby:-

We can find out what is happening within a set distance of a feature or event by mapping what is nearby using geoprocessing tools like Buffer,

Example:- Below we see a map of drive times from a central location in the city of Madison. We can use streets as a network and add specific criteria like speed limit and intersection controls to determine how far a driver can typically get in 5, 10 or 15 minutes.

Mapping change:-

we can map the change in a specific geographic area to anticipate future conditions, decide on a course of action or policy.

Example:- Below we see land use maps of Barnstable, mapping change showing changes in residential development from 1951 to 1999. the dark green shows forest, while bright yellow shows residential development. Application like this can help inform community planning processes and policies.

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13/07/22

Three view of information system:-

Each GIS has three key parts that are used to work with geographic information these three GIS views are represented in ArcGIS by the catalog and the geodatabase (a GIS is a collection of geographic datasets), the map (a GIS is an intelligent map view), and the toolbox (a GIS is a set of geoprocessing tools).

This combination has resulted in a technology that is science based, trusted and easily communicated across cultures, social classes, languages and displines.

To support this vision, GIS combines three fundamental aspects or views:-

* <u>The geodatabase view:-</u>

A GIS manages geographic information. One way to think of a GIS is as a spatial database containing data-sets that represent geographic information in terms of a generic GIS data model - features, rasters, attributes, topologies, networks and so forth.

GIS datasets are like map layers; they are geographically referenced so that they overlay onto the earth's surface. In many cases, the features (points, lines and polygons) share spatial relationship with one another. For example, adjacent features share a common boundary. Many linear features connect at their end points. Many point locations fall along linear features (address locations along roads).

<u>The map view:-</u>

A GIS is a set of intelligent maps and other views that show features and feature relationships on the earth's surface. Various map views of the underlying geographic information can be constructed and used as windows into the geographic database to support query, analysis, and editing of geographic information. each GIS has a series of two dimensional (2D) and three-dimensional (3D) map application that provide rich tools for working with geographic information through these views.

Geoprocessing view:-

A GIS is a set of information transformation tools that derive new information from existing data sets. these geoprocessing functions take information from existing datasets, apply analytic function, and write results into new derived datasets. Geoprocessing involves the ability to string together a series of operations so that users can perform spatial analysis and automate data processing all by assembling an ordered sequence of operations.

There are numerous spatial operators that can be applied to GIS data. the ability to derive new information within a GIS analysis process is one of the fundamental capabilities in GIS.

Database or table view, map view and model view.

The following rules apply when you add data from a database to the map.

* Data types that do not map to an ArcGIS data type will not appear in ArcGIS.
* Feature classes must contain one spatial reference and one type of geometry, either points, lines or polygons. ArcGIS determines the spatial reference and geometry type from the table metadata or from the first row in the table. If the spatial reference can not be determined you will be prompted to provide spatial reference information. Only features of that type and spatial reference will be displayed.
* ArcGIS does not place delimiters around table and field names. Tables with names or field names that require delimiters can not be displayed on the map.

Spatial Data model :-