



ANNAMACHARYA INSTITUTE OF TECHNOLOGY & SCIENCES

(AUTONOMOUS)

UTUKUR (P), C. K. DINNE (V&M), KADAPA, YSR DIST.

Approved by AICTE, New Delhi & Affiliated to JNTUA, Anantapuramu.

Accredited by NAAC with 'A' Grade, Bangalore.



Surveying

(23HES0102)

Lecture Notes

By

J V Sai Sumanth

Assistant Professor

Dept. Of Civil Engg.

AITS, Kadapa



ANNAMACHARYA INSTITUTE OF TECHNOLOGY & SCIENCES::KADAPA

HM23 Regulation

(AUTONOMOUS)

(Approved by AICTE New Delhi & Affiliated to JNTUA, Anantapuramu)

Accredited by NAAC with 'A' grade, Bangalore

II Year B.Tech. CE – I Semester

L	T	P	C
3	0	0	3

(23HES0102) SURVEYING

Course Objectives:

The objective of this course are to:

- Know the principle and methods of surveying and measuring of horizontal and vertical- distances and angles
- Identification of source of errors and rectification methods
- Know surveying principles to determine areas and volumes
- Setting out curves and use modern surveying equipments for accurate results
- Know the basics of Photogrammetry Surveying

Course Outcomes:

CO	Statement	Bloomslevel
CO 1	Apply the principle and methods of surveying and measuring of horizontal and vertical- distances and angles	L2
CO 2	Identify the source of errors and rectification methods	L3
CO 3	Apply surveying principles to determine areas and volumes	L2
CO 4	Setting out curves and using modern surveying equipments	L3
CO 5	Apply the basics of Photogrammetry Surveying in field	L4

Syllabus:

UNIT - I

Introduction and Basic Concepts: Introduction, Objectives, classification and principles of surveying, Surveying accessories. Introduction to Compass, leveling and Plane table surveying.

Lineardistances- Approximate methods, Direct Methods- Chains- Tapes, ranging, Tape corrections.

Prismatic Compass- Bearings, included angles, Local Attraction, Magnetic Declination, and dip –systems and W.C.B and Q.B systems of locating bearings.

UNIT - II

Leveling- Types of levels, methods of levelling, and Determination of levels, Effect of Curvature of Earth and Refraction.

Contouring- Characteristics and uses of Contours, methods of contour surveying.

Areas - Determination of areas consisting of irregular boundary and regular boundary.

Volumes -Determination of volume of earth work in cutting and embankments for level section, capacity of reservoirs.

UNIT - III

Theodolite Surveying: Types of Theodolites, temporary adjustments, measurement of horizontal angle by repetition method and reiteration method, measurement of vertical Angle, Trigonometrical leveling when base is accessible and inaccessible.

Traversing: Methods of traversing, traverse computations and adjustments, Introduction to Omitted measurements.



ANNAMACHARYA INSTITUTE OF TECHNOLOGY & SCIENCES::KADAPA

HM23 Regulation

(AUTONOMOUS)

(Approved by AICTE New Delhi & Affiliated to JNTUA, Anantapuramu)

Accredited by NAAC with 'A' grade, Bangalore

UNIT - IV

Curves: Types of curves and their necessity, elements of simple, compound, reverse curves. Introduction to Tacheometric Surveying.

Modern Surveying Methods: Principle and types of E.D.M. Instruments, Total station- advantages and Applications. Introduction to Global Positioning System. Introduction to Drone survey and LiDAR Survey (Light Detection And Ranging).

UNIT - V

Photogrammetry Surveying:

Introduction, Basic concepts, perspective geometry of aerial photograph, relief and tilt displacements, terrestrial photogrammetry, flight planning; Stereoscopy, ground control extension for photographic mapping- aerial triangulation, radial triangulation, methods; photographic mapping- mapping using paper prints, mapping using stereo-plotting instruments, mosaics, map substitutes.

Text Books:

1. Surveying (Vol – 1 & 2) by Duggal S K, Tata McGraw Hill Publishing Co. Ltd. New Delhi, 5th edition, 2019.
2. Textbook of Surveying by C Venkatramaiah, Universities Press 1st Edition, 2011.

Reference Books:

1. Surveying (Vol – 1), by B. C. Punmia, Ashok Kumar Jain and Arun Kumar Jain - Laxmi Publications (P) ltd., New Delhi, 18th edition 2024.
2. Surveying (Vol – 2), by B. C. Punmia, Ashok Kumar Jain and Arun Kumar Jain - Laxmi Publications (P) ltd., New Delhi 17th 2022.
3. Surveying (Vol – 3), by B. C. Punmia, Ashok Kumar Jain and Arun Kumar Jain - Laxmi Publications (P) ltd., New Delhi 16th 2023.
4. Plane Surveying and Higher Surveying by Chandra A M, New age International Pvt. Ltd., Publishers, New Delhi, 3rd Edition, 2015.
5. Surveying and Levelling by N. Basak Tata McGraw Hill Publishing Co. Ltd. New Delhi, 4th edition, 2014.
6. Surveying (Vol 1, 2 & 3), by Arora K R, Standard Book House, Delhi. Edition: 12th, 2015.

Web Resources:

https://koha.srmap.edu.in/cgi-bin/koha/opac-detail.pl?biblionumber=11522&shelfbrowse_itemnumber=23066

SURVEYING:-

Surveying is the art of determining the relative positions of points on, above or beneath the surface of earth by means of direct or indirect measurements of distance, direction (angle) and elevation.

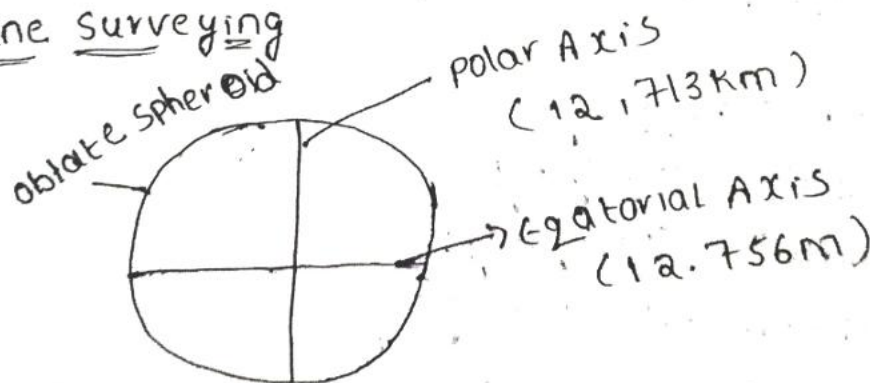
* objectives of surveying:-

- * To collect field data
- * To prepare plan or map
- * To analyse and to calculate the field parameters for setting out the operation of actual engineering works.

classifications of surveying:-

* based on shape of earth:-

- * plane surveying
- * Geodetic Surveying

* plane Surveying

* here the curvature of earth is not taken into consideration

* The surface of earth is considered as plane surface.

* The Triangle formed by any three points is considered as plane triangle and the angles of the triangle are plane angles.

* It is done for an area less than 250 sq km or 195 sq km

Geodetic Surveying:

* Here the curvature of the earth is taken into consideration

* The Triangle formed by any three points is considered as to be spherical and the angle of the triangle are assumed to spherical angles.

* It is carried out if area is greater than 250 sq km or 195 sq km

* Classification based upon instruments used:

- * chain surveying
- * compass surveying
- * theodolite surveying
- * tachometric surveying
- * plane table surveying
- * photogrammetric surveying
- * Aerial Surveying
- * Total Station etc...

* principles of surveying:-

1. To work from whole to part (to minimize the errors)
2. To locate a new station by at least two measurements (linear or angular) from fixed reference points

* chain surveying:-

* It is the simplest and at the same time, most accurate method of surveying, when the area of land to be surveyed is limited.

* Here only linear measurements are taken in the field with the help of chain, tape, and offset rod etc.

* principle of chain surveying:-

The principle of chain surveying is Triangulation.

As it is done with a chain is called chain triangulation.

* According to its principle the area to be surveyed is divided into number of small triangles, connected with each other.

possible the triangle formed should be well conditioned triangle.

* A triangle is said to be well conditioned when no angle unit is less than 30° or more than 120° .

* Scales:

It is not always possible to represent the actual length of an object on a drawing.

* So, it is required to reduce the object in order to accommodate it on the drawing in some proportion. The ratio by which the actual length of the object is reduced or increased is known as scale.

* Types of Scales:-

1. Full sized scale: if the actual length of the object is shown on the drawing, the scale used is said to be a full sized scale.

2. Reducing scale:-

if the actual length of the object is reduced in order to accommodate it ^{on} the drawing sheet the scale used is said to be a reducing scale.

3. Increasing or enlarging scale:-

if the actual length of an object is enlarged so as to bring out its details more clearly on the drawing the scale used is said to be an enlarging scale.

* Representative Fraction: (RF) Factor (VF):

The ratio of the distance on the drawing to the corresponding actual length of the object is known as 'RF'.

$$\therefore RF = \frac{10 \text{ cm}}{100 \text{ cm}} = \frac{1}{10}$$

$$\therefore RF = \frac{\text{distance on drawing of object}}{\text{corresponding actual distance of object}}$$

both distances in same units.

eg:- if a scale is 1cm : 10m then, $RF = ?$

$$RF = \frac{1\text{cm}}{10\text{m}} = \frac{1\cancel{\text{cm}}}{1000\cancel{\text{cm}}} = \frac{1}{1000}$$

eg:- 1: To construct a scale to show metres & decimeters when 1met is represented by 2.5cm, the scale should be long enough to measure 6met.

sol: Scale $\Rightarrow 2.5\text{cm} : 1\text{met} : 100\text{cm}$

$$\therefore RF = \frac{2.5\text{cm}}{100\text{cm}} = \frac{1}{40}$$

length of scale = $(RF) \times (\text{distance})$

$$= \frac{1}{40} \times 6\text{met}$$

$$= \frac{1}{40} \times 600\text{cm} = 15\text{cm}$$

eg: A plan represents an area of $93,750\text{m}^2$ & measures $6\text{cm} \times 6.25\text{cm}$. Find scale RF .

sol: Here, $6\text{cm} \times 6.25\text{cm} \longrightarrow 93,750\text{m}^2$

$$\longrightarrow 6 \times 6.25\text{cm}^2 = 93,750\text{m}^2$$

$$\longrightarrow 1\text{cm}^2 = \frac{93,750}{6 \times 6.25}\text{m}^2$$

$$= 2,500\text{m}^2$$

$$\Rightarrow 1\text{cm} = \sqrt{2500\text{m}^2}$$

$$\Rightarrow 1\text{cm} = 50\text{m}$$

$$\therefore RF = \frac{1\text{cm}}{50\text{m}} = \frac{1\text{cm}}{50 \times 100\text{cm}} = \frac{1}{5000}$$

* plan:-

Large scale representation of small area is called as "plan".

eg:- plan of a building.

Scale: 1:50; 1:150 etc

* Map:-

Small scale representation of large area is map.

eg:- Road map of India

Scale: 1:2000; 1:2,25,000 etc.

* Errors due to shrinkage:-

It is the plan is drawn, shrinkage due to variation in the atmospheric condition, then.

$$\text{Shrinkage Ratio (or) Factors} = \frac{\text{Length of shrinkage}}{\text{Actual length}}$$

$$\rightarrow \text{Shrinkage R.F.} = (\text{Shrinkage factor}) \times (\text{original R.F.})$$

$$\therefore \text{Correct dist.} = \frac{(\text{measured dist.})}{(\text{shrinkage factor})}$$

$$\text{correct Area} = \frac{(\text{measured area})}{(S.F.)^2}$$

eg:- The plan of an area has shrunk such that a line originally 10cm, now measuring 9.5cm. If the original scale of the plan, 1cm = 10mt, the correct dist. corresponding to a measured dist of 190mt is ?

$$SF = \frac{-(\text{Shrinkage length})}{(\text{Actual length})}$$

$$= \frac{9.5}{1000} = 0.95$$

$$\therefore \text{correct dist} = \frac{(\text{measured distance})}{SF}$$

$$= \frac{190}{0.95}$$

$$= 200 \text{ m}$$

* Equipment used for chain surveying :-

1. Chain :-

Chains are prepared of galvanised metal steel wire. Straight links bent into rings at the ends joined each other by three small circular (or) oval wire rings.

→ The ends of the chain are provided with brass handle at each end.

The following are the various types of chains :-

- i) metric chain
- ii) Gunter's chain (or) Surveyor's Chain
- iii) Engineer's Chain
- iv) Revenue chain
- v) Steel Band (or) Band Chain

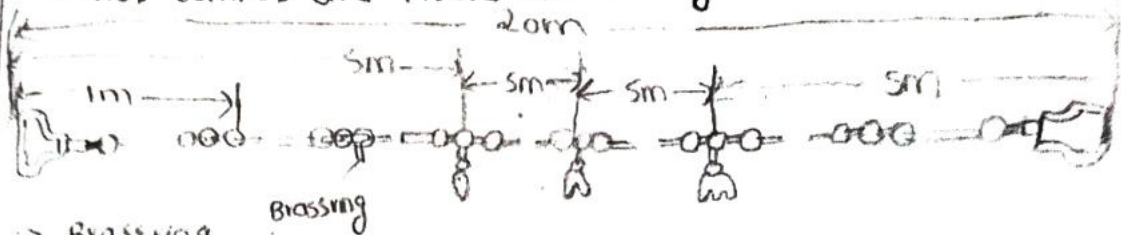
i) metric chain :-

→ In general, metric chains are available in 20m & 30m.

→ 20m chain is divided into 100 links & 30m chain is divided into 150 links.

→ length of each link : 0.2 m (or) 20cm

→ Small brass rings are provided at every 1m length
brass tallies are fixed at every 5m length.



Brass ring at every meter length

Tally at 5m length

Tally at 10m length

Tally at 15m length

iii) Gunter's chain (or) Surveyor's chain:-

→ This chain is 66 feet long & having 100 links
100 links - 66 feet

$$\therefore 1 \text{ link} = 0.66 \text{ ft}$$

→ It is very useful for measuring the distances in miles & furlongs.

1 Gunter's chain = 22 yards

$\therefore 1 \text{ yard} = 3 \text{ ft}$

10 Gunter's chain = 1 furlong

80 Gunter's chains = 1 mile = 8 furlongs

1 sq. Gunter chain = 484 sq. yards [10 cents]

10 sq. Gunter chains = 4840 sq. yards [100 cents]

= 1 acre

iii) Engineer's chain:-

→ It is 100 ft long & divided into 100 links

$$\therefore \boxed{1 \text{ link} = 1 \text{ ft}}$$

→ It was previously used for all engineering works

iv) Revenue chain

→ This chain is 33 ft long & divided into 16 links

$$16 \text{ links} = 33 \text{ ft}$$

$$\begin{array}{r} 16 \overline{) 33} 2 \\ \underline{32} \\ 1 \end{array}$$

$$\therefore 1 \text{ link} = \frac{33}{16} \text{ feet} = 2 \frac{1}{16} \text{ feet}$$

→ 1 Sq. Revenue chain = 1 Righa

v) Steel Band (or) Band chain:-

→ It consists of a ribbon of steel of width 16mm & length 20m (or) 30m.

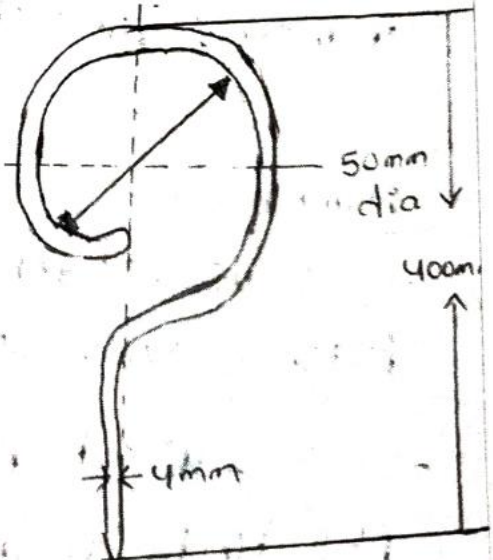
→ It has brass handle at both ends.

→ main disadvantage is that they get broke easily & are difficult to repair in field.

2. Arrows:-

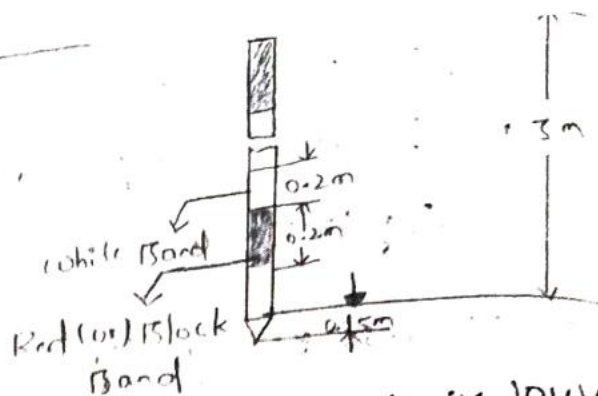
Arrows are used for counting the no. of chains completed, while measuring a chain line.

→ Generally, 10 arrows are supplied with a chain.



3. Ranging Rods:-

These are wooden (or) metal poles 2m (or) 3m long & having dia of 30mm



- They are provided with iron shoes at their lower ends to facilitate easy driving into ground.
- Ranging rods are used to define a station point & also for ranging a line.

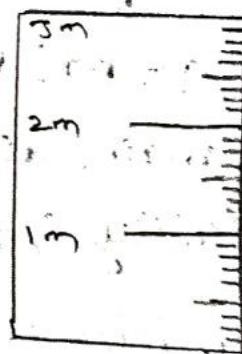
4. pegs:-

These are made from 'hard' timbers & tapered at one end

- The length varies from 120-600mm
- These are driven into the ground to mark the instrument stations.

5. Offset Rod:-

It is similar to ranging rod having slightly longer length & is graduated in metres & one tenth of metre.



6. Plumb Bob:-

It is used to define the vertical line while measuring distance along slopes.

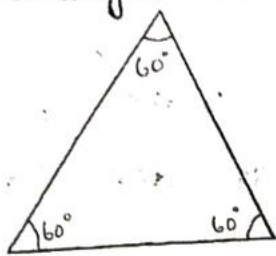
- It is also helpful in centering, to transform pts to the ground.

7. Tapes:-

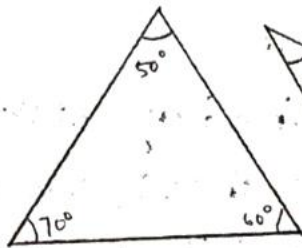
Types:-

- i) Cloth (or) Linen tape
- ii) metallic tape
- iii) steel tape
- iv) Invar tape $\rightarrow$ used for max. precision [accurate]

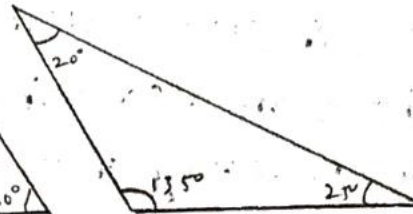
$\rightarrow$ If Triangle is said to be well-conditioned when no angle in it is less than 30° (or) more than 120° .



a) Ideal triangle



b) well conditioned



c) Ill-conditioned

*Terms used in chain surveying:-

① Survey Stations:-

Survey stations are the important points on the chain line. They can be at the beginning at the end (or) at anywhere on the chain line.

$\rightarrow$ It may be marked on the ground by driving a peg.

Types:-

i) main stations:-

Stations located along the boundary of an area as controlling pts are known as main stations.

→ Generally, these are present at the starting & end of a survey line, represented by A, B, C, D etc

ii) Subsidiary stations:-

Stations located on main survey lines (or) any other survey lines are known as subsidiary stations, denoted by "O", with letters S_1, S_2, S_3 ... etc.

→ These stations are marked to run check lines through them.

iii) Tie stations:-

These are also subsidiary stations, located on the main survey lines. These stations are denoted by "O", with letters T_1, T_2 ... etc.

→ These stations are marked to run tie lines through them.

2. Survey lines:-

The lines joining the survey stations are called survey lines.

→ These are denoted by ————

Types:-

i) Main Survey lines:-

The lines joining the main survey stations are called main survey lines.

→ These lines should cover the whole area to be surveyed.

ii) Base line:

The longest of the main survey lines of an area is known as Base line

→ It is an important line in chain surveying as various stations are plotted with reference to base line.

→ It is an important line in chain surveying as various stations are plotted with reference to base line.

→ The NW of triangle are built-up on it

∴ This line should be measured more accurately

iii) Check line: - (or) proof line:-

A check line may be laid by joining the apex of the triangle to any point on the opposite side (or) by joining 2 pts on any 2 sides of Δ 's.

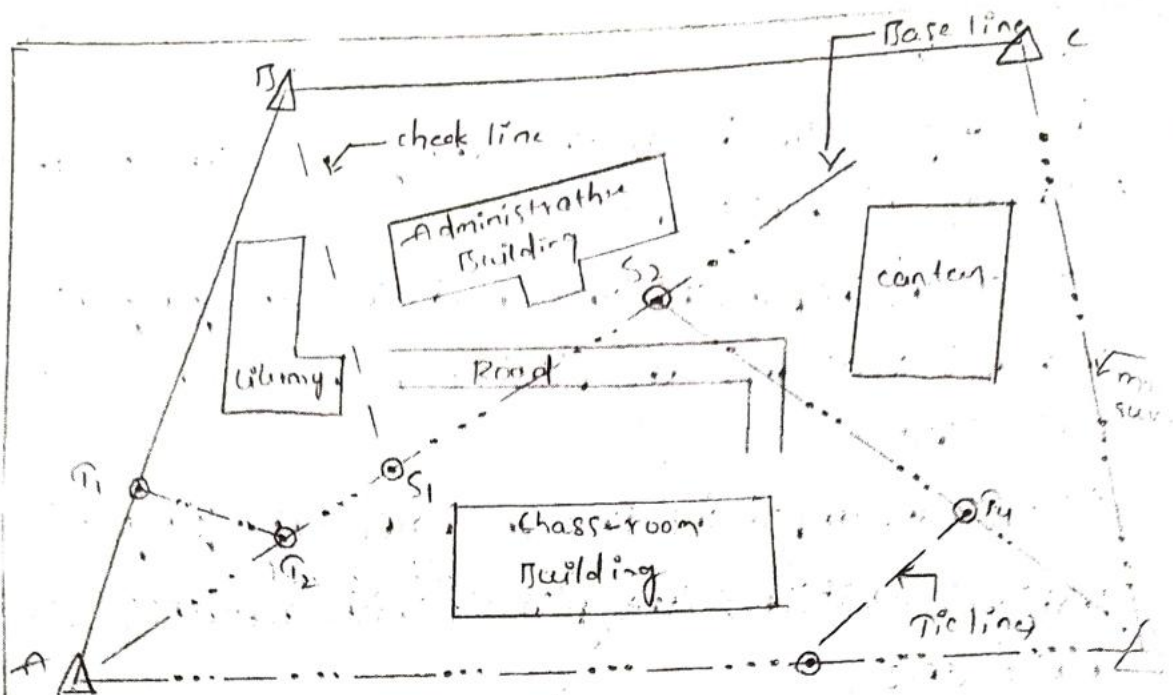
→ The purpose of check line is to check the accuracy of the frame work.

iv) Tie lines:-

The lines joining subsidiary station (or) tie station on the main survey lines are Tie lines.

→ The object of running a tie line is to locate the details which are far away from main survey lines.

→ It also survey the purpose of check line.



* offset :-

The lateral measurement taken from an object to the survey line is known as offset.

-> offsets are taken to locate the objects with reference to the survey line.

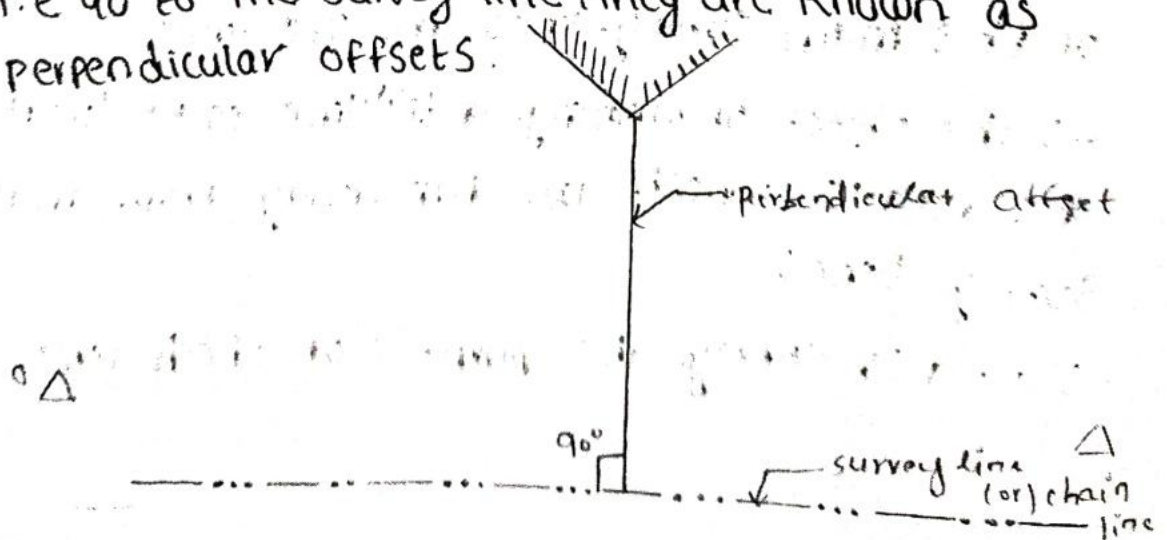
Types :-

i) perpendicular offsets

ii) oblique offsets.

i) perpendicular offsets :-

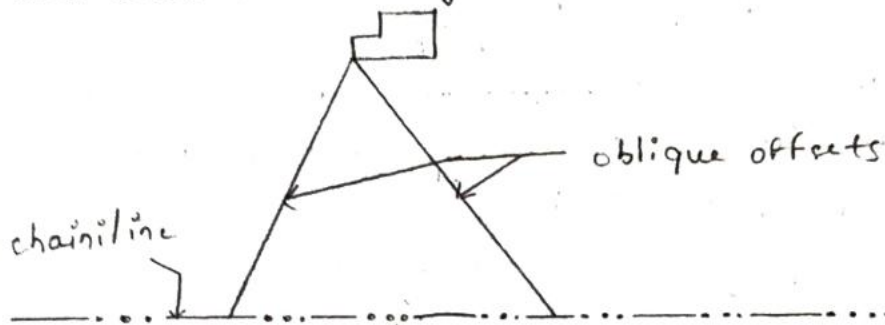
If the lateral measurements taken at right angles i.e. 90° to the survey line, they are known as perpendicular offsets.



2) oblique offsets:- (other than 90°)

offsets not perpendicular to the chain line is said to be oblique offsets.

→ these offsets are taken when the objects are located at a long distance from chain line



*Ranging:- (line- Ranger is used)

The process of establishing intermediate points on a straight line between 2 end points is known as Ranging.

→ Ranging must be done before a survey line is chained.

Ranging can be done by

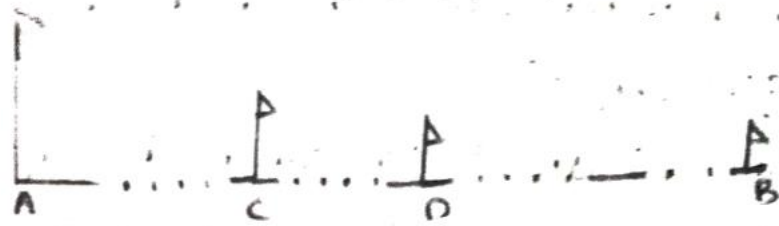
i) Direct Ranging

ii) Indirect (or) Reciprocal Ranging.

i) Direct Ranging:-

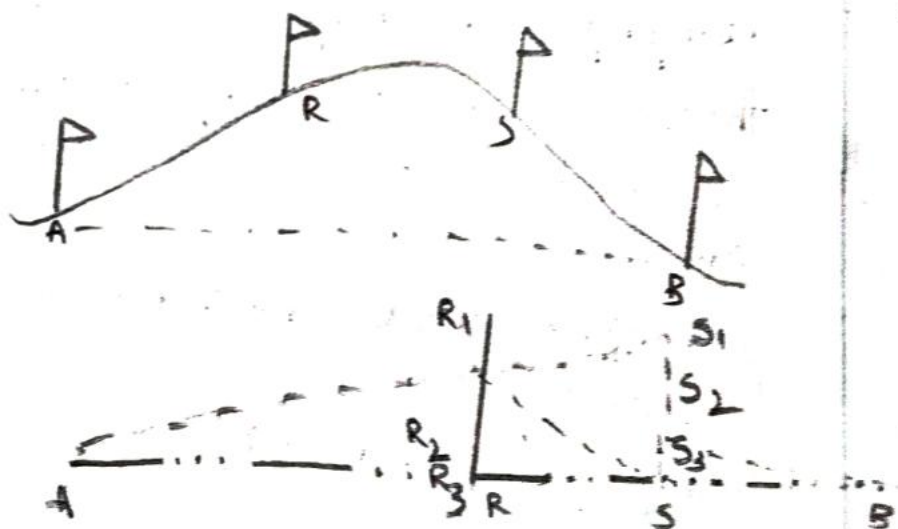
when intermediate ranging rods are fixed on a straight line by direct observation from end-stations, the process is known as Direct Ranging

→ Direct Ranging is possible when the end stations are intervisible.



i) Indirect (or) Reciprocal Ranging:-

Indirect ranging is used when the end stations are not intervisible due to there being high ground between them or when the distance between the stations is so large that they are not clearly intervisible.



* Chaining a survey line:-

chaining a survey line is carried out by two chain men, one of whom is known as leader & other as follower.

→ The follower spreads the chain on the ground with one handle touching the station & stands with (one handle) his heel firmly on the handle.

→ The leader takes other handle, a ranging rod & bunch of ten arrows with him & walks in the direction of the survey line to the measured.

→ Then, ranging rod is perfectly aligned.

→ Now, the leader takes the handle in both hands & pulls it by a severe jerk, so that the chain is straight without links & then he fixes an arrow at the end of the chain marking a cross at that point.

* Instruments used for setting out right-angles:-

a) cross-staff:-

i) open-cl/s (or) wooden cl/s:-

can set two lines at right angles to each other.

ii) French cl/s:-

can set lines at 45° (or) 90°

iii) Adjustable cl/s:-

can set any angle.

b) optical square:-

more convenient & accurate than cl/s for setting right angles.

c) prism square:-

more modern & precise than optical square.

* Mtds of chaining on sloping ground:-

Horizontal distances are required in surveying. So, in chaining along a sloping ground, the horizontal distances b/w 2 stations are

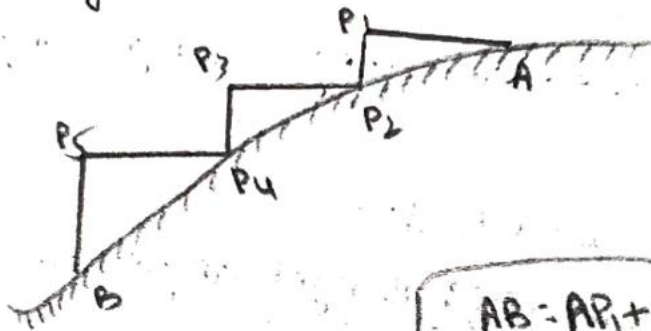
measured carefully by applying some convenient mlds

1) Direct mld (or) stepping mld:-

This mld is applied when the slope of the ground is very steep.

→ In this mld, sloping ground is divided into a no. of horizontal & vertical strips, like steps.

→ the lengths of horizontal portions are measured & added to get total horizontal dist b/w pts



$$AB = AP_1 + P_1P_2 + P_2P_3 + P_3P_4$$

2) Indirect mld:-

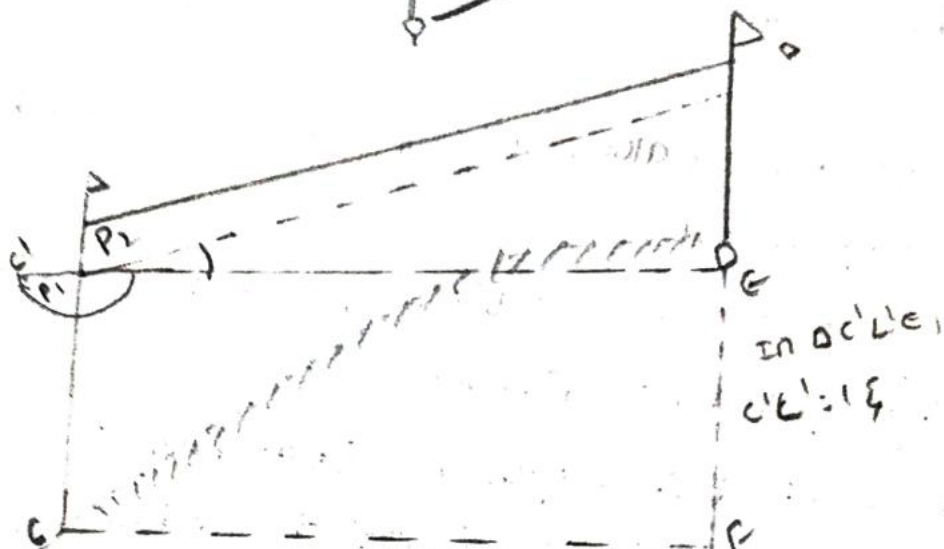
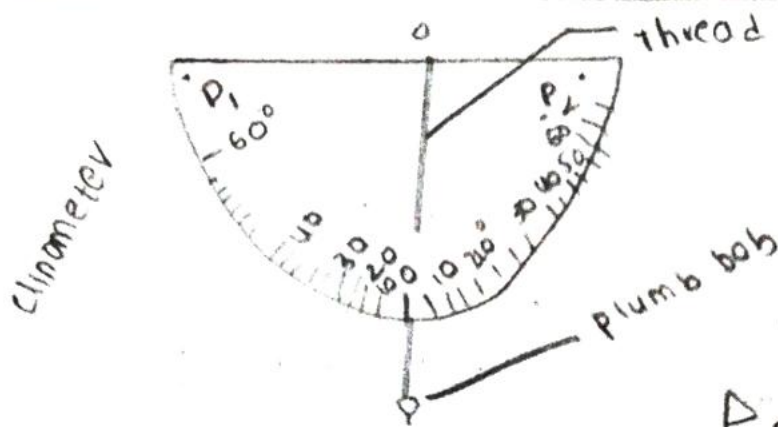
It is used when the slope of ground surface is long & gentle, where the stepping mld is not suitable

i) measuring the slope with a clinometer:-

Clinometer is a graduated semi-circular Protractor

It consists of two pins

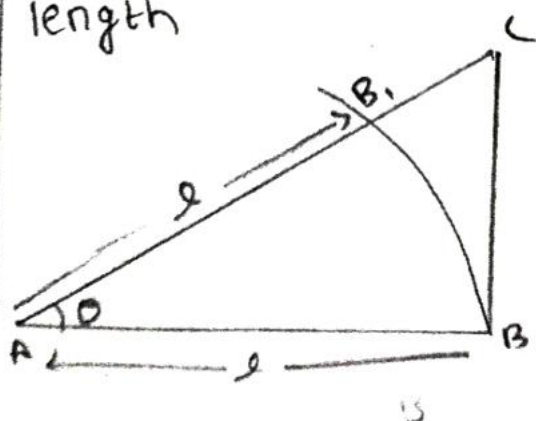
[P1 & P2] for sighting the object



ii) Applying Hypotenusal Allowance:-

In this mtd, the slope of the ground is first found out by using Clinometer.

→ Hypotenusal allowance is then made for each chain length



θ - Angle of slope

l - chain length

$$AB_1 = AB = 20 \text{ m} = 100 \text{ links}$$

$$\text{In } \triangle ABC, \cos \theta = \frac{AB}{AC} = \frac{l}{(AB + B_1C)} = \frac{l}{l + B_1C}$$

$$\Rightarrow l + B_1C = \frac{l}{\cos \theta} = l \sec \theta$$

$$\Rightarrow B_1C = (l \sec \theta - l)$$

$$= l (\sec \theta - 1) = 20 (\sec \theta - 1) \text{ m}$$

$$= 100 (\sec \theta - 1) \text{ links}$$

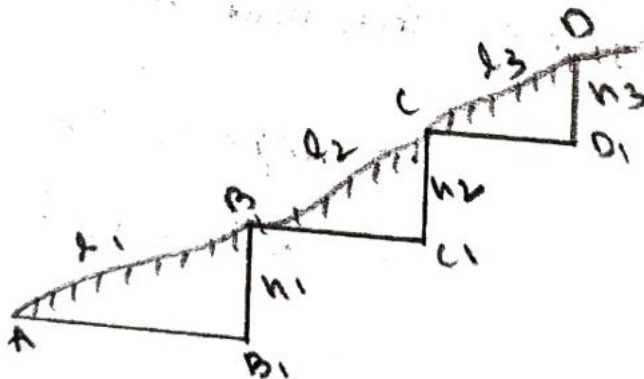
→ the amount $100(\sec \theta - 1)$ is said to be Hypotenusal allowance.

→ while chaining along the slope, one chain would be actually located at B_1 ,

but the arrow should be placed at "C", after making Hypotenusal allowance.

→ The next chain length will start from "C". The same principle is followed until the end of the line is reached.

iii) Knowing the Difference of Level



→ let A, B, C, D are different points on sloping ground. The difference of level b/w these pts is determined by a leveling instrument.

→ let, the respective differences be $h_1, h_2, h_3, \dots$

→ the sloping distances of AB, BC & CD are

l_1, l_2, l_3 respectively.

$$\therefore AB_1 = \sqrt{l_1^2 - h_1^2}$$

$$BC_1 = \sqrt{l_2^2 - h_2^2}$$

$$CD_1 = \sqrt{l_3^2 - h_3^2}$$

$$\therefore \text{Total Horizontal distance} = AB_1 + BC_1 + CD_1$$

*obstacles in chaining:-

i) when chaining is free, but vision is obstructed.

eg :- Rising ground or forest.

ii) when chaining is obstructed but vision is free.

eg :- pond (or) river.

iii) when chaining & vision both are obstructed.

eg :- A huge building.

eg. The length of a line to be found to be 600m when measured with 20m chain. if the chain is 15cm short find the correct length.

Given

$$L = 20m, e = 15cm, 0.15m$$

$$L' = L - e$$

$$= 20 - 0.15m$$

$$\boxed{L' = 19.85m}$$

$$mL = 600m$$

$$TL = \left(\frac{L'}{L} \right) \times mL$$

$$= \frac{19.85}{20} \times 600m$$

$$= 19.85 \times 30$$

$$= 595.50$$

2) The length of the line measured with a 20m chain was found to be 475m. True length of the line was 476.20m find the error in chain.

Sol: Given

$$L = 20m \text{ (correct length of chain)}$$

$$L' = ? \text{ (incorrect length)}$$

$$TL = 476.20m \text{ (True length)}$$

$$mL = 475m$$

$$TL = \left(\frac{L'}{L} \right) \times mL$$

$$476.20 = \frac{L'}{20} \times 23.75$$

$$476.20 = L' \times 23.75$$

$$\frac{476.20}{23.75} = L'$$

$$L' = 20.05$$

$$L' - L = \pm e$$

$$20.05 - 20 = \pm e$$

$$0.05m = \pm e$$

$e = 5m$ too long

2. A certain field was measured with 20m chain and found to have an area of 60m². It was after words found that the chain was 0.1m too short what is the true area of the field.

Given

Correct length of the chain (L) = 20m

measured area (MA) = 60m²

error (e) = 0.1m

$$TA = \left(\frac{L'}{L} \right) \times MA$$

$$= \left(\frac{19.9}{20} \right)^2 \times 60$$

$$= 59.40m^2$$

* correction due to incorrect length of chain or Tape:-

1. correction to incorrect length:-

True length, T.L = $\left(\frac{L'}{L}\right) \times (\text{measured length})$

where, L = Std (or) correct length of chain

L' = Incorrect

= $(C.L) \pm (\text{error})$

$L' = L \pm e$

→ use +ve Sign, when chain (or) tape is too long
& -ve Sign when it is too short. -

2. correction to incorrect Area:-

True Area = $\left(\frac{L'}{L}\right)^2 \times (\text{measured Area})$

3. Correction to incorrect Volume:-

True Volume = $\left(\frac{L'}{L}\right)^3 \times (\text{measured volume})$

* chain & Tape corrections:-

1. Temperature correction (C_t):-

length of the tape or chain may be increased or decreased due to raise or fall of temperature during measurement.

* It can be either "tve" or "-ve" $\{C_t = \alpha (T_m - T_0) L\}$

$\Delta L = \alpha L \Delta T$

where C_t = correction for temperature in meters

α = Coefficient of linear expansion or thermal expansion.

T_m : temperature during measurement (in $^{\circ}C$)

T_0 : temperature at which tape or chain was standardised (in $^{\circ}C$)

L : length of chain or tape in meters

2. pull correction :- (C_p) :-

The applied pull may be either more or less than the pull at which the chain or tape was standardised.

* it can be either positive or negative

$$C_p = \frac{(P_m - P_0)L}{AE}$$

$$\Delta L = \frac{P.L}{AE}$$

where C_p : (~~re correction~~) pull correction in meters

P_m : pull applied during measurement.

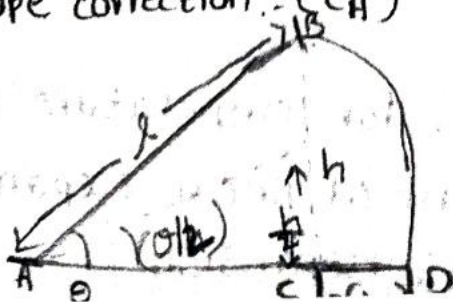
P_0 : pull at which chain or tape was standardised.

L : length of chain or tape in meters

A : cross sectional area of chain or tape

E : young's modulus or modulus of elasticity.

3. slope correction :- (C_H)



it is always "-ve".

$$CD = AD - AC$$

$$= l - l \cos \theta$$

$$CD = l(1 - \cos \theta)$$

$$= l(2 \sin^2 \theta/2)$$

$$CD = l \times 2 \times \left(\frac{h}{2l}\right)^2 = \cancel{l} \times \frac{h^2}{2l}$$

$$CD = \frac{h^2}{2l}$$

* Sag correction:-

This correction is necessary when the measurement is taken with tape in suspension.

It is always negative.

* If unit weight is given

$$C_s = \frac{L(w \cdot L)^2}{24m^2 p^2} \text{ mt}$$

* If total weight is given

$$C_s = \frac{LW^2}{24n^2 p^2} \text{ mt}$$

Where L = length of chain or tape in meters

w = weight of chain or tape per unit length
(N/m) or (kg/m)

W = total weight of chain or tape
(N or kgs)

n = no of spans

p = pull applied during measurement

5. Normal tension: $-(P_n)$:-

The tension at which the effect of pull is neutralised by the effect of sag is known as Normal tension:

here,

$$\left\{ \text{Elongation due to Pull} \right\} = \left\{ \text{Shortening due to sag} \right\}$$

$$\frac{(P_m - P_0)L}{AE} = \frac{LW^2}{2un^2P_m^2}$$

where, $n=1$

$$\frac{(P_m - P_0)L}{AE} \times \frac{2uP_m^2}{LW^2}$$

$$(P_m - P_0)(2uP_m^2) = AEW^2$$

$$2uP_m^2(P_m - P_0) = AEW^2$$

$$P_m^2(P_m - P_0) = \frac{W^2 AE}{2u}$$

by substituting the values of P_0, W, A, E , we can obtain P_m by trial and error method

eg:- A steel was exactly 30m long at 20°C , when supported throughout its length (underating) under a pull of 10kg. A line was measured with this tape under a pull of 15 kgs and at a mean temperature of 32°C and found to be 780m long. The cross sectional Area of tape = 0.03 cm^2

$W = 0.693 \text{ kgs}$, $\alpha_{\text{steel}} = 11 \times 10^{-6}/^{\circ}\text{C}$, $E_{\text{steel}} = 2.1 \times 10^6$
 kg/cm^2 . compute the true length of line if
tape was supported during measurement.

i) at every 30 m

ii) at every 15 m

sol Given

Steel long (L) = 30 m

Temperature of tape (T_0) = 20°

Temperature during measurement

pull chain standardised (T_m) = 32°C

pull applied during measurement (P_0) = 10 kg

measured length (P_m) = 15 kg

(m_2) = 780 m

$A = 0.03 \text{ cm}^2$

$W = 0.693 \text{ kg}$

$\alpha = 11 \times 10^{-6}$

$E = 2.1 \times 10^6 \text{ kg/cm}^2$

now

Temperature correction

$$C_t = \alpha (T_m - T_0) L$$

$$= 11 \times 10^{-6} (32 - 20) 30$$

$$= 11 \times 10^{-6} (12) 30$$

$$C_t = 3.96 \times 10^{-3}$$

pull correction

$$C_p = \frac{(P_m - P_0)L}{\Delta \rho}$$

$$C_p = \frac{(15 - 10) \times 30}{0.03 \times 2.1 \times 10^4}$$

$$C_p = \frac{(5 \times 30)}{0.03 \times 2.1 \times 10^4}$$

$$C_p = \frac{150}{0.010} = 15000 \quad \frac{15.0}{63000} = 2.380 \times 10^{-3}$$

sag correction

$$C_s = \frac{LW^2}{24n^2P_m^2}$$

$$= \frac{30 \times (0.693)^2}{24(1)(15)^2}$$

$$= \frac{14.40}{6400}$$

$$C_s = 2.66 \times 10^{-3}$$

$$e = C_t + C_p - C_s$$

$$= 3.96 \times 10^{-3} + 2.380 \times 10^{-3} - 2.66 \times 10^{-3}$$

$$= 3.68 \times 10^{-3}$$

$$L' = L + e$$

$$= 30 + 3.68 \times 10^{-3}$$

$$L' \approx 30.003$$

$$TL = \frac{L}{L} \times m \times \rho$$

$$= \frac{30.003}{30} \times 780$$

$$= 1.0001 \times 780$$

$$TL = 780.078$$

ii) at every 15m

Temperature correction:-

$$C_t = \alpha (T_m - T_0) L$$

$$= 11 \times 10^{-6} (32 - 20) 30$$

$$= 11 \times 10^{-6} (12 \times 30)$$

$$C_t = 3.96 \times 10^{-3}$$

pull correction

$$C_p = \frac{(P_m - P_0) L}{A E}$$

$$C_p = \frac{(15 - 10) \times 30}{0.03 \times 2.1 \times 10^6}$$

$$= \frac{5 \times 30}{0.03 \times 2.1 \times 10^6}$$

$$= \frac{150}{63000}$$

$$C_p = 2.380 \times 10^{-3}$$

sag correction

$$C_s = \frac{Lw^2}{24n^2 P_m^2}$$

$$= \frac{30 \times (0.693)^2}{24(2)^2 (15)^2}$$

$$= \frac{14.40}{24 \times 4 \times (15)^2} = \frac{14.40}{21600}$$

$$C_s = 1/540 \quad C_s = 6.66 \times 10^{-4}$$

$$e = C_t + C_p - C_s$$

$$= 3.96 \times 10^{-3} + 2.380 \times 10^{-3} - 6.66 \times 10^{-4}$$

$$e = 5.674 \times 10^{-3}$$

$$L' = L + e$$

$$= 30 + 5.674 \times 10^{-3}$$

$$L' = 30.005$$

$$T_L = \frac{L'}{L} \times 780 \quad T_L = \frac{L'}{L} \times m \ell$$

$$= \frac{30.005}{30} \times 780$$

$$T_L = 780.13$$

ii) at every 10m

Temperature correction

$$C_t = \alpha (T_m - T_0) L$$

$$= 11 \times 10^{-6} (32 - 20) 30$$

$$= 11 \times 10^{-6} (12 \times 30)$$

$$C_t = 3.96 \times 10^{-3}$$

Pull correction

$$C_p = \frac{(P_m - P_0) L}{A \rho}$$

$$= \frac{(15 - 10) \times 30}{0.03 \times 2.1 \times 10^6}$$

$$= \frac{5 \times 30}{0.03 \times 2.1 \times 10^6}$$

$$= \frac{150}{63000}$$

$$C_p = 2.380 \times 10^{-3}$$

Sag correction :-

$$C_s = \frac{L w^2}{24 n^2 P m^2}$$

$$= \frac{30 \times (0.693)^2}{24 (3)^2 (15)^2}$$

$$= \frac{14.40}{24 \times 9 \times (15)^2}$$

$$= \frac{14.40}{48600}$$

$$C_s = 2.96 \times 10^{-4}$$

Introduction to E.D.M.

The distance can be measured between two points by the following methods:

1) Direct distance measurement (DDM) by using chain or tape.

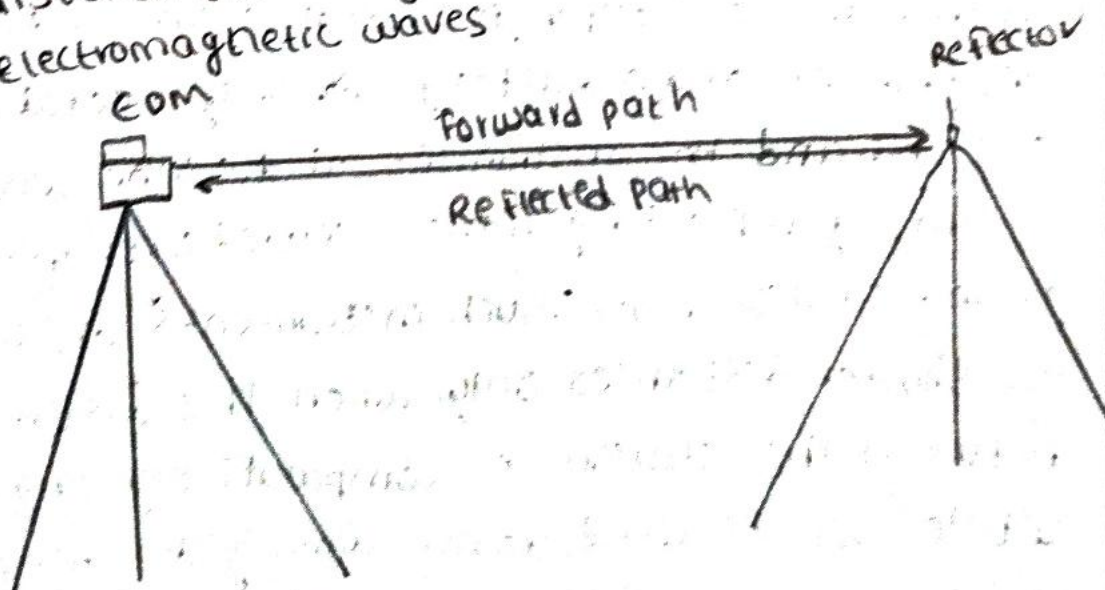
2) Optical distance measurement (ODM) by using tachometer, or horizontal subtense method using optical wedge attachments.

3. Electro magnetic distance measurement (EDM), is done by Geodimeter, Tellurometer or distomat.

The method of direct distance measurement is unsuitable in difficult terrain. In the optical distance measurement method the range is limited to 150 meters and accuracy also may be upto 1 in 1000.

But electromagnetic distance measurement, enables the accuracy upto 1×10^6 over ranges upto 100 km.

EDM is a surveying instrument used for measuring distance electrically between two point through electromagnetic waves.



Basic functions of an E.D.M Instrument:-

1. The generation of carrier wave and measuring wave frequencies
2. The modulation and demodulation of the carrier wave
3. The measurement of the phase difference between the transmitted and received frequency waves
4. The display in some form of the result of the measurement.

Classification of E.D.M Instruments:-

The E.D.M Instruments for surveying may be divided into the two categories:

1. Those in which a beam of light is the carrier and which is reflected back from a mirror.
2. Those which transmit, short radio waves (microwaves) as carrier.

1. Those instruments in which a beam of light is the carrier and which is reflected back from a mirror are less expensive because one active instrument and a battery are only needed at one end and the instrument at the other end is simply reflecting mirror centred over the ground station mark. Such instruments are used for shorter distances only when the distance between the stations is comparatively large, a telescope is used as for aiming the beam at the mirror and laser is used as a mirror

because this has a very small angle or spread

2. Those instruments which transmit short radio waves (microwaves) as carrier and in these instruments an active instrument and skilled operator are needed at each end of the line. The signal is retransmitted on another carrier wave back to the originating instrument. The wavelengths of the carrier of such instruments varies 8 to 10 mm. As the spread angle of carrier waves is between 5° to 10° these instruments do not require precise directional setting. Instruments using microwaves are generally used for the long distances required in primary triangulation and precise traversing.

Principle used in E.D.M Instruments:-

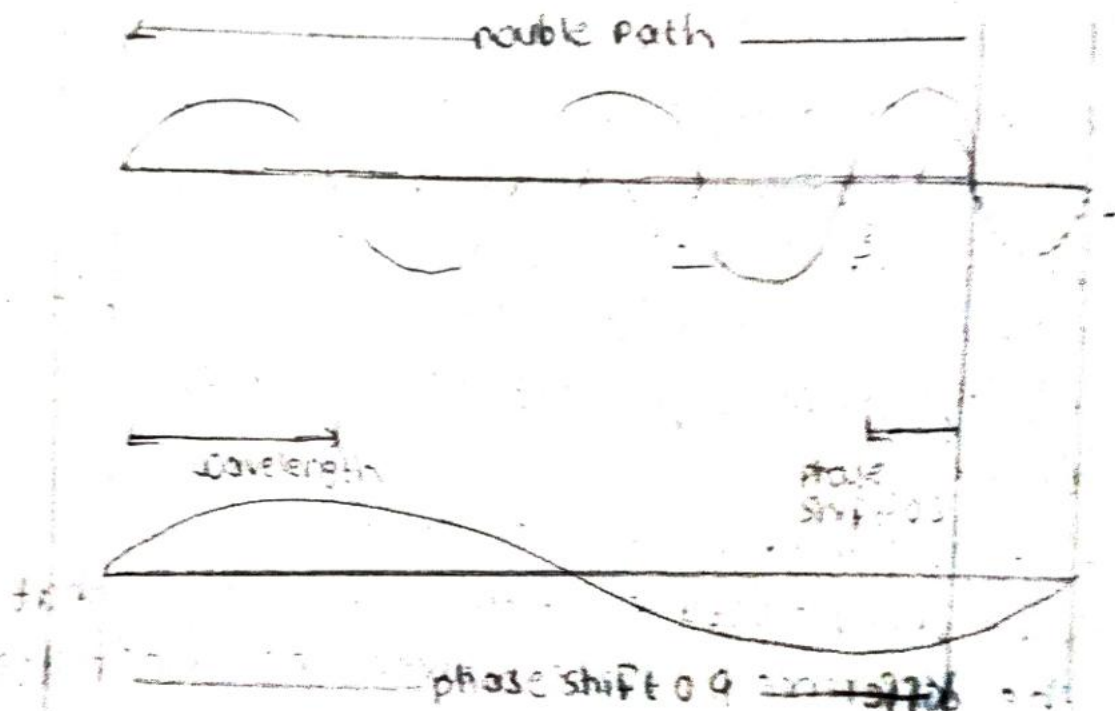
* The principle used in E.D.M instruments is that the difference in phase between the transmitted and received waves represents only a fraction of the wave lengths and that by using several

wave lengths of varying frequencies the distance can be measured.

* Distance (D) = Velocity $\times$ Time = $V \times t$

* This can be explained referring to the Fig. using wavelength 'A'. The double path measurement is 3.5 wave lengths. The

Instrument will record only 0.5. By using another wave of wavelength four times greater than λ let the phase shift be measured as 0.9. The whole number of wavelengths will be 3.6 (i.e., the smaller wavelengths will give more accurate distance).



The electromagnetic waves are transmitted to a retro reflector which returns them to the transmitting instrument. The instrument measures phase shift. By comparison of the phase shift between the transmitted and reflected signals the time and thus the distance can be determined.

Compass Surveying :-

* Meridian:

Meridian is a fixed direction on the surface of the earth.

→ The bearings of the survey lines are measured with reference to the meridians.

(* True Meridian :- (or) Geographical Meridian :-

It is a line (or) plane passing through the true North pole & South pole any pt on the surface of the earth.

* Magnetic Meridian:-

It is the direction established by a freely suspended magnetic needle, unaffected by magnetic substances.

→ The direction of it, through a pt can be established by a compass.

* Arbitrary Meridian:-

Sometimes, for the survey of a small area, a convenient direction is assumed as a meridian known as Arbitrary meridian.

* Grid Meridian:-

Sometimes for preparing a map, some state agencies assume several lines parallel to the true meridian for a particular zone.

These are the kinds of Angles in the
Surveying of the ground

* Bearing -

The bearing is a horizontal angle
is measured with reference to a some fixed line
(meridian)

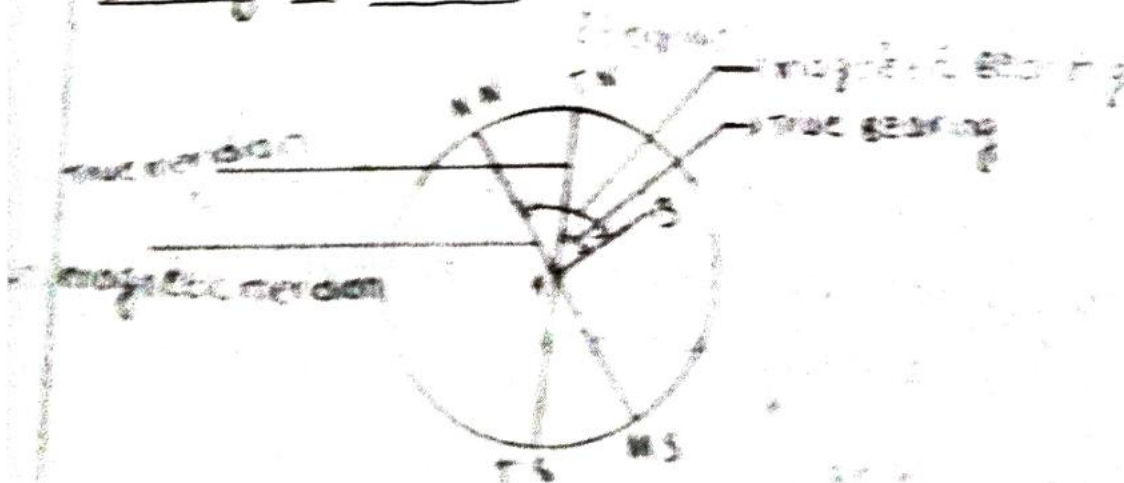
* True Bearing -

The horizontal angle made by a survey line
with reference to the true meridian is called

True bearing of a line

* Magnetic Bearing -

The horizontal angle made by a survey line with
reference to magnetic meridian is called magnetic
bearing of a line



* Principle of compass surveying -

The principle of compass surveying is traversing
which involves a series of connected lines.

→ The magnetic bearings of the lines are measured by prismatic compass & the distances of the lines are measured by chain. Such survey doesn't require the formation of NW of triangles. Compass surveying is recommended when:-

- A large area to be surveyed.
- The area is crowded with many details & triangulation is not possible.
- The course of a rivers (or) coast line is to be surveyed.

* ~~Traverse~~ Traversing:-

The surveying which involves a series of connected lines is known as "Traversing". The sides of the traverse are known as traverse legs.

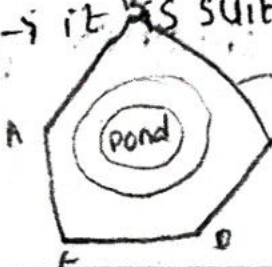
- In Traversing, the lengths of the lines are measured by chain & the directions are fixed by compass (or) theodolite.

Types:-

1) Closed Traverse:-

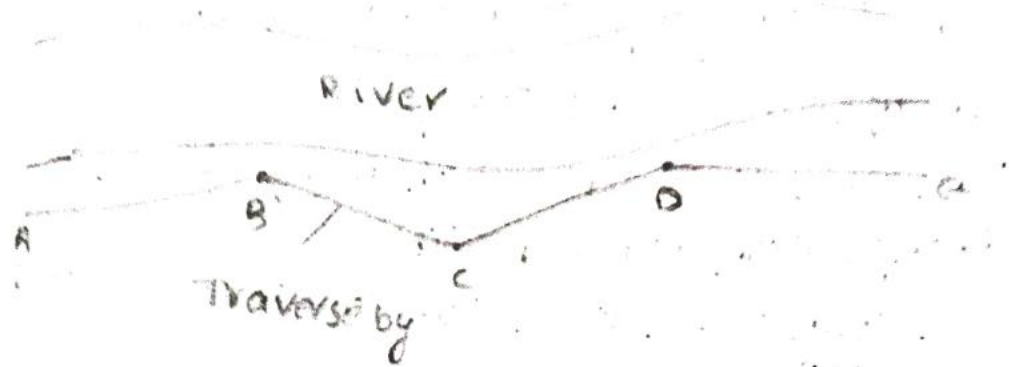
When a series of connected lines form a closed circuit i.e. when the finishing pt coincides with the starting pt of a survey, it is called closed Traverse.

- it is suitable for ponds, forests etc.



2) open Traverse:-

when a sequence of connected lines extends along a general direction & doesn't return to the starting Pt. is known as open Traverse (or) unclosed Traverse.



→ It is suitable for the survey of roads, rivers, coast lines etc.

* Designation of magnetic Bearing:-

magnetic Bearings are designated by two systems.

i) whole circle Bearing System (W.C.B)

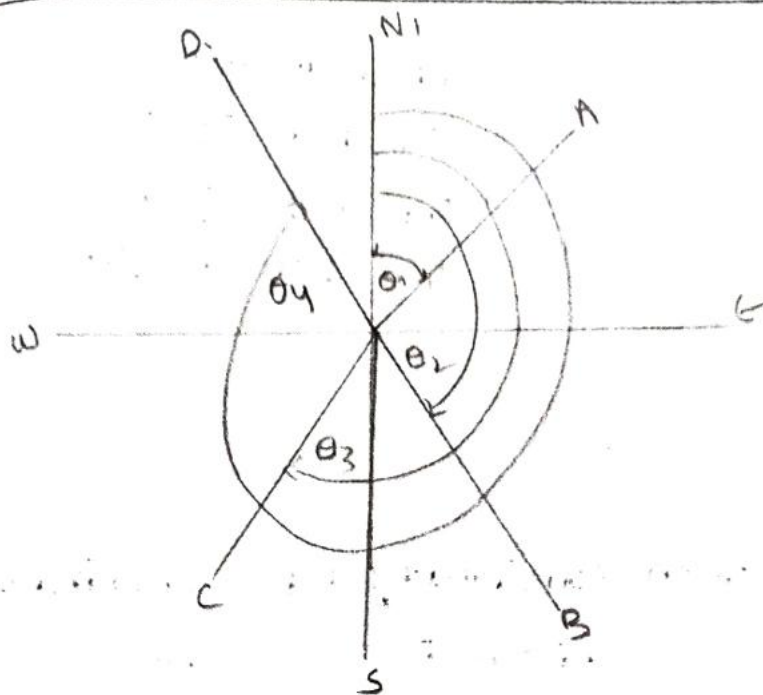
ii) Quadrantal Bearing System (Q.B)

i) whole circle Bearing (or) Azimuthal Bearing system

The mag bearing of a line measured clockwise from the North pole towards the line is known as whole circle Bearing of that line.

→ such a bearing may have any value between 0° & 360°

→ The whole C.B of a line is obtained by prismatic compass



W.C.B of OA = θ_1 ; WCB of OC = θ_3

W.C.B of OB = θ_2 ; WCB of OD = θ_4

iii) Quadrantal bearing (or) Reduced Bearing System:-

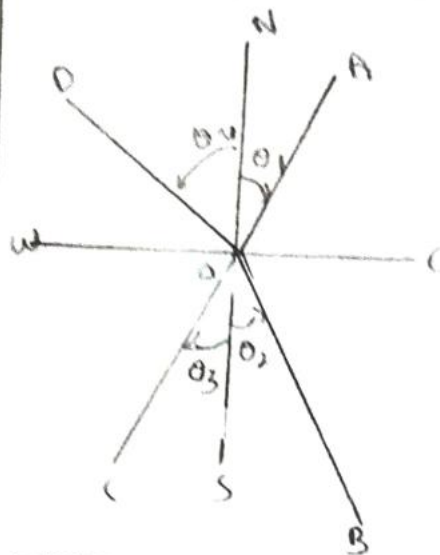
The mag bearing of a line measured clockwise (or) anticlockwise from the North pole (or) South pole whichever is nearest to the line towards the East (or) West, is known as Quadrant Bearing of line.

→ This system consists of four quadrants -

NE, SE, SW & NW

→ The value of a quadrant bearing ~~lies~~ lies blw 0° & 90°

→ Q.B. are obtained by surveyor's compass



Q.B of OA = $N\theta_1 E$

Q.B of OB = $S\theta_2 E$

Q.B of OC = $S\theta_3 W$

Q.B of OD = $N\theta_4 W$

WCB Between	Corresponding R.B	Quadrant
$0^\circ \text{ \& } 90^\circ$	$RB = WCB$	NE
$90^\circ \text{ \& } 180^\circ$	$RB = 180^\circ - WCB$	SE
$180^\circ \text{ \& } 270^\circ$	$RB = WCB - 180^\circ$	SW
$270^\circ \text{ \& } 360^\circ$	$RB = 360^\circ - WCB$	NW

eg-1:- Convert the following WCB's to QB's

i) WCB of AB = $45^\circ 30'$

$\therefore$ Q.B of AB = $N 45^\circ 30' E$

ii) WCB of BC = $125^\circ 45'$

Q.B of BC = $180^\circ 00' - 125^\circ 45' = S 54^\circ 15' E$

iii) WCB of CD = $222^\circ 15'$

iv) WCB of DE = $320^\circ 30'$

eg-2:- Convert the following Q.B's to WCB

i) Q.B OF AB = $S 36^{\circ} 30' W$

WCB OF AB = $180^{\circ} 00' + 36^{\circ} 30'$

= $216^{\circ} 30'$

ii) Q.B OF BC = $S 43^{\circ} 30' E$

Q.B OF BC = $180^{\circ} 00' - 125^{\circ} 45' = S 54^{\circ} 15' E$

iii) Q.B OF CD = $N 26^{\circ} 45' E$? -

iv) Q.B. OF DE = $N 40^{\circ} 15' W$? -

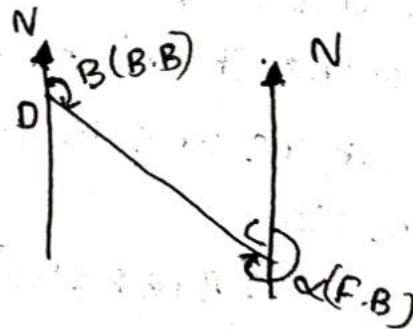
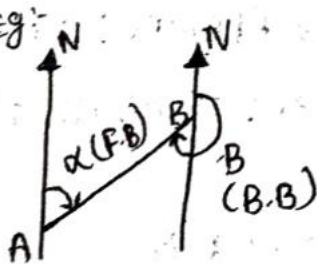
* Fore Bearing

The bearing of a line measured in the direction of the progress of survey is called fore bearing (FB) of the line.

* Back Bearing:-

The bearing of a line measured in the direction opp to the survey is called Back Bearing (BB) of the lines

Eg:-



=> in W.C.B system, diff b/w FB & BB should be exactly 180°

$$\therefore \boxed{BB = FB \pm 180^{\circ}}$$

$$FB < 180^\circ \Rightarrow B.B = FB + 180^\circ$$

$$FB > 180^\circ \Rightarrow B.B = FB - 180^\circ$$

→ In Q.B.S, FB & BB are ~~numerically~~ numerically equal but quadrants are just opp.

$$N \Rightarrow S, E \Rightarrow W$$

eg: FB of AB is $N 30^\circ E$

$\therefore$ BB of AB = $S 30^\circ W$

* Local Attraction:-

The magnetic needle doesn't indicate the North direction when it is under the influence of attractive bodies in its surroundings. Such a disturbing influence is termed as local attraction. The disturbing magnetic substances may be iron, steel structures, rails, electric carrying current etc.

→ To detect presence of local attraction, F.B & B.B of a line should be taken. If difference of FB & B.B of the line is 180° , then there is no local attraction.

$$F.B + B.B \neq 180^\circ = 180^\circ \text{ There is no L.A}$$

→ If FB & BB of a line ~~do not~~ do not differ by 180° , the needle is said to be affected by local attraction [provided there is no instrumental error]

Application of correction for local attraction:-

1. Direct mtd:-

ally

from the given table, the unaffected line is first detected

→ Then, commencing from the unaffected line the bearings of the other affected lines are corrected by finding the amount of correction at each station.

This mtd is simple & generally employed.

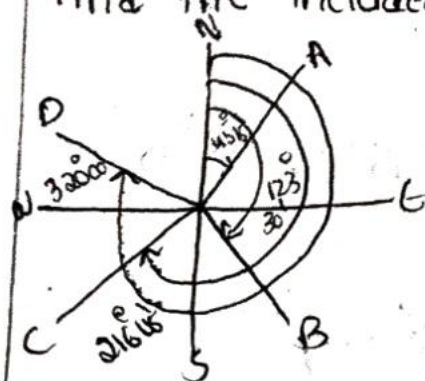
2. Indirected mtd:-

→ The interior angles of a ~~traverse~~ traverse are calculated from the observed bearings. Then, an angular check is applied.

→ The sum of the interior angles should be equal to $(2n-4) \times 90^\circ$

If it is not so, the total errors is equally distributed among all the angles of the traverse

eg:- The bearings of the lines OA, OB, OC & OD are $48^\circ 45' 15''$, $123^\circ 30'$, $320^\circ 00'$, $216^\circ 45'$ respectively
find the included angles $\angle AOB$, $\angle BOC$, $\angle COD$, $\angle DOA$



from the figure,

$$\begin{aligned}\angle AOB &= (\text{Bearing of OB}) - (\text{B of OA}) \\ &= (123^\circ 30') - (48^\circ 45' 15'') \\ &= 78^\circ 15'\end{aligned}$$

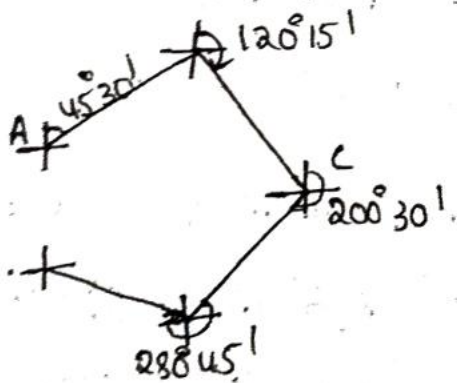
$$\begin{aligned}\angle BOC &= (B \text{ of } OC) - (B \text{ of } OB) \\ &= 216^\circ 45' - 123^\circ 30' \\ &= 93^\circ 15'\end{aligned}$$

$$\begin{aligned}\angle COD &= (B \text{ of } OD) - (B \text{ of } OC) \\ &= (320^\circ 00') - (216^\circ 45') \\ &= 103^\circ 15'\end{aligned}$$

$$\begin{aligned}\angle DOA &= (360^\circ 00' - B \text{ of } OA) + (B \text{ of } OA) \\ &= (360^\circ 00' - 320^\circ 00') + (45^\circ 15') \\ &= (40^\circ 00') + (45^\circ 15') \\ &= 85^\circ 15'\end{aligned}$$

eg:- F.B's of the lines AB, BC, CD & DE are $45^\circ 30'$, $120^\circ 15'$, $200^\circ 30'$ & $280^\circ 45'$ respectively.

Find $\angle B$, $\angle C$, $\angle D$?



$$\angle B = B \text{ of } AB - F.B \text{ of } BC$$

$$= (45^\circ 30' + 180^\circ 00') - (120^\circ 15') = 105^\circ 15'$$

$$\angle C = B \text{ of } BC - F.B \text{ of } CD$$

$$= (120^\circ 15' + 180^\circ 00') - (200^\circ 30')$$

$$= 99^{\circ} 45'$$

$$\angle D = (BB \text{ of } CD) + (360^{\circ} - FB \text{ of } DE)$$

$$= (20^{\circ} 30') + (360^{\circ} - 280^{\circ} 45')$$

$$= 99^{\circ} 45'$$

Magnetic declination:-

The horizontal angle b/w True meridian & mag. meridian is known as mag. declination.

$$\therefore \text{True Bearing} = (\text{mag Bearing}) \pm (\text{mag declination})$$

→ The lines passing through pts of equal declination are known as Isogonic lines.

→ The line passing through pts of zero declination is said to be the Agonic line.

eg:- The bearings observed in a traverse survey, at a place where local attraction was suspected are given below.

Line	F.B	B.B
PQ	$124^{\circ} 30'$	$304^{\circ} 30'$
QR	$68^{\circ} 15'$	$246^{\circ} 00'$
RS	$310^{\circ} 30'$	$135^{\circ} 15'$
SP	$200^{\circ} 15'$	$17^{\circ} 45'$

At what stations do you suspect local attraction. Find corrected bearings of the lines

sol: Direct mtd:

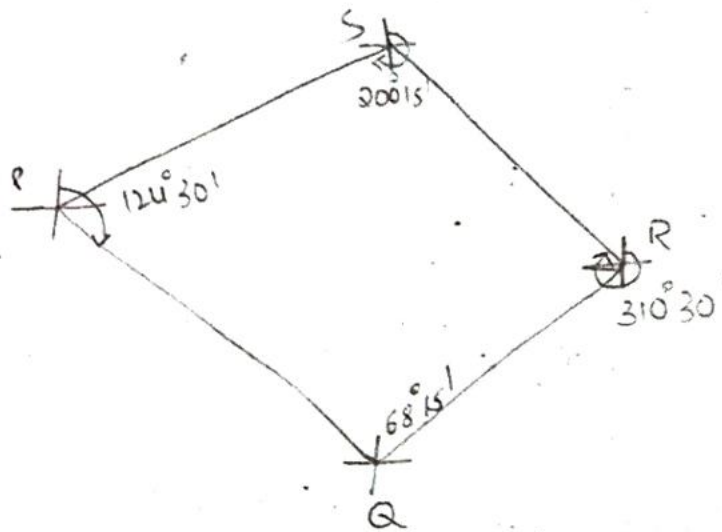
Line FB BB Diff in FB & BB

PQ - $124^{\circ}30'$ $304^{\circ}30'$ $180^{\circ}00'$

QR - $68^{\circ}15'$ $246^{\circ}00'$ $177^{\circ}45'$

RS - $310^{\circ}30'$ $135^{\circ}15'$ $175^{\circ}15'$

SP - $200^{\circ}15'$ $174^{\circ}5'$ $182^{\circ}30'$



→ FB & BB of line PQ differ by 180° . So, P & Q are free from local attraction & observed FB & BB of line PQ are correct.

∴ FB of QR observed from stns Q & B.B of SP observed from stns P are considered

→ FB of line QR = $68^{\circ}15'$

Actual BB of line QR shld be = $68^{\circ}15' + 180^{\circ}$
= $248^{\circ}15'$

But Observed B.B of QR = $246^{\circ}00'$

$\therefore$ Errors at stn 'R' = (Observed bearing) - (Actual Bearing)

$$= 246^{\circ}00' - 248^{\circ}15'$$

$$\text{errors} = -2^{\circ}15'$$

$\therefore$ Correction at R = $+2^{\circ}15'$

So, correct FB of RS = (Observed FB of RS) + (correction at stn 'R')

$$= 310^{\circ}30' + 2^{\circ}15'$$

$$= 312^{\circ}45'$$

$\therefore$ Actual B.B of RS shld be = $312^{\circ}45' - 180^{\circ}00'$
 $= 132^{\circ}45'$

But observed B.B of RS = $135^{\circ}15'$

$\therefore$ Errors at stn 'S' = (Observed bearing) - (Actual Bearing)

$$= 135^{\circ}15' - 132^{\circ}45'$$

$$\text{error} = +2^{\circ}30'$$

$\therefore$ Correction at 'S' = $-2^{\circ}30'$

correct FB of SP = (Observed FB of SP) + (correction at stn 'S')

$$= 200^{\circ}15' - 2^{\circ}30'$$

$$= 197^{\circ}45'$$

check:-

$$\begin{aligned}\text{B.B of SP} &= 197^{\circ}45' - 180^{\circ}00' \\ &= 17^{\circ}45' \text{ [check is OK]}\end{aligned}$$

$$\therefore \text{Correction at R} = +2^{\circ}15'$$

$$\text{So, correct FB of RS} = (\text{Observed FB of RS}) + (\text{Correction at stn R})$$

$$= 310^{\circ}30' + 2^{\circ}15'$$

$$= 312^{\circ}45'$$

$$\begin{aligned}\therefore \text{Actual B.B of R.S shld be} &= 312^{\circ}45' - 180^{\circ}00' \\ &= 132^{\circ}45'\end{aligned}$$

$$\text{But observed B.B of R.S} = 135^{\circ}15'$$

$$\therefore \text{Error at stn 'S'} = (\text{Observed bearing}) - (\text{Actual bearing})$$

$$= 135^{\circ}15' - 132^{\circ}45'$$

$$\text{error} = +2^{\circ}30'$$

$$\therefore \text{Correction at 'S'} = -2^{\circ}30'$$

$$\begin{aligned}\text{Correction FB of SP} &= (\text{Observed FB of SP}) + \\ &\quad (\text{Correction at stn 'S'})\end{aligned}$$

$$= 200^{\circ}15' - 2^{\circ}30'$$

$$= 197^{\circ}45'$$

check:-

$$\begin{aligned}\text{B.B of SP} &= 197^{\circ}45' - 180^{\circ}00' \\ &= 17^{\circ}45' \text{ [check is OK]}\end{aligned}$$

Line	F.B (or) B.B	Observed Bearing	Correction	Corrected Bearing
PQ	- FB	$124^{\circ}30'$	0° at P	$124^{\circ}30'$
PQ	- BB	$304^{\circ}30'$	0° at Q	$304^{\circ}30'$
QR	- FB	$68^{\circ}15'$	0° at Q	$68^{\circ}15'$
QR	- BB	$246^{\circ}00'$	$+2^{\circ}15'$ at R	$248^{\circ}15'$
RS	- FB	$310^{\circ}30'$	$+2^{\circ}15'$ at R	$312^{\circ}45'$
RS	- BB	$135^{\circ}15'$	$-2^{\circ}30'$ at S	$132^{\circ}45'$
SP	- FB	$200^{\circ}15'$	$-2^{\circ}30'$ at S	$197^{\circ}45'$
SP	- BB	$17^{\circ}45'$	0° at P	$17^{\circ}45'$

The Bearings observed in a Traverse surveying at a place where local attraction was suspected are given below

Line	F.B	B.B
PQ	$124^{\circ}30'$	$304^{\circ}30'$
QR	$68^{\circ}15'$	$246^{\circ}00'$
RS	$310^{\circ}30'$	$135^{\circ}15'$
SP	$200^{\circ}15'$	$17^{\circ}45'$

at what stations do you suspect local attraction
Find corrected bearings of the lines.

QR

$$FB = 68^{\circ}15'$$

$$\text{Observed BB} = 246^{\circ}00'$$

$$\begin{aligned}\text{Actual BB} &= 68^{\circ}15' + 180^{\circ}00' \\ &= 248^{\circ}15'\end{aligned}$$

$$\text{Error} = (\text{Obs. B}) - (\text{Act. B})$$

$$= 246^{\circ}00' - 248^{\circ}15'$$

$$= -2^{\circ}15'$$

$$\text{correction at stn 'R'} = +2^{\circ}15'$$

$$\therefore \text{Actual FB of RS} = 310^{\circ}30' + 2^{\circ}15' = 312^{\circ}45'$$

$$\text{Actual BB of QR} = 246^{\circ}00' + 2^{\circ}15' = 248^{\circ}15'$$

RS

$$\text{Observed BB of RS} = 135^{\circ}15'$$

$$\text{Actual BB of RS} = 312^{\circ}45' - 180^{\circ}00'$$

$$= 132^{\circ}45'$$

$$\text{Error at 'S'} = (\text{Observed Bearing}) - (\text{Act. Bearing})$$

$$= 135^{\circ}15' - 132^{\circ}45' = +2^{\circ}30'$$

$$\text{correction at stn 'S'} = -2^{\circ}30'$$

$$\text{Actual FB of SP} = 200^{\circ}15' - 2^{\circ}30' = 197^{\circ}45'$$

Actual BB of RS = $135^{\circ}15' - 2^{\circ}30' = 132^{\circ}45'$

2. Levelling

* Levelling:-

The art of determining the relative heights of diff. pts on (or) below the surface of the earth. is known as leveling.

Thus, leveling deals with measurements in the vertical plane.

* level surface:-

Any surface parallel to the mean spheroidal surface of the earth is said to be a level surface. Such a surface is obviously curved.

→ The water surface of a still lake is also considered to be a level surface.

level line:-

Any line lying on a level surface is called a level line.

This line is normal to the plumb line [direction of gravity] at all pts.

* Horizontal plane:-

Any plane tangential to the level surface at any pt is known as horizontal plane.

* Horizontal line:-

Any line lying on the horizontal plane is said to be a horizontal line.

- it is a straight line tangential to the level line.

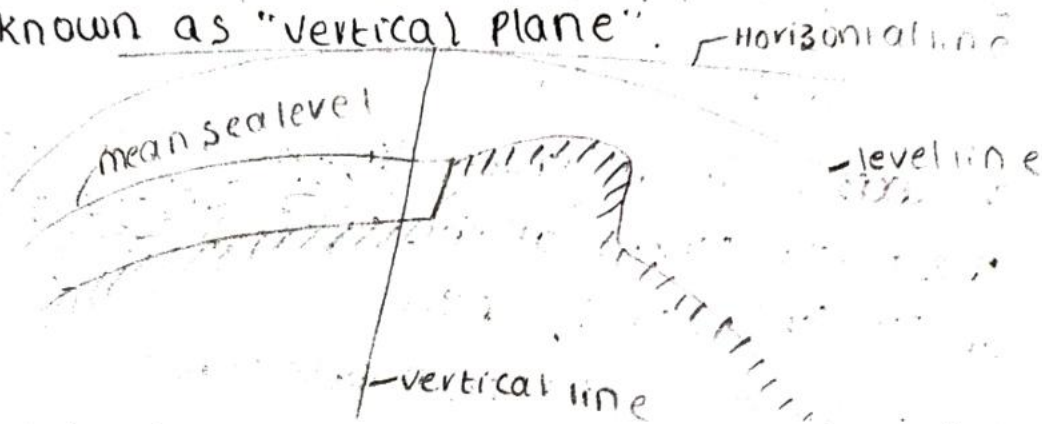
PF * Vertical line.

The direction indicated by a plumb line is known as vertical line.

→ this line is perpendicular to the horizontal line.

* vertical plane:-

Any plane passing through the vertical line is known as "vertical plane".



* Datum line:-

This is an imaginary level line from which the vertical distances of diff pts [above or below this line] are measured.

→ In India, the datum adopted for the Great Trigonometrical Survey (G.T.S) is the mean sea level at Mumbai (old-Karachi).

* Reduced level (RL):-

The vertical dist of a pt above (or) below the datum line is known as RL of that pt.

→ R.L. of a pt is +ve if it above the datum & is -ve if it below the datum.

* Line of collimation

It is an imaginary line passing through the intersection of the cross-hairs at the diaphragm & the optical centre of the object glass & its continuation.

→ it is also known as line of sight.

* Axis of the telescope:-

This axis is an imaginary line passing through the optical centre of the object glass & the optical centre of eye-piece.

* Axis of Level Tube (or) Bubble Tube:-

It is an imaginary line tangential to the longitudinal curve of the bubble tube & its middle point.

* Back sight (BS):-

This is the first staff reading taken in any set up of the instrument after the levelling has been perfectly done. This reading is always taken on a pt of known R.L. i.e. on a Bench-mark (or) change point.

* Fore sight (F.S):

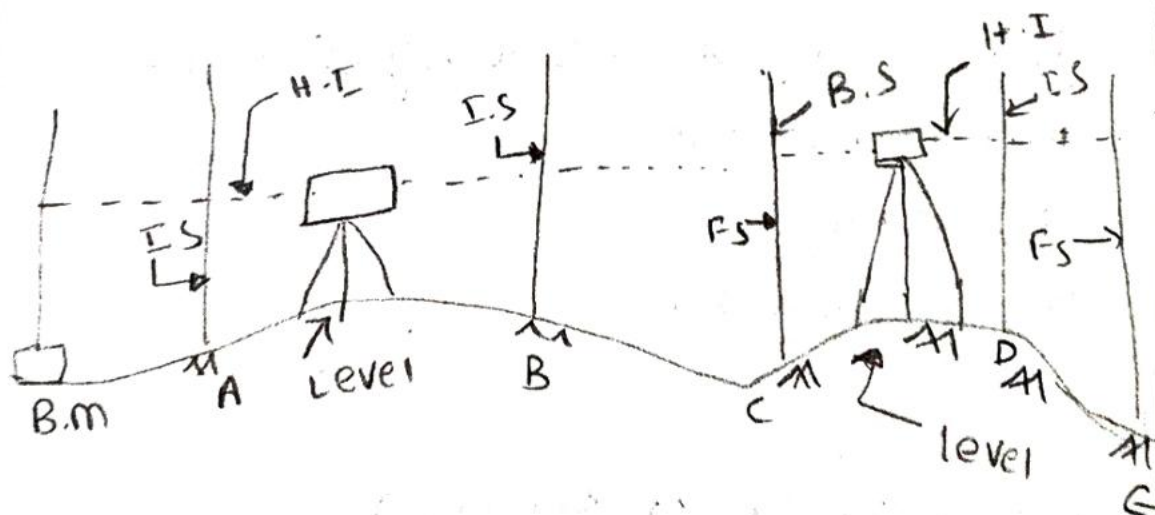
It is the last staff reading in any set up of the instrument, & indicates the shifting of the latter.

Intermediate sight (I.S):-

It is any staff reading taken on a pt of unknown elevation b/w B.S & F.S.

change pt (CP) (or) turning pt (TP):-

This pt indicates the shifting of the instrument. At this pt, an F.S is taken from one setting & a B.S from the next setting.



* Ht. of instrument (H.I):-

When the leveling instrument is properly levelled, the R.L of the line of collimation is known as Height of the Instrument.

→ This is obtained by adding the B.S reading to the R.L of B.M (or) CP on which the staff reading was taken

* Bench-marks (B.M)

These are fixed pts (or) marks of known R.L determined with reference to the datum line. These are very imp marks.

→ They serve as reference pts for finding the R.L of new pts (or) for conducting levelling operations in projects involving roads, railways etc.

Types:-

- i) Great Trigonometric Survey (G.T.S) B.M
- ii) permanent B.M
- iii) Arbitrary B.M
- iv) Temporary B.M

* Focussing:-

The operation of setting the eye-piece & the object glass a proper distance apart for clear vision of the object is known as focussing. This is done by turning the focussing screw clockwise (or) anticlockwise

→ focussing is done in two steps:-

i) focussing the eye-piece.

ii) focussing the object glass.

* parallax:-

The apparent movement of the image relative to the cross-hairs is known as parallax.

→ This occurs due to imperfect focussing, when the image does not fall in the plane of diaphragm.

* Different types of levels:-

1. The dumpy level:-

The telescope of the dumpy level is rigidly fixed to its supports. It cannot be removed from its supports nor can it be rotated about its longitudinal axis.

3. Cook's Reversible level:-

This is a combination of the dumpy level & Y-level. It is supported by 2 rigid sockets.

→ the telescope can be rotated about its longitudinal axis, withdrawn from the sockets, replaced from one end of the telescope to the other end.

4. Cushing's level:-

Here telescope connect be removed from the sockets

→ the eye-piece & object glass are removable, can be interchanged from one end of the telescope to the other end.

5. modern tilting level:-

The telescope can be tilted slightly about its horizontal axis with the help of a tilting screw.

→ In this instrument, the line of collimation is made horizontal for each observer by means of the tilting screw.

6. Automatic Level:-

This is also known as self-aligning level. This instrument is levelled automatically within a certain tilt range by means of a compensating device (tilt compensator).
Description of dumpy level:-

*leveling staff:-

It is graduated rod used for measuring the vertical distances but the points on the ground & the line of collimation.

Types:-

1. Target Staff:-

For very precise works & sight, target staff are used.

-> A movable target is provided in this staff.

-> A vernier is provided on target to give precise reading.

-> In target staff, level man directs the staff man to move the target up & down until it bisects by the line of sight.

2. Self-Reading Staff:-

i.) Telescopic metric Staff:-

It is arranged in three length placed one into the other. It can be extended to its full length by pulling.

=> The top portion is solid & of length 1.25m, the central box portion is hollow & of length

1.25m & the bottom box portion is hollow & 1.5m long.

→ The staff is graduated in such a way that smallest division is of 5mm.

ii) Folding metric staff:

This staff is made of well seasoned timber, is of width 75mm, thickness 18mm & length 3m.

→ It is divided into two parts of length 2m having a locking arrangement.

It can be folded or detached when required.

iii) One-length staff:

It is solid & made of seasoned timber. It is 3m long & graduated in the same way as the telescopic staff.

iv) Invar staff:

It is used for precise levelling work.

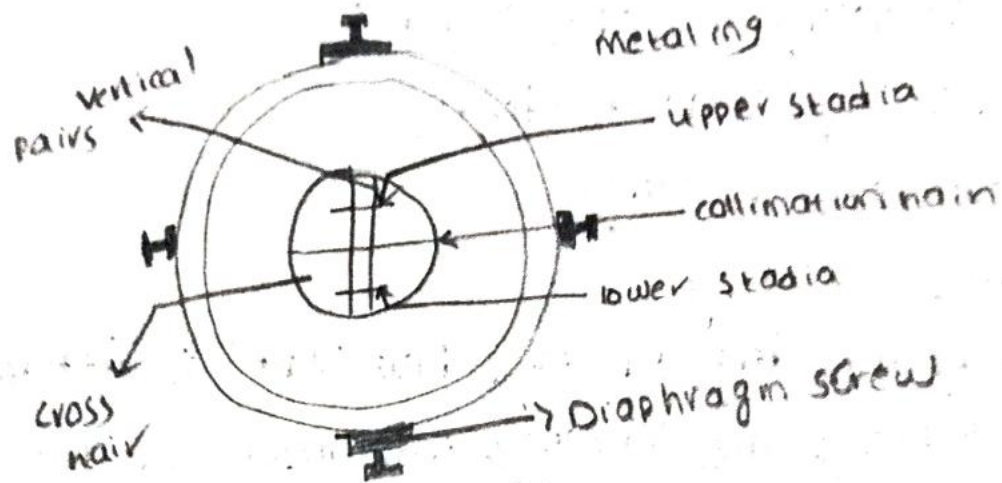
→ An invar band is fitted to a wooden staff. It is 3m long.

Diaphragm:

It is a brass ring fitted inside the telescope just in front of the eye-piece.

→ The ring carries the cross-hairs which

It gets magnified when viewed through the eye-piece.



Temporary Adjustments of level:

The adjustments made at very set up of the level before the staff reading are taken are known as temporary adjustments.

The following are the diff. steps to be followed in temporary adjustments.

(The following are the diff.)

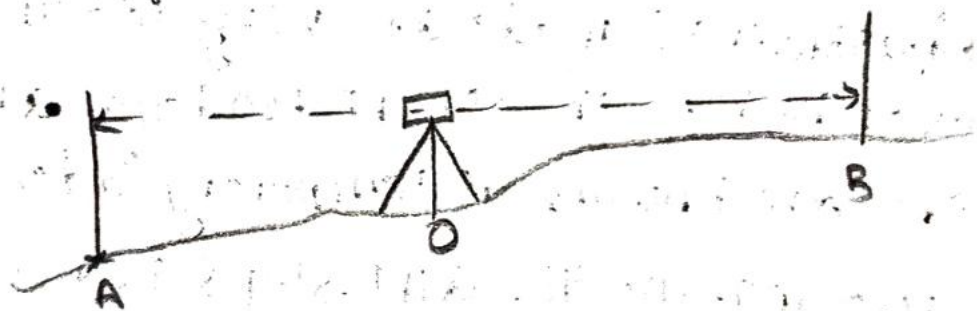
1. selection of suitable position
2. fixing level with tripod stand
3. Approximate levelling by legs of tripod stand
4. Perfect levelling by foot screws

5. focussing the eye-piece
6. focussing the object piece
7. focussing the object glass
8. taking staff readings.

Methods of levelling

1) Simple leveling:-

when the diff of level b/w 2 pts is determined by setting the levelling instrument mid-way b/w the pts, the process is called simple levelling.



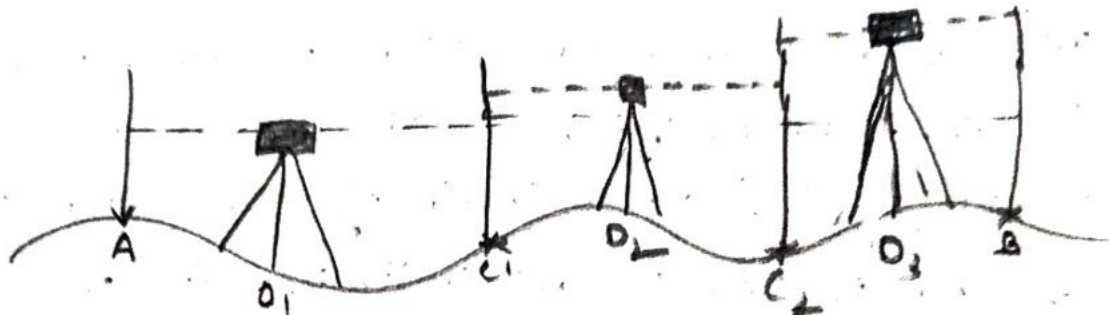
Differential levelling:-

this method is also known as compound levelling or continuous levelling.

→ In this method the level is set up at several suitable positions & staffs readings are taken at all of these.

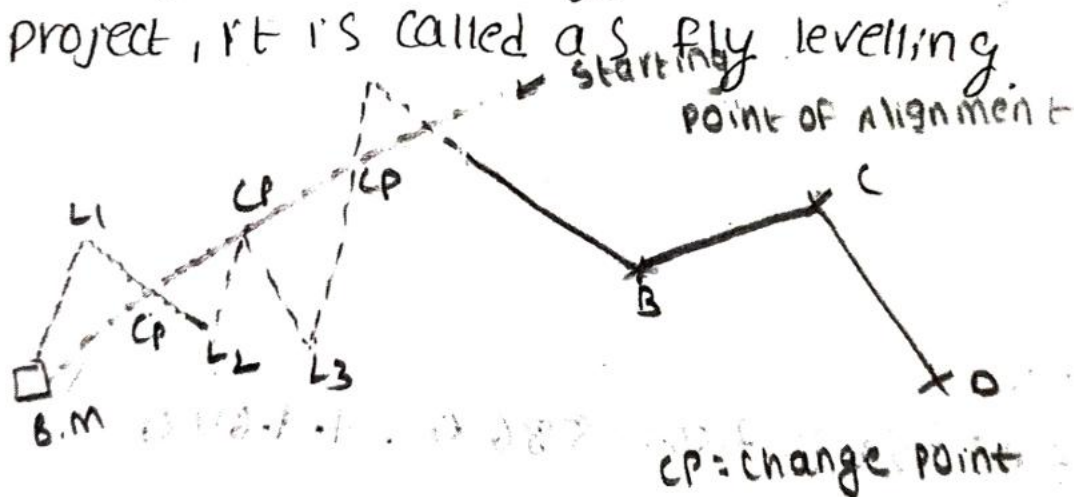
→ This method is adopted when :-

- i) the pts are at a great dist apart
- ii) Diff of elevation b/w pts is large
- iii) There are obstacles b/w the pts



fly leveling :-

when the differential levelling is done in order to connect a bench-mark to the starting pt of the alignment of any project, it is called as fly levelling.



4) Profile (or) longitudinal leveling :-

The operator of taking levels along the center line of any alignment [road, railway etc] at regular intervals is known as

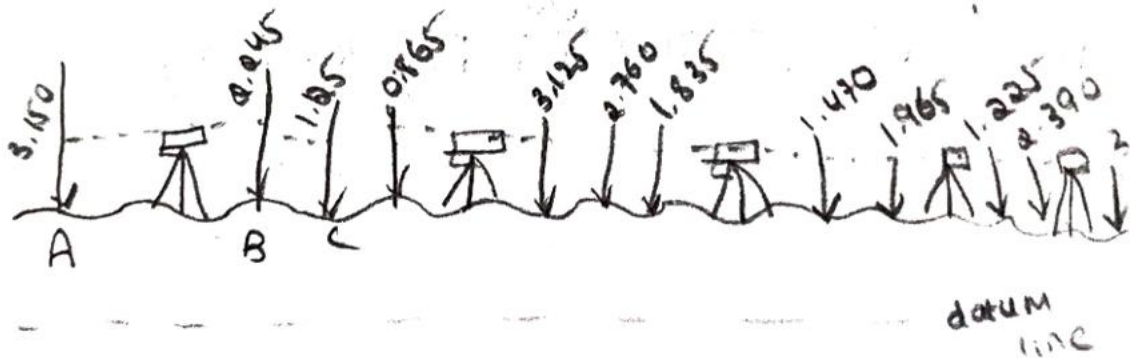
Profile levelling.

eg-① Height of instrument (or) line of collimation m/d.

The following consecutive readings were taken, with a following consecutive readings were taken common interval of 15m. The first reading was at a chainage of 165m where the RL is 98.005 the instrument was shifted after 4th

readings:-

3.150, 2.245, 1.125, 0.850, 3.125, 2.760, 1.835, 1.470, 1.965, 1.225, 2.390 & 3.035m



Check :-

$$\sum BS - \sum FS = 7.500 - 5.860 = +1.640$$

$$(\text{Last RL}) - (1^{\text{st}} \text{RL}) = 99.725 - 98.085 = +1.640$$

$$\sum BS - \sum FS = (\text{Last RL}) - (1^{\text{st}} \text{RL})$$

$$L.H.S = R.H.S$$

Hence checked

st pt	chainage	B.S	I.S	F.S	H.I	R.L	Remarks
A	165	3.150			101.235	98.085	B.M
B	180		2.245			98.990	
C	195		1.125			100.110	
D	210	3.125		0.866	103.500	100.375	LP
E	225		2.760			100.740	
F	240		1.835			101.665	
G	255		1.470			102.030	
H	270	1.225		1.965	102.760	101.535	LP
I	285		2.390			100.370	
J	300			3.035		99.728	

$\Sigma B.S = 7.500$

$\Sigma F.S = 5.866$

Rise & Fall m/d :-

The following, consecutive readings were taken with a levelling instrument at intervals of 20m.

2.375, 1.730, 0.615, 3.45-

The Instrument was Shifted after Fourth Eight readings. The last reading was taken B.M of R.L 110.200m
Find R.L's of all pts.

Q.1-

St. pt.	Change	B.S	I.S	F.S	Rise (+)	Fall (-)	R.L	Remarks
A	0	2.375					112.620	
B	20		1.730		0.645		113.265	
C	40		0.615		1.115		114.380	
D	60	2.835		2.450		2.835	115.45	C.P.
E	80		2.070		0.765		112.310	
F	100		1.835		0.235		112.545	
G	120	0.485		0.985	0.850		113.395	C.P.
H	140		1.630			1.175	112.200	
I	160		2.290			0.625	111.575	
J	180			3.630		1.375	110.200	on B.M.

$$\sum BS = 5.645 \quad \sum FS = 8.065 \quad \sum Rise = 3.610 \quad \sum Fall = 6.030$$

$$\therefore \sum BS - \sum FS = 5.645 - 8.065 = -2.420$$

$$\sum Rise - \sum Fall = 3.610 - 6.030 = -2.420$$

$$\text{W.K.T, } \sum BS - \sum FS = \sum Rise - \sum Fall = \text{Last R.L.} - \text{1st R.L.}$$

$$\Rightarrow \text{Last R.L.} - \text{1st R.L.} = -2.420$$

$$\Rightarrow \text{1st R.L.} = (\text{Last R.L.}) + 2.420$$

$$= (110.200) + (2.420)$$

$$= 112.620$$

* Correction to be Applied:

1) Curvature Correction:

For long sights, the Curvature of Earth affects Staff reading.

The line of Sight is horizontal, but the line is curved & Parallel to the mean Sphere ideal Surface of the Earth.

→ The vertical distance b/w line of sight & the level line at a particular place is called - the Curvature Correction.

It is always Subtractive (i.e. negative)

∴ Curvature Correction, $c_c = \frac{D^2}{2R} = 0.0785 D^2 m$
where D = Horizontal dist in km

∴ True Staff Reading = [Observed Staff reading] -

2) Refraction, Correction: [Curvature Correction]

Rays of light are refracted when they pass through layers of air of varying density.

→ So, when long sights are taken, the line of sight is refracted towards the surface of the earth is curved path.

→ Due to the effect of refraction, object appears higher than they really are, the refraction correction is taken as one-seventh of the Curvature Correction.

$$C_r = \frac{1}{7} \times \frac{D^2}{2R} = \frac{1}{7} \times 0.0785 D^2 = 0.0112 D^2 m$$

Refraction Correction is always addition

∴ True Staff reading = (Observed Staff reading) + (Refraction Correction)

3) Combined Correction:

Combined Correction = (Curvature Correction) + (Refraction Correction)

$$= (0.0785 D^2) + (0.0112 D^2)$$

$$= -0.0673 D^2 m$$

Combined Correction is always Subtractive (i.e. negative)

∴ True Staff reading = (Observed Staff reading) - (Combined Correction)

Contouring:-

* contour line:-

The line of intersection of a level surface with the ground surface is known as contour line (or)

Simply the contour.

→ It is also defined as a line passing through pts of equal reduced levels.

eg:- A contour of 100m indicates that all the pts on this line have an RL of 100m.

→ A map showing only the contour lines of an area is called a contour map.

* contour Interval:-

The vertical distance between any 2 consecutive contours is known as contour interval.

→ it depends upon:-

i) Nature of ground (flat (or) steep)

ii) Scale of map

iii) purpose of survey

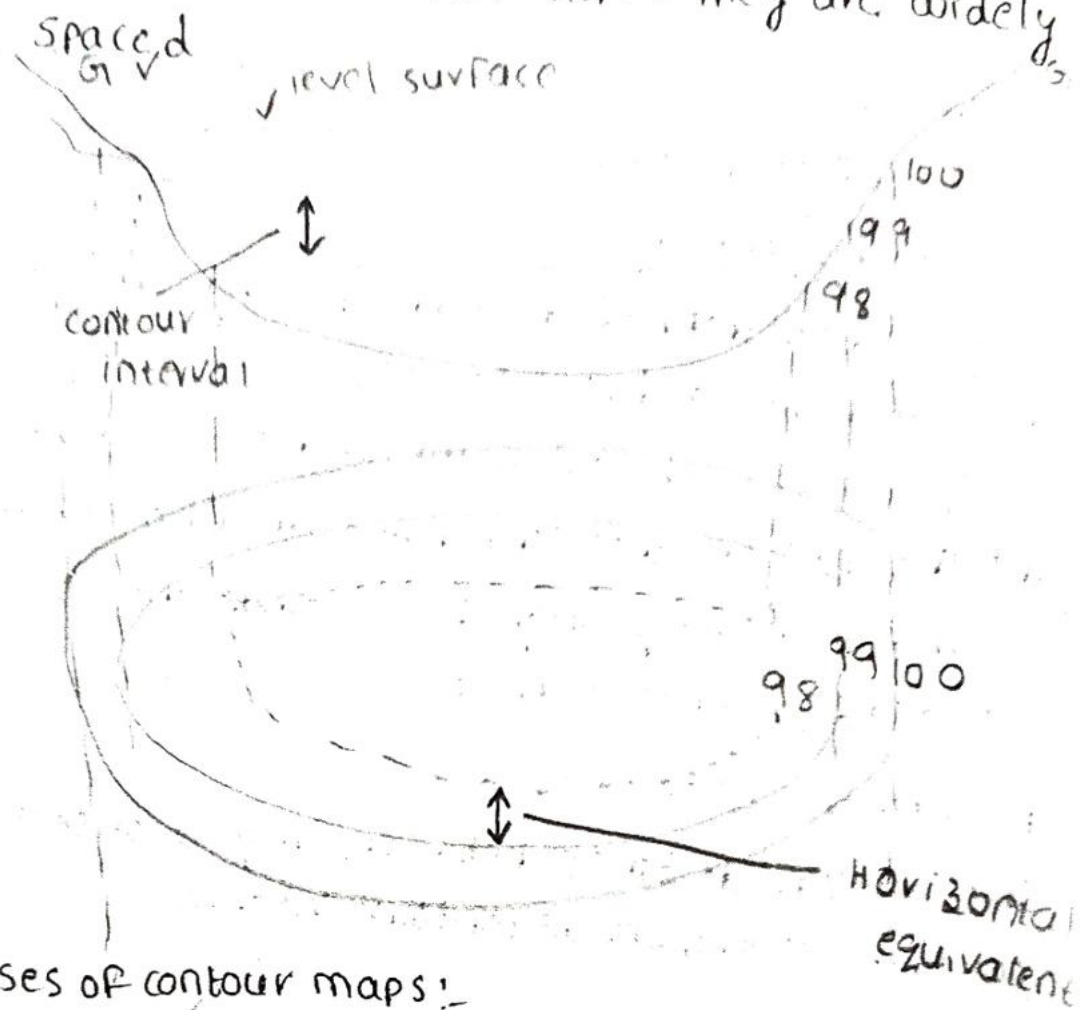
→ C.I. for a particular map is always constant

* Horizontal Equivalent:-

The horizontal distance between any 2 consecutive contours is known as Horizontal Equivalent.

→ It is not constant, varies according to the steepness of the ground.

→ For steep slopes the contour lines run close together & for flatter slopes they are widely spaced



uses of contour maps:-

The nature of the ground surface of a country can be understood by studying a contour map which helps in possible route of communication b/w different places

- * A suitable alignment can be selected for any engineering project
- * The capacity a reservoir (or) area of a catchment can be computed approx.

* characteristics of contour :-

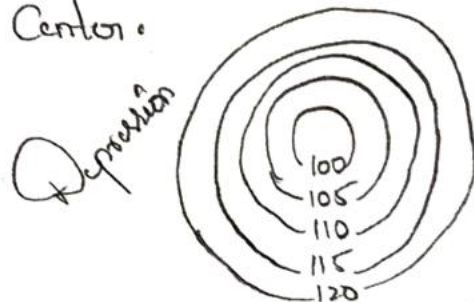
- The Contour lines are closer near the top of a hill (or) higher ground & wide apart near the foot.

This indicates a very steep slope towards the peak & a flatter slope towards the foot.



- The Contour lines are closer near the Pond (or) depression & wide apart towards the center.

This indicates a steep slope near the flatter slope at the center.

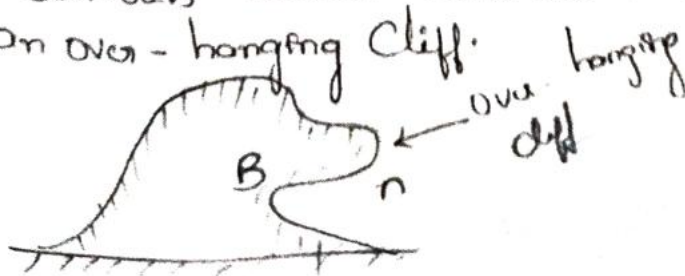


- Uniformly Spaced Contour lines indicate a Uniform Slope.

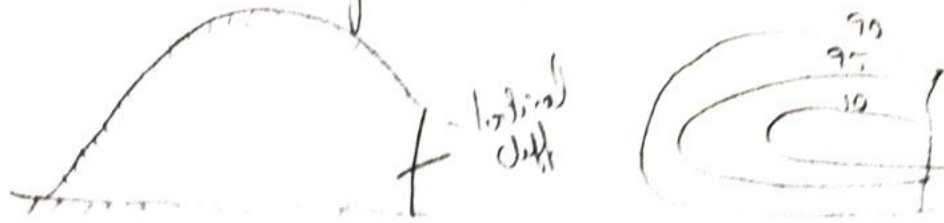


- Contours always form a closed circuit.

- Contours cannot cross one another, except in the case of an over-hanging cliff.

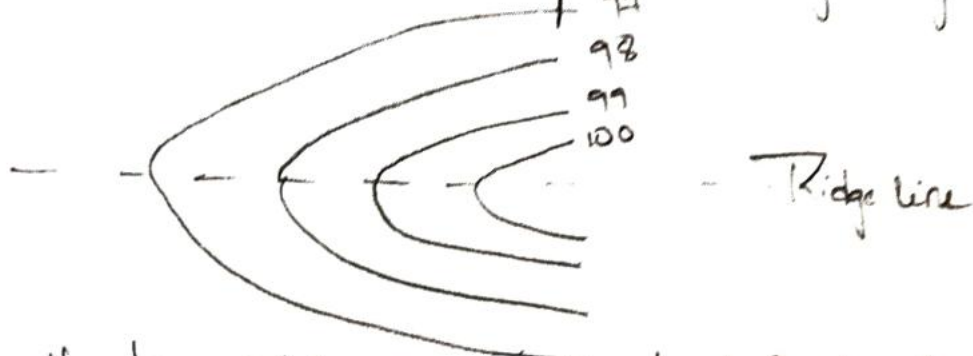


→ Contour lines meeting at a pt indicates a Vertical Cliff



→ When the higher values are inside the loop, it indicates a ridge line.

Contour line cross the ridge lines at right angles.



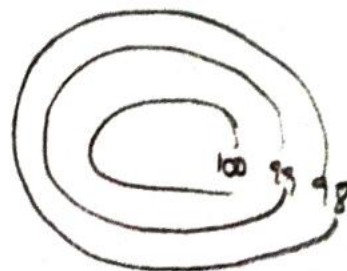
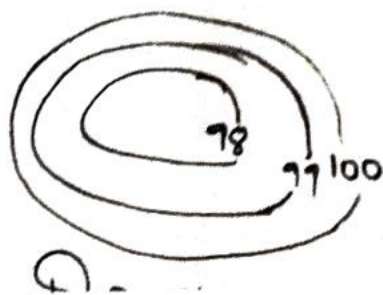
→ When the lower values are inside the loop, it indicates a valley line.

Contours cross the valley line at right angles.

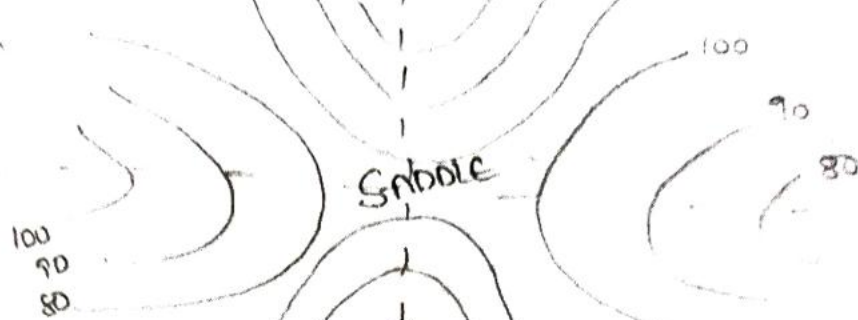


→ A series of closed contours always indicates depression or Summit.

The lower values being inside the loop indicates a depression & the higher values being inside the loop indicates a Summit.



→ Depressions & Summits are called 'Saddles'.



* M/ds & of * Contouring: 80 90 100

i) Direct & indirect:

case ①: When the area is oblong & cannot be controlled from a single stn.

Procedure:

- A temp B.M is set up near the site by taking off fly level readings from a Permanent B.M.
- The levelling instrument is then set up at a suitable position 'L' from where max area can be covered.
- The Plane table is set up at a suitable S-In 'P' from where the above area can be plotted.
- A backsight reading is taken on the TBM. Let TBM RL be 249.50m & its BS reading be 2.250m

$$\therefore +I = \left(\text{RL}_{\text{TBM}} \right) + (B.S) = 249.50 + 2.250$$

$$= 251.750\text{m}$$

- If a contour of 250m is reqd, the staff reading should be 1.750m.

The Staff man holds the staff at diff. pts of the area by moving up & down (or) left & right,

Until the Staff reading by is exactly 1.750

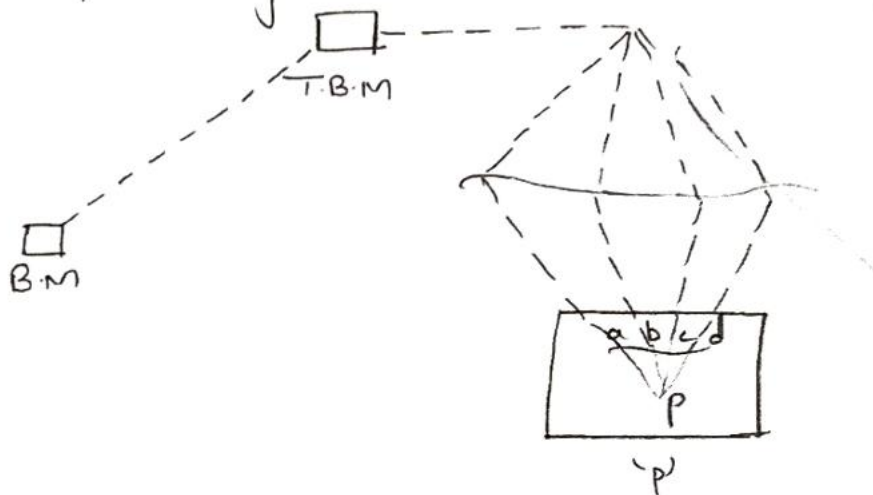
→ Then the pts are marked by Pegs & let these pts are: A, B, C, D

→ A Suitable pt 'P' is Selected on the Sheet to represent the Stn 'P'. Then with the alidade - tracing 'P' rays are drawn at A, B, C, & D.

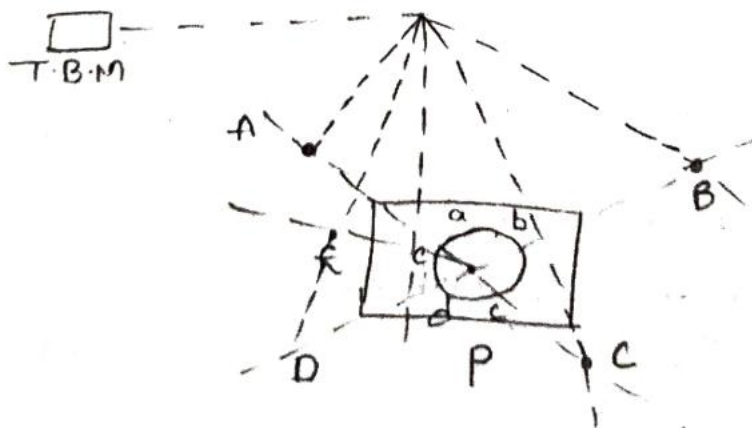
→ The distances PA, PB, PC & PD are measured & Plotted to a Suitable Scale.

→ In this manner, the pts. a, b, c & d of the Contour line of RL 250.00m are obtained.

→ These pts are joined to obtain the Contour of 250.00m.



Case (2): When the area is Small & can be Controlled from a Single Stn.



Direct mtd is very slow & tedious, But they give accurate contour lines.

② Indirect mtd

In this mtd, the Pts of diff. planes taken at regular intervals along a series of lines set up on the ground. The positions of these plots are plotted on a sheet to any suitable scale.

Then the Pts of Contour lines are found out by interpolation, after they are joined to get the reqd contour lines.

→ Although this mtd is very quick, it gives only approximate positions of the Contour lines.

→ This mtd is adopted in two ways:

(a) Cross-Sections

(b) Squares

* mtd of Interpolation of Contours:-

The Process of locating Contours proportionately b/w the plotted Pts is termed as Interpolation.

→ It can be done by:

(a) Arithmetic calculation:

(b) The graphical mtd:

Eg:-

x	y
Temp	Pre

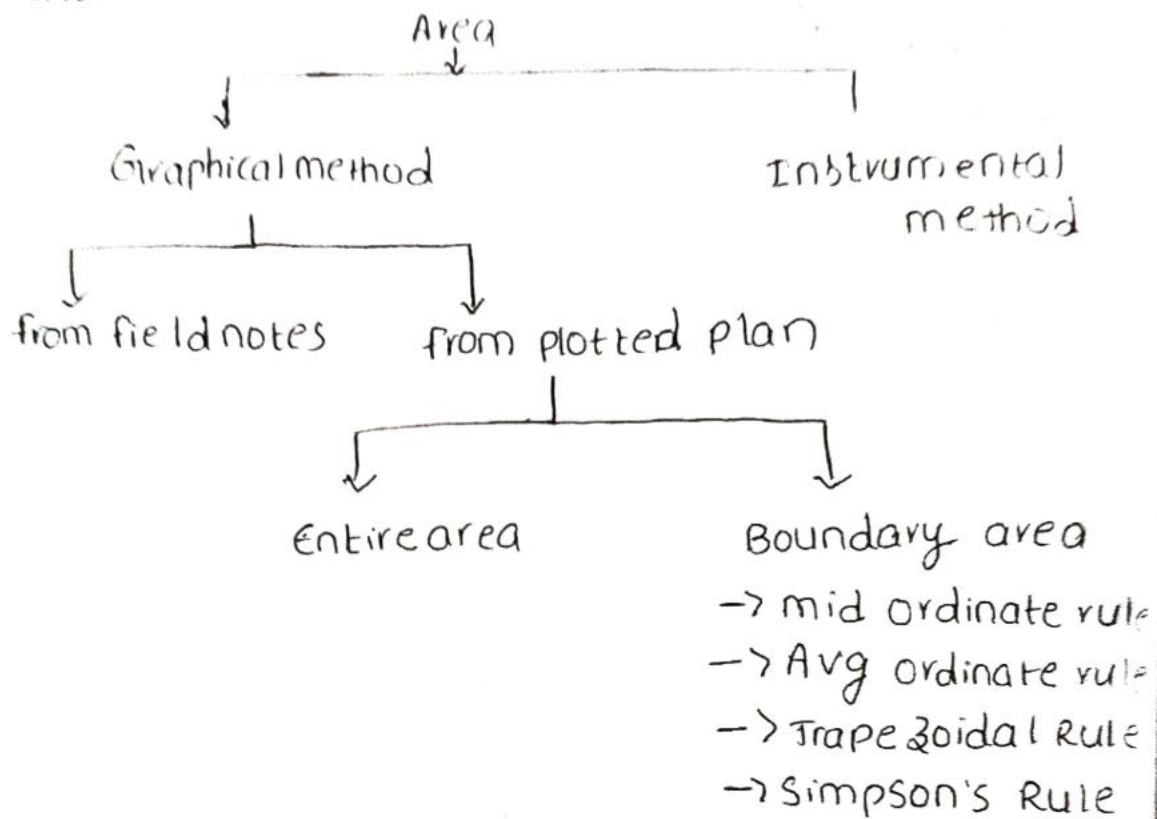
A → 10 — 20 Kpa

B → 20 — 50 Kpa

$$X = \frac{\begin{matrix} T_B & T_x & P_A & T_x & T_A & P_B \\ (20-17) & 80 & + & (17-10) & 50 \end{matrix}}{\begin{matrix} (20-10) \\ T_B - T_A \end{matrix}} = \frac{3(20) + 7(50)}{10} = 41$$

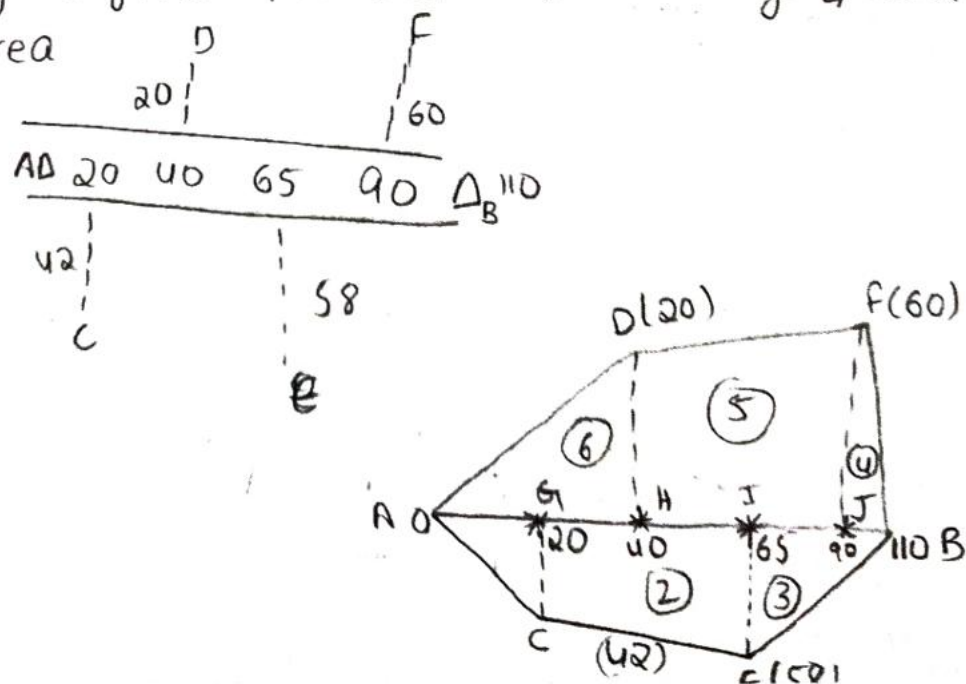
Computation of Areas:

The various methods of computation of area are:

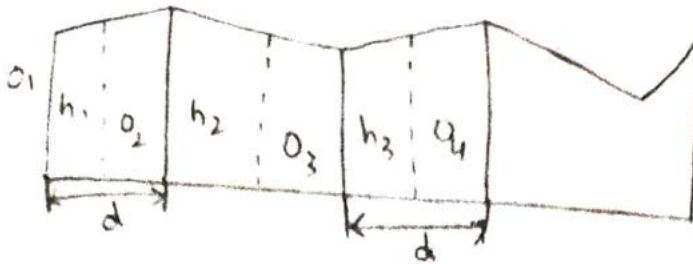


from field notes:

eg:- A page of field book of a cross-staff survey is given. plot the required fig & calculate its area



* mid-ordinate Rule:



Let $o_1, o_2, o_3, \dots, o_n$ = ordinates at equal intervals

d = common distance b/w ordinates

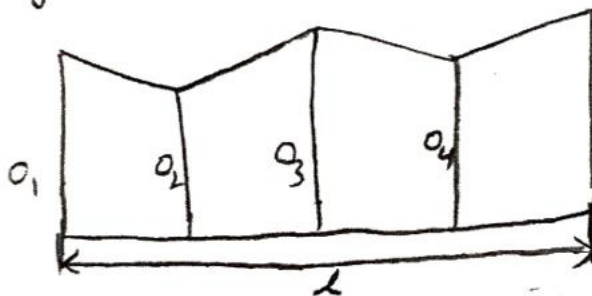
$h_1, h_2, h_3, \dots, h_n$ = mid-ordinates

$$\therefore \text{Area of plot} = (h_1 \cdot d) + (h_2 \cdot d) + (h_3 \cdot d) + \dots + (h_n \cdot d)$$

$$= d(h_1 + h_2 + \dots + h_n)$$

$$\therefore \text{Area} = (\text{common distance}) \times (\text{Sum of mid ordinates})$$

* Avg-ordinate Rule:-

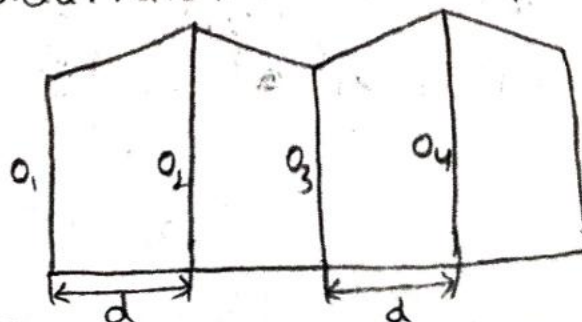


L = length of Base line

$$\text{Area} = \frac{(o_1 + o_2 + o_3 + \dots + o_n)}{(n)} \times L$$

$$\therefore \text{Area} = \frac{(\text{Sum of ordinates})}{(\text{No of ordinates})} \times (\text{length of Base line})$$

Trapezoidal rule:-



$$\therefore \text{Area} = \frac{(\text{Common distance})}{3} \left[(\text{1st Ord + last ord}) + 4(\text{sum of even ordinates}) + 2(\text{sum of odd ordinates}) \right]$$

Trapezoidal Rule	Simpson's Rule
→ Boundaries b/w ordinates are considered as st. lines → It can be applied for any no. of ordinates → It gives an approximate results.	→ Boundaries b/w ordinates are considered as parabola → To apply this rule, the no. of ordinates must be odd. → It gives more accurate results.

eg: The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m.
0.2, 5.0, 3.5, 5.0, 4.6, 3.2, 0 m.

* Instrumental method:-

The instrument used for computation of area from a plotted map is the planimeter.

→ The area obtained by planimeter is more accurate than that obtained by Graphical method.

→ There are various types of planimeter in use but the Amsler polar planimeter is the most commonly used.

ii) Avg - Ordinate Rule:-

$$\text{Area} = d \times \frac{(\text{sum of ordinates})}{(\text{No. of ordinates})}$$

$$= 60 \times \left[\frac{(0 + 2.5 + 3.5 + 5 + 4.6 + 3.2 + 0)}{7} \right]$$

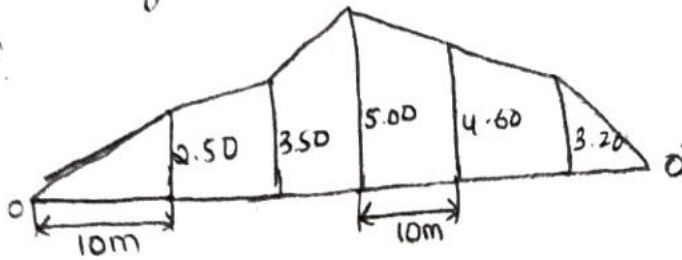
$$= 60 \times \left(\frac{18.80}{7} \right) = 161.14 \text{ m}^2$$

$$\text{Area} = \frac{d}{2} [(O_1 + O_n) + 2(O_2 + O_3 + O_4 + \dots + O_{n-1})]$$

$$\therefore \text{Area} = \frac{(\text{common dist})}{2} [1^{\text{st}} \text{ ordinate} + \text{last ordinate} + 2 \times (\text{sum of other ordinates})]$$

* Compute the area b/w chain line, the irregular boundary line & the end offsets.

Sol:-



Given data :-

$$O_1 = 0$$

$$O_2 = 2.5$$

$$O_3 = 3.5$$

$$O_4 = 5.0$$

$$O_5 = 4.60$$

$$O_6 = 3.20$$

$$O_7 = 0$$

$$h_1 = \frac{0 + 2.5}{2} = 1.25 \text{ m}$$

$$h_2 = \frac{2.5 + 3.5}{2} = 3 \text{ m}$$

$$h_3 = \frac{3.5 + 5}{2} = 4.25 \text{ m}$$

$$h_4 = \frac{5 + 4.60}{2} = 4.80 \text{ m}$$

$$h_5 = \frac{4.60 + 3.20}{2} = 3.90 \text{ m}$$

$$h_6 = \frac{3.20 + 0}{2} = 1.60 \text{ m}$$

$$d = 10 \text{ m} \Rightarrow L = 10 \times 6 = 60 \text{ m}$$

i) mid-ordinate rule:-

$$\text{Area} = d \times (h_1 + h_2 + \dots + h_6)$$

$$= 10 \times (1.25 + 3 + 4.25 + 4.80 + 3.90 + 1.60)$$

$$= 10 \times 18.80 = 188 \text{ m}^2$$

* Simpson's Rule (or) parabolic Rule:-

$$\text{Area} = \frac{d}{3} [(O_1 + O_n) + 4(O_2 + O_4 + O_6 + \dots) + 2(O_3 + O_5 + \dots)]$$

iii) Trapezoidal Rule:-

$$\text{Area} = \frac{d}{2} \times \left[(1^{\text{st}} \text{ ordinate} + \text{last ordinate}) + 2(\text{sum of other ordinate}) \right]$$

$$= \frac{10}{2} \times \left[(10+0) + 2(2.5+3.5+5+4.6+3.2) \right]$$

$$= 5 \times 37.60 = 188 \text{ m}^2$$

iv) Simpson's Rule:-

$$\text{Area} = \frac{d}{3} \times \left[(o_1 + o_n) + 4(o_2 + o_4 + \dots) + 2(o_3 + o_5 + \dots) \right]$$

$$= \frac{10}{3} \times \left[(10+0) + 4(2.5+5+3.2) + 2(3.5+4.6) \right]$$

$$= 5 \times 37.607 = \frac{10}{3} \times 59 = 196.66 \text{ m}^2$$

here $b = 10 \text{ mt}$

$$s = 2$$

$$h = 2.5 \text{ mt}$$

$$\therefore \text{cls area} = (b + sh) \cdot h$$

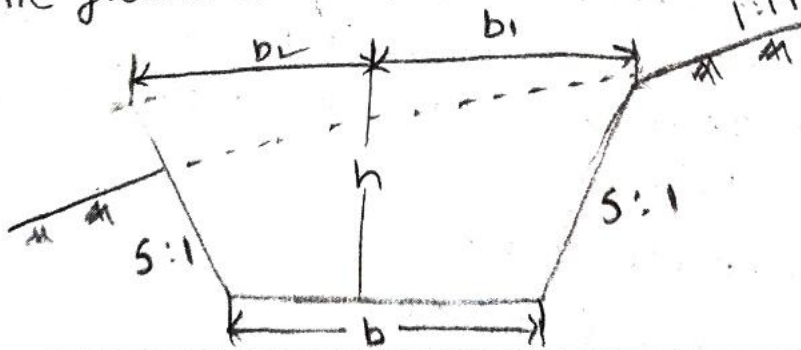
$$= [10 + 2(2.5)] \times 2.5$$

$$= (10 + 5) 2.5$$

$$= 37.5 \text{ m}^2$$

* Two-level Section:-

when the ground surface has a transverse slope



$$\therefore \text{Area} = \frac{1}{2} \cdot \left[\left(\frac{b}{2s} + h \right) \cdot (b_1 + b_2) - \frac{b^2}{2s} \right]$$

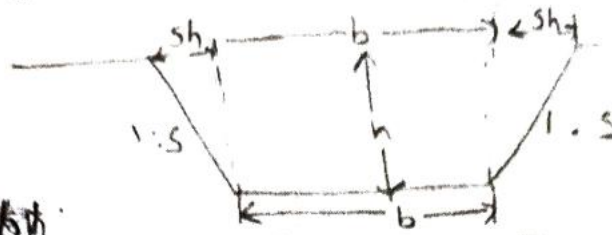
$$\text{where } b_1 = \frac{b}{2} + \frac{n \cdot s}{(n-s)} \cdot \left(h + \frac{b}{2n} \right)$$

$$b_2 = \frac{b}{2} + \frac{n \cdot s}{(n+s)} \cdot \left(h - \frac{b}{2n} \right)$$

Computation of volume :-

1. level Section.

when the ground is level along the transverse direction

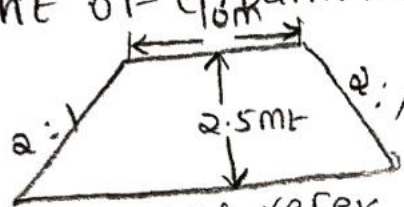


C/S area = (bh)

$$\text{C/S area} = (bh) + 2 \cdot \left[\frac{1}{2} \times sh \times h \right]$$

$$= (b + sh) \cdot h$$

eg: Calculation C/S area of an embankment 10m wide, with a side slope 2:1. The ground is level in a transverse direction to the centre line. The central ht of embankment is 2.5 mt.



A. The following notes refer to a three-level section.

Stn
①

$$\begin{array}{r} \text{C/S} \\ + 0.95 \\ \hline 4.55 \end{array}$$

$$\begin{array}{r} + 1.50 \\ \hline 0 \end{array}$$

$$\begin{array}{r} + 2.90 \\ \hline 6.50 \end{array}$$

②

$$\begin{array}{r} + 1.75 \\ \hline 5.50 \end{array}$$

$$\begin{array}{r} + 2.00 \\ \hline 0 \end{array}$$

$$\begin{array}{r} + 3.20 \\ \hline 8.30 \end{array}$$

Find C/S area at Stn ① & ②, assuming a formation width of 8m

sol. Data for C/S at Stn ①:

$$b = 8m,$$

$$b_2 = 4.55m$$

$$b_1 = 6.50m$$

$$h_2 = 0.95m$$

$$h = 1.5m$$

$$h_1 = 2.9m$$

∴ C/S area at Stn ①,

$$\Delta_1 = \frac{b}{2} (b_1 + b_2) + \frac{b}{4} (h_1 + h_2)$$

$$= \frac{1.5}{2} (6.5 + 4.55) + \frac{8}{2} (2.9 + 0.95)$$

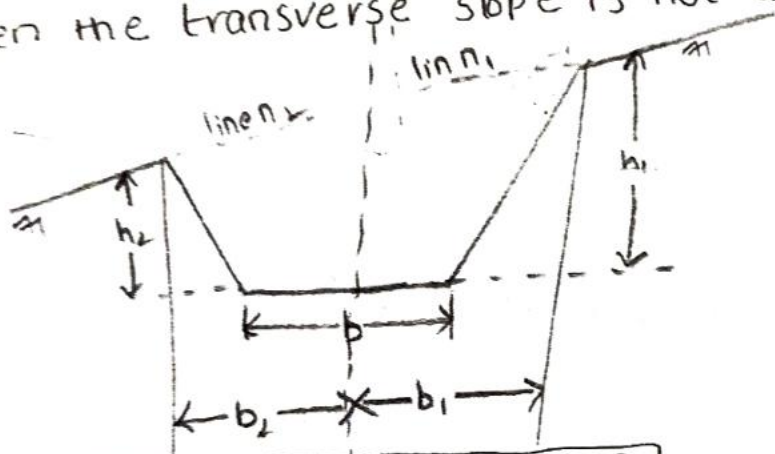
$$= 15.99 \text{ m}^2$$

Data for clsat-stn ②:

$$\begin{array}{l|l} b_2 = 8 \text{ m} & h_2 = 1.75 \text{ m} \\ b_2 = 5.5 \text{ m} & h = 2.50 \text{ m} \\ b_1 = 8.3 \text{ m} & h_1 = 3.20 \text{ m} \end{array}$$

$$\begin{aligned} A_2 &= \frac{h}{2} (b_1 + b_2) + \frac{b_u}{4} (h_1 + h_2) \\ &= \frac{2.50}{2} (8.3 + 5.5) + \frac{8}{4} (3.20 + 1.75) \\ &= \end{aligned}$$

* Three-level Section:-
when the transverse slope is not uniform.



$$\text{Area} = \left\{ \frac{h}{2} (b_1 + b_2) + \frac{b_u}{4} (h_1 + h_2) \right\}$$

where $h_1 = h + \frac{b_1}{n_1}$; $h_2 = h - \frac{b_2}{n_2}$

$$b_1 = \frac{n_1 \cdot s}{(n_1 - s)} \times \left(h + \frac{b}{2s} \right)$$

$$b_2 = \frac{n_2 \cdot s}{(n_2 + s)} \times \left(h + \frac{b}{2s} \right)$$

eg:- The width at the formation level of a certain cutting is 10 m & side slope 1:1. The surface of ground has a uniform slope of 1 in 6 in the transverse direction. Find 1/s area where the depth of cutting at centre is 3 m.

$$n = 6 \quad ; \quad h = 3 \text{ m}$$

$$b_1 = \left(\frac{b}{2} \right) + \left(\frac{n \cdot s}{n-1} \right) \left(h + \frac{b}{2n} \right)$$

$$= \left(\frac{10}{2} \right) + \left[\frac{6(1)}{6-1} \right] \cdot \left[3 + \frac{10}{2(6)} \right]$$

$$= 9.6 \text{ m}$$

$$b_2 = \left(\frac{b}{2} \right) + \left(\frac{n \cdot s}{n+1} \right) \left(h - \frac{b}{2n} \right)$$

$$= \left(\frac{10}{2} \right) + \left[\frac{6(1)}{6+1} \right] \cdot \left[3 - \frac{10}{2(6)} \right]$$

$$= 6.85 \text{ m}$$

$$\therefore \text{Area} = \frac{1}{2} \cdot \left[\left(\frac{b}{2s} + h \right) \cdot (b_1 + b_2) - \frac{b^2}{2s} \right]$$

$$= \frac{1}{2} \left[\left(\frac{10}{2(1)} + 3 \right) \cdot (9.6 + 6.85) - \frac{(10)^2}{2(1)} \right]$$

$$= 40.8 \text{ m}^2$$

2) Prismoidal formula:-

volume (cutting or fitting) $V = \frac{D}{3} \left[(A_1 + A_n) + u(A_2 + A_4 + \dots + A_{n-1}) + 2(A_3 + A_5 + \dots + A_{n-2}) \right]$

$\therefore \text{Volume} = \frac{(\text{common dist})}{3} \times \left[\frac{(\text{Area of 1st section}) + (\text{Area of last section}) + u(\text{sum of area of even sections})}{2} + \frac{2(\text{sum of areas of odd sections})}{5} \right]$

eg:- An embankment of width 10m & side slopes 1 1/2 : 1 is required to be anode on a ground which is level in a direction traverse to the centre line. The central heights at 10m intervals are as follows:

0.90, 1.25, 2.15, 2.50, 1.85, 1.35 & 0.85

cal vol of earth-work by i) Trapezoidal formula
ii) Prismoidal formula.

Sol: Given, width, $b = 10\text{m}$; $D = 40\text{m}$

$$S : 1 : 1\frac{1}{2} : 1 : 1.5 : 1$$

cls area, $A = (b + sh) \cdot h$

$$\therefore A_1 = [10 + 1.5(0.90)] \times (0.90) = 10.22\text{m}^2$$

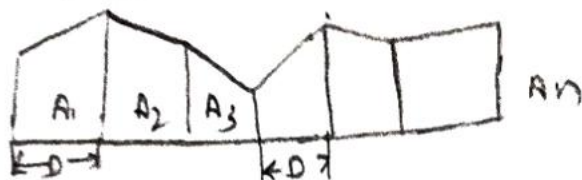
$$= \text{cls area at stn (2), } A_2 = \left\{ \frac{h}{2} (b_1 + b_2) + \frac{b}{4} (h_1 + h_2) \right\}$$

$$= A_2 = \left\{ \frac{2.00}{2} (8.30 + 5.50) + \frac{8}{4} (3.20 + 1.75) \right\}$$

$$= 1(13.80) + 2(4.95)$$

$$= 23.70\text{m}^2$$

* Formula for calculation of volume:—



D = common dist; $A_1, A_2, \dots, A_n$ = area of section

① Trapezoidal Rule [Avg End Area Rule]:—

$$\text{Volume (Cutting or Filling)} = \frac{D}{2} \left[(A_1 + A_n) + 2(A_2 + A_3 + \dots + A_{n-1}) \right]$$

$$\therefore \text{Volume} = \frac{\text{Common distance}}{2} \times \left[\text{Area of 1st section} + \text{Area of last section} + 2(\text{sum of area of other sections}) \right]$$

$$A_2 = [10 + 1.5(1.25)] \times (1.25) = 14.84\text{m}^2$$

$$A_3 = [10 + 1.5(2.15)] \times (2.15) = 28.43\text{m}^2$$

$$A_4 = [10 + 1.5(2.50)] \times (2.50) = 34.38\text{m}^2$$

$$A_5 = [10 + 1.5(1.85)] \times (1.85) = 23.63\text{m}^2$$

$$A_6 = [10 + 1.5(1.35)] \times (1.35) = 16.23\text{m}^2$$

$$A_7 = [10 + 1.5(0.85)] \times (0.85) = 9.58\text{m}^2$$

i) trapezoidal formula:-

$$V = \frac{D}{2} \times [(A_1 + A_n) + 2(A_2 + \dots + A_{n-1})]$$

$$= \frac{40}{2} \times [(10.22 + 9.58) + 2(14.84 + 28.43 + 34.38 + 23.63 + 16.23)]$$

$$= 20 \times (19.80 + 235.02)$$

$$= 5096.4 \text{ m}^3$$

ii) prismoidal formula:-

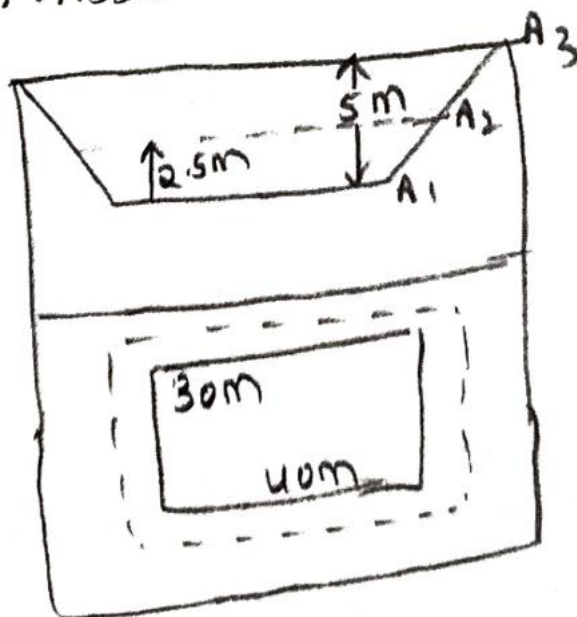
$$V = \frac{D}{3} \times [(A_1 + A_n) + 4(A_2 + A_4 + \dots) + 2(A_3 + A_5 + \dots)]$$

$$= \frac{40}{3} \times [(10.22 + 9.58) + 4(14.84 + 34.38 + 16.23) + 2(28.43 + 23.63)]$$

$$= 20 \times (19.80 + 235.02)$$

$$= 5142.9 \text{ m}^3$$

eg:- An excavation is to be made for a reservoir 40m long & 30m wide at the bottom. The side slope of the excavation has to be 2:1 calculate volume of earth-work if the depth of excavation is 5m. Assume level ground at site.



at Bottom Section,

$$l_1 = 40\text{m} ; b_1 = 30\text{m}$$

$$\therefore \text{Area, } A_1 = l_1 \cdot b_1 = 40 \times 30 = 1200\text{m}^2$$

at mid - Section,

$$l_2 = l_1 + 2sh = 40 + 2(2)(2.5) = 50\text{m}$$

$$b_2 = b_1 + 2sh = 30 + 2(2)(2.5) = 40\text{m}$$

$$\therefore \text{Area, } A_2 = l_2 \cdot b_2 = 50 \times 40 = 2000\text{m}^2$$

at top Section,

$$l_3 = l_1 + 2sh = 40 + 2(2)(5) = 60\text{m}$$

$$b_3 = b_1 + 2sh = 30 + 2(2)(5) = 50\text{m}$$

$$\therefore \text{Area, } A_3 = l_3 \cdot b_3 = 60 \times 50 = 3000\text{m}^2$$

here, we are taking common distance, $D = 2.5\text{m}$

$\therefore$ From prismoidal formula, $V = \frac{D}{3} \{ (A_1 + A_3) + 4(A_2) \}$

$$\Rightarrow V = \frac{2.5}{3} \times [(1200 + 3000) + 4(2000)]$$

$$= 10,166.66\text{m}^3$$

Theodolite Surveying

Theodolite:

The theodolite is an instrument used mainly for accurate measurement of horizontal & vertical angles upto $10''$ (or) $20''$, depending upon the least count of the instrument.

* Different purposes of theodolite :-

- i) measuring horizontal angles.
- ii) measuring vertical angles
- iii) measuring deflection angles
- iv) measuring magnetic bearings
- v) measuring horizontal distance b/w 2 points.
- vi) finding vertical height of an object.
- vii) finding difference of elevation b/w various pts.
- viii) Ranging a line

* Theodolites may be of two types:-

i) Transit theodolites

ii) Non transit

→ In the transit theodolite, the telescope can be revolved through a complete revolution about its horizontal axis in a vertical plane.

→ In the non-transit theodolite, the telescope cannot be revolved through a complete revolution in the vertical plane.

But it can be revolved to a certain extent in the vertical plane in order to measure the angle of elevation (or) depression.

→ The size of the theodolite is defined according to the dia of the main horizontal graduated circle. In engineering survey 8cm to 12cm theodolites are generally used.

For eg. in a 10cm theodolite, the dia of the main graduated circle is 10cm.

→ Theodolites may be classified as:

- i) vernier theodolite - when fitted with vernier scale
- ii) micrometer theodolite when fitted with micrometer.

Definition S:-

1) centring:-

the setting of a theodolite exactly over a stn mark by means of a plumb-bob is known as centring.

2) transitting:-

The method of turning the telescope about its horizontal axis in a vertical plane through 180° is termed as transitting.

3. Face left:- It is also referred as Bubble up.

Face left means that the vertical circle of the theodolite is on the left of the observer at the time of taking readings.

The observation taken in the face left position is called face left observation.

→ It is also known as Telescope normal (or) telescope direct.

4) Face right:-

It means that the vertical circle of the instrument is on the right of the observer when the readings

Temporary Adjustments:

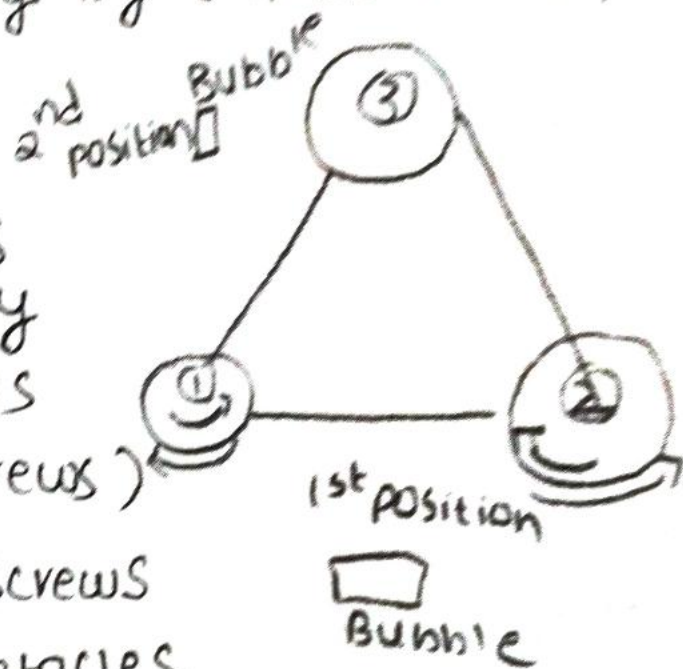
The setting of the theodolite over a station at the time of taking any observation is called temporary adjustment.

This adjustment is necessary for every set up of the instrument.

The following steps are involved in the temporary adjustments :-

- 1) Setting the theodolite over the station.
- 2) Approximate levelling by tripod stand.
- 3) Centering
- 4) levelling :-

→ The bubble tube is placed parallel to any pair of foot screws (let 1st & 2nd foot screws)



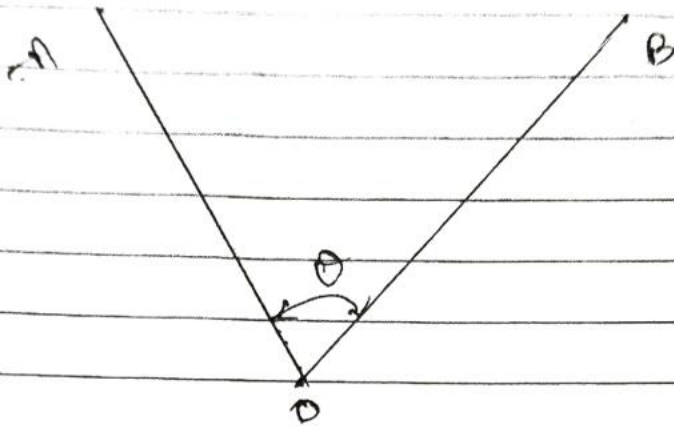
By turning both these screws equally inward (are obstacles).

The bubble is brought to the centre

→ The bubble tube is turned through 90° , so that it is perpendicular to the line joining 1st & 2nd foot screws. Then by turning 3rd foot screw either clockwise or anticlockwise the bubble is brought to the centre.

3rd unit

⇒ Direct method of measuring horizontal angle :-



- The instrument is set up over 'O'. It is centred & levelled perfectly according to the procedure
- Let the instrument was initially in the face-left position
- The lower clamp is kept fixed, The upper clamp is loosened & by turning the telescope in the clockwise direction vernier-A is set to 0° & vernier B to 180° .
- The upper clamp is tightly fixed the lower one is loosened & the telescope is directed to the left hand object 'A'. The lower clamp is tightened & by turning the lower tangent screws, the ranging rod at 'A' is bisected accurately.
- The lower clamp is kept fixed, the upper clamp is loosened & the telescope is turned clockwise to approximately bisect the ranging rod at 'B' by properly focusing the telescope.
- The upper clamp is tightened & the ranging rod at 'B' is

bisected accurately by turning the upper tangent screw
 → The readings on verniers 'A' & 'B' are noted vernier 'A' gives the angle directly but in case of vernier 'B', the angle is obtained by subtracting the initial reading from the final reading

Sl. No.	Object	Angle	Observation	Reading on vernier		Angle on vernier		Mean angle of vernier	Mean angle of observation
				A	B	A	B		
O	A	$\angle AOB$	Face left	0°0'0"	180°0'0"	30°20'0"	30°20'0"	30°20'30"	30°20'30"
	B			30°20'0"	210°20'0"				
O	A	$\angle AOB$	Face right	0°0'0"	180°0'0"	30°20'40"	30°20'0"	30°20'30"	30°20'30"
	B			30°20'40"	210°20'20"				

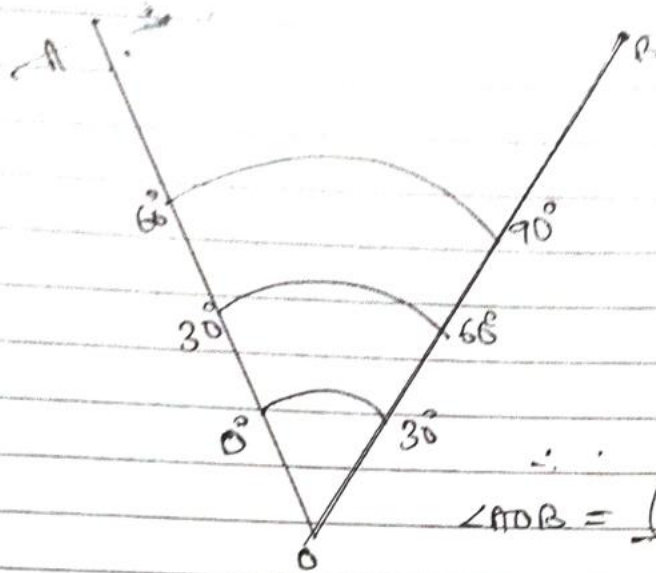
These are two methods of measuring the horizontal angles:-

① Repetition method

- Here the angle is added a no of times the total is divided by the no of readings to get the angle
- The angle should be measured clockwise in the face left and face right positions, with three repetition at each face
- The final reading of the first observation will be the initial reading of the second observation & so on.

procedure :-

- set up the theodolite at 'O'
- the instrument is centred & levelled properly vernier 'A' is set to 0° & vernier B to 180°
- the upper clamp is fixed & the lower one is loosened
- By turning the telescope the ranging rod at 'A' is perfectly bisected with the hair of lower clamp screw & lower tangent screw
- the upper clamp is loosened & the telescope is turned clockwise to perfectly bisect the ranging rod at 'B'
- let the reading on vernier 'A' is 30°
- now the upper clamp is clamped & the lower is loosened. the telescope is turned anticlockwise to bisect the ranging rod at 'A'. here the initial reading is 30° for 2nd observation
- the lower clamp is tightened the upper one is loosened & the telescope is turned clockwise to bisect the ranging rod at 'B'. let the reading on vernier 'A' be 60°
- the initial reading for 3rd observation is set to 60°
 $\angle AOB$ is measured again
let, the final reading on the vernier 'A' be 90°
(accumulated angle)
- The face of the instrument is changed. the previous is followed, the mean of the two observations gives the actual angle $2\angle AOB$.



$$\angle AOB = \frac{\text{accumulated angle}}{\text{no of readings}}$$

$$= \frac{120^\circ}{4} = 30^\circ$$

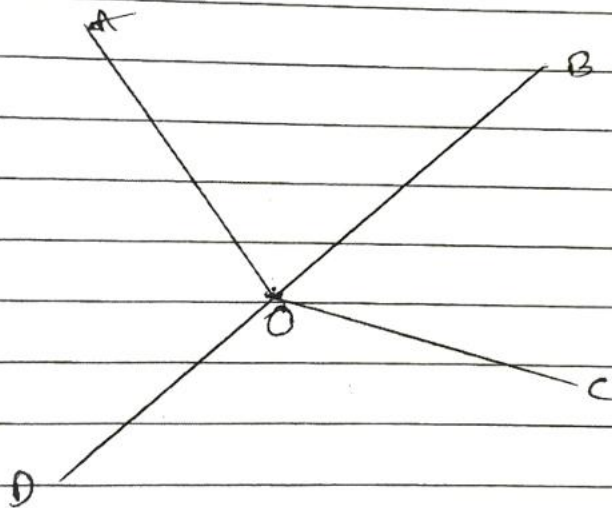
Sl. No.	Object	Face	Angle	no of readings	initial angle on vernier		final angle on vernier		angle on vernier		mean angle of vernier	no of observations
					A	B	A	B	A	B		
1	A	Left Face	$\angle AOB$	1	0°0'0"	180°0'0"	30°10'0"	210°10'0"	30°10'	30°10'	30°23'26"	
				2	30°40'00"	210°40'00"	61°40'00"	240°40'00"	30°40'	30°40'		
				3	61°40'00"	240°40'00"	93°40'00"	270°40'00"	46°	6"		
2	A	Right Face	$\angle AOB$	1	0°0'0"	180°0'0"	30°10'0"	210°10'0"	30°10'	30°10'	30°23'26"	
				2	30°40'00"	210°40'00"	61°40'00"	240°40'00"	30°40'	30°40'		
				3	61°40'00"	240°40'00"	93°40'00"	270°40'00"	46"	0"		

Teacher's Signature

⇒ Measurement of horizontal angle by reiteration method

Reiteration method is adopted when several angles are to be measured at a station the angles are measured in succession and finally "horizon is closed" to check the accuracy of the work it may be noted that the final reading after closing horizon should be same as the initial reading

let it be required to measure angles $\angle AOB$, $\angle BOC$ and $\angle COD$



⇒ procedure

- The theodolite is set up at 'O' and all the temporary adjustments are made. the reading on vernier 'A' is then adjusted to $0^\circ - 0' - 0''$ and the object 'A' is bisected with the help of lower clamp and lower tangent screw. lower clamp is then tightened

- The upper clamp is then released and the telescope is rotated in horizontal plane in clock wise direction and the next object B is bisected exactly by upper clamp and upper slow motion screw.
- The above procedure is repeated to bisect the objects C and D in succession and both the verniers are read and the corresponding included angles are worked out.
- Lastly the horizon is closed by bisecting the same initial object P. The final reading now should be the same as the initial one.
- If not discrepancy is noted and equally distributed in all angles. If the discrepancy is beyond the permissible limits the above readings are to be deleted and new set of readings are to be taken.
- The mean of the above two sets of observations gives required horizontal angles, the readings are entered in a tabular form.
- ⇒ Errors eliminated by Reiteration method.
- Graduation errors are eliminated.
- Errors due to non adjustment of line of collimation and horizontal axis are eliminated by taking face left and right face observations.

Trigonometrical levelling

Introduction

Trigonometrical levelling is the process of determining the differences of elevations of stations from observed vertical angles and known distances, which are assumed to be either horizontal (or) geodetic lengths at mean sea level. The vertical angles may be measured by means of an accurate theodolite and the horizontal distances may either be measured

We shall discuss the trigonometrical levelling under two heads.

- (i) observations for heights and distances, and
- (ii) Geodetical observations.

→ In the first case, the principles of plane surveying will be used, it is assumed that the distances between the points observed are not large so that either the effect of curvature and refraction may be neglected or proper corrections may be applied linearly to the calculated difference in elevation.

Teacher's Signature

In the geodetical observations of trigonometrical levelling, the distance between the points measured is geodetic and is large. The ordinary principles of plane surveying are not applicable. The corrections for curvature and refraction are applied in angular measure directly to the observed angles. The geodetical observations of trigonometrical levelling have been dealt with in the second volume.

HEIGHTS AND DISTANCES

In order to get the difference in elevation between the instrument station and the object under observation, we shall consider the following cases:

Case 1:- Base of the object accessible.

Case 2:- Base of the object inaccessible: instrument stations in the same vertical plane as the elevated object

Case 3:- Base of the object inaccessible: instrument stations not in the same vertical plane as the elevated object

⇒ Base of the object accessible

Let it be assumed that the horizontal distance between the instrument and the object can be measured accurately.

P = instrument station

Q = point to be observed

A = centre of the instrument

Q' = projection of Q on horizontal plane through A

$D = A Q'$ = horizontal distance between P and Q

h' = height of the instrument at P

$h = Q Q'$

s = reading of staff kept at B.M. with line of sight horizontal

α = angle of elevation from A to Q

from triangle $A Q Q'$: $h = D \tan \alpha$

R.L of Q = R.L of instrument axis + h

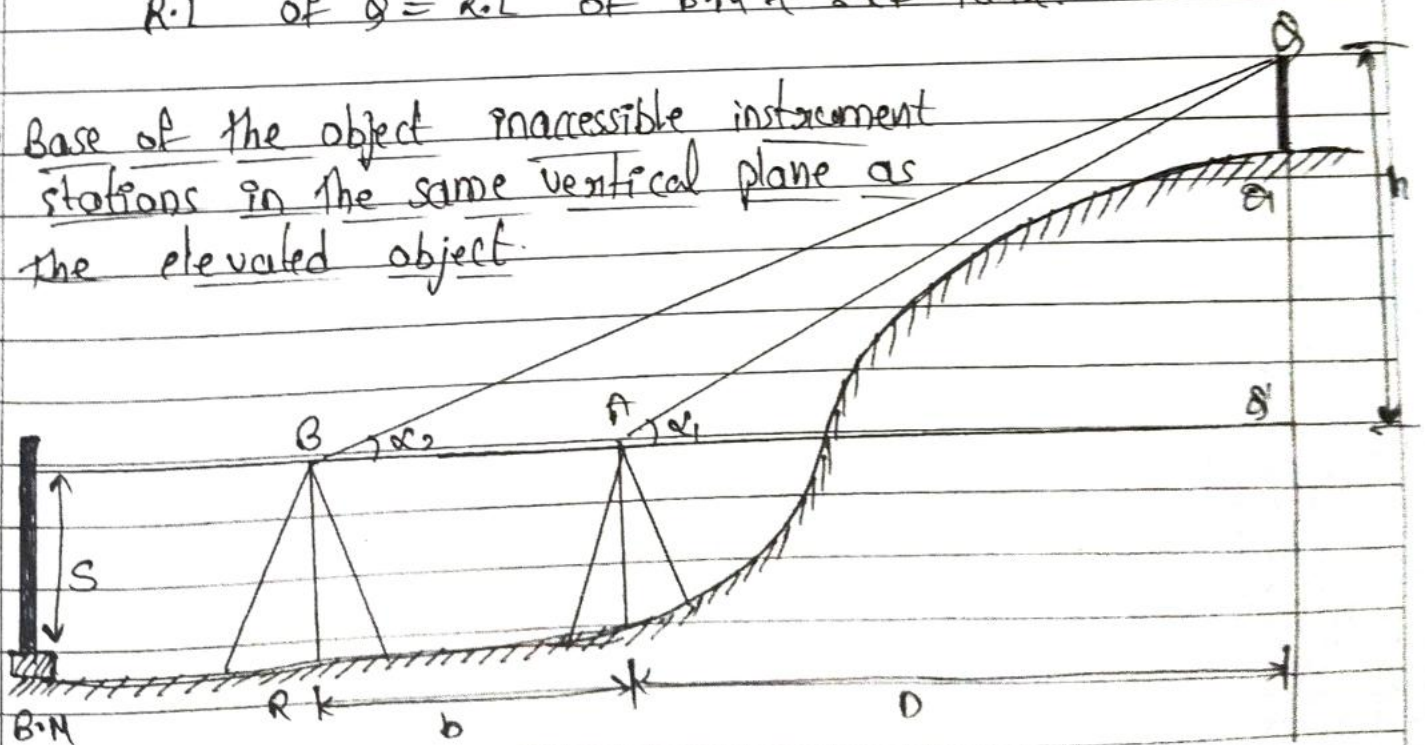
if the R.L of P is known

R.L of Q = R.L of P + h' + $D \tan \alpha$

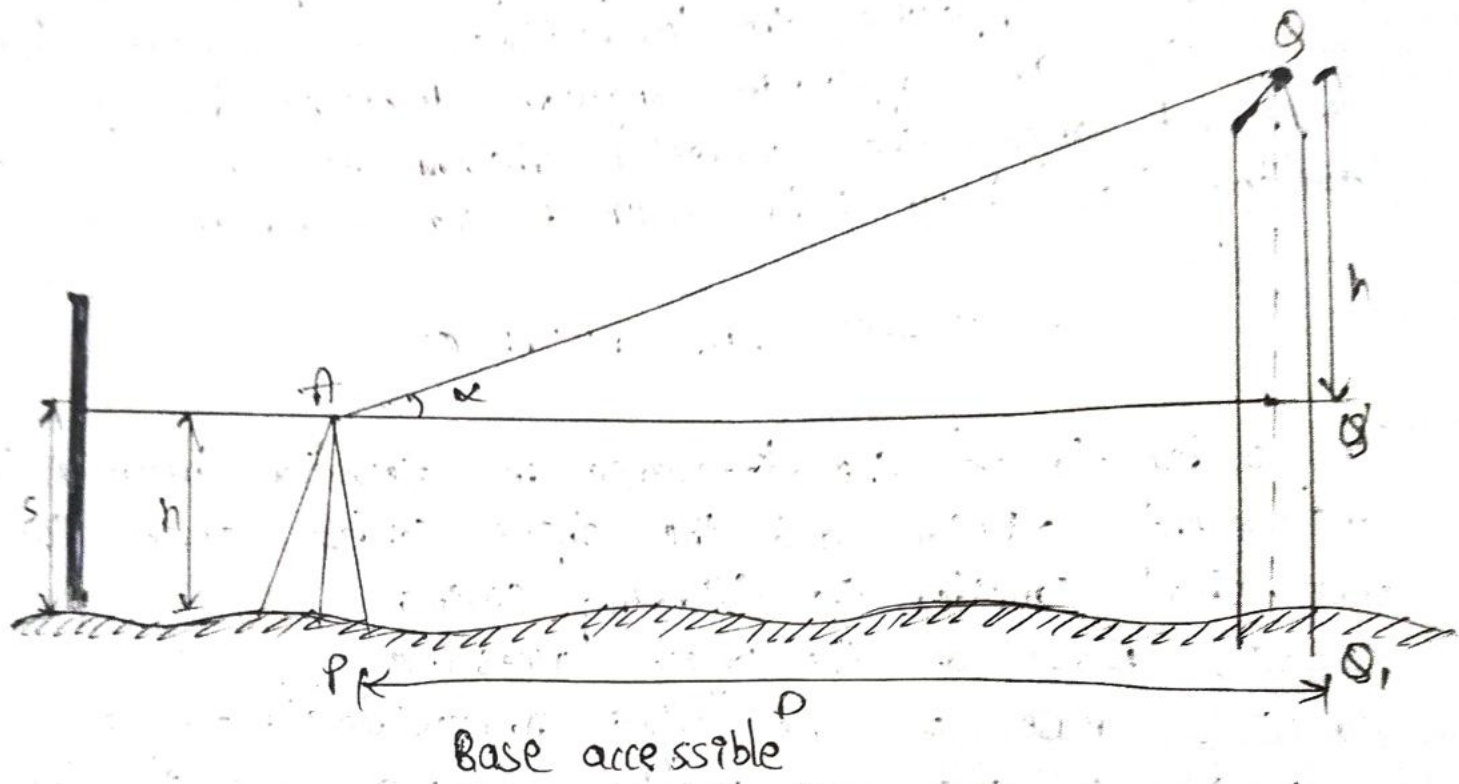
if the readings on the staff kept at the B.M. is s with the line of sight horizontal.

R.L of Q = R.L of B.M + s + $D \tan \alpha$

⇒ Base of the object inaccessible instrument stations in the same vertical plane as the elevated object.



Teacher's Signature.....



If the horizontal distance between the instrument and the object can be measured due to obstacles etc. two instrument stations are used so that they are in the same vertical plane as the elevated object.

Procedure

- Set up the theodolite at P and level it accurately with respect to the altitude bubble.
- Direct the telescope to mark Q and bisect it accurately clamp both the plates. read the vertical angle α .
- Transit the telescope so that the line of sight is reversed mark the second instrument station R on the ground measure the distance RP accurately.
- repeat steps (2) and (3) for both face observations. the mean values should be adopted.
- with the vertical vernier set to zero reading, and the altitude bubble in the centre of its run take the reading on the staff kept at the nearby B.M.
- with the vertical vernier set to zero reading and the altitude bubble in the centre of its run take the reading on the staff kept at the nearby B.M.

@ when the instrument axes of A and B are at the same level.

Teacher's Signature:

- (b) when they are at different levels but the difference is small and
 (c) when they are at very different levels
 (a) Instrument axes at the same level

$$h = OQ'$$

α_1 = angle of elevation from A to Q

α_2 = angle of elevation from B to Q

s = staff reading on B.M taken from both A and B the reading being the same in both the cases

b = horizontal distance between the instrument stations

D = horizontal distance between P and Q

from triangle Agg' , $h = D \tan \alpha_1$

from triangle Bgg' $h = (b+D) \tan \alpha_2$

equating (1) and (2) we get

$$D \tan \alpha_1 = (b+D) \tan \alpha_2$$

$$D = \frac{b \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2}$$

$$h = D \tan \alpha_1 = \frac{b \tan \alpha_1 \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = \frac{b \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)}$$

$$R.L \text{ of } Q = R.L \text{ of B.M} + s + h$$

Teacher's Signature.....

⑤ Instrument axes at different levels

When the instrument axes are at different levels if S_1 and S_2 are the corresponding staff readings on the staff top of B.M., the differences in levels of the instrument axes will be either $(S_2 - S_1)$ if the axis at B is higher or $(S_1 - S_2)$ if the axis at A is higher. Let Q' be the projection of Q on horizontal line through A and Q'' be the projection on horizontal line through B. Let us derive the expressions for when S_2 is greater than S_1 .

From triangle QAQ' $h_1 = D \tan \alpha_1$

From triangle BQQ'' , $h_2 = (b + D) \tan \alpha_2$

Subtracting (2) from (1), we get

$$(h_1 - h_2) = D \tan \alpha_1 - (b + D) \tan \alpha_2$$

$h_1 - h_2 =$ difference in level of instrument

axes $= S_2 - S_1 = S$ (say)

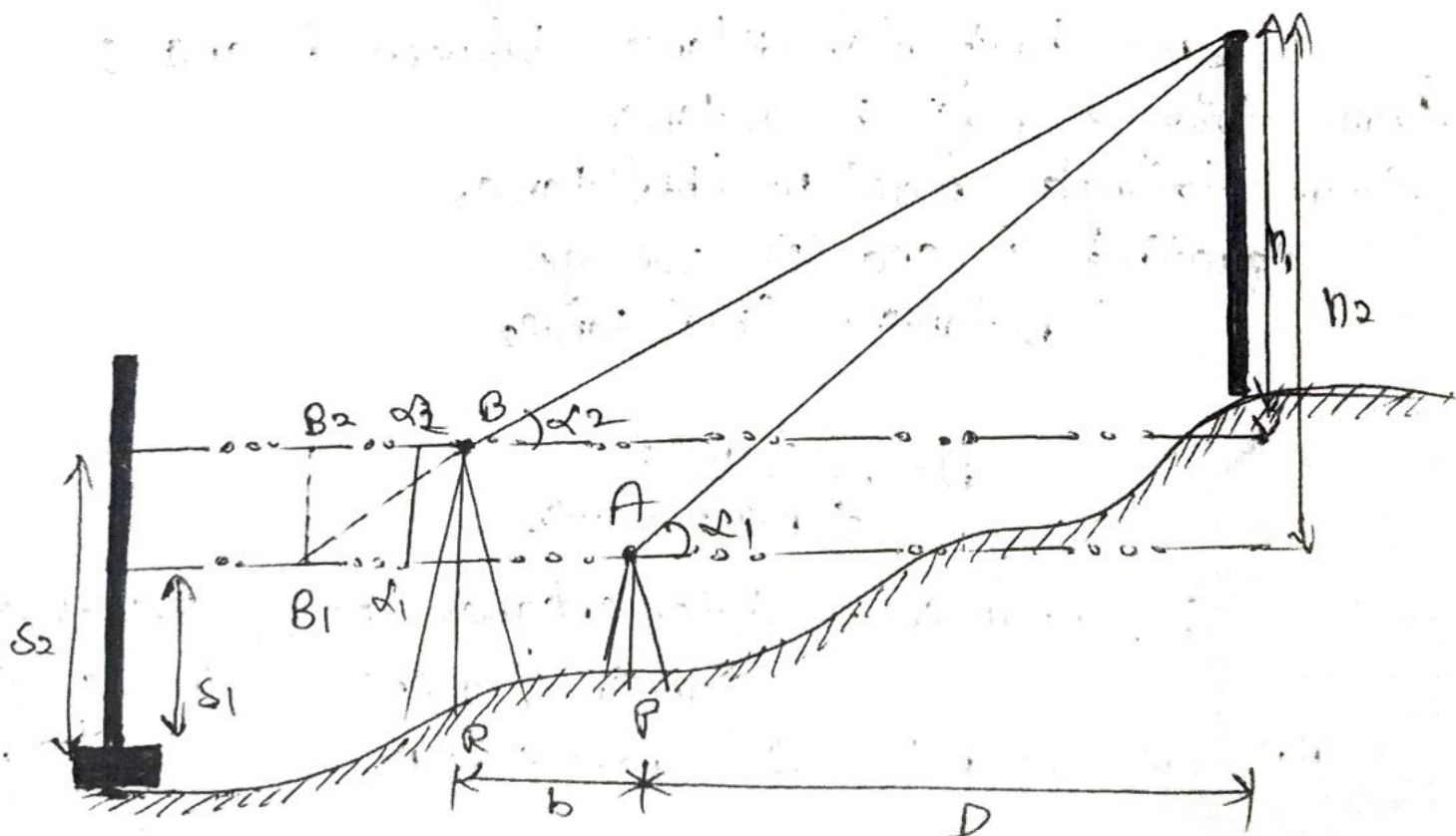
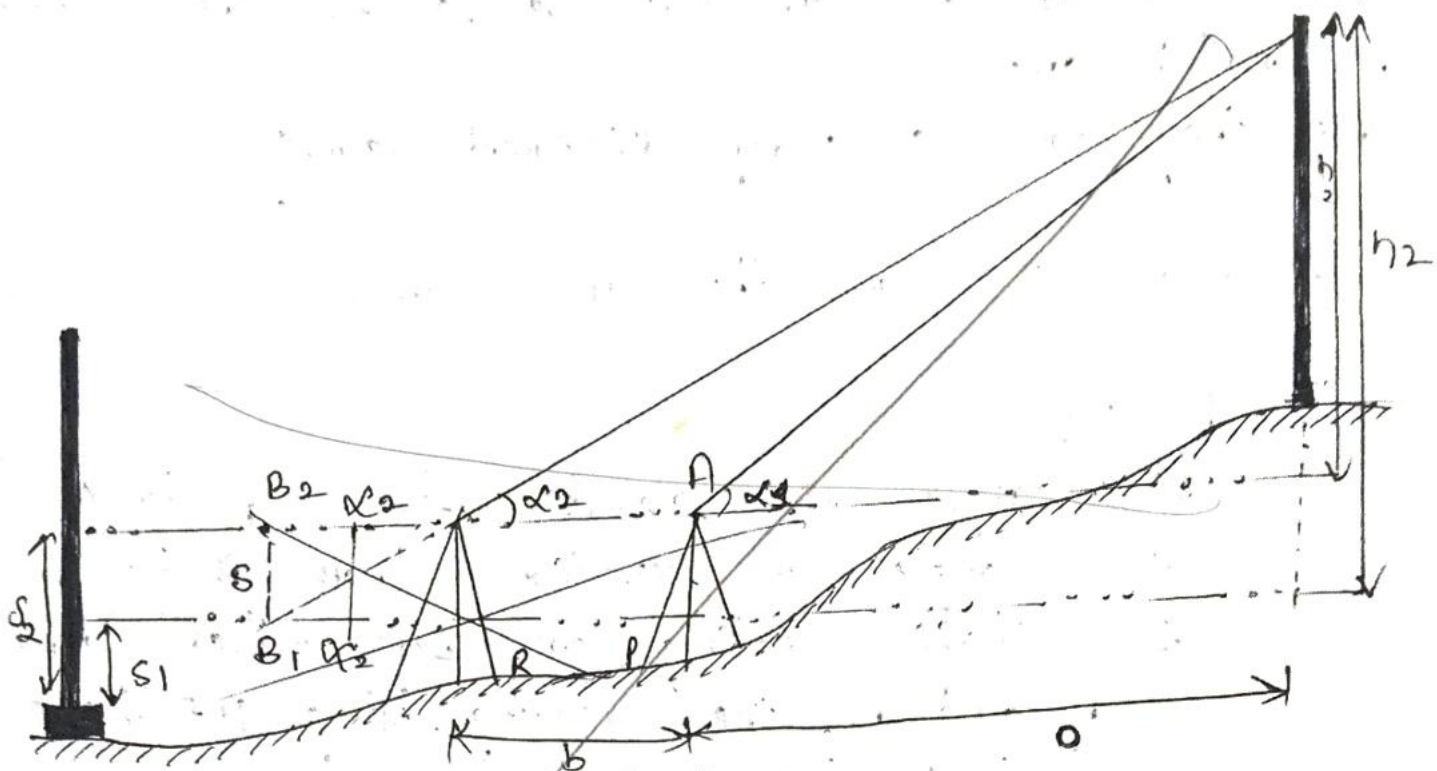
$$S = D \tan \alpha_1 - b \tan \alpha_2 - D \tan \alpha_2$$

Now $h_1 = D \tan \alpha_1$

$$h_1 = \frac{(b + S \cot \alpha_2) \tan \alpha_1 \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2}$$

$$= \frac{(b + S \cot \alpha_2) \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)}$$

~~expression is~~



② could also be obtained by producing the lines of sight BO backwards to meet the line of A in B , drawing B_1B_2 as vertical to meet the horizontal line of B in B_2 . It is clear that with the same angle of elevation if the instrument axis were the distance at which the axes are at the same level is $AB_1 = b + BB_2 = b + s \cot \alpha_2$ substituting this value of the distance between the instrument stations in equation we get

$$D = \frac{(b + s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} \text{ which is the same as equation}$$

proceeding on the same lines for the case where the instrument at O is higher, it can be proved.

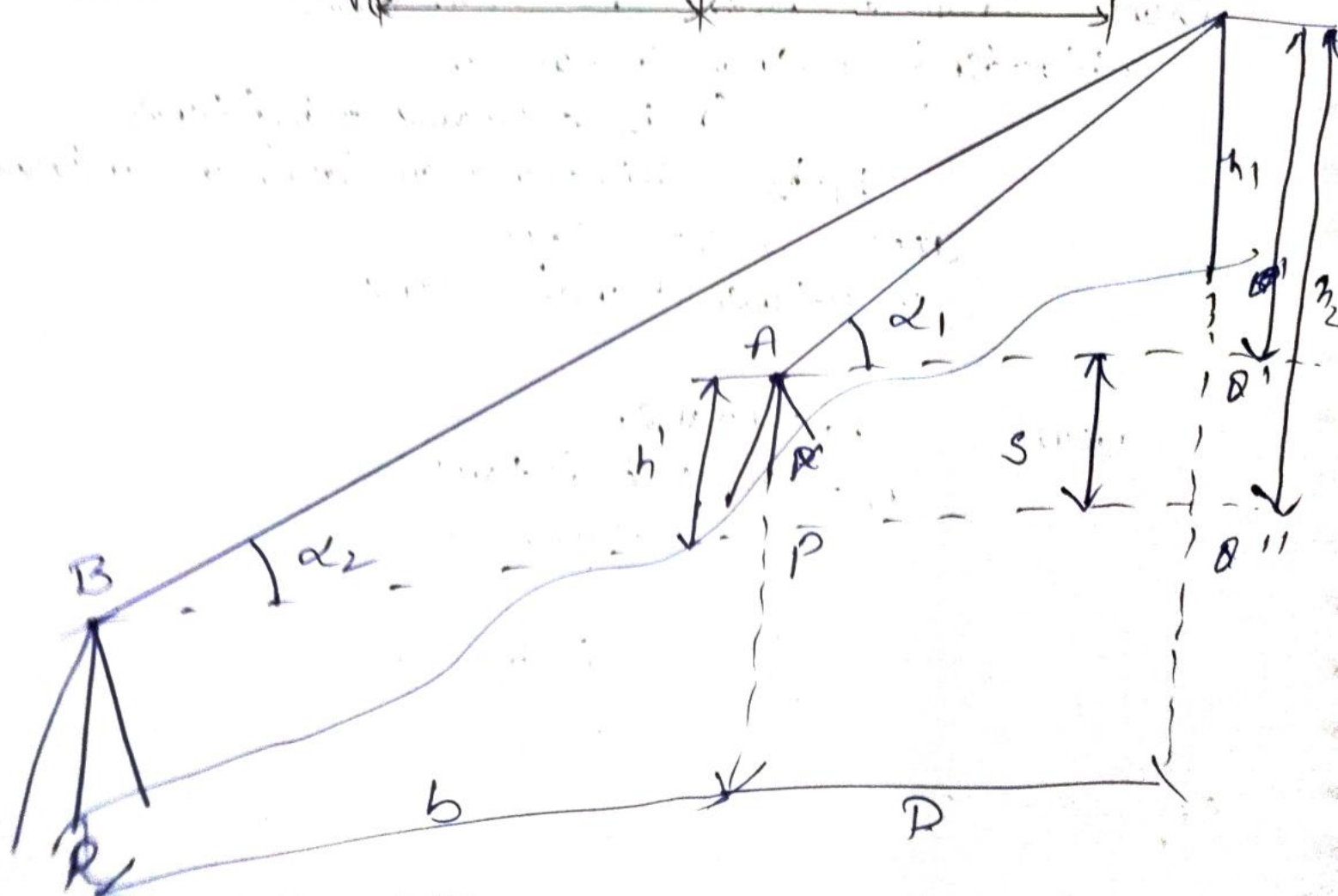
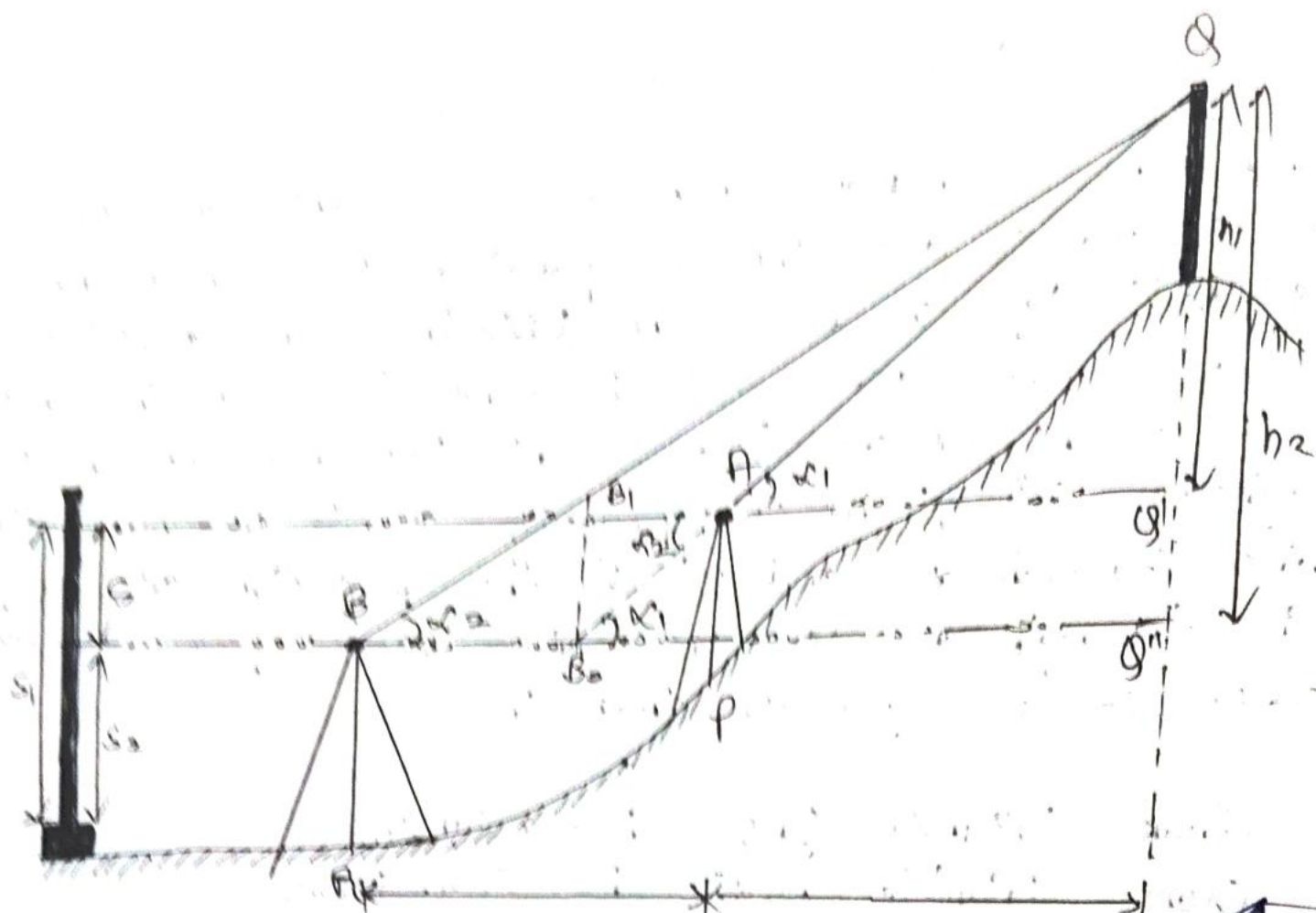
$$D = \frac{(b - s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2}$$

$$\text{and } h_1 = \frac{(b - s \cot \alpha_2) \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)}$$

Thus, the general expressions for D and h_1 can be written as:

$$D = \frac{(b \pm s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2}$$

Teacher's Signature.....



$$h_1 = (b \pm \text{scot } \alpha_2) \sin \alpha_1 \sin \alpha_2$$

$$\sin (\alpha_1 - \alpha_2)$$

use + sign with $\text{scot } \alpha_2$ when the instrument axis at A is lower and - sign when it is higher than at B
 $R.L. \text{ at } Q = R.L. \text{ at } B.M. + S_1 + h_1$

⇒ Base of the object inaccessible instrument stations not in the same vertical plane as the elevated object

Let P and R be the two instrument stations not in the same vertical plane as that of Q. the procedure is as follows

- (i) set the instrument at P and level it accurately with respect to the altitude bubble measure the angle elevation α_1 to Q
- (ii) sight the point R with reading on horizontal circle as zero and measure the angle $\angle RPQ$, i.e. the horizontal angle θ_1 at P
- (iii) take a backsight S on the staff kept at B.M
- (iv) Shift the instrument to R and measure α_2 and θ_2 there, AQ' is the horizontal line through A, Q' being the vertical projection of Q. thus AQQ' is a vertical plane. Similarly BQQ'' is a vertical plane Q'' being the vertical

Teacher's Signature.....

plane Q'' being the vertical projection of Q on a horizontal line through B . PRQ is a horizontal plane, Q_1 being the vertical projection of Q , and R vertical projection of B on a horizontal plane passing through P . θ_1 and θ_2 are the horizontal angles, and α_1 and α_2 are the vertical angles measured at A and B respectively.

From triangle APQ_1 $QQ_1 = h_1 = D \tan \alpha_1$

From triangle BRQ $\angle PQR = 180^\circ - (\theta_1 + \theta_2) = \pi - (\theta_1 + \theta_2)$

From the sine rule.

$$\frac{PQ_1}{\sin \theta_2} = \frac{RQ_1}{\sin \theta_1} = \frac{RP}{\sin (\pi - (\theta_1 + \theta_2))} = \frac{b}{\sin (\theta_1 + \theta_2)}$$

$$PQ_1 = D = \frac{b \sin \theta_2}{\sin (\theta_1 + \theta_2)}$$

$$RQ_1 = \frac{b \sin \theta_1}{\sin (\theta_1 + \theta_2)}$$

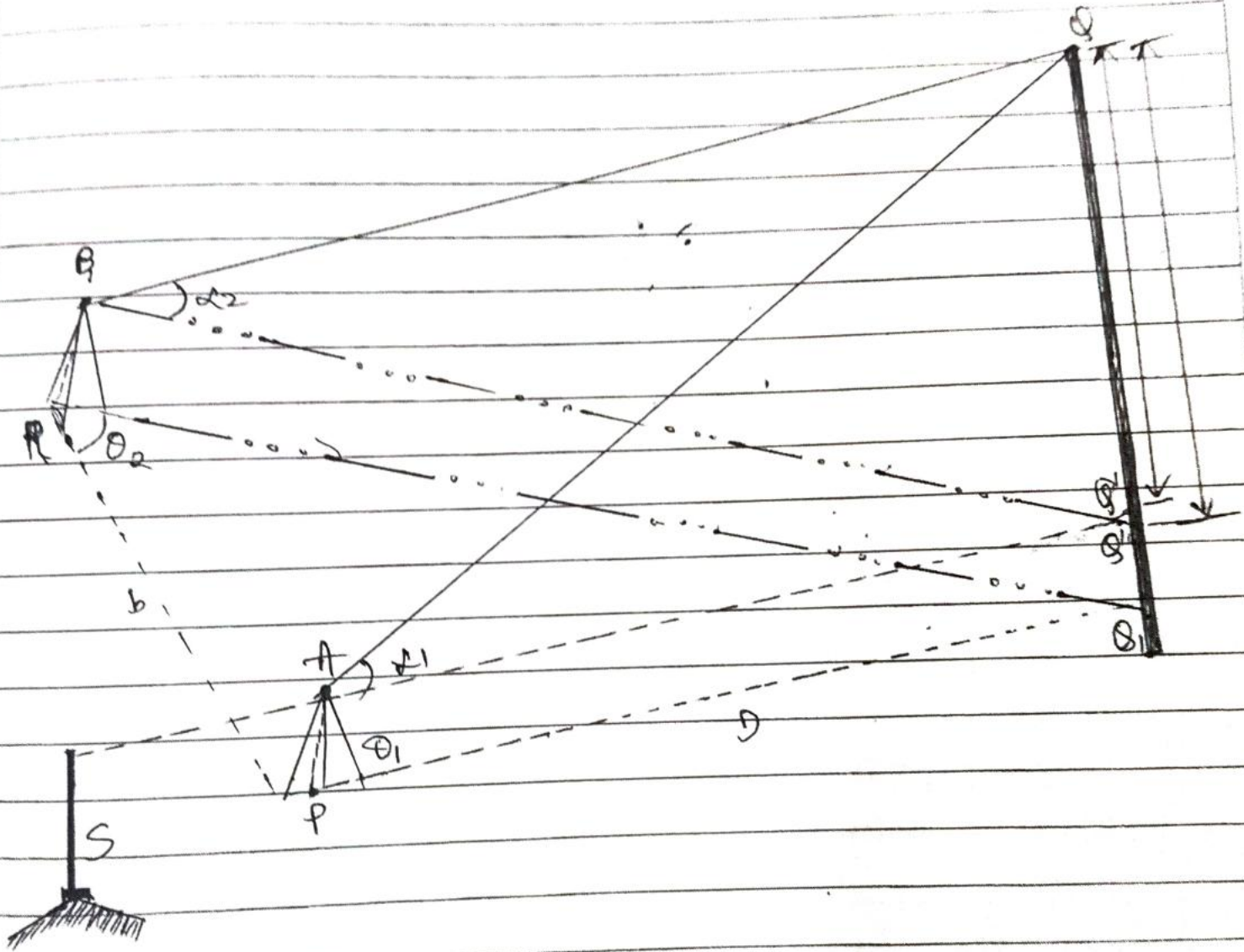
substituting the value of D in (i) we get
 $h_1 = D \tan \alpha_1 = \frac{b \sin \theta_2 \tan \alpha_1}{\sin (\theta_1 + \theta_2)}$

R.L of $Q = P.L$ of B.M + S + h_1

As a check, $h_2 = RQ_1 \tan \alpha_2 = \frac{b \sin \theta_1 \tan \alpha_2}{\sin (\theta_1 + \theta_2)}$

Teacher's Signature.....

If a reading on B.M is taken from B, the R.L of S can be known by adding h_2 to R.L of B



Teacher's Signature.....

4th unit :- Tacheometric surveying

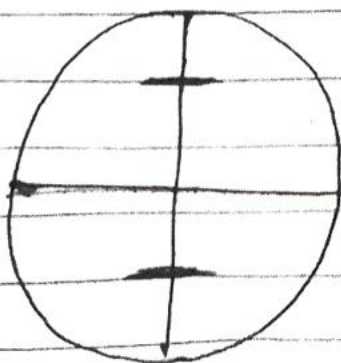
Introduction

It is a branch of surveying in which horizontal and vertical distances are determined by taking instrumental observations alone, with an instrument known as a tachometer. The chaining operation is completely eliminated in such survey. It is adopted in rough and difficult terrain where direct levelling and chaining are either not possible (or) very tedious.

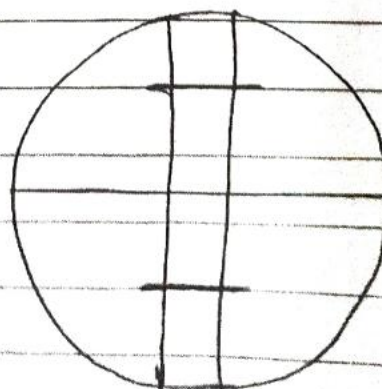
The following accessories are required for performing Tacheometry survey:

- (a) Tacheometer
- (b) Levelling staff and stadia rod
- (c) ranging rods, pegs, plumb-bob etc.

(a) Tacheometer :- A Tacheometer is a transit theodolite fitted with a stadia diaphragm and an anallatic lens. The following fig. 3.1, shows the different forms of stadia diaphragm generally fitted in a tachometer.



(a)



(b)

Teacher's Signature.....

(b) levelling staff and station rod :

Generally ordinary levelling staves are used for short distances. marked with a least count of 0.005 m and 4 long. for greater distances a specially designed station rod of 4 m long may be used the graduations are comparatively bold and clear and the least count by be

⇒ CHARACTERISTICS OF GOOD TACHEOMETER

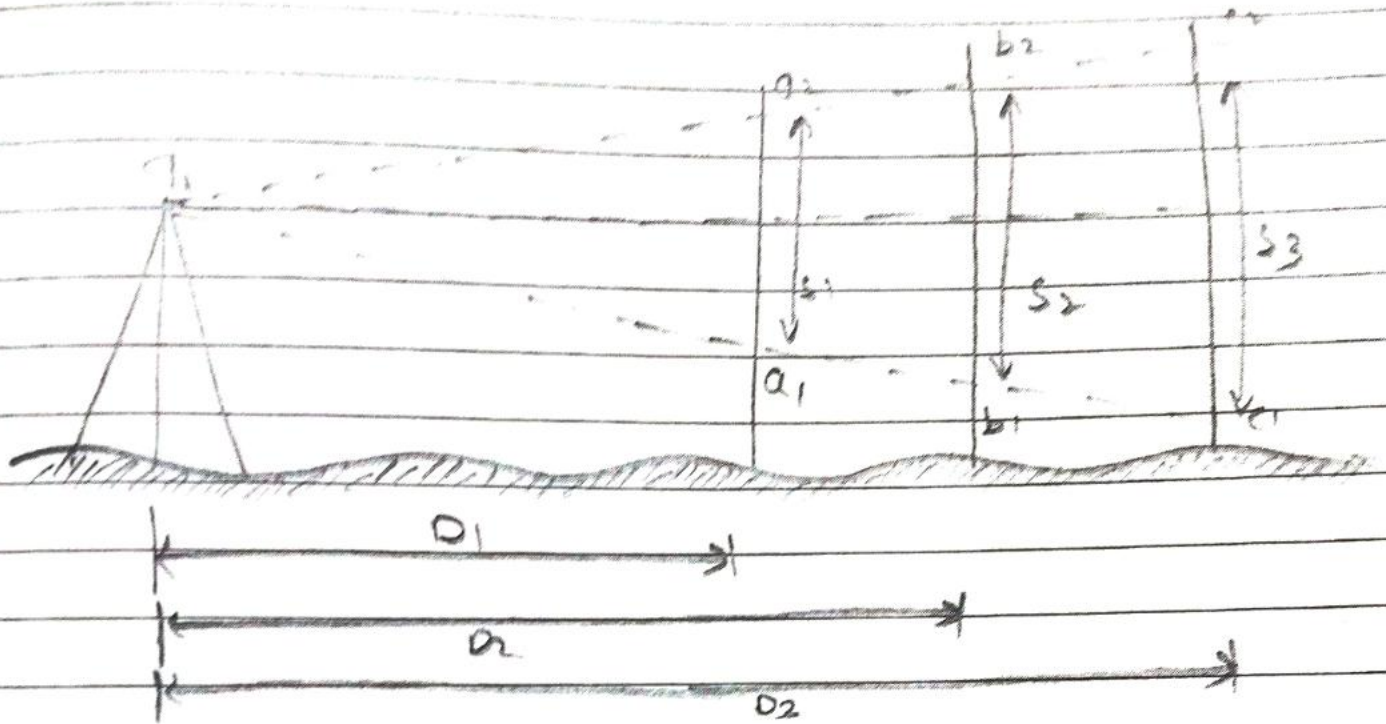
- The value of the multiplying constant f/i (or) k should be 100
- the telescope should have a magnification of at least 20 to 30 diameters
- The eyepiece should be of greater magnifying power than usual, so that it is possible to obtain a clear staff reading from a long distance

⇒ PRINCIPAL USES OF TACHEOMETRY

1. principle:- The principle of tacheometry is based on the property of isosceles triangles where the ratio of the distance of the base from the apex and the length of the base is always constant.
- In figure $\triangle A_1 a_1 b_1$, $\triangle A_2 b_2 c_2$ and $\triangle A_3 c_3 d_3$ are all isosceles triangles where D_1 , D_2 and D_3 are the distances

Teacher's Signature.....

of the true bases & from the apex T , and s_1 , s_2 and s_3 are the length of the bases (staff intercepts)



→ As per the principle stated above, we have a relation.

$$\frac{D_1}{s_1} = \frac{D_2}{s_2} = \frac{D_3}{s_3} = \frac{f}{i} \quad (\text{const}) \quad \text{or} \quad (\text{Constant})$$

The f/i (or) k is known as the multiplying constant, where f = focal length of objective and i = stadia intercept.

uses of Tacheometric surveying

- distance and elevations are obtained where direct procedures are either impossible (or) difficult
- contouring can be done even in difficult terrain with hills and other natural obstructions.

- It is advantageous when it is required to prepare small scale maps to study the topography of the area.
- It is advantageously adopted, where speed is more important than high accuracy.

⇒ METHODS OF TACHEOMETRY

Tacheometry is mainly divided into:

1. stadia Tacheometry
2. Tangential Tacheometry

1. stadia Tacheometry:- In this method the telescope of the tacheometer is provided with two stadia hairs one upper and another lower. Looking through the telescope the stadia hair readings are taken and noted. The difference between upper and lower stadia hair reading is known as staff intercept (S). To determine the distance between the instrument station and the staff, the staff intercept (S) is multiplied by the stadia constant ($1/f$ or k generally 100). The stadia method is further divided into two types:

- (a) fixed hair method:- This is the most commonly used method of tacheometry. In this method the stadia hair are kept at fixed interval and the value of staff intercept varies with the distance from instrument station. This method is also used even when horizontal sights are not possible for inclined sights, readings may be

taken holding the staff either vertical or normal to the line of sight

- (b) moveable hair method:- In this method the staff intercept on the levelling staff is kept constant by fixing two targets (or) vanes at a known distance apart. During observation, the distance between stadia hairs is so adjusted that the upper hair coincides the upper target and the lower hair coincides with the lower target, by adjusting the micrometer screw fitted on the diaphragm. The variable stadia intercept is measured and the required distance is then calculated. Generally, this method is not adopted.

2) Tangential Tacheometry

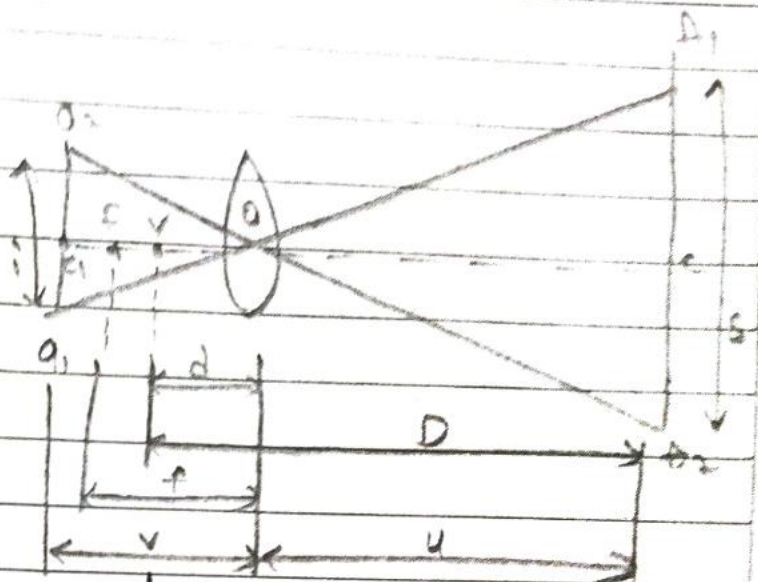
This method is generally adopted when the theodolite is not provided with stadia diaphragm. In this method the horizontal and vertical distances are calculated from the observed vertical angles to two vanes fixed at a known distance apart on the staff.

This method is also not preferred, because atleast two vertical angles are required to be taken. But in case of stadia tacheometry only one vertical angle is sufficient.

⇒ Theory of stadia tachometry

Determination of Distance between the instrument and staff station, when the line of sight is horizontal and staff held vertical.

Notations used (refer fig. 3.3)



φ_1, φ_2, C = bottom, top and central hairs of diaphragm
 A_1, A_2, C = reading on staff cut by three hairs.

$a_1, a_2 = i$ = length of image

$A_1A_2 = s$ = staff intercept

O = optical centre of object glass

f = focus

V = vertical axis of instrument

F = focal length of object glass

d = distance between optical centre and vertical axis of instrument

u = distance between optical centre and staff

v = distance between optical centre and image

from fig 2.3 triangles Δ_1 and Δ_2 and Δ_1, O, Δ_2 are similar, $\frac{i}{s} = \frac{u}{v} \Rightarrow u = \frac{iv}{s}$

from the properties of lenses, we have

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

Substituting the value of 'u' in equation (2), we get

$$\frac{1}{\frac{1}{u}} + \frac{1}{u} = \frac{1}{f} \quad \text{or} \quad \frac{s}{1u} + \frac{1}{u} = \frac{1}{f}$$

taking $\frac{1}{u}$ common, $\frac{1}{u} \left(\frac{s}{1} + 1 \right) = \frac{1}{f}$

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

But distance,

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

it can also be written as.

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

The values $\left(\frac{f}{1} \right)$ and $(f + d)$ are known as tacheometric

constants $\left(\frac{f}{1} \right)$ is called multiplying constant, and $(f + d)$ the additive constant

⇒ Anallatic lens

A concave lens generally provided in a telescope between the object lens and eye piece, to eliminate the additive constant $(f + d)$ from the tacheometric distance equation is known as an anallatic lens. generally all latest model theodolites are fitted with anallatic lens

⇒ The main advantage of introducing anallatic lens is the calculations of distances and height is made simple

Teacher's Signature.....

Here the additive constant becomes zero. hence the horizontal distance can be obtained by simply multiplying the staff intercept by multiplying constant (f/i)

one disadvantage of using the anallatic lens is that the brightness of the image is much reduced due to absorption of light.

⇒ Determination of constants of a tachometer

The constant may be determined by:

- ① Laboratory method
- ② Field method

① Laboratory method: The focal length 'F' of the objective lens can be determined by conducting laboratory test and using relation $\frac{1}{F} = \frac{1}{u} + \frac{1}{v}$

The stadia intercept 'i' can be measured from the diaphragm with the help of a vernier calliper

The distance 'd' between the optical centre and the vertical axis of the instrument can also be measured

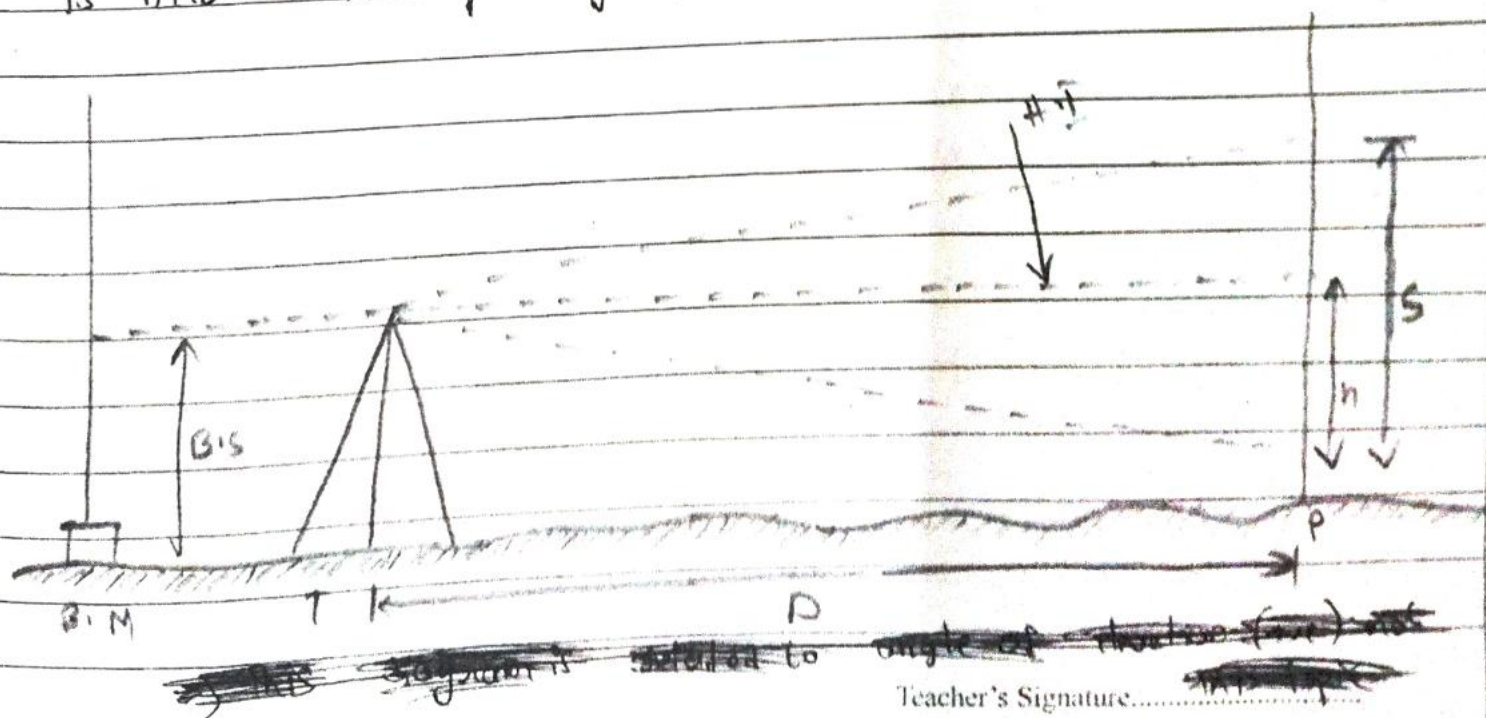
Then the multiplying constant (f/i) and additive constant ($f+d$) can be calculated

② Field method:

- A fairly level ground is selected. set up the instrument at one end say P of a straight line PQ about 150 m long
- Drive pegs at $D_1 = 50\text{m}$, $D_2 = 100\text{m}$ and $D_3 = 150\text{m}$ from P on PQ
- By substituting the value of D_1 , D_2 & D_3 and S_1 , S_2 & S_3 in general equation $D_1 = \left(\frac{f}{r}\right) S_1 + (f+d)$ and $D_2 = \left(\frac{f}{r}\right) S_2 + (f+d)$ and so on
- By solving the equations in pairs, several values of (f/r) and $(f+d)$ are obtained the mean of these values gives the required constants.

⇒ Fixed hair method

Case 1:- when the line of sight is horizontal and staff is held vertically (Fig 3.9)



Teacher's Signature.....

when the line of sight is horizontal, the general tacheometric equation for distance is given by $D = \left(\frac{f}{i}\right)s + (f + d)$
 R.L of line of collimation (H.I) = R.L of B.M + back sight
 R.L of staff station P = H.I - h
 where h = central hair reading

Case 2: Distance and elevation formulae for inclined sights, with staff held vertical

1. considering angle of elevation (positive):

'I' instrument station

'P' is staff station

$s = AC \equiv$ staff intercept

$h =$ central hair reading

$v =$ vertical distance of central hair above line of axis

$D =$ Horizontal distance between I and P

$L =$ inclined distance

$\theta =$ angle of elevation

$\alpha =$ Angle made by upper and lower stadia rays with central ray

$A, B, C =$ staff reading cut by stadia hairs

Let $A'C'$ is drawn perpendicular to I_1P

Now, inclined distance $I_1B = L = \left(\frac{f}{i}\right) \times (A'C') + (f + d)$

Horizontal distance,

$$D = L \cos \theta$$

Teacher's Signature

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

Now $A'C'$ is expressed in terms of $AC(s)$ in triangles ABA' and CBC'

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

$$\angle A'A'B = 90^\circ + \theta \text{ and } \angle B'C'C = 90^\circ - \theta$$

The angle θ is generally very small. if the ratio of TAB and AC is ∞ , the value of θ equal to 17.11° . ignoring θ both angle $\angle A'A'B$ and $\angle B'C'C$ may be taken equal to 90°

$$\therefore A'C' = AC \cos \theta = s \cos \theta$$

substitute in equation (1)

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

$$D = \frac{f}{i} (s \cos^2 \theta) + (f+d) \cos \theta \quad \text{--- (2)}$$

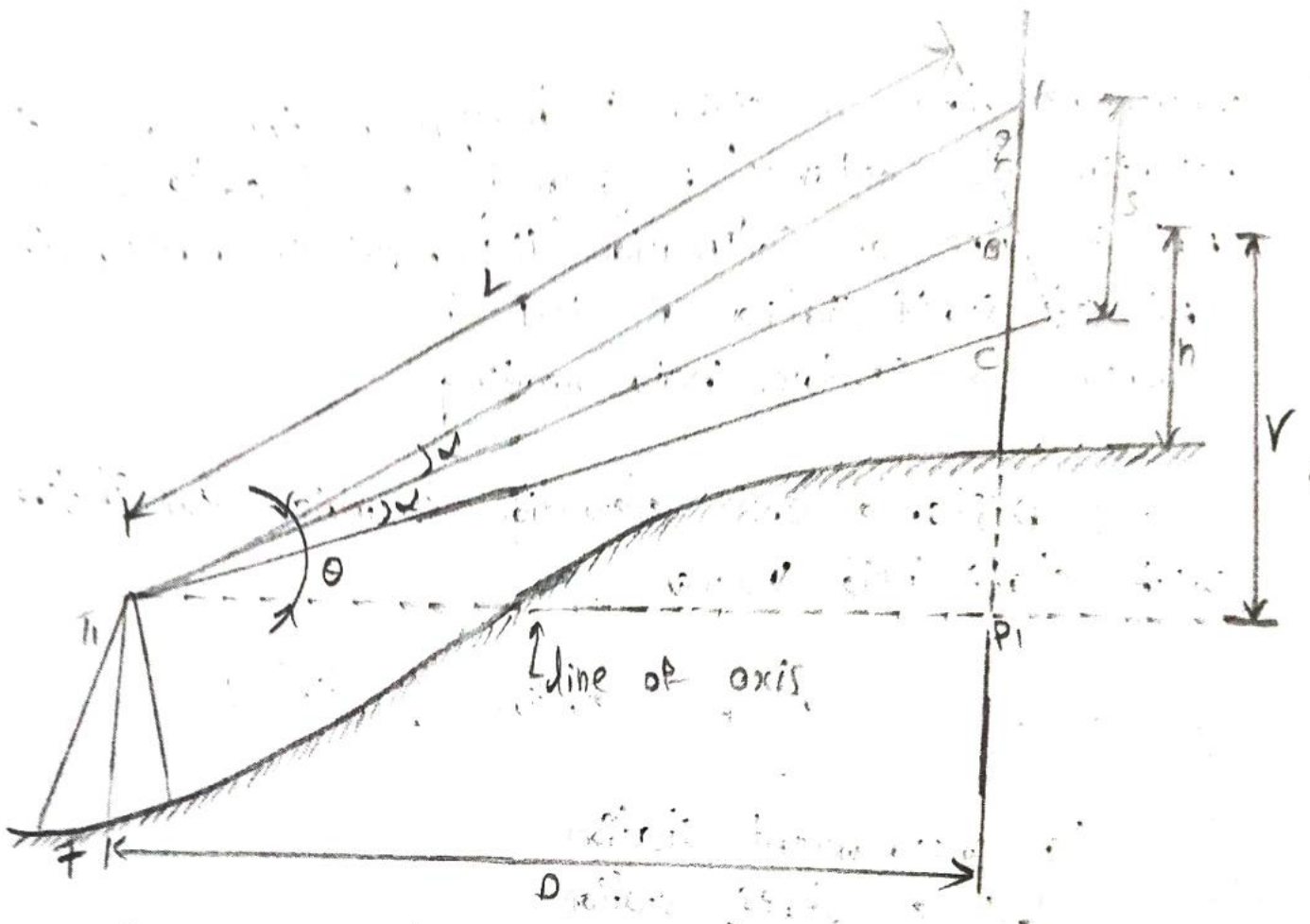
Now vertical distance $v = L \sin \theta$

substitution 'L' value

$$v = \left[\frac{f}{i} s \cos^2 \theta + (f+d) \right] \sin \theta$$

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

$$v = \frac{f}{i} \times s \frac{\sin 2\theta}{2} + (f+d) \sin \theta \quad \text{--- (3)}$$



angle of elevation (positive)

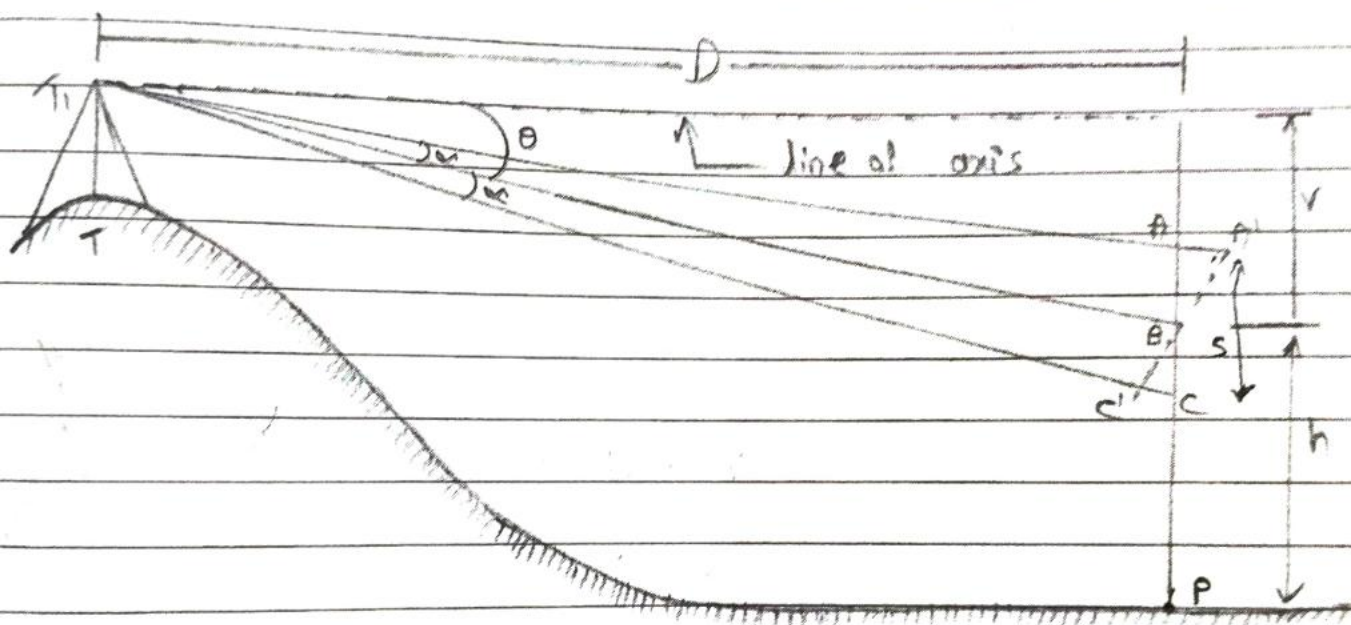
② Considering angle of depression (-ve)

Here also same expressions for D and v can be used

$$D = \frac{f}{i} \times s \cos \theta + (f + d) \cos \theta \quad \text{--- (5)}$$

$$v = D \tan \theta$$

$$R.L \text{ of station } P = R.L \text{ of line of axis} - v \quad \text{--- (6)}$$



⇒ Sources of errors in stadia tachometry

The following are the different sources of error in stadia measurement.

① Instrumental errors:

1. The permanent adjustment of the tachometer may not be perfect.

Teacher's Signature

2. The graduation of the staff (or) stadia rod may not be uniform
3. The value of the multiplying constant may not be correct

(b) observational errors:

- Errors may be caused by incorrect Centering and Levelling
- The staff may not be held vertically
- Imperfect focussing of the telescope
- The sight distance may be much greater than the range of the telescope

(c) Natural errors :-

- High wind, causes incorrect staff observations may be taken due to vibration of instrument
- Temperature in a hot day the different parts of the barometer may expand, affecting staff readings.
- Due to the glare of the sun, the visibility of the target (a) staff may be affected.

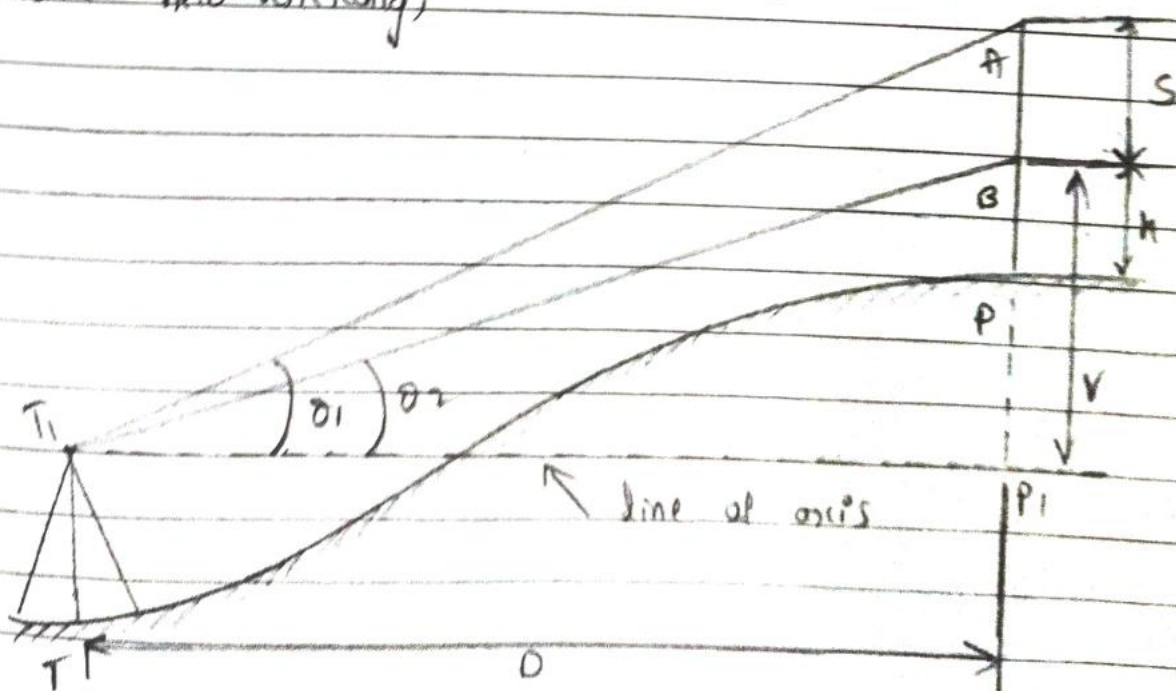
4) Tangential method of theodolite

This method is adopted when the theodolite is a simple transit type and does not carry a stadia diaphragm. In this method, horizontal and vertical distances from the instrument to the staff position are computed from the observed vertical angles to two targets fixed at a known distance 's' on the staff.

Depending upon the vertical angles, three cases may arise

- ① when both angles of target are angles of elevation
- ② when both angles of target are angles of depression
- ③ when one angle is that of elevation and the other that of depression

Case 1:- when both angles of target are angles of elevation (Staff held vertically)



Teacher's Signature

T = Instrument station

T_1 = Instrument axis

P = staff station

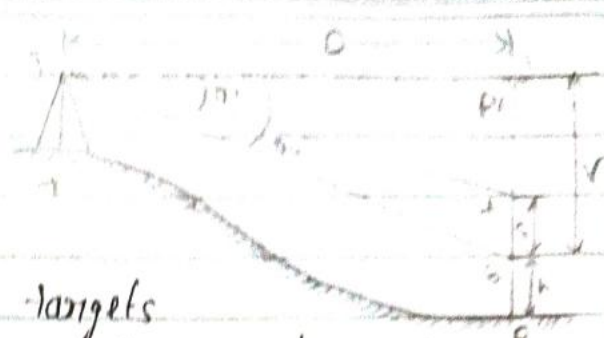
S = distance between targets

v = vertical distance between lower vane and axis of instrument

θ_1 = Vertical angle made by upper target

θ_2 = vertical angle made by lower target

h = height of lower vane above the staff station



$$VTS = D \tan \theta_1$$

$$v = D \tan \theta_2$$

$$S = D (\tan \theta_1 - \tan \theta_2)$$

$$D = \frac{S}{\tan \theta_1 - \tan \theta_2}$$

$$v = D \tan \theta_2 = \frac{S \tan \theta_2}{\tan \theta_1 - \tan \theta_2}$$

$$R.L \text{ of } P = R.L \text{ of instrument axis} + v - h$$

Case-II

when both angles of target are angles of depression
(staff held vertically)

$$v = D \tan \theta_2$$

$$v - s = D \tan \theta_1$$

$$s = D (\tan \theta_1 - \tan \theta_2)$$

Teacher's Signature.....

$$D = \frac{S}{(\tan \theta_2 - \tan \theta_1)}$$

$$V = D \tan \theta_2 = \frac{S \tan \theta_2}{(\tan \theta_2 - \tan \theta_1)}$$

R.L of P = R.L of instrument axis - v-h

Case-III: when one angle is that of elevation and the other that of depression (staff held vertically)

$$V = D \tan \theta_2$$

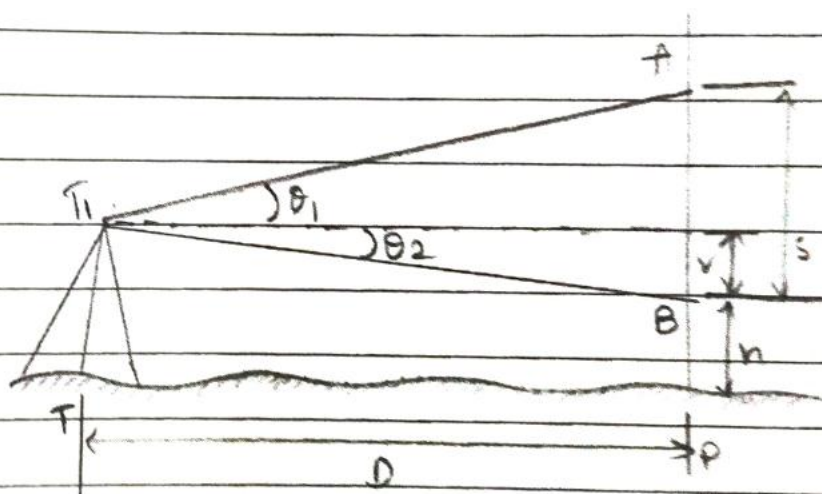
$$S - V = D \tan \theta_1$$

$$\therefore S = D \tan \theta_1 + V$$

$$S = D \tan \theta_1 + D \tan \theta_2$$

$$S = D (\tan \theta_1 + \tan \theta_2)$$

$$D = \frac{S}{\tan \theta_1 + \tan \theta_2}$$



$$V = D \tan \theta_2 = \frac{S \tan \theta_2}{\tan \theta_1 + \tan \theta_2}$$

R.L of P = R.L of instrument axis - v-h

⇒ Disadvantages of tangential method of tacheometry

The tangential method of tacheometry has the following disadvantages:-

- ① It lacks speed
- ② Two vertical angles are to be observed to compute distance
- ③ This method involves more calculations to calculate horizontal distance and reduced level
- ④ During the method of observing vertical angles the instrument might get disturbed unnoticed
- ⑤ Due to above disadvantages this method is considered inferior to the stadia method and is generally not adopted

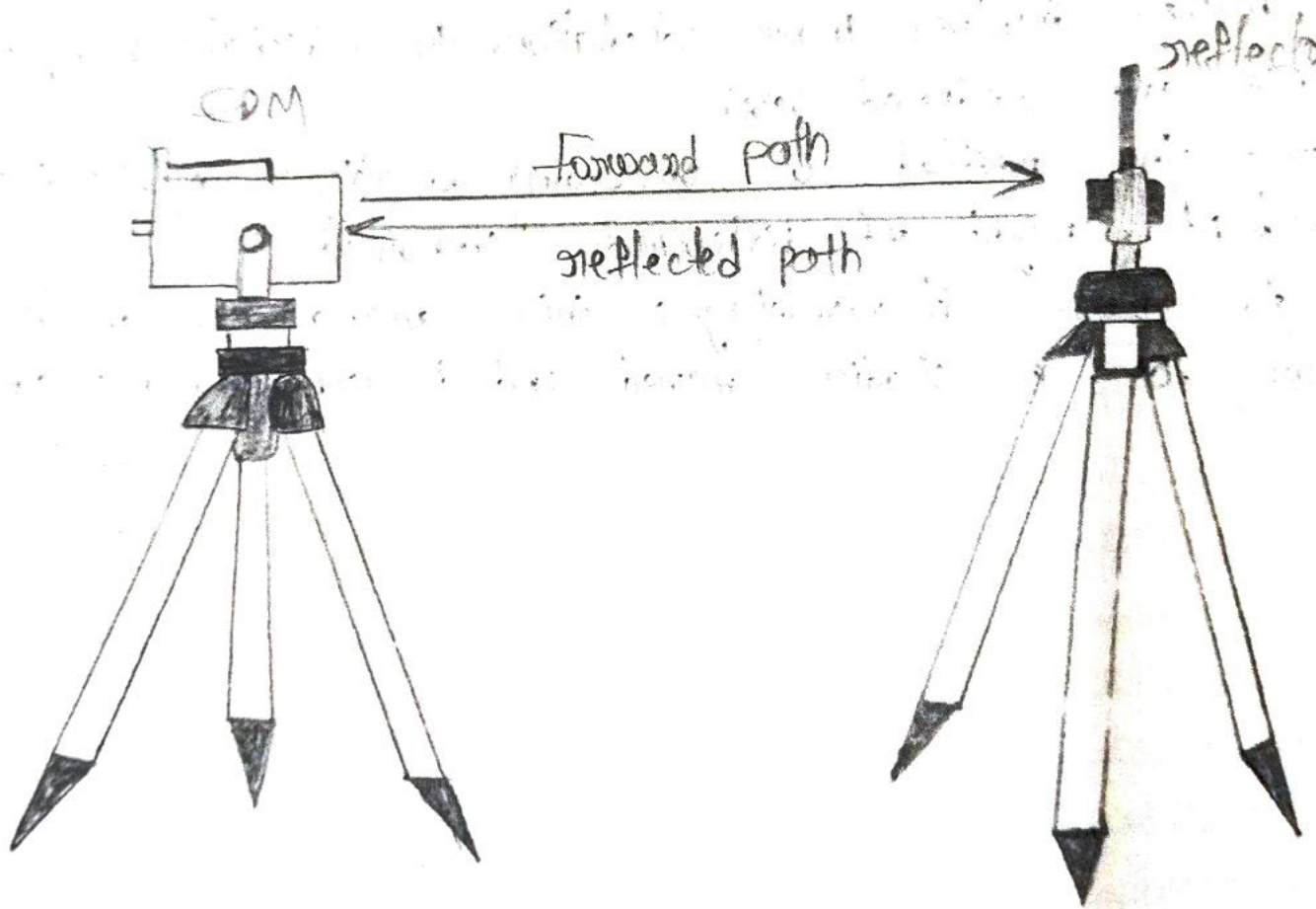
5th unit :- Advanced surveying instruments

Introduction to EDM:

The distance can be measured between two points by the following methods:

1. direct distance measurement (DDM) by using chain or tape
2. optical distance measurement (ODM) by using tachometer, or horizontal subtense method using optical wedge attachments
3. electro magnetic distance measurement (EDM) is done by geodimeter, tellurometer or distomat.

⇒ The method of direct distance measurement is unsuitable in difficult terrain. In the optical distance measurement method the range is limited to 150 metres and accuracy also may be upto 1 in 1000. But electromagnetic distance measurement enables the accuracy upto 1×10^5 over ranges upto 100 km. EDM is a surveying instrument used for measuring distance electrically between two points through electromagnetic waves.



⇒ Basic functions of an F.D.M Instrument :

- The generation of carrier wave and measuring wave frequencies
- The modulation and demodulation of the carrier wave
- The measurement of the phase difference between the transmitted and received frequency waves
- The display in some form, of the result of the measurement.

⇒ Classification of F.D.M Instruments

The F.D.M instruments for surveying may be divided into two categories :

1. Those in which a beam of light is the carrier and which is reflected back from a mirror
 2. Those which transmit short radio waves (microwaves) as carrier.
- Those instruments in which a beam of light is the carrier and which is reflected back from a mirror are expensive because one active instrument and a battery are only needed at one end and the instrument at the other end is simply reflecting mirror centred over the ground station mark. Such instruments are used for shorter distances only. When the distance between the stations is comparatively large a telescope is used for

The beam of the mirror and laser is used as a carrier, because this has a very small angle of spread. Those instruments which transmit short radio waves as carrier and in these instruments, an active instrument and a skilled operator are needed at each of the line. The signal is retransmitted on another carrier wave back to the originating instrument, the wavelengths of the carrier of such instruments varies 8 to 10 mm. As the spread angle of carrier waves is between 5° to 10° these instruments using microwaves are generally used for the long distances required in primary triangulation and precise traversing.

⇒ principle used in EDM instruments :

The principle used in EDM instruments is that the difference in phase between the transmitted and received waves represents only a fraction of the wavelength and that by using several wave lengths of varying frequencies the distance can be measured.

$$\text{Distance (d)} = \text{velocity} \times \text{time} = v \times t$$

⇒ Reduction of EDM lines :- The distance between two points A and B obtained by an EDM instrument be D . If the reduced distance at m.s.l be K . Let the spherical

Distance be S . Let H_1 and H_2 be the respective elevations of A and B and R be the radius of the earth

$$K = R \sqrt{\frac{(D - (H_2 - H_1))(D + (H_2 - H_1))}{(R + H_1)(R + H_2)}} \quad \text{--- (1)}$$

and $S = K + \frac{K^3}{24R^3}$

→ A modified form of equation 2 is used to suit the various instruments

$$S = K + \frac{K^3}{43R^3} \quad (\text{for tellurimeters})$$

$$S = K + \frac{K^3}{38R^3} \quad (\text{for gradimeters})$$

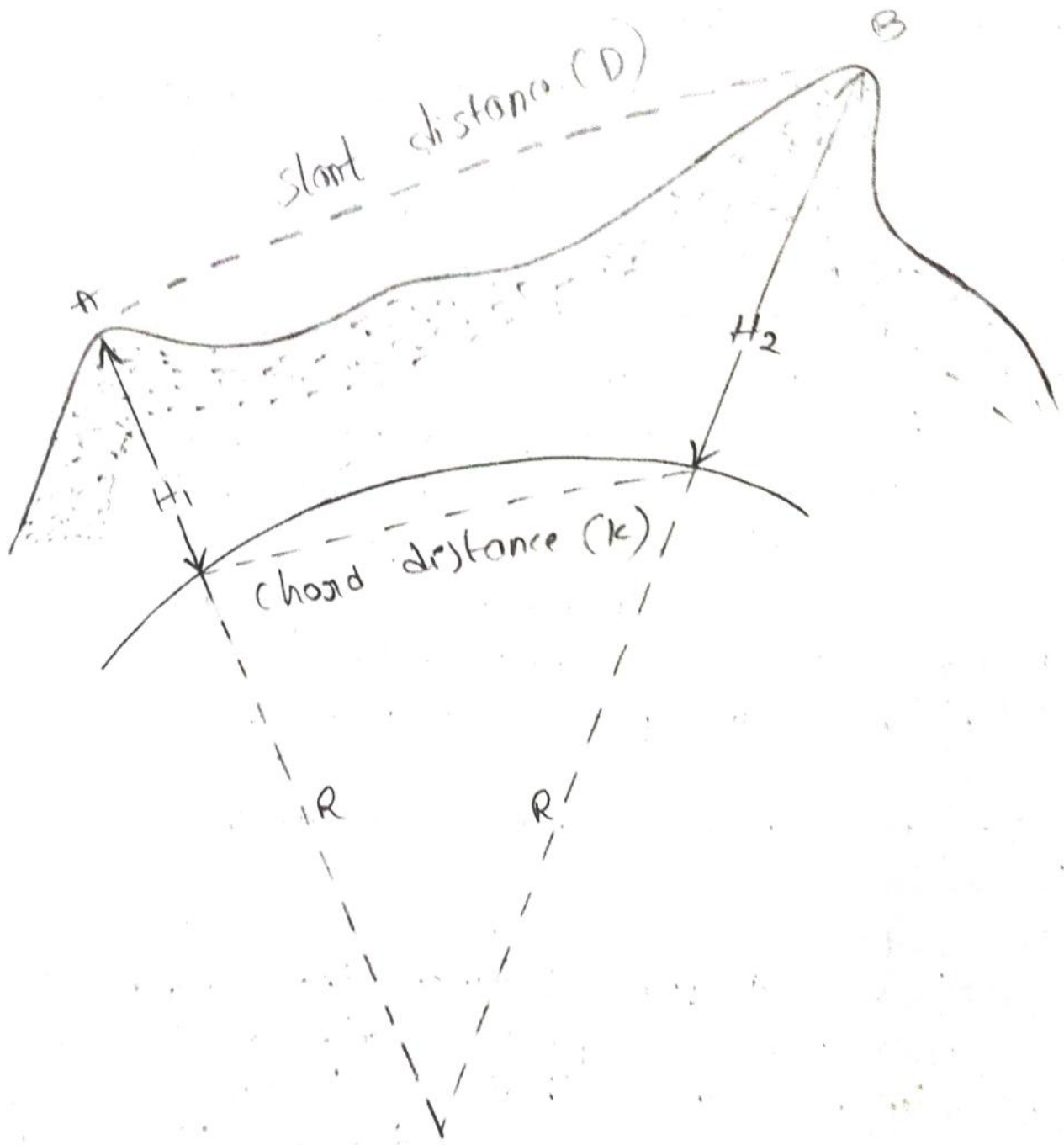
⇒ Distomat

A distomat is the most precise and modern EDM instrument. 'Distomat' is a registered trade name used by Wild for their electronic distance measurement (EDM) instruments. The distomat has an advantage over the other EDM instruments. It can measure distances from 20m to 150km.

⇒ Electronic Theodolite

Electronic theodolites resemble a conventional optical theodolite.

Teacher's Signature.....



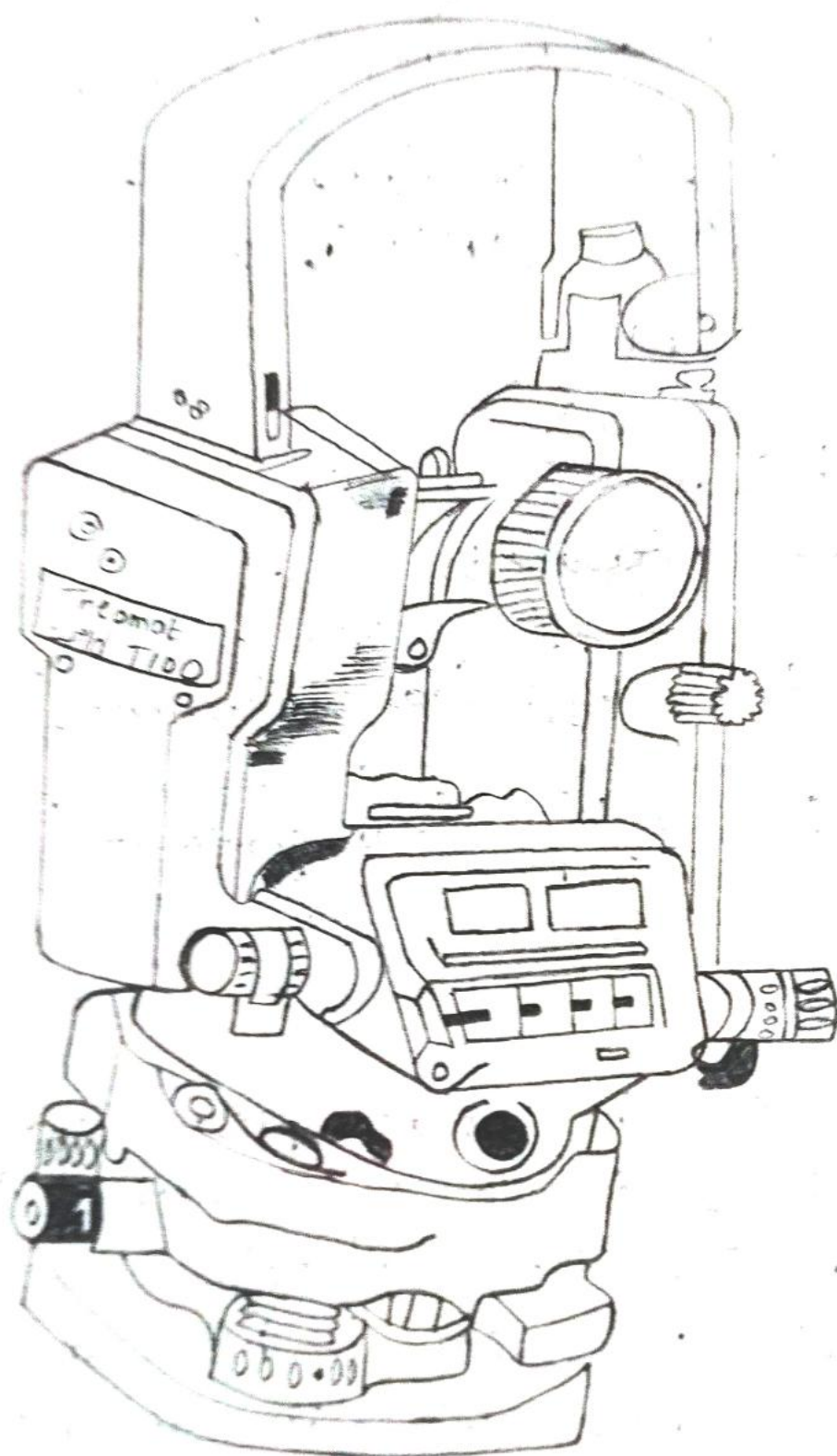
Reduction of electromagnetic distance

In size and weight but with wild 1-1000 electronic speed and efficiency.

This is also called 'Thecomat'. It measures electronically and opens the way to electronic data acquisition and data processing. It has 30 x telescope which gives a bright high contrast, exact image pointing is fast and bright high cont precise even in poor observing conditions.

The Theodolite has two control panels each with keyboard and two liquid crystal displays. It can be used easily and quickly in both positions. The keyboard has just six multifunctional keys. The main operation require only a single key stroke, colour coding and easy to follow key sequences and commands make the instrument remarkably easy to use. The theodolite has an absolute electronic reading system with position coded circle, integrated circuits and microprocessors ensure a high level to performance and operating comfort.

- T-1000 theodolite has electronic clamp for circle setting and repetition measurements using simple commands one can set the horizontal circle to read zero, horizontal, collimation and vertical index errors can be determined and stored permanently.



Displayed heights are corrected for earth curvature and mean refraction. The typical commands can be obtained from the control panel. The power for T-1000 theodolite is obtained from a small, rechargeable 0.45 Ah Ni Cd battery which plugs into the theodolite standards.

USES :- ① It can be used for angle measurements only

② It combines with wild distomat for angle and distance measurement

③ It connects to GRE3 data terminal for automatic data acquisition

④ It is compatible with wild theodolite accessories

⑤ It connects to computer with RS 232 interface

⇒ Global positioning system

This system is satellite based one. This has been initially developed by U.S. Department of Defence (DOD). The G.P.S. technology overcomes the various limitations of a conventional survey, requirement of visibility of one survey station from another, dependence on weather and difficulty in night observation. If we consider the advantages over the conventional techniques such as the high accuracy it achieves in three dimensional positioning and the economy of

Teacher's Signature

Its operation

GPS has three advantages

- (i) GPS may be used to identify or define the geographical co-ordinates associated with satellite imagery.
- (ii) GPS can be used in the ground to get satellite images. When a particular satellite image has a region of unusual or unrecognised reflectivity the coordinates of that region can be loaded into GPS receiver

⇒ Segments of GPS

① space ② control ③ user

① Space segment (satellites used in GPS)

The space segment consists of 24 satellites including three active spares. The satellites are placed in near circular orbits in six orbital planes with an orbital inclination of 55 degree and at 20200 km above the earth. The operational GPS satellite has design life of 4.5 years. The satellites are spaced in orbit so that at any time a minimum of 4 satellites will be in view to users anywhere in the world

② Control segment

The control segment consists of a master control station in Colorado Springs, Colorado with five monitor stations and three control up link stations located throughout the world. Monitor stations track all GPS satellites in view and collect ranging information from the satellite broadcasts. The monitor stations send information they collect from each of the satellites back to the master control station which computes extremely precise satellite orbits. The monitor stations receive all satellite signals from which they determine the pseudo ranges to all visible satellites and transmit the ranged data along with local meteorological data via data link to the master control station.

③ User segment (GPS receivers) :-

Appropriate GPS receivers are required to use GPS signals for navigation and positioning. The user segment consists of the receivers processors and antennas that allow land, sea or airborne operators to receive the GPS satellite broadcasts and compute their precise position, velocity and time.

The GPS concept of operation is based upon satellite ranging users determine their position by measuring

Their distance from the group of satellites in space

⇒) Function of GPS receiver:-

- Measurements collected simultaneously from four satellites are processed to solve for the three dimensions of positions such as latitude longitude and altitude and time position measurements are in the world wide WGS 84 geodetic reference system and time is with respect to a world common U.S. Naval observatory time reference.
- Appropriate GPS receivers are requested to use GPS signals for navigation and positioning
- GPS receivers collect signals from satellites in view they display the users position velocity and time as needed for their marine terrestrial or aeronautical applications
- The GPS standard positioning service is available to civil users world wide for their peaceful transportation scientific and other uses free of direct user charges.

⇒) Components of GPS receiver

- (i) Antennas with preamplifier
- (ii) RF section with signal identification and signal processing
- (iii) microprocessor for receiver control data sampling data processing

Teacher's Signature.....

- (iv) precision oscillator
- (v) power supply
- (vi) user interface, command & display panel

Memory data storage

The antenna detects the electromagnetic waves arriving from the satellites converts the wave energy into electric current current amplifies the signal strength and hands over to the signals over the receiver electronics. GPS signal structure requires that all GPS antennas must be circularly polarized.

- ⇒ Field procedure :- The signals the GPS receivers get
- from the satellites are converted into position, velocity and time estimates.
 - In general for computing the 4 dimensions of any point i.e. x, y, z and time 4 satellites are required. The position in x, y, z is converted within the receiver to geodetic latitude, longitude and height. The velocity is computed from the change in the position of the receiver over time.

⇒ GPS data creation (observation and processing):

The final stage of the process of GPS based data collection for GPS is to transfer the data from

Field device to the target data base.
→ This data transfer is most often accomplished by running a translation program that will convert the data from the compact internal storage format to the database interchange format of the user's choice.

⇒ Map definitions—map projection data entry importance

The map is still the most elegant and compact method of displaying spatial data. The role of the map is to communicate spatial information to the user. This information may include location, size, shape, pattern, distribution and trends in spatial objects.

⇒ Types of maps :-

- These are two general types of maps.
- Those that show the distribution of a specific topic are thematic or statistical maps. A map showing population distribution by a country is a thematic map.
- Although both reference and thematic maps can be constructed from the data contained in a GIS, this document will focus on thematic maps.

⇒ Geographic information system (GIS):-

It is a computer system build to capture, store, manipulate, analyze, manage and display all kinds of spatial or geographical data.

- Most of the time GIS is used to create maps and to print. To perform the basic task in GIS, layers are combined, edited and designed.
- GIS can be used to solve the location based queries such as "what is located here" or where to find particular features.
- GIS helps to find new information.

⇒ Components of GIS

- ① Hardware
- ② software
- ③ data
- ④ people
- ⑤ methods

① Hardware :- It is the computer on which GIS software runs. Nowadays there are a different range of computer it might be desktop or server based.

- a) motherboard
- b) hard drives
- c) processor
- d) graphics card
- e) printer and so on

② Software :- It is one of the component in GIS which provides tools to run and edit spatial information. It helps query, edit, run and display GIS data it uses RDBMS (relational database management system) to store the data.

③ Data :- It is the most important and expensive component of GIS which is generally known as fuel for GIS. The process of creating the GIS data from the analog data on paper format is called digitization.

- Raster data :- It can store information in a cell based manner. It can be aerial photo, satellite image. It stores continuous data.
- Vector data :- It is discrete, it stores information in x, y co-ordinate format.

④ people :- They are the users of GIS. They run the GIS software. hardware and software made people easy to run the GIS software. people are the main component for successful GIS.

⑤ methods :- for successful GIS operation a well-designed plan and business operation rules are important methods. Can vary with different organizations.

Teacher's Signature

⇒ GIS Data types

- ① spatial data :- They are used to provide the visual representation of geographic space and is stored as raster and vector types
- ② Attribute data :- These are descriptions, measurements and classification of geographic features in a map. It is the detailed data used in combination with spatial data to create GIS.

⇒ Applications of G.I.S in civil engineering

- GIS is advanced to apply in different fields of engineering civil engineering departments perform survey, storm drains street lighting rights of way
- GIS computer aided design and drafting and related technologies are commonly used to support the project planning design.
- Community development departments include planning and zoning, economic development, building permits and inspection redevelopment agency and resource management

⇒ Major Applications in civil Engineering

① Environmental planning

- providing effective analysis and modelling tools for an environment impact assessment.

- Air/water quality modelling and monitoring
- ground water movement modelling
- environmental hazard identification & evaluation.

② local and municipal authorities

- land use inventory and planning
- growth monitoring
- planning and zoning
- cadastral and parcel mapping
- Tax records management

③ Transport planning

- fleet monitoring and navigation
- highway design ~~data~~ and customisation
- routing optimisation and demand modelling
- analysis of accident prone areas.

④ public utilities

- modelling communication and distribution network.
eg:- electric power, telephone, gas, water, and sewer lines
- Asset inventory and management
- Recreation and parks management.

⇒ Conclusions

Most of the geographic information systems in our country are regional resource and environment based which are useful for ensuring sustainable development strategy. Geographic information systems are capable of monitoring and analysing environmental issues like industrial pollution.

⇒ photogrammetry :- photogrammetry or photogrammetric surveying is the science and art of obtaining accurate measurements by use of photographs for various purposes. photogrammetry used for acquisition of military intelligence and the preparation of composite pictures of the ground.

① Terrestrial photogrammetry :- Terrestrial photogrammetry is that branch of photogrammetry where in the photographs are taken from a fixed position on or near the ground.

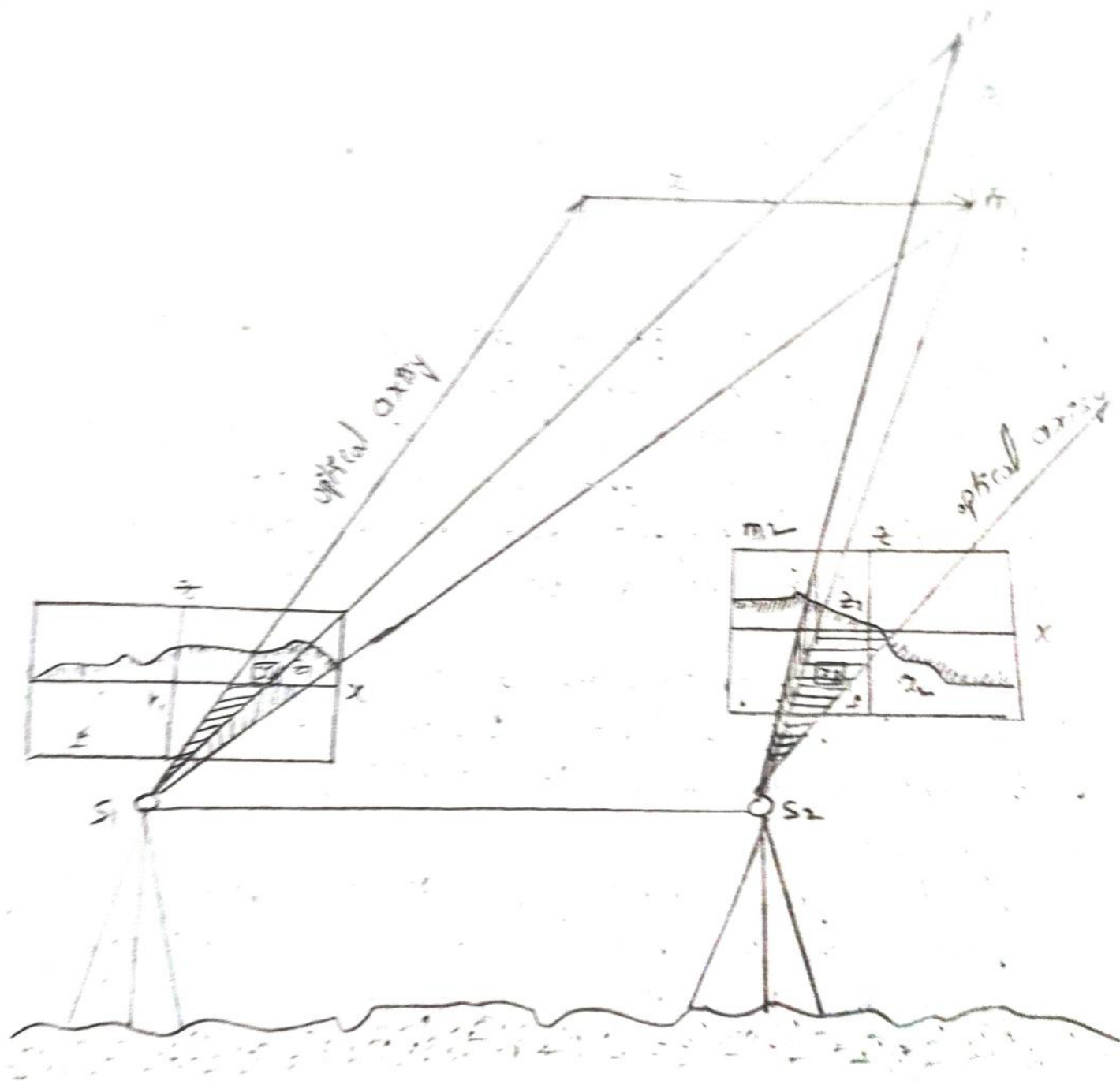
② Aerial photogrammetry :- Aerial photogrammetry is that branch of photogrammetry where the photographs are taken by a camera mounted on aircraft flying over the area.

⇒ Terrestrial photogrammetry and its principle

- In this photogrammetry photos are taken with the camera supported on the ground. In this method photographs are taken by photo theodolite which is a combination of camera and theodolite. Maps are then compiled from the photographs.
- Terrestrial photography is performed with a phototheodolite which is a combination of a theodolite and a precise camera.

principle :- The fiducial marks in the shape of teeth or small round holes are provided in the plate carrier. The focal length which is about 20cm and fiducial marks are printed on the plate or film as well as on photograph. The fiducial marks are made in such a way that the point of intersection of co-ordinate axes on the photograph should coincide with the foot of a perpendicular dropped from the centre of the objective onto the plane of plate carrier. This foot of perpendicular on the photograph gives its principal point.

The origin of the co-ordinates on the photograph should coincide with the principal point of the photograph. The line passing through the optical centre of the objective and the principal point represents the optical axis of the camera.



⇒ Total station

- A total station is an electronic instrument used in modern surveying it is an electronic theodolite integrated with an electronic distance meter (EDM), plus internal data storage and external data collector.
- Data can be downloaded from the total station to a computer and application software used to compute result and generate a map of the surveyed area

Applications of total station

- ① detail survey i.e data collection
- ② control survey (transverse)
- ③ height measurement
- ④ resection
- ⑤ area calculation
- ⑥ missing line measurement

components of total station :-

- 1) electronic of four main components
- 2) electronic theodolite
- 3) microprocessor
- 4) electronic display

⇒ Electronic distance measurement (EDM) :- The function of EDM is to measure distance from the total station to a Reflector. it uses electromagnetic (EM) energy to determine the length of a line.

⇒ Electronic Theodolite :- The function of theodolite is to measure horizontal and vertical angles. a standalone theodolite works by combining the optical plummet. The calibrated circles one vertical and one horizontal enable the user to read out angles in both axes.

⇒ Microprocessor

It saves a lot of work to surveyor. it performs the following functions

- Average multiple angle & distance measurements
- Compute horizontal and vertical distances
- Compute x, y, & z co-ordinates

⇒ Electronic display :- It provides a visual of what is happening in the total station. it also enables input of instrument and reflector heights as well as access to stored data, difference in elevations of two observed points and all the three coordinates of the observed points (x, y, z)

⇒ Accessories for total station

- 1) Tripods
- 2) prisms
- 3) batteries and charger
- 4) prism poles
- 5) Tribraches
- 6) data and power cables

⇒ Features of total station :-

- 1) It provides total solution for surveying work.
- 2) most accurate and user friendly
- 3) gives position of a point (xyz) w.r.t known point
- 4) digital display
- 5) EDN is fitted inside the telescope.

⇒ Characteristics of total station

Electronic display : It is capable of displaying various values when respective keys are pressed. The system is capable of displaying horizontal and vertical distance, horizontal and vertical angles.

⇒ operation of total station :- ① Since the total station contains delicate electronic components they are not as rugged as ordinary theodolite. They must be packed

And transported carefully. handled gently and carefully removed from their cases.

- 2) The setting of total station over the station mark is similar to an ordinary theodolite and it includes.
- centering
 - levelling
 - removal of parallax

3) Distance measurement :- EDM instrument is a major part of total station. its range varies from 2.5 km to 4.2 km. The accuracy of measurement varies from 5mm to 10mm per km measurement. The distance measured is always sloping distance from instrument to the object.

4) Angle measurement :- The electronic theodolite part of total station is used for measuring vertical and horizontal angle. for vertical angle measurement vertical upward direction is taken as reference direction. The accuracy of angle measurement varies from 2 to 6 seconds.