

(2)

6th Oct,
MONDAY

SURVEYING (4 marks)

1. Basic Concepts - Map Projections
2. Linear Measurements - Chain Surveying
3. Angular Measurements - Compass Surveying
4. Plane Table Survey
5. Levelling
6. Contouring
7. Areas & Volumes
8. Minor Instruments
9. Theodolite
10. Theodolite Traversing
11. Omitted measurements
12. Trigonometrical Levelling
13. Tacheometric Surveying
14. Curves

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1. BASIC CONCEPTS

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→ Objective of Surveying

- (i) To take measurements for determining the locations of existing ground features.
- (ii) To mark the positions of objects w.r.t. assumed datum.
- (iii) To calculate the related quantities like areas & volumes

→ Primary Divisions of Surveying:

(i) Plane Surveying

- neglect the curvature of earth.
- distances less than 18.5 km and areas less than 250 km².
- less accurate.

(ii) Geodetic Surveying.

- consider the curvature of Earth.
- large areas and more accurate.
- fixing the control points and boundary points of a field.

→ Classification of Surveys

* Based on Function:-

1. Control Survey

2. Land Survey

3. City Survey

4. Engineering Survey

5. Topographic Survey

6. Geological Survey

7. Archaeological Survey

8. Astronomical Survey

9. Hydrographic Survey

10. Gravity Survey.

11. Mining Survey

12. Military Survey

13. Satellite Survey

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Topographic Survey:- It is carried out to delineate features such as hills, rivers, forests and man made features like villages, buildings, transmission lines and roads.

Hydrographic Survey:- Related with water bodies like low water level, high flood level etc.

Gravity Survey:- Fluctuation of gravity value from place to place.

* Based on Instrument:

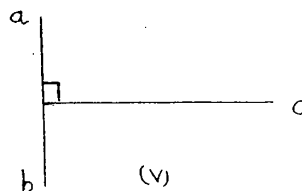
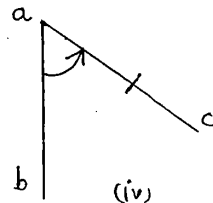
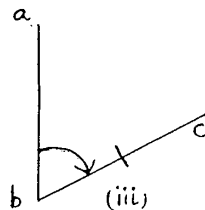
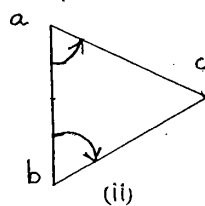
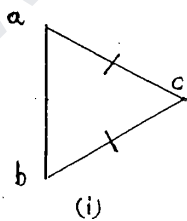
1. Chain Surveying
2. Compass Surveying
3. Plane table Surveying.
4. Levelling Surveying.
5. Theodolite Surveying
6. Photogrammetry.
7. EDM's. - Electronic Distance Measuring equipment.

- **Triangulation:-**

It is the process of measuring the sides of a triangle with the help of EDMs; esp

→ **Principles of Surveying**

1. To locate the point with two known points.



2. Working from whole to the part but not from part to a whole.

Accumulation of errors can be reduced.

→ Basic Measurements in Survey

1 mm $\Rightarrow$ 1000 mm.

(i) Horizontal Distance.

Done by chain, tape, tacheometer, total station.

(ii) Vertical Distance.

Level, total station, tacheometer & sextant, Abney level (minor instrument)

(iii) Horizontal Angle

Compass, theodolite, clinometer (minor instrument), total station.

(iv) Vertical Angles.

Theodolite, sextant and total station.

→ Scale of a Map

It is the ratio b/w distance on the map to the distance on the ground.

$$\text{Scale} = \frac{\text{distance on the map}}{\text{distance on the ground.}}$$

1 : 1000 $\Rightarrow$ 1 unit on map = 1000 units on ground.

(i) Large Scale 1 cm = 10 m

(ii) Medium Scale. 1 cm = 100 m

(iii) Small Scale. 1 cm $\geq$ 100 m

(iv) Engineer's Scale 1 cm = 50 m

→ Error due to Shrinkage of a Map:

$$\text{Shrinkage factor, } SF = \frac{\text{distance on the map.}}{\text{corresponding dist. on ground}}$$

$$= \frac{\text{distance during measurement}}{\text{corresponding actual distance.}}$$

$$SF < 1$$

$$\text{Shrunk scale} = \text{Original scale} \times SF$$

$$\text{Shrunk RF} = \text{Original RF} \times SF$$

$$\text{Corrected distance, } CD = \frac{MD}{SF}$$

$$\text{Corrected area, } CA = \frac{MA}{(SF)^2}$$

$$\text{Corrected volume, } CV = \frac{MV}{(SF)^3}$$

→ Error due to Wrong Scale:

$$CD = MD \left(\frac{\text{RF of WS}}{\text{RF of CS}} \right)$$

$$CA = MA \left(\frac{\text{RF of WS}}{\text{RF of CS}} \right)^2$$

$$CV = MV \left(\frac{\text{R.F of WS}}{\text{RF of CS.}} \right)^3$$

→ Accuracy:

Degree of accuracy :- It is the ratio between units of error to the units of measured quantity.

Degree of accuracy = 1 in n .
 ie 1 unit of error in n units of measured value.

→ Precision.

It is closeness to the some other measured quantity.

→ Sources of Errors

- (i) Instrumental errors :- when instrument is not calibrated at regular intervals. by permanent adjustments.
- (ii) Personal errors :-
- (iii) Natural errors.

→ Types of Errors

- (i) Mistakes.
- (ii) Systematic errors
- (iii) Accidental or Random error.

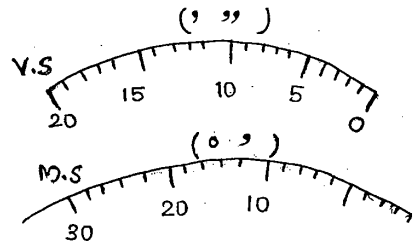
Random errors are directly proportional to square root of ' n '; where n is total no. of observations

$$\text{Random error} \propto \sqrt{n}$$

→ Vernier Scales

$$\text{Least Count, LC} = \frac{S}{n}$$

$$= \frac{1 \text{ MSD}}{\text{No. of VSD's}}$$



* Types of Verniers

(i) Direct Vernier.

' n ' divisions of vernier = $(n-1)$ divisions of MS

(ii) ~~Extended~~ Retrograde Vernier

' n ' divisions of vernier = $(n+1)$ divisions of MS

(iii) ^{extended.} Retrograde Vernier

'n' division of vernier = $(2n-1)$ div. of MS

Q.9. Scale = 1 : 1000

1 mm $\Rightarrow$ 1000 mm. (i) To take measurements for distance

0.25 mm $\Rightarrow$?

$$= \frac{0.25}{1} \times 1000$$

$$= 250 \text{ mm} = \underline{\underline{0.25 \text{ m}}}$$

Q.10. Representative fraction, RF = $\frac{0.5 \text{ cm}}{10 \text{ m}} = \frac{0.5}{10 \times 100} = \underline{\underline{\frac{1}{2000}}}$

$$\begin{aligned} \text{Q.11} \quad CD &= MD \left(\frac{\text{R.F of WS}}{\text{RF of CS}} \right) = 468 \left(\frac{\frac{1}{2000}}{\frac{1}{4000}} \right) \\ &= \underline{\underline{936 \text{ m}}} \end{aligned}$$

Q.12. 1 MSD = S = 30'

No. of VSD, n = 60

$$LS = \frac{30'}{60} = \frac{(30 \times 60)''}{60} = \underline{\underline{30''}}$$

Q.13. 1 MSD = S = $\frac{1}{6} \times 60' = 10'$

$$n = 20$$

$$LC = \frac{(10 \times 60)''}{20} = \underline{\underline{30''}}$$

14 $S = 1' = 60''$

For extended vernier,

$$2n - 1 = 11$$

$$n = \underline{\underline{6}}$$

$$\therefore LC = \frac{60'}{6} = \underline{\underline{10'}}$$

15. n div. of 'V' = $(n+1)$ div. of 'S'

$$10 V = 11 S$$

16 $RF = 1/2500$

$$1 \text{ cm} = 2500 \text{ cm}$$

$$\Rightarrow \underline{\underline{1 \text{ cm} = 25 \text{ m}}}$$

18. $SF = \frac{90}{100} = 0.9$

$$SRF = \text{Original RF} \times SF$$

$$= \frac{1}{1000} \times 0.9 = 9 \times 10^{-4}$$

$$= \frac{1}{1111}$$

$$\Rightarrow \underline{\underline{1 : 1111}}$$

24. $SRF = \frac{1}{2500} \times \frac{24}{25} = \frac{1}{2604.16}$

$$\Rightarrow \underline{\underline{1 : 2600}}$$

25 $SF = \frac{9}{10} = 0.9$

$$CA = \frac{MA}{SF^2} = \frac{81}{(0.9)^2} = 100 \text{ cm}^2 \text{ (on the plan)}$$

$$CA \text{ on the field} = 100 \times 10 \times 10 = 10000 \text{ m}^2 \quad \left(\text{SCALE } 1 : 10 \frac{\text{m}}{\text{cm}} \right)$$

7th Oct,
TUESDAY

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2. LINEAR MEASUREMENTS

1. Direct Method : Chain or tape.

2. Optical Method : EDM Measurements in Survey

EDMs are classified as:

(i) Light Waves - Geodimeter, Mekameter & Range Finder

(ii) Microwaves - Distomat, Decca navigator, Lambda Omega, Tellurometer etc.

3. Approximate Methods:

a) Pacing : 75 cm to 95 cm

b) Pedometer : gives the no. of foot steps covered.

c) Pedometer

d) Perambulator / Odometer

e) Speedometer

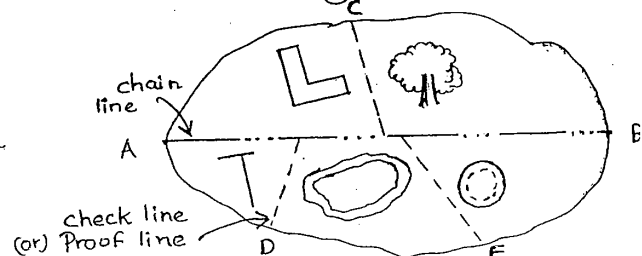
→ Chain Survey

→ Principle

(i) Triangulation.

Baseline: Longest line laid approximately through middle of field. It is a chain connecting main survey stations.

Offsets: lateral distances measured from chain line (base line) to objects.



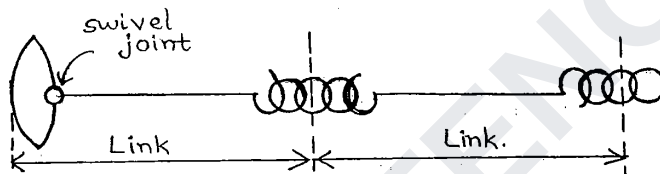
Offsets are of two types:

- (i) $\perp^r$ offset.
- (ii) Oblique offset.

→ Instruments for Chain Surveying

- | | |
|--------------------|---------------------------------|
| (i) Chains | (vi) Cross Staff |
| (ii) Tape | (vii) Plumb bob |
| (iii) Ranging rods | (viii) Wooden Peg |
| (iv) Arrows | (ix) Plasterers, laths & whites |
| (v) Offset Rod. | |

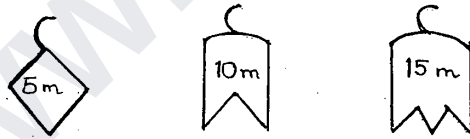
→ Chain (metric)



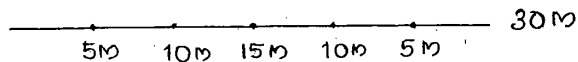
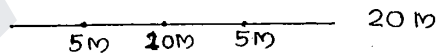
In metric chain, link = 0.2 m
= 20 cm

- Brass rings are provided every 1 m

- Tallies are provided for every 5 m.



Tallies	5m	10m	15m
20m	2	1	0
30m	2	2	1



- Standard temp : 20°C
Allowable pull : 8 kg

- 20 m $\Rightarrow \pm 5\text{mm}$
30 m $\Rightarrow \pm 8\text{mm}$

→ Types of Chains

- (i) Metric Chain : 20 m, 30 m
- (ii) Gunter's chain : 66 ft, 100 links (Surveyor's chain)
- (iii) Revenue Chain : 33 ft, 16 links
- (iv) Engineer's chain : 100 ft, 100 links.

→ Sources of Errors

→ Tapes :

- Least count : 1 mm.

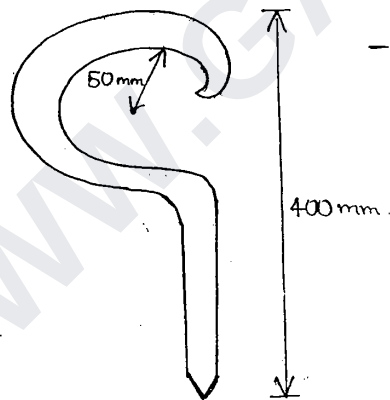
- Types of tapes :-

- (i) Cloth or linen tape :- 10 m, 15 m, 20 m, 30 m
- (ii) Metallic tape : Survey works, construction works
- (iii) Steel tape : 2 m, 5 m, 10 m, 15 m, 20 m, 30 m, 50 m
- (iv) Invar tape : steel (64%) & nickel (36%)

Invar tape is used for baseline measurements.

$$\alpha = 1.2 \times 10^{-6} / ^\circ\text{C} \Rightarrow \alpha = \frac{1}{10} \alpha_s$$

→ Arrows :



- At the end of every chain length, an arrow is fixed.

→ Ranging Rods:

- purpose of ranging rods is to range a line.
- they are available at 2 m & 3 m length.

→ Offset Rod:

- maximum length is 5 m.

→ Cross Staff:

- (i) Open Type - 90°
- (ii) French type - 45° & 90°
- (iii) Adjustable - @ 15° interval

→ Plumb bob:

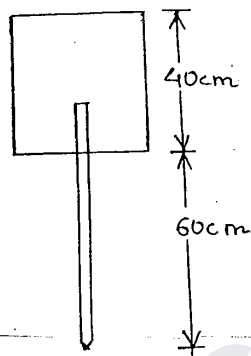
$$\therefore AA = (100 \text{ sec } 67' - 100) \text{ links} = 100$$

To check the verticality of ranging rods, cross staff etc.

→ Wooden Pegs:

To mark the terminal stations

→ Plasterers Lath's & whites



— used to mark the intermediate station in an open level ground online with a base line.

→ Ranging out Survey Lines:

— Ranging is required when the length of a line to be measured is greater than the chain length

— methods of ranging:

(i) Direct Ranging: It is possible when two stations are intervisible,

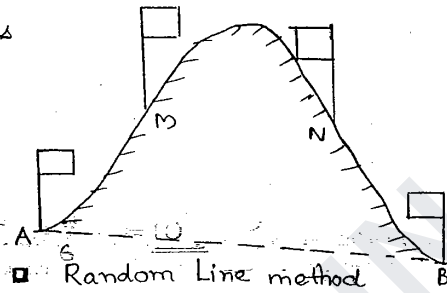
a) By eye judgement

b) By using Line Ranger

(ii) Indirect Ranging / Reciprocal Ranging:

— when stations are not intervisible.

Random Line method is used to establish intermediate stations by reciprocal ranging.



→ Error due to Incorrect Chain

- When chain is too long, measured distance is less, error is -ve correction is +ve.
- If chain is too short, measured distance is more, error is +ve correction is -ve.

$$\left. \begin{array}{l} \text{Correction to length} \\ \text{Corrected length} \end{array} \right\}, l = l' \left(\frac{L'}{L} \right)$$

where $l' \rightarrow$ measured length

$L' \rightarrow$ incorrect length of a chain

$L \rightarrow$ true/designated length of a chain.

$$L' = L + \Delta L$$

$$\frac{L'}{L} = \frac{L + \Delta L}{L} = 1 + \frac{\Delta L}{L}$$

$$\Rightarrow 1 + \frac{\Delta L}{L} = e ; e \rightarrow \text{error}$$

$$l = l' (1 + e)$$

$$\text{Corrected area, } A = A' \left(\frac{L'}{L} \right)^2$$

$$A = A' (1 + 2e)$$

$$\text{Corrected volume, } V = V' \left(\frac{L'}{L} \right)^3$$

$$V = V' (1 + 3e)$$

Case (i): zero at begin
& 'e' at end.

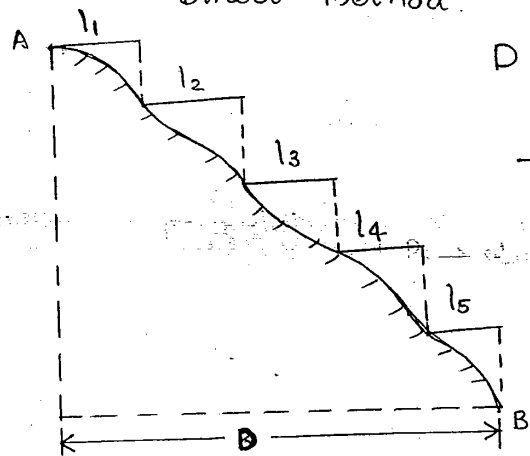
$$e_{\text{avg}} = \frac{0 + e}{2} = \frac{e}{2}$$

Case (ii): e_1 at beginning
& e_2 at end.

$$e_{\text{avg}} = \frac{e_1 + e_2}{2}$$

→ Chaining on Uneven / Sloping Ground:

(i) Direct Method.



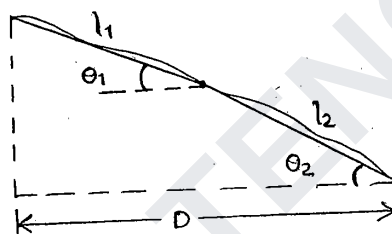
$$D = \sum_{i=1}^n l_i$$

- measuring down the hill is easier than up the hill.

- standard pull. $E \rightarrow$ Young's modulus

(ii) Indirect Method.

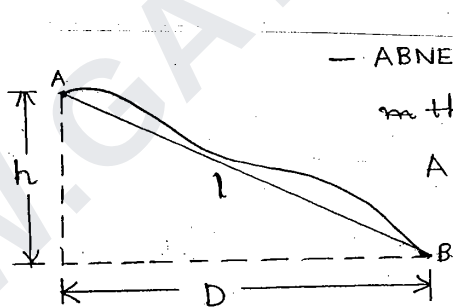
a)



- Clinometer is used to measure the angles.

$$D = \sum_{i=1}^n l_i \cos \theta_i$$

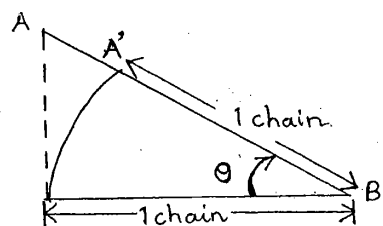
b)



- ABNEY ^{level} is used to measure the level difference b/w A & B.

$$D = \sqrt{l^2 - h^2}$$

c) Hypotenusal Allowance



It is the correction to be applied in the field at every chain length and every point, where the slope changes

This facilitates in locating & surveying the intermediate points.

(9) (8)

Hypotenusal Allowance, $AA' = AB - BA'$

$$\cos \theta = \frac{BC}{AB} = \frac{100 \text{ links}}{AB}$$

$$\therefore AB = 100 \sec \theta \text{ link}$$

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

AA' is the correction for 1 chain ($= 20 \text{ m} = 100 \text{ links}$).
This can be extended to any length.

$$\Rightarrow AA' = 50 \cdot 100 \left(1 + \frac{\theta^2}{2} + \dots - 1 \right)$$

$$\therefore AA' = 50 \theta^2 \text{ links. } (\theta \text{ radians})$$
$$= 0.015 \theta^2 \text{ links } (\theta \text{ degrees})$$

• When slope is given as 1 in n ($\theta \approx \tan \theta \approx \frac{1}{n}$)

$$AA' = \frac{50}{n^2} \text{ links.} = \frac{50}{n^2} \times 0.02 \text{ m.}$$

NOTE:

$$\text{For } 30 \text{ m chain, } AA' = 150 (\sec \theta - 1) \text{ links.}$$
$$= 75 \theta^2 \text{ links; } (\theta \text{ radians})$$

→ Errors in Chaining:

(i) Cumulative Errors

Cumulative error is the one which occurs in the same direction and get accumulate.

(ii) Compensating Errors

Compensating error may occur in either direction and tends to compensate.

1. Erraneous Length of a Chain/tape: cumulative ' $\pm$ '
2. Bad Ranging cumulative ' $\pm$ '
3. Careless holding & marking compensating ' $\pm$ '

4. Bad straightening.
Non horizontality &
Sag in Chains. } Cumulative '+'

5. Variation in temp : cumulative '+'

6. Variation in pull : cumulative '+'

1st Nov,

SATURDAY

TAPE CORRECTIONS

1. Correction for Standardisation

$$C_a = \frac{L \cdot c}{l}$$

$L \rightarrow$ measured length of a line.

$C \rightarrow$ correction for tape length.

$l \rightarrow$ designated length of a tape.

$C_a \rightarrow +ve$ if tape or chain is too long

$C_a \rightarrow -ve$ if tape or chain is too short.

2. Correction for Slope

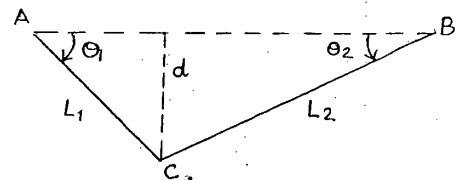
$$C_{SL} = L - \sqrt{L^2 - h^2} = \frac{h^2}{2L}$$

$$C_{SL} = L - L \cos \theta = L(1 - \cos \theta) = 2L \sin^2\left(\frac{\theta}{2}\right)$$

C_{SL} is always -ve.

3. Correction for misalignment.

$$C_{MA} = \frac{d^2}{2L_1} + \frac{d^2}{2L_2}$$



$$C_{MA} = L_1(1 - \cos \theta_1) + L_2(1 - \cos \theta_2) \quad \left\{ C_{MA} \text{ is always } -ve \right\}$$

4. Correction for temperature

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

where $T_m \rightarrow$ temperature during measurement.
 $T_o \rightarrow$ standard temperature.

C_t is +ve ($T_m > T_o$) & C_t is -ve ($T_m < T_o$)

5. Correction for Pull.

$$C_p = \frac{(P - P_o) \times L}{AE}$$

C_p is +ve ($P > P_o$)

C_p is -ve ($P < P_o$)

$P_o \rightarrow$ standard pull.

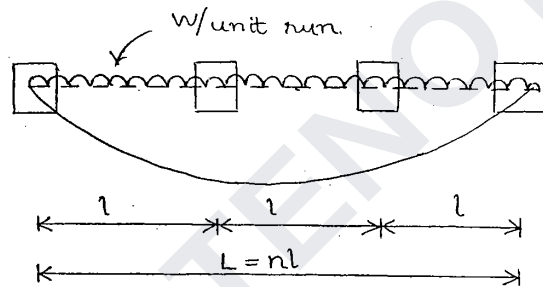
$E \rightarrow$ Young's modulus of tape.

($= 2 \times 10^5$ if not given)
 MPa

$A \rightarrow$ c/s area of tape

$P \rightarrow$ pull applied during measurement.

6. Correction for Sag.



(i) If both the supports are at same level.

$$C_{\text{sag}} = \frac{(wl)^2 l}{24 p^2} \quad ; \text{ for length 'l'}$$

$$= \left(\frac{(wl)^2 l}{24 p^2} \right) \times n \quad ; \text{ for 'n' no. of bays.}$$

$$= \frac{w^2 \left(\frac{L}{n} \right)^2 L}{24 p^2} = \frac{(wL)^2 L}{24 n^2 p^2} \quad ; \text{ for length 'L'}$$

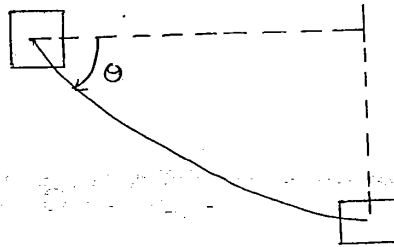
$$= \frac{W^2 L}{24 n^2 p^2}$$

$$W = wL$$

= weight of tape

Correction for sag is always -ve

(ii) If ends are at different level.



$$C'_{sag} = C_{sag} \times \cos^2 \theta.$$

7. Correction for normal tension.

$$P_n = \frac{0.204 w \sqrt{AE}}{\sqrt{P_n - P_0}}$$

where $P_n \rightarrow$ normal tension.

$P_0 \rightarrow$ standard pull.

$w \rightarrow$ total weight of tape

$A \rightarrow$ c/s area of tape

$E \rightarrow$ young's modulus of material of tape.

8. Correction for Mean Sea Level.

$$C_{MSL} = \frac{Lh}{R}.$$

where $L \rightarrow$ length of a tape.

$h \rightarrow$ height of object above or below MSL

$R \rightarrow$ radius of curvature of earth. ($= 6370$ km).

$C_{MSL} \rightarrow +ve$; if object lies above MSL

$C_{MSL} \rightarrow -ve$; if object lies below MSL

→ Limiting Length of Offset

The min. length of an offset in plotting is 0.25 mm

The length of offset on ground depends on scale value that we are using.

Eg: 1) Scale 1: 100.

Length of offset on ground = 25 mm

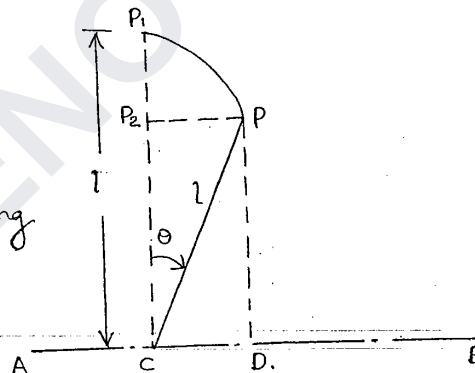
2) Scale 1: 500

Length of offset on ground = 125 mm.

Let the scale of a map be

1 cm = 'S' m.

Degree of accuracy in limiting
the length of an offset
= 1 in r



- Displacement of point P
parallel to the chain line = PP_2

$$PP_2 = l \sin \theta$$

$$= \frac{l \sin \theta}{S} \text{ cm}$$

$$\frac{l \sin \theta}{S} = 0.025$$

ie limiting length of an offset, $l = 0.025 S \operatorname{cosec} \theta$.

$$l = 0.025 \times S \times \operatorname{cosec} \theta$$

- Displacement of point P perpendicular to chain line
= P_1P_2

$$P_1 P_2 = l - l \cos \theta$$

$$= \frac{l(1 - \cos \theta)}{S}$$

$$\therefore P_1 P_2 = \frac{l(1 - \cos \theta)}{S}$$

Degree of accuracy can be calculated from,

$$r = \operatorname{cosec} \theta.$$

Limiting length of an offset by considering both linear and angular displacements,

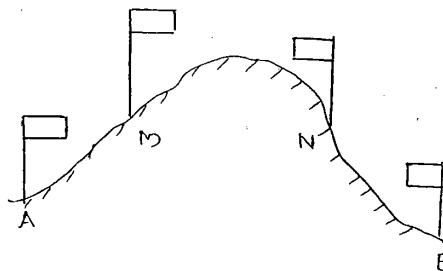
$$l = \frac{S \cdot r}{40 \sqrt{2}}$$

→ Instruments for setting Perpendicular Offsets.

1. Cross-staff : 90° only.
2. Optical square : 90° only.
3. Prism square : 45° & 90° only
4. Side square : 90° only.

→ Obstacles in Chain Surveying

1. Obstacle to Ranging but not chaining

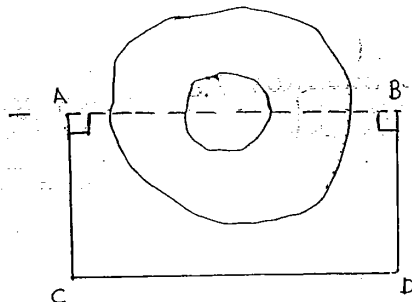


Eg: Hill.

2. Obstacle to Chaining but not Ranging

Eg: Pond, river.

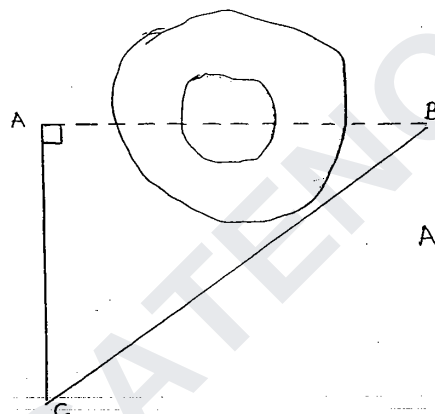
(i)



$$BD = AC$$

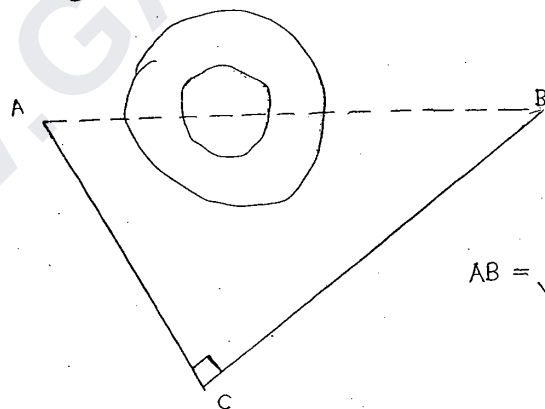
$$\therefore AB = CD.$$

(ii)



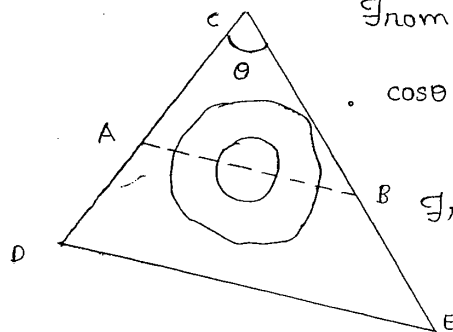
$$AB = \sqrt{BC^2 - AC^2}$$

(iii)



$$AB = \sqrt{AC^2 + BC^2}$$

(iv)



From triangle DCE,

$$\cos \theta = \frac{CD^2 + CE^2 - DE^2}{2 \times CD \times CE} \rightarrow (1)$$

From triangle ACB,

$$\cos \theta = \frac{CA^2 + CB^2 - AB^2}{2 \times CA \times CB} \rightarrow (2)$$

Complete Class Note Solutions
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Abids, Hyd.
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Equating ① & ②, find AB.

→ Cross - staff Survey

It is done to locate the boundaries of field and also to calculate the area.

$$A_1 = \frac{1}{2} \times 35.2 \times 3.2 = 56.32 \text{ m}^2$$

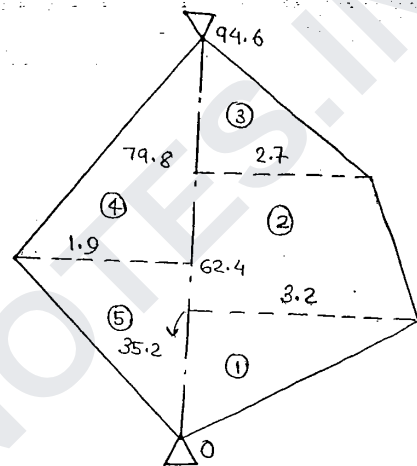
$$A_2 = \frac{1}{2} (79.8 - 35.2) (3.2 + 2.7) = 131.57 \text{ m}^2$$

$$A_3 = \frac{1}{2} (94.6 - 79.8) (2.7) = 19.98 \text{ m}^2$$

$$A_4 = \frac{1}{2} (94.6 - 62.4) (1.9) = 30.59 \text{ m}^2$$

$$A_5 = \frac{1}{2} (62.4) (1.9) = 59.28 \text{ m}^2$$

$$\begin{aligned} \text{Total area, } A &= A_1 + A_2 + A_3 + A_4 + A_5 \\ &= \underline{\underline{297.74 \text{ m}^2}} \end{aligned}$$



07. Correction for temperature, $C_t = L \alpha (T_m - T_0)$

$$\begin{aligned} &= 20 \times (6 \times 10^{-6}) \times (30 - 55) \\ &= \underline{\underline{-0.003 \text{ m}}} \end{aligned}$$

11. $L' = \frac{20.10 + 20.30}{2} = \underline{\underline{20.2 \text{ m}}}$

$$A = A' \left(\frac{L'}{L} \right)^2 = 32.56 \left(\frac{20.2}{20} \right)^2 = 33.21 \text{ cm}^2$$

$$10 \text{ cm} = 8 \text{ m} \Rightarrow 1 \text{ cm} = 0.8 \text{ m}$$

$$\begin{aligned} 33.21 \text{ cm}^2 &= 33.21 \times 0.8 \times 0.8 \\ &= \underline{\underline{21.256 \text{ m}^2}} \end{aligned}$$

12. With 20 m chain:

$$\text{Corrected distance} = 1200 \times \frac{20.1}{20} = 1206 \text{ m}$$

With 25 m chain:

$$1206 = 1212 \times \frac{L'}{25}$$

$$\Rightarrow L' = \underline{24.88 \text{ m}}$$



$$\begin{aligned} 13. \quad W &= \gamma V = (7.86 \times 0.08 \times 3000) \times 10^{-3} \\ &= 1.8864 \text{ kg.} \end{aligned}$$

$$\begin{aligned} \text{Correction for sag, } C_{\text{sag}} &= \frac{w^2 L}{24 K^2 p^2} = \frac{1.886^2 \times 30}{24 \times 3^2 \times 10^2} \\ &= 0.0049 \text{ m.} \end{aligned}$$

C_{sag} is always negative.

$$\therefore C_{\text{sag}} = -0.0049 \text{ m}$$

$$\begin{aligned} 15. \quad \text{Degree of accuracy, } r &= \operatorname{cosec} \theta \\ &= \operatorname{cosec} 1^\circ 30' \\ &= 38.25 \end{aligned}$$

$$\begin{aligned} \text{DA} &= 1 \text{ in } r \\ &= 1 \text{ in } 38.25 \quad (\approx 1 \text{ in } 39). \end{aligned}$$

$$16. \quad l = \frac{S \cdot r}{40 \sqrt{2}} = \frac{20 \times 40}{40 \sqrt{2}} = \underline{14.14 \text{ m}}$$

$$S: 1 \text{ cm} = 20 \text{ m}$$

$$r: 1 \text{ in } 40.$$

20. $1 \text{ mm} = 1000 \text{ mm}$

$0.1 \text{ mm} = 100 \text{ mm}$

$\therefore 0.1 \text{ mm} = \underline{\underline{0.1 \text{ m}}}$

P-8

19. $\tan \theta = \frac{1}{20}$

$\therefore \theta = 2.86^\circ$

$\cos \theta = 0.998$

Correction for slope, $C_{SL} = L(1 - \cos \theta)$

$= 60(1 - 0.998) = 0.075 \text{ m}$

$= \underline{\underline{7.5 \text{ cm}}}$

1st NOV,
SATURDAY

03. COMPASS SURVEY

* Principle

Direction of a line can be measured

→ Types of Meridian

(i) True meridian

It is at a point a great circle passing through the geographical north and south pole of earth surface.

(ii) Magnetic meridian

It is a direction shown by a magnetic north when it is freely suspended.

(iii) Grid meridian

It is a reference line established by state governments in the middle of the state for their ~~cross~~ projects in various departments.

(iv) Arbitrary meridian

It is a local reference point taken for measurement.

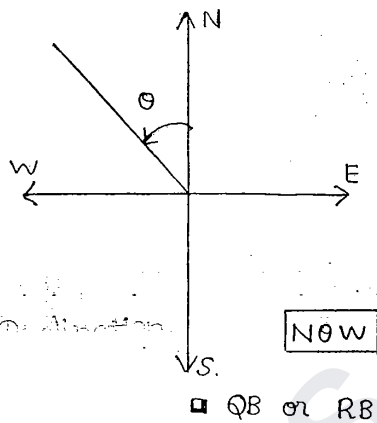
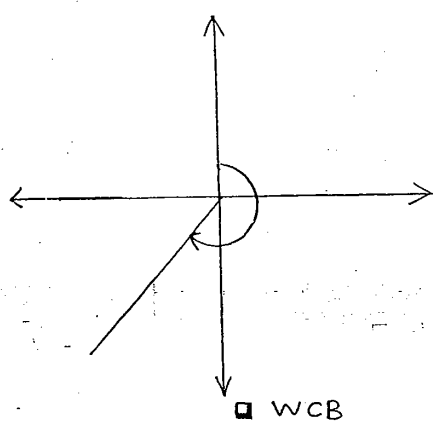
→ Bearing of a Line

It is the horizontal angle made by a line with any type of reference meridian.

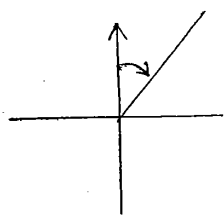
→ Systems of Bearing

1. Whole Circle Bearing (WCB) System (Azimuthal System)
2. Quadrantal (or) Reduced Bearing System.

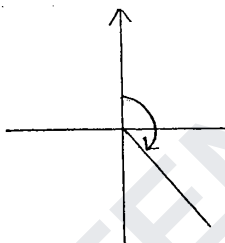
Bearing of line will be measured with N or S whichever is nearer.



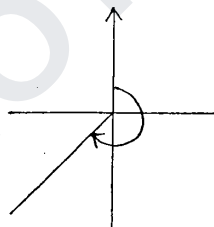
* Conversion of wcb into QB.



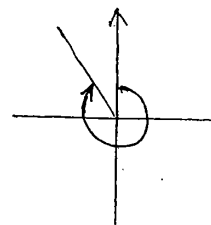
$N\theta E$



$S(180-\theta)E$

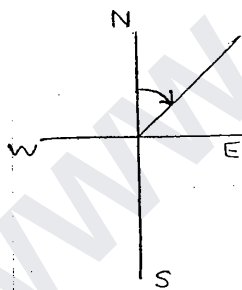


$S(\theta-180)W$

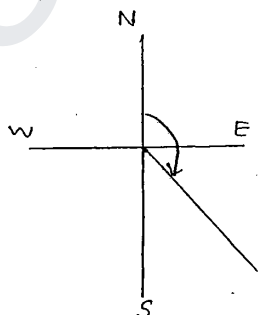


$N(360-\theta)W$

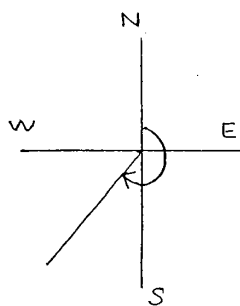
* Conversion of QB into wcb.



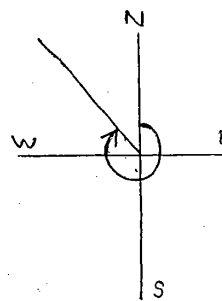
θ



$180-\theta$



$180+\theta$



$360-\theta$

→ Forebearing & Backbearing of a Line.

Forebearing of a line is the bearing of a line measured in the direction of progression of a survey.

Backbearing of a line is the bearing of a line measured opposite to the direction of progression of survey.

- Backbearing of line AB
 = Bearing of line BA.
 = $\theta + 180$
 = $FB + 180$

$$BB = FB \pm 180^\circ$$

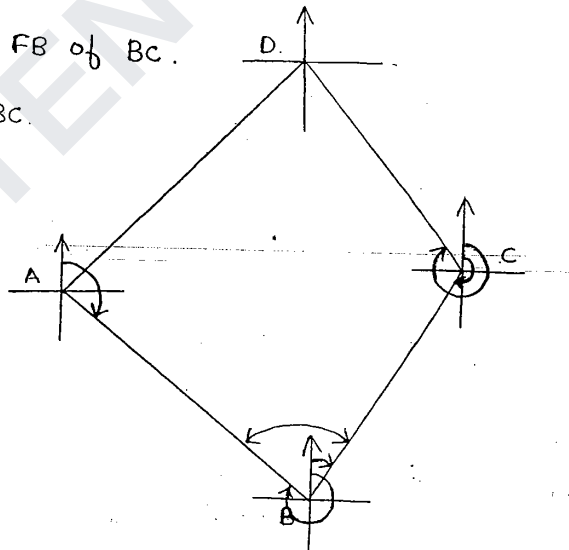
Use '+', if $FB < 180^\circ$

Use '-', if $FB > 180^\circ$

* Calculation of Interior Angles from given bearings.

$$\angle B = (360 - BB \text{ of } AB) + FB \text{ of } BC.$$

$$\angle C = FB \text{ of } CD - BB \text{ of } BC.$$

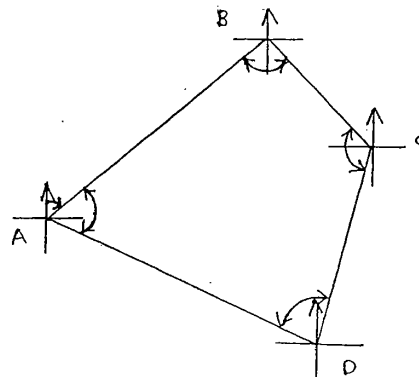


* Calculation of Bearings from given interior angles.

$$(FB)_{BC} = (BB)_{AB} - \angle B.$$

$$360 - (FB)_{DA} + (BB)_{CD} = \angle D.$$

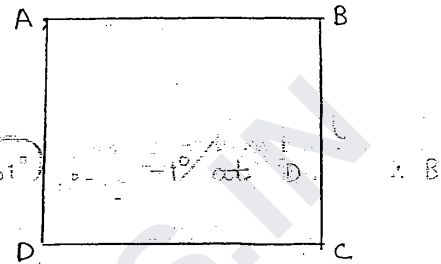
$$(FB)_{DA} = 360 - \angle D + (BB)_{CD}.$$



NOTE :

① In a rectangle or square ABCD shown in fig,

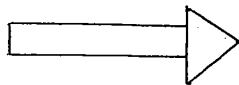
- bearing of AB = bearing of DC.
- bearing of AD = bearing of BC



→ Differences b/w Prismatic Compass & Surveyor's Compass.

Prismatic Compass

1. Broad type of magnetic needle.



2. Graduated cord ring is attached to the needle.
3. Graduations marked are inverted.
4. WCB system is followed.
 0° at S, 180° at N, 90° at W,
 270° at E

Surveyor's Compass

1. Edge bar type



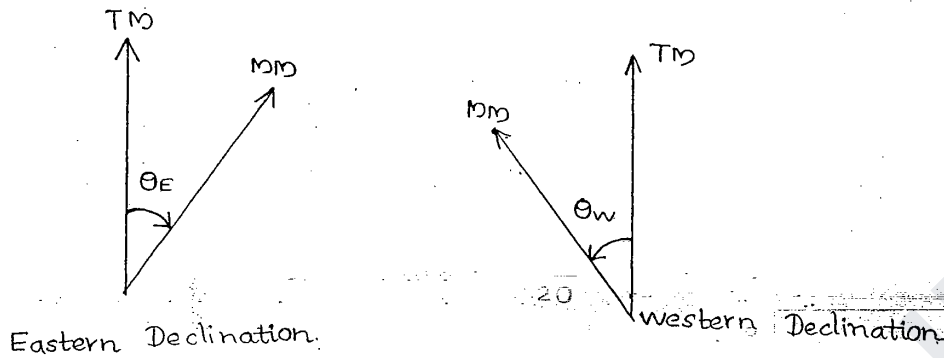
2. Graduated chord ring is attached to compass box.
3. Direct graduations are marked.
4. QB system is followed.
 0° at N & S, 90° at E & W.

→ Temporary Adjustments.

Centering, leveling & focussing the prism.

→ Magnetic Declination.

It is at the place, horizontal angle b/w true meridian and magnetic meridian shown by needle at the time of observation.



$$TB = MB \pm \text{Declination.}$$

Use +, if declination is towards east.
 -, if declination is towards west.

$$MB = TB \mp \text{Declination.}$$

Use -, if eastern declination.
 +, if western declination.

→ Diurnal Variation.

- calculated for 24 hours.
- Ranges blw 3' to 12'

→ Annual Variation

- calculated for 365 day.
- Ranges blw 1' to 2'

→ Secular Variation

- calculated for 250 years.
- blw 5' to 10' per year.

→ Irregular Variation.

- calculated during natural calamities, magnetic storm.
- observed as 2'

→ Isogonic Lines

It is a line joining the points of same declination.

→ Agonic Lines

It is a line joining points of zero declination (when MN & TN coincides).

→ Dip.

Inclination of magnetic needle with horizontal.

Dip is zero at equator and 90° at S & N magnetic pole

NOTE :

- ① TB is also called as 'Azimuth'.
- ② TB of sun at noon (12.00 hrs) is 180°
- ③ If longitude is greater than the standard meridian, the difference b/w them will be added to the standard time to get the local mean time.

2nd Nov,
SUNDAY

→ Local Attraction:

It is the deviation of magnetic needle with the influence of magnetic attracted materials like fencing, steel materials etc

- Detection of local attraction:-

If the difference b/w FB & BB is not 180° , stations represented by that line are affected by local attraction.

- Correction for Local Attraction:-

a) For bearings

Line	FB	BB	Line	FB	BB
AB	$120^\circ 30'$	299°	CD	80°	261°
BC	$140^\circ 30'$	$320^\circ 30'$	DA	$100^\circ 30'$	281°

Find the correction for bearings:

(17) (18)

Ans.	Line	FB	BB	Correction
	AB	$120^{\circ}30'$ 119°	299°	0° at B.
	BC	$140^{\circ}30'$	$320^{\circ}30'$	0° at C.
	CD	80°	261° 260°	-1° at D.
	DA	$100^{\circ}30'$ -1°	281°	$-1^{\circ}30'$ at A.
		$99^{\circ}30' + 180 = 279^{\circ}30'$		

$\left\{ (BB)_{BC} - (FB)_{BC} = 180^{\circ} \right.$
 $\therefore B \text{ \& } C \text{ are free from corrections.}$

b) For Interior Angles.

Step 1:

Calculate the interior angles at all stations from the given bearings

Step 2:

Check for a closed traverse, i.e., sum of interior angles $= (2n-4)90$ where n is no. of sides in a closed traverse.

For exterior angles, check will be applied

Sum of exterior angles $= (2n+4)90$; for a closed traverse

Step 3:

If check is not verified, total correction obtained from the check verification, is distributed equally to all interior angles. and calculate corrected interior angles.

Sum of interior angles

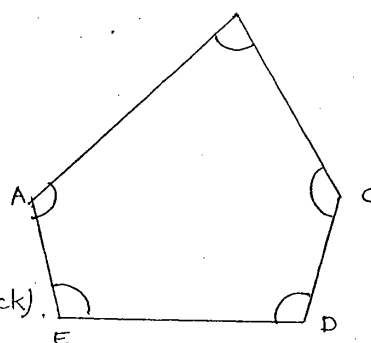
$$= 538^{\circ}30' \text{ (given)}$$

Sum of interior angles

$$= (2n-4)90$$

$$= (2 \times 5 - 4)90 = 540^{\circ} \text{ (check)}$$

$$\text{Correction} = \frac{540 - 538^{\circ}30'}{5} = 0^{\circ}18'0''$$



Step 4:

Calculate correct bearings of lines in a closed traverse by taking FB of first line (AB) as correct, and the corrected interior angles.

NOTE:

If there is no line that is unaffected by local attraction the line whose FB and BB differs least from 180° , find mean bearing of that line by distributing half the error each of FB & BB.

P-29

01. $MB = S 28^\circ 30' E$
 $= 151^\circ 30'$

$$TB = MB + \text{Declination}$$
$$= 151^\circ 30' + 5^\circ 38' = 157^\circ 08'$$
$$= \underline{\underline{S 22^\circ 52' E}}$$

02. $TB = 48^\circ 24' + 5^\circ 38' = \underline{\underline{54^\circ 02'}}$

03. $\text{Declination} = 184^\circ - 180^\circ$
 $= \underline{\underline{4^\circ W}}$

04. $TB = 180 - 89 = 91^\circ$

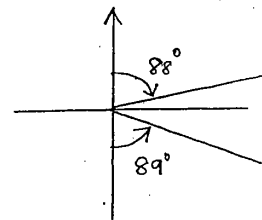
$$\text{Declination} = \underline{\underline{3^\circ E}}$$

05. True bearing = Azimuth = 260°

06. $S 10^\circ W = 180 + 10 = 190^\circ$ (in WCB).

08. $MB < 180$ (Sun in Eastern Hemisphere).

$MB > 180$ (Sun in Western Hemisphere).



09. If FB is given as N 0 E,
then BB is obtained as S Q W and vice versa.

$$FB = 225^\circ$$

$$BB = 225 - 180 = 45^\circ$$

$$= N 45 E$$

$$12. \angle BAC = 120^\circ - 30^\circ = \underline{90^\circ}$$

$$13. \angle ABC = (BB)_{AB} - (FB)_{BC}$$

$$= (180 + 50) - 310 = \underline{80^\circ}$$

$$14. (FB)_{AB} = N 70 W \quad (FB)_{BC} = N 70^\circ W = 290$$

$$(BB)_{AB} = S 70 E = 110$$

$$\angle ABC = 290 - 110 = \underline{180^\circ}$$

$$15. (FB)_{AB} = N 38 E = 38^\circ \quad (FB)_{BC} = S 70 E = 110.$$

$$(BB)_{AB} = 180 + 38 = 218^\circ$$

$$\angle ABC = 218 - 110 = \underline{108^\circ}$$

$$17. (FB)_{AB} = N 30 E = 30^\circ$$

$$\angle ABC = 90^\circ \text{ (Square)}$$

$$(FB)_{BC} = (BB)_{AB} - 90^\circ = 210 - 90 = 120^\circ = \underline{S 60^\circ E}$$

$$18. (FB)_{AB} = 30^\circ \quad (FB)_{BC} = 150 \quad (FB)_{CA} = 270$$

$$(BB)_{AB} = 210 \quad (BB)_{BC} = 330 \quad (BB)_{CA} = 90$$

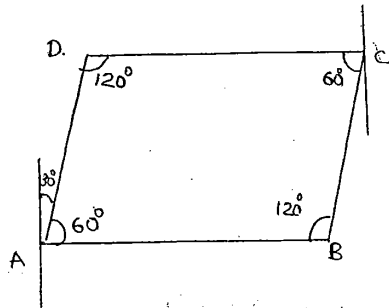
$$\angle ABC = 210 - 150 = 60^\circ$$

$$\angle BCA = 330 - 270 = 60^\circ$$

$$\angle CAB = 90 - 30 = \underline{60^\circ}$$

$\Rightarrow$ equilateral triangle

20.



$$(FB)_{AB} = 30^\circ$$

$$(BB)_{AB} = 210^\circ$$

$$(FB)_{BC} = 210 - 120 = 90^\circ$$

$$(BB)_{BC} = 210^\circ - 15^\circ = \underline{\underline{(FB)_{CD}}}$$

22. MB of AB = $89^\circ + 1^\circ = 90^\circ$

MB of BA = $360 - 90 = \underline{\underline{270^\circ}}$

23.

PQ	FB 59° ✓	BB 235°	239°
QR	125° 30' ✓	309° 30'	
	129° 30'		

For an open traverse, first reading is assumed as correct

$$\begin{aligned} \angle PQR &= (BB)_{PQ} - (FB)_{QR} \\ &= 239 - 129^\circ 30' = \underline{\underline{109^\circ 30'}} \end{aligned}$$

24. MB = S 45 E = 135°

Declination = $5^\circ W$.

$$TB = 135 - 5 = 140^\circ - 5^\circ = 130^\circ$$

$$\Rightarrow \underline{\underline{S 50^\circ E}}$$

25. TB = MB + declination - correction

$$= 185 + 3.5 - 1.5 = \underline{\underline{187}}$$

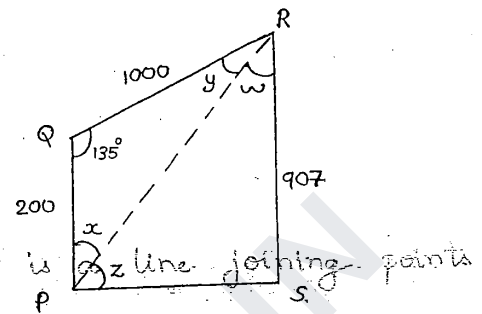
28.

Line	AB	BC	CD	DA
FB	120° 30' ✓	78° 15' ✓	300° 30' + 2° 15'	210° 15' = 207° 45'
BB	300° 30' ✓	256° ✓ 258° 15'	125° 15' (122° 45')	120° 45' = 27° 40'

P-28

Q.51.

Line	Length (m)	Bearing (wcb)
PQ	200	0°
QR	1000	45°
RS	907	180°
SP	?	?



$$\begin{aligned}\angle PQR &= (BB)_{PQ} - (FB)_{QR} \\ &= (180+0) - 45 = 135^\circ\end{aligned}$$

$$\begin{aligned}PR^2 &= PQ^2 + QR^2 - 2PQ \cdot QR \cos 135^\circ \\ &= 200^2 + 1000^2 - 2 \times 200 \times 1000 \cos 135^\circ\end{aligned}$$

$$\Rightarrow PR = \underline{\underline{1150 \text{ m}}}$$

Applying sine rule in $\triangle PQR$,

$$\frac{\sin x}{1000} = \frac{\sin y}{200} = \frac{\sin 135}{1150}$$

$$\Rightarrow \sin x = \frac{1000 \sin 135}{1150}$$

$$\therefore x = \underline{\underline{37.94^\circ}}$$

$$\text{Similarly } y = 7.06^\circ$$

$$\begin{aligned}\angle QRS &= (BB)_{QR} - (FB)_{RS} \\ &= (180+45) - 180 = \underline{\underline{45^\circ}}\end{aligned}$$

$$\therefore y + w = 45^\circ$$

$$\text{or } w = 45 - 7.06 = \underline{\underline{37.94^\circ}}$$

$$SP^2 = PR^2 + RS^2 - 2PR \cdot RS \cos 37.94^\circ$$

$$= 1150^2 + 907^2 - 2 \times 1150 \times 907 \cos 37.94^\circ$$

$$\Rightarrow SP = \underline{\underline{707.06 \text{ m}}}$$

Applying sine rule in ΔPRS

$$\frac{\sin Z}{907} = \frac{\sin 37.94}{707.06}$$

$$\Rightarrow Z = \underline{\underline{52.06^\circ}}$$

$$\begin{aligned}\angle QPS &= (BB)_{QP} - (FB)_{PS} \\ &= 0 - (FB)_{PS}\end{aligned}$$

$$\text{But } \angle QPS = x + z = 52.06 + 37.94 = \underline{\underline{90^\circ}}$$

$$\therefore \angle QPS = (FB)_{PS} = (BB)_{SP} = \underline{\underline{90^\circ}}$$

$$(FB)_{SP} - (BB)_{SP} = 180^\circ$$

$$(FB)_{SP} = 180^\circ + 90^\circ = \underline{\underline{270^\circ}}$$

52. Find lengths PQ & QR.

$$x = 1000 - 200 = 800$$

$$y = 1000 - 100 = 900$$

$$\tan u = \frac{900}{800}$$

$$\therefore u = \tan^{-1}\left(\frac{9}{8}\right) = 48.36$$

$$\therefore v = 90 - u = 41.64$$

$$PR = \sqrt{x^2 + y^2} = \sqrt{800^2 + 900^2} = 1204.16 \text{ m.}$$

$$30^\circ + u + \angle QPR = 90^\circ$$

$$\therefore \angle QPR = \underline{\underline{11.64^\circ}}$$

$$\angle PQR = (BB)_{PQ} - (FB)_{QR} = 210 - 45 = 165^\circ$$

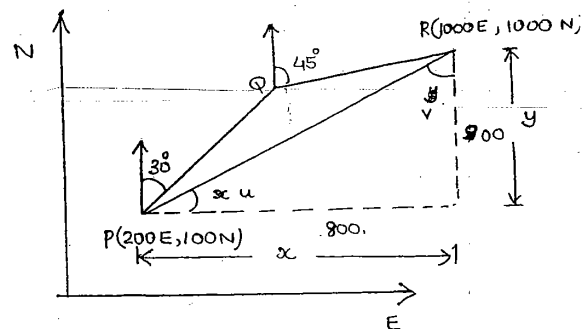
$$\therefore \angle QRP = 180 - (11.64 + 165) = \underline{\underline{3.36^\circ}}$$

Applying sine rule,

$$\frac{\sin 165}{PR} = \frac{\sin(3.36)}{PQ} = \frac{\sin(11.64)}{QR} \Rightarrow$$

$$PQ = \frac{\sin 3.36}{\sin 165} \times PR = \underline{\underline{272.68 \text{ m}}}$$

$$QR = \frac{\sin 11.64}{\sin 165} \times PR = \underline{\underline{938.7 \text{ m}}}$$



24 hours $\rightarrow$ 360°
 1 hour $\rightarrow$ 15°

Degree System Hour System.

15° $\rightarrow$ 1 hour
 $15'$ $\rightarrow$ 1 min.
 $15''$ $\rightarrow$ 1 sec.

Difference = $90^\circ 40' - 82^\circ 30'$
 $= 8^\circ 10'$

$\frac{8^\circ}{15} \Rightarrow 0 \text{ hour}$

Local mean time = $6 \text{ hr } 30 \text{ m } 0 \text{ s}$
 $0 \text{ hr } 32 \text{ m } 40 \text{ s}$
 $7 \text{ hr } 02 \text{ m } 40 \text{ s}$

$\frac{40'}{15} \Rightarrow 32 \text{ min.}$

$\frac{10 \times 60}{15} \Rightarrow 40 \text{ sec.}$

Line	FB	BB	
AB	$131^\circ 30'$ $126^\circ 45'$	$311^\circ 30'$ 308°	$3^\circ 30' @ B.$
BC	$45^\circ 15'$ $48^\circ 45'$	$227^\circ 30'$ $\checkmark$	
CD	$340^\circ 30'$	$161^\circ 45'$ $\checkmark$	
DE	$258^\circ 30' \checkmark$	$78^\circ 30'$ $\checkmark$	
EA	$216^\circ 30' \checkmark$	$31^\circ 45'$ $36^\circ 30'$	$+4^\circ 45' @ A.$

After applying correction for local attraction, correct bearing of line BC = ?

$\Rightarrow (FB)_{BC} = \underline{\underline{48^\circ 45'}}$

2nd Nov,
SUNDAY

11. PLANE TABLE SURVEYING

- It is most suitable for traverse surveying in which plotting and measurement can be done simultaneously.
- It can be used for areas which are affected by local attraction.

→ Accessories of Plane table Surveying

a) Drawing table

(i) Traverse table.

(ii) Johnson's table

(iii) Coast Survey table.

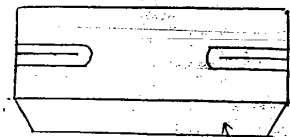
45 cm x 60 cm.

45 cm x 75 cm.

b) Alidade - used for sighting the objects.

(i) Plane

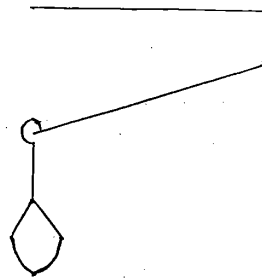
(ii) Telescopic - long & inclined sights



fiducial edge

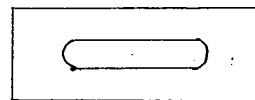
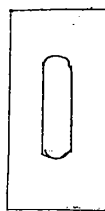
c) U-frame Plumbing Fork.

- used to transfer points from plan to ground and vice versa.



d) Spirit Level.

e) Sough Compass.



21
19

→ Temporary Adjustments:

- (i) Fixing
- (ii) Levelling
- (iii) Centering

(iv) Orientation — it is the process of putting the plane table in a fixed direction so that a line representing a certain direction on the plan is parallel to the direction of same line on the ground.

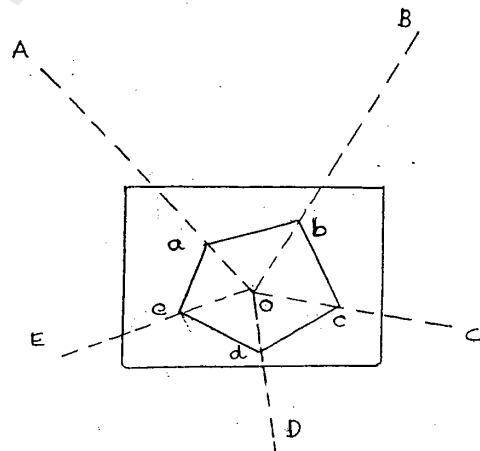
NOTE:

Orientation is more essential when more than one instrument station is to be set up.

→ Methods of Plane Tableing

1. Radiation method.

It is used for locating the points from a single instrument station. It can also be used for calculation of area of field bounded by the points.



It is most suitable when the points are accessible and visible.

2. Intersection Method.

It is suitable when the points are inaccessible but visible. It is particularly used for calculating the distance b/w two inaccessible points.

3. Traversing

Any surface, its shape & elevations.

It is most suitable for taking the details of objects as there is a check from each traversing station.

4. Resection.

It is the process of determining the plotted position of the station occupied by the plane table by means of sights taken towards known points, locations of which have been plotted already.

(i) 3-point problem

a) Mechanical method / Tracing paper method.

b) Bessel's graphical.

c) Lehmann's method (or) Trial & Error method.

(ii) 2-point problem.

→ Advantages of Plane table surveying:

(i) It is quite suitable for plotting small scale maps directly on the field.

(ii) Plotted map can be compared with the actual features on the ground.

(iii) Contour maps and topographical maps can be prepared and can be checked with ground features.

(iv) Errors in measurement and plotting can easily be detected in the field by running check lines.

(v) It is best suited for traversing.

→ Disadvantages

- (i) It is not accurate, (accurate upto 85%)
- (ii) not possible to replot to some other scale.
- (iii) It is used only for short sights.

2nd nov,
SUNDAY

06. LEVELLING

→ Principle.

Measurements in vertical plane.

when unequally reflected by a lens that

→ Objectives

- To find RL of different objects lying on (or) below (or) above the Earth's surface.
- To establish points at a given elevation with datum.

→ Uses

- to prepare the contour map.
- altitudes of different points in a hill.
- to prepare layout of water distribution system, drainage system etc.
- to prepare longitudinal sectioning and cross sectioning of a project.

→ Terms.

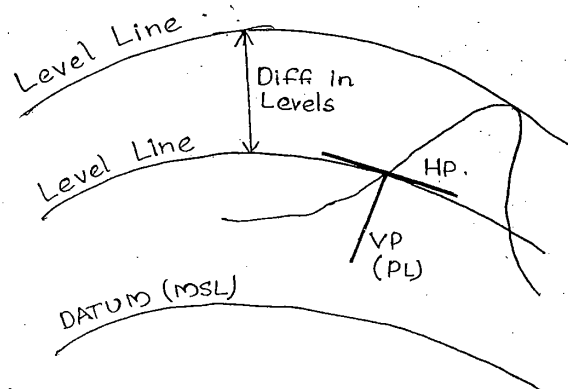
1. Level Surface:

Curved surface perpendicular to direction of gravity.

Eg: still water in a lake, pond.

2. Horizontal Plane:

Tangential to level surface and $\perp$ to plumb line.



3. Vertical Plane.

It is normal to horizontal plane and shown by plumb bob.

4. Datum.

Any surface to which elevations are referred is called datum.

5. Mean Sea Level:

Average height of sea for all the stages of tides considered for 19 years period.

6. Bench Mark:

It is a permanent point of reference whose elevation with datum is known.

(i) Permanent BM

(ii) Temporary BM

(iii) GTS BM

(iv) Arbitrary BM

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7. Reduced Level

Elevation of a point above or below datum

4th NOV. → Methods of Levelling

TUESDAY

1. Barometric Levelling.

RL difference b/w any two points can be measured by taking the pr. difference b/w them. Pressure can be measured with the help of barometer, hypsometer, altimeter also.

2. Indirect Levelling. / Trigonometrical Levelling.
Heights and distances of various points can be calculated by taking the vertical angles.

3. Spirit Levelling / Direct Levelling.

Concept of ^{elevation} telescope and spirit level is coupled. It is called as a plus sign. bcc

→ Levelling Instruments.

1. Level.

- Dumpy Level.
- Auto level.
- Reversible level.
- Tilting level.
- Wye Level.

2. Levelling Staff.

(i) Self Reading

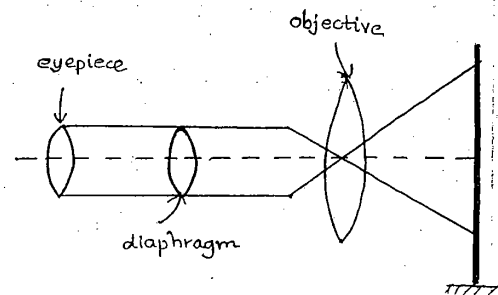
- Solid - 3m
- Folded. - 4m
- Telescopic - 4m, 5m

(ii) Target Levelling Staff.

* Level.

It consists of :

- Telescope
- Levelling head.
- Level tube.
- Tripod.



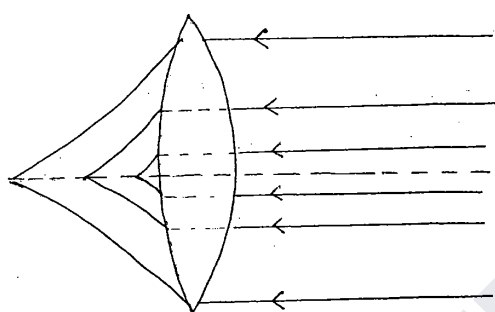
* Telescope is adoption of Kepler, eyepiece is of Ramsdon's type.

○ Optical Defects of Lenses.

a) Aberration - It is the deviation of rays of light when unequally refracted by a lens so that they do not converge and meet at a focus, but separate forming an indistinct image of the object.

(i) Spherical Aberration.

(ii) Chromatic Aberration.



Optical defects of lens can be avoided by providing compound lens.

Definition:

It is the capability of producing the sharp image.

Magnification:

It is the ratio b/w focal length of objective to the eyepiece.

$$\text{Magnification} \propto \frac{1}{\text{Illumination of lenses}}$$

Illumination depends on the quality of lens, magnification power of lens and no. of lenses used.

External Focussing Telescope:

It is the one in which focussing is done by the external movement of either objective or eyepiece.

Internal Focusing Telescope:

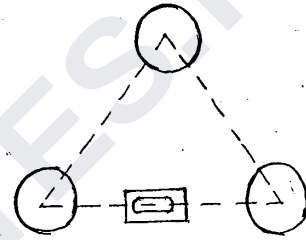
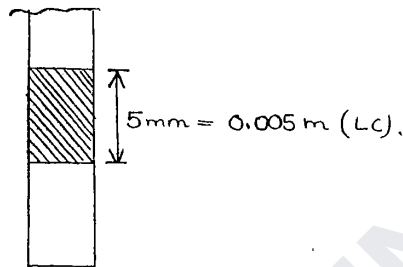
It is the one in which focussing is done internal by negative lenses.

NOTE:

Internal focusing telescope is advantageous than external focussing telescope.

(iii) Rising Gradient

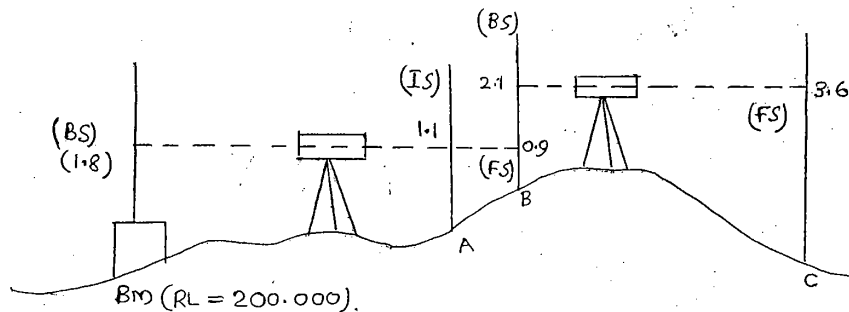
* Levelling Staff.



■ Tribrach

→ Types of Levelling

- (i) Differential Levelling / Fly levelling / Check levelling.
- (ii) Profile Levelling.
- (iii) Longitudinal sectioning & Cross sectioning.
- (iv) Precise levelling
- (v) Reciprocal levelling



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* Terms :-

1. Station - location of levelling staff.
2. Height of instrument - it is the reduced level of LOS.
3. Backsight - It is the sight taken on a point of known elevation. It is called as plus sight because HI can be calculated by adding BS to the known RL.

$$HI = RL + BS$$

4. Foresight - it is the sight taken on a point of unknown elevation. It is called as minus sight because RL of any point can be calculated by deducting FS from HI.

$$RL \text{ of any point} = HI - FS$$

5. Intermediate sight - it is the sight taken b/w FS & BS.

$$RL \text{ of any point} = HI - IS$$

6. Line of sight - it is an imaginary line passing through the optical centres of objective and intersection of cross hairs.

→ Methods of Reduction of Levels.

(i) HI method (or) Collimation method.

- It is not suitable for intermediate sights because there is no check for it.

Stn	BS	IS	FS	HI	RL	Remarks
Bm	1.8			201.800	200.000	RL of B.
A		1.1			200.700	
B	2.1		0.9	203.00	200.900	CP
C			3.6		199.400	

Check:

$$\begin{aligned}\sum BS - \sum FS &= \text{Last RL} - \text{First RL} \\ &= \underline{\underline{0.6}}\end{aligned}$$

Stn.	BS	IS	FS	Rise (+)	Fall (-)	RL	Remarks
Bm	1.8					200.00	
A		1.1		0.7		200.7	
B	2.1		0.9	0.2		200.9	
C			3.6		1.5	199.4	

If succeeding reading < preceding reading $\Rightarrow$ RISE

If succeeding reading > preceding reading $\Rightarrow$ FALL

Check:

$$\sum BS - \sum FS = \sum \text{Rise} - \sum \text{Fall} = \text{LRL} - \text{FRL} = \underline{\underline{-0.6}}$$

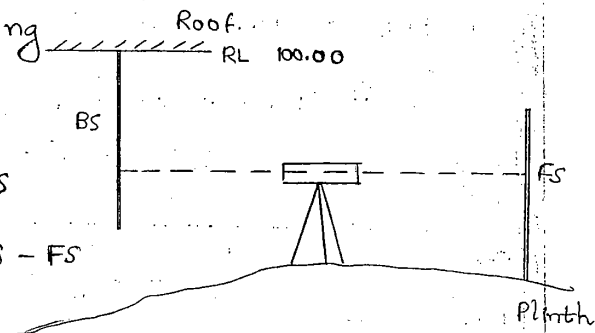
NOTE:

① Rise & Fall Method is superior than HI method; as it is more verified. Rise and fall method is used for small areas, preparation of contour maps and LS and CS.

$\rightarrow$ Inverted Staff Reading

① Difference in height of roof and plinth = BS + FS

② RL of plinth = $(RL)_{\text{roof}} - BS - FS$



Eg: 0.71, 0.85, 2.2, 0.9, 2.4, 3.6, 0.2
(FS) (FS) (BS)

$\Rightarrow$ instrument is shifted after 3rd & 5th readings

→ Levelling on Slopes

(i) Falling Gradient.

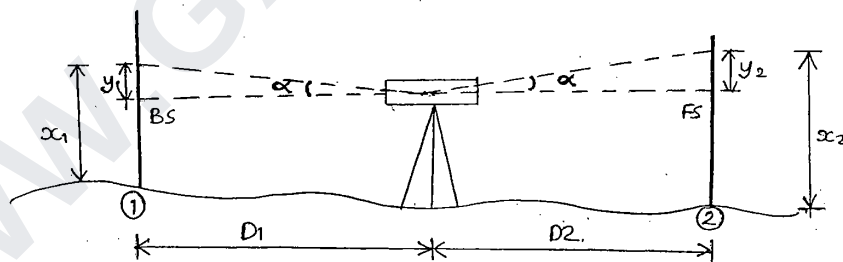
BS $\textcircled{0.7}$, 1.4, 2.6, FS $\textcircled{3.7}$, BS $\textcircled{1.2}$, 1.9, FS $\textcircled{3.4}$

(ii) Rising Gradient.

BS 3.9, 2.4, FS $\textcircled{1.6}$, BS $\textcircled{0.2}$, 3.6, 2.9, FS $\textcircled{1.7}$

→ Balancing Backsight & Foresight.

When it is required to evaluate the RL difference b/w any two points taking BS on point 1 and FS on point 2. It is necessary to make the distance equal from instrument to each of the stations to eliminate the error due to line of collimation and its adjustment, error due to curvature and refraction.



$$\text{Correct staff reading on ①} = (x_1 - y_1).$$

$$\text{Correct staff reading on ②} = (x_2 - y_2).$$

True difference in levels b/w point ① & ②

$$= (x_1 - y_1) - (x_2 - y_2).$$

$$= x_1 - x_2 \quad ; \quad \text{if } y_1 = y_2$$

$$y_1 = y_2$$

$$D_1 \tan \alpha = D_2 \tan \alpha$$

$\Rightarrow \underline{D_1 = D_2}$; instrument must be centrally located b/w stations.

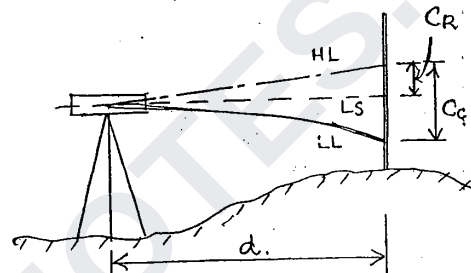
→ Correction for Curvature.

$\Rightarrow$ collimation error

It is the difference b/w horizontal line and level line.

The apparent reading is more and the object appears to be lower than it really is.

Hence correction for curvature is negative.



$$C_c = \frac{d^2}{2R} \quad (-)$$

where $R = 6370 \text{ km}$.

$$C_c = -0.07857 d^2$$

C_c obtained in m when $d = \text{km}$

→ Correction for Refraction.

The LOS is deviated from horizontal line, hence, the effect of refraction is to make the object appear higher than it really is. $\therefore C_R$ is positive.

$$C_R = \frac{1}{7} C_c = \frac{d^2}{14R}$$

$$C_R = 0.01122 d^2 \quad (\text{when } d \text{ in km})$$

* Combined Correction, $C = C_c + C_R = -0.07857 d^2 + 0.01122 d^2$

$$\Rightarrow \underline{C = -0.06735 d^2}$$

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→ Distance to Visible Horizon.

P → point of observation.

A → horizon

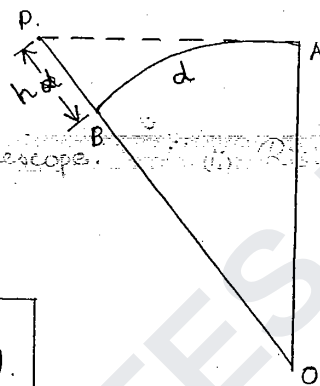
d → distance blw P & A.

h = combined correction

$$= 0.06735 d^2$$

$$\therefore d = \sqrt{\frac{h}{0.06735}} \quad (h \text{ in m}).$$

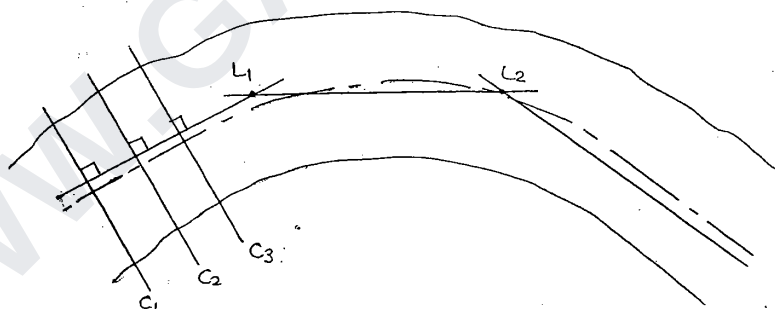
(km)



→ Profile Levelling.

- Profile levelling is used to locate the centre line of path. Centre line can be straight or curved.

- LS & CS can give you an idea of cross sections from which volumes can be calculated.

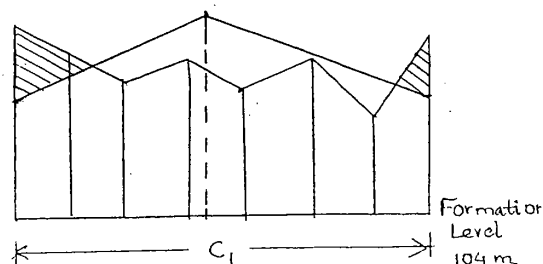


LS : Interval 5m to 10m

CS : Interval 1m to 2m.

 cutting

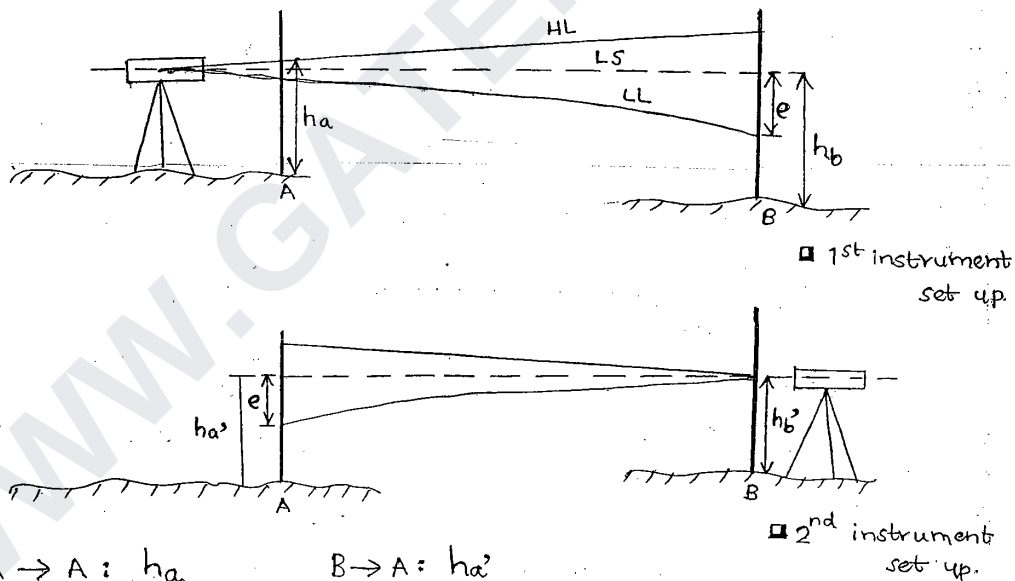
 filling.



→ Reciprocal Levelling

When it is necessary to carry the levelling across a river or ravine or any obstacle requiring long sight between two points so situated that no place for the level can be found from which the length of FS & BS will be even. approximately equal, the special method, i.e., reciprocal levelling must be opted to obtain the accuracy and to eliminate the following errors:-

- (i) Error in line of collimation and instrument adjustment.
- (ii) Error due to curvature.
- (iii) Error due to refraction. (partly eliminated).



A → A : h_a

B → A : h_a'

A → B : h_b

B → B : h_b'

(i) True difference in levels b/w A & B :

$$H = \frac{1}{2} [(h_a - h_b) + (h_a' - h_b')]$$

It is average of difference of apparent readings from both stations

(ii) $RL \text{ of } B = RL \text{ of } A \pm H.$

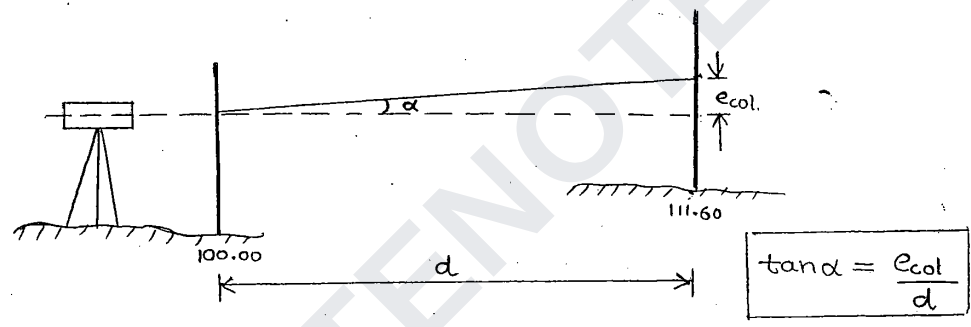
(iii) Total error, $e = -\frac{1}{2} [(h_a - h_b) - (h'_a - h'_b)]$

$e = e_{col} + e_{cur} + e_{ref}$

$e_{col} \rightarrow$ collimation error $(= 0.07857 d^2; d \text{ in km})$

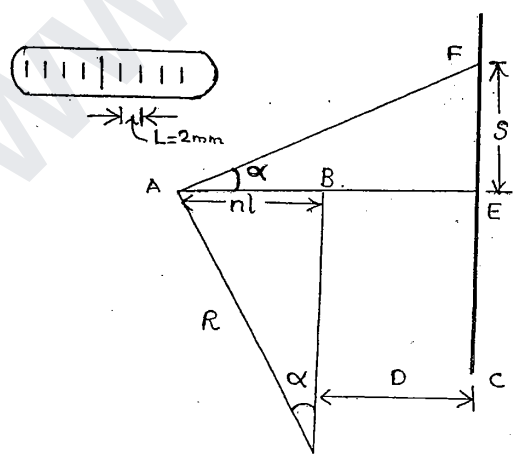
$e_{cur} \rightarrow$ curvature error $\rightarrow$

$e_{ref} \rightarrow$ refraction error. $(= -0.01122 d^2; d \text{ in km}).$



$\rightarrow$ Sensitivity of Bubble tube

It is an angular value of one division marked on the level tube. It is an amount the horizontal axis has to be tilted to cause the bubble to move from one graduation to other.



$n \rightarrow$ no. of divisions disturb.

$S \rightarrow$ diff. in staff readings before and after disturb the level.

$S = CF - CE; l = 2 \text{ mm.}$

In $\triangle FAE,$

$\tan \alpha = \frac{S}{D}.$

$\Rightarrow \alpha = \frac{S}{D}$

Arc of AOB:

$$nl = R\alpha$$

$$\therefore \alpha = \frac{nl}{R}$$

$$\Rightarrow \text{Radius of curvature of level tube, } R = \frac{nD}{S}$$

$$\alpha' = \frac{L}{R} \text{ rad}$$

$$= 206265 \frac{L}{R} \text{ Sec.}$$

$$\therefore \alpha' = \frac{l}{R} = \frac{l}{\frac{nD}{S}} = 206265 \frac{S}{nD} \text{ Seconds.}$$

Unit : Seconds/2 mm.

$$\text{Sensitivity, } \alpha' = 206265 \frac{S}{nD}$$

Sensitivity depends on:

- (i) increasing the radius of level tube.
- (ii) increasing the diameter of the tube.
- (iii) increasing the length of level tube.
- (iv) decreasing the roughness of wall.
- (v) decreasing the viscosity of liquid in level tube.

P-52

04. $(BS)_A = 1.535 \text{ m. (normal staff)}$

$(FS)_B = -1.837 \text{ m (inverted staff.)}$

$$\begin{aligned} \text{Difference in elevation b/w A \& B} &= 1.535 + 1.837 \\ &= \underline{\underline{3.37 \text{ m}}} \end{aligned}$$

06. RL of forward station = $200 + 1.525 + 3.175$
 $= \underline{204.7 \text{ m}}$

07. $\Sigma BS - \Sigma FS = LRL - FRL$

→ Correction for Curvature

$6.475 - 8.565 = (RL)_2 - 560.5$

$\Rightarrow (RL)_2 = \underline{558.41 \text{ m}}$

10. Distance AB = 1 km.

$e = -\frac{1}{2} [(h_a - h_b) - h_a' - h_b']$

$= -\frac{1}{2} ((1.625 - 2.545) - (0.725 - 1.405))$

$= \underline{+0.12}$

$e = e_{col} + e_c + e_R$

$e_c = 0.07857 \times 1^2$

$e_R = -0.01122 \times 1^2$

$0.12 = e_{col} + 0.07857 - 0.01122$

$\Rightarrow e_{col} = \underline{0.0527}$

20. $d = \sqrt{\frac{h}{0.06735}} = \sqrt{\frac{120}{0.06735}} = \underline{42.226 \text{ km}}$

23. HI = RL + IS.

$= 79.1 + 2.885 = 81.985$

RL = HI - FS = $81.985 - 0.68$

$= \underline{81.305 \text{ m}}$

$$25. \quad D = 200 \text{ m}, n = 2.5, \alpha' = \frac{206265 S}{nD} = 30s$$

$$\Rightarrow S = \underline{\underline{0.073 \text{ m}}}$$

$$26. \quad H = \frac{1}{2} [(h_a + h_b) + (h_a' - h_b')]$$

$$= \frac{1}{2} [(1.485 - 1.725) + (1.190 - 1.415)]$$

$$= -7.5 \times 10^{-2} - 0.2325$$

$$\text{RL of A} = \text{RL of B} + H.$$

$$= 55.18 + 0.2325 = \underline{\underline{55.4125 \text{ m}}}$$

$$27. \quad e = -\frac{1}{2} [(1.485 - 1.725) - (1.190 - 1.415)]$$

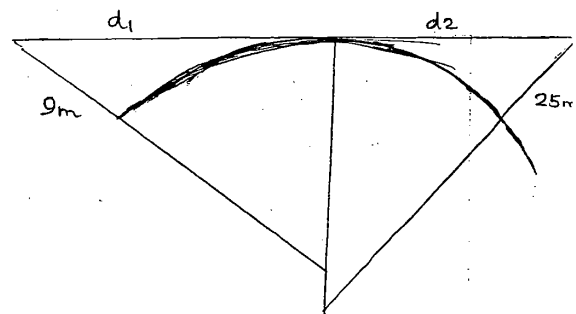
$$= 0.0075 \text{ m}.$$

$$e = e_{\text{col}} = 0.0075 \text{ m} (\because e_c = e_R = 0)$$

$$28. \quad d_1 = \sqrt{\frac{9}{0.06735}}$$

$$d_2 = \sqrt{\frac{25}{0.06735}}$$

$$d = d_1 + d_2 = \underline{\underline{30.84 \text{ km}}}$$



$$32. \quad h = 0.06735 \times D^2$$

$$= 0.06735 \times 60^2 = \underline{\underline{242.46 \text{ m}}}$$

$$\begin{aligned} 34 \quad \text{RL of plinth} &= 100 - 2.105 - 1.105 \\ &= 96.79 \text{ m} \end{aligned}$$

$$\begin{aligned} 35 \quad H &= \frac{1}{2} [(1.03 - 1.630) + (0.950 - 2.74)] \\ &= \underline{\underline{-1.195 \text{ m}}} \end{aligned}$$

found from which the length of FS is as

$$\begin{aligned} \text{RL of Q} &= \text{RL of P} + H \\ &= 450 - 1.195 = \underline{\underline{448.805 \text{ m}}} \end{aligned}$$

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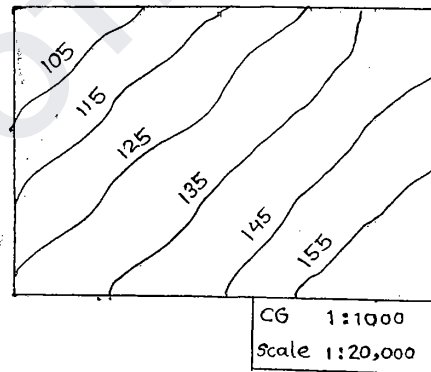
7th nov,
FRIDAY

06. CONTOURS

- Imaginary line passing through points of same elevation.
- It is a line in which surface of ground intercepted by level surface.

→ Contour interval

- It is the RL difference b/w two adjacent contours.
- It is a constant for the given map
- It is small for flat grounds and large for undulated grounds
- contour interval is inversely proportional to scale of map.



* Suggested contour intervals

(i) Building sites	0.2 to 0.5
(ii) Town planning & reservoir	0.5 to 2
(iii) Highways & railways	2
(iv) Location surveys	2 to 3
(v) Geological surveys	6 to 15

→ horizontal equivalent

It is the horizontal distance b/w any two points on two consecutive contours.

$$\text{Contour gradient (CG)} = \frac{\text{Contour Interval (CI)}}{\text{Horizontal equivalent (HE)}}$$

$$HE = \frac{CI}{CG}$$

→ contour gradient

- It is a line lying on the ground surface and maintaining a constant inclination to the horizontal surface.
 ⇒ Radius of curvature of level

* Grade contours

Lines having equal gradient along a slope.

NOTE:

• Horizontal equivalent is variable from point to point on a contour map depending on the steepness of ground.

• HE is less for steeper grounds.

* Radius of an arc required for contour path

$$= \frac{\text{Horizontal equivalent}}{\text{Scale.}}$$

For the fig given above,

$$HE = \frac{10}{\frac{1}{1000}} = 10,000$$

$$\text{Radius} = \frac{10,000}{20,000} = \underline{\underline{0.5 \text{ m}}}$$

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$$5. HE = \frac{10}{\frac{1}{100}} = 1000$$

$$\text{Radius} = \frac{1000}{10,000} = \underline{\underline{0.1 \text{ m}}}$$

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6. $HE = \frac{20}{0.04} = 500 \text{ m.}$

Radius = $\frac{500}{20,000} = 0.025 \text{ m}$
 $= \underline{\underline{2.5 \text{ cm}}}$

1 Acre = $4046.7 \text{ m}^2 = 4840 \text{ sq. yds}$

→ Characteristics of Contours

- (i) Two contour lines of different elevations can cross each other in case of overhanging cliff.
- (ii) Two contour lines of different elevations can unite to form a line in case of vertical cliff.
- (iii) Contour lines close together indicates steep slope, and they are far apart represents a gentle slope.
- (iv) A closed contour line with one or more higher ones inside, it represents a hill. A closed contour line with one or more lower ones inside represent a depression without an out.
- (v) Contour lines cross a ridge ^{at right angles} or watershed line and form U-shape contours.
- (vi) Contour lines cross a valley line at right angles and form V-shape contours.
- (vii) A contour line must close upon itself though not necessarily within the limits of map.

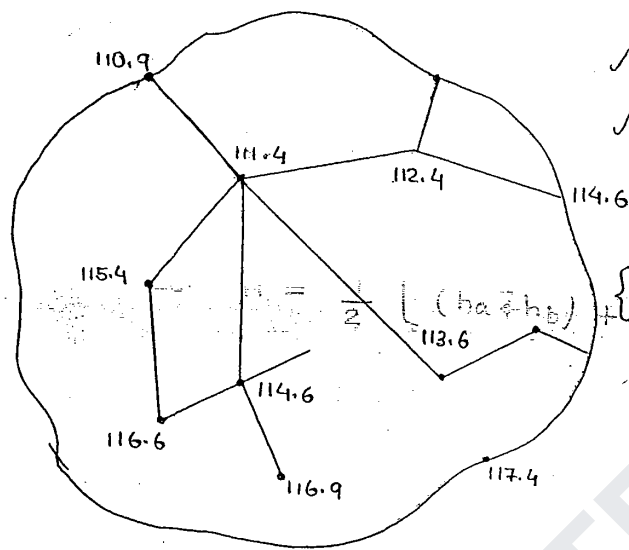
→ Methods for tracing Contours

1. Direct Method.

- a) By spot-levels
- b) By radial lines

2. Indirect Method.

- a) By squares
- b) By rectangular
- c) By grids
- d) Tacheometric methods.



Min RL = 110.6

Max RL = 117.4

CI = 1 m.

$\left\{ \begin{array}{l} 111, 112, 113, 114, 115 \\ 116, 117 \end{array} \right\}$

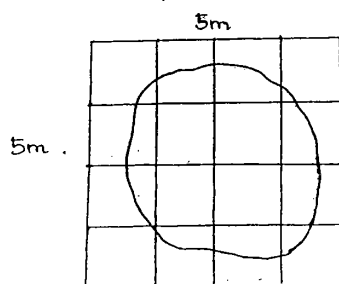
□ spot - levels.

Q. RLS of points P & Q are 49.6 & 51.8 m respectively. Distance PQ is 20 m. Distance in m from P at which 51 m contour cuts the line PQ is — ?

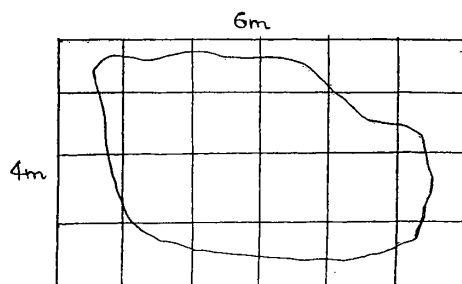
$$\text{Distance from P} = \frac{(51 - 49.6)}{(51.8 - 49.6)} \times 20 = \underline{12.73 \text{ m}}$$

- Direct method is most accurate when used for small areas where high accuracy is required
- Indirect method is not accurate but used for large areas where less accuracy is required.
- Direct method is not suitable for hilly areas.

* By Square



* By Rectangle.



→ Uses of Contours

(i) From contour map, sections may be easily drawn in any direction

(ii) Intervisibility from two ground points plotted on the map can be ascertained.

(iii) It provides a suitable and economical site for any engg. project. A route of a given grade can be traced out on the map.

(iv) Catchment area, capacity of reservoir, quantity of earthwork can be calculated from contour map.

8th nov,
SATURDAY

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10. AREAS & VOLUMES

1 hectare = $10,000 \text{ m}^2$

1 Acre = $4046.7 \text{ m}^2 \approx 4840 \text{ sq. yards}$

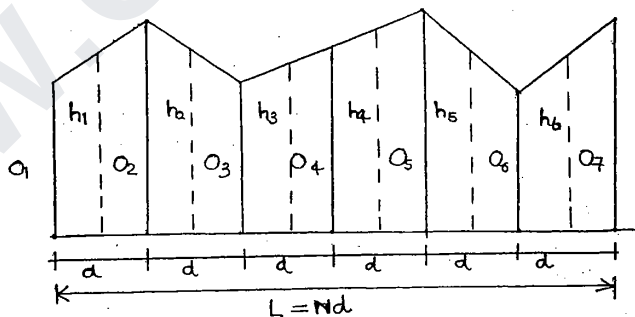
Areas.

Instrumental method.

- (i) Cross Staff survey
- (ii) Compass survey.
- (iii) Plane table survey.
- (iv) Theodolite traversing

Graphical method.

- (i) Field notes
- (ii) End areas. (Boundary areas).
 - a) Mid Ordinate method.
 - b) Average Ordinate method.
 - c) Trapezoidal rule.
 - d) Simpson's rule.



$N \rightarrow$ total no. of ~~ordinates~~ + common intervals.

$d \rightarrow$ common interval.

Total no. of ordinates = $N+1$

(i) Mid - Ordinate Method.

$$h_1 = \frac{O_1 + O_2}{2}, h_2 = \frac{O_2 + O_3}{2} \dots$$

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

(ii) Average - Ordinate method.

$$A = \left(\frac{\text{Sum of Ordinates}}{\text{Total no. of Ordinates}} \right) * \text{Base length}$$

$$A = \left(\frac{O_1 + O_2 + O_3 + \dots O_n}{(N+1)} \right) L$$

(iii) Trapezoidal Rule (Parabolic Rule).

- it is most suitable for curved boundaries
- there is no restriction to no. of ordinates.

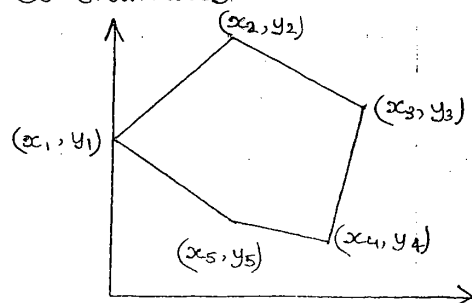
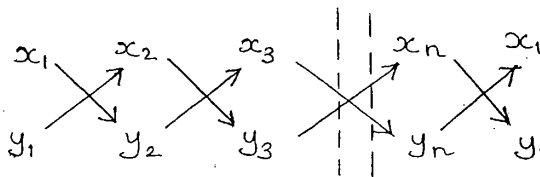
$$A = d \left[\left(\frac{O_1 + O_n}{2} \right) + (O_2 + O_3 + \dots O_{n-1}) \right]$$

(iv) Simpson's Rule.

- it can be used only ^{for} odd number of ordinates
- it can also be used for curved boundaries

$$A = \frac{d}{3} \left[(\text{first} + \text{last}) + 4(\text{even}) + 2(\text{odd}) \right]$$

→ Calculation of area from Co-ordinates.



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32

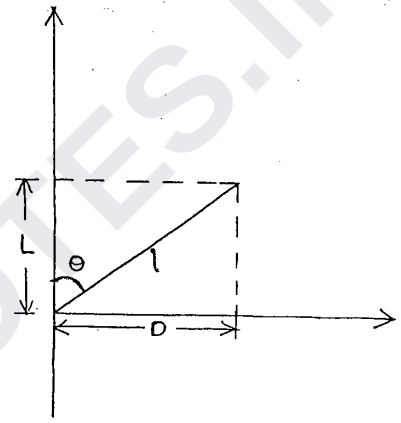
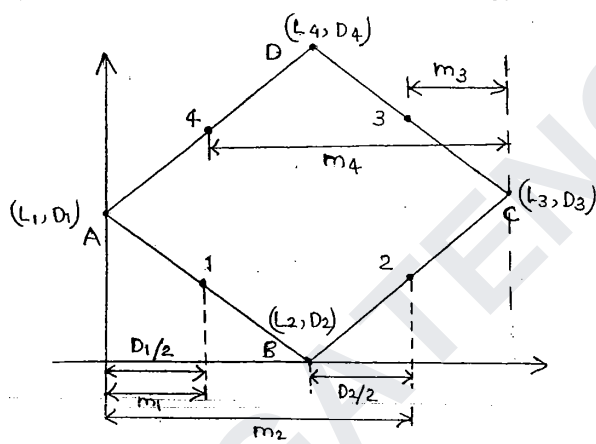
$$A = \frac{1}{2} \left[(x_1 y_2 - x_2 y_1) + (x_2 y_3 - y_2 x_3) + (x_3 y_n - y_3 x_n) + (x_n y_1 - y_1 x_n) \right]$$

→ Calculation of areas from Latitudes & Departures.

Latitude, $L = l \cos \theta$

Departure, $D = l \sin \theta$

(i) Meridian Distance method.



Meridian distance of a line is defined as the meridian distance of its midpoint.

Meridian distance of a line = meridian distance of preceding line
+ half of departure of preced. line
+ half of departure of line itself.

$$m_1 = \frac{D_1}{2} ; \quad m_2 = m_1 + \frac{D_1}{2} + \frac{D_2}{2}$$

$$m_3 = -\frac{D_3}{2} ; \quad m_4 = -D_3 - \frac{D_4}{2}$$

$$A = \sum m L$$

(ii) Double meridian distance Method.

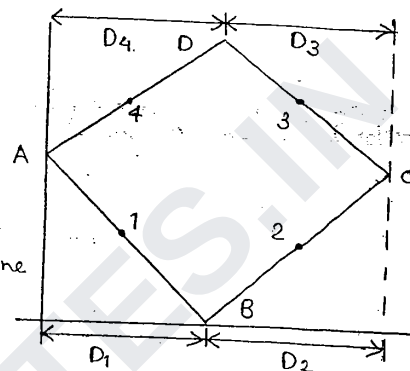
Double meridian distance of a line is equal to sum of the meridian distances of two extremities

$$M_1 = D_1$$

$$M_2 = M_1 + D_1 + D_2 = 6 \times 4 = 24 \text{ m}$$

DMD of a line = DMD of prec. line
+ Departure of prec. line
+ departure of line
itself.

$$A = \frac{1}{2} \sum ML$$



Line	L	D
AB	108	4
BC	15	249
CD	-123	4
DA	0	-257

Single meridian distance method.

Line	L	D	D/2	m	A = mL
AB	108	4	2	2	216
BC	15	249	124.5	128.5	1927.5
CD	-123	4	2	255	-3136.5
DA	0	-257	-128.5	128.5	0

$$\text{Area} = \sum mL = -29221 \text{ m}^2 = \underline{\underline{29221 \text{ m}^2}}$$

Double meridian distance method.

Line	L	D	M	A = $\frac{ML}{2}$
AB	108	4	4	432/2
BC	15	249	257	3855/2
CD	-123	4	510	-62730/2
DA	0	-257	257	0

$$\text{Area} = \frac{1}{2} \sum ML = \underline{\underline{29221 \text{ m}^2}}$$

→ Volumes

* Calculation of area from Level Sections.

$$A = \frac{1}{2n} \left[\left(\frac{b}{2} + nh \right) (w_1 + w_2) - \frac{b^2}{2} \right]$$

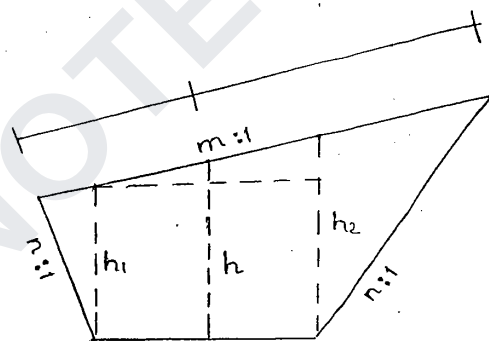
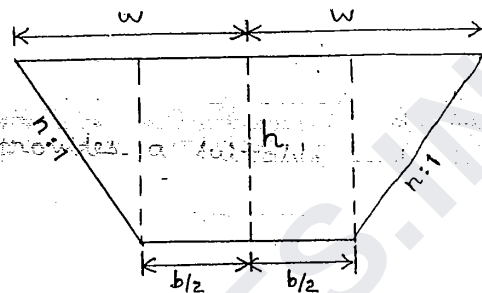
where,

$$w_1 = \left(\frac{b}{2} + nh \right) \left(\frac{m}{m-n} \right)$$

$$w_2 = \left(\frac{b}{2} + nh \right) \left(\frac{m}{m+n} \right)$$

$$* w = \frac{b}{2} + nh$$

$$A = (b + nh)h \quad \left\{ m = 0 \right\}$$



* Calculation of Volumes.

Let $A_1, A_2, \dots, A_n$ be the ^{areas of} cross sections, h be the contour interval of cross section. The volume can be calculated by:

1. Mean Area method.

$$\text{Volume, } V = \left(\frac{A_1 + A_2 + A_3 + \dots + A_n}{n} \right) * L$$

$L \rightarrow$ distance b/w end sections.

2. Prismoidal Formula (Simpson's Rule).

$$V = \frac{h}{3} \left[\left(\text{first} + \text{last} \right) + 4(\text{even areas}) + 2(\text{odd areas}) \right]$$

3. Trapezoidal Formula (Average End area method).

$$V = h \left[\left(\frac{\text{first} + \text{last}}{2} \right) + \text{Remaining areas} \right]$$

* Prismoidal correction

- It is the difference b/w prismoidal formula and trapezoidal value

- Prismoidal correction is always negative.

- The volume calculated by trapezoidal formula must be deducted from volume calculated by prismoidal formula.

Let $A, w_1, w_2, h_1, h_2 \dots$ be the c/s at one end and $A', w_1', w_2', h_1', h_2' \dots$ be the c/s at other end.

Prismoidal correction, $C_p = \frac{dn}{6} (h-h')^2 \rightarrow$ single LS

$$C_p = \frac{d}{6n} (w_1 - w_1')(w_2 - w_2') \rightarrow 2 \text{ LS.}$$

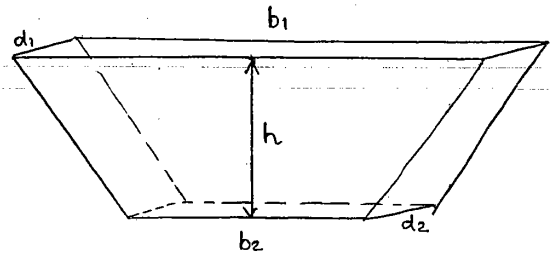
* Volume of water stored.

$$\text{Volume, } V = \frac{h}{6} [A_1 + 4A_m + A_2]$$

$$A_1 = b_1 d_1$$

$$A_2 = b_2 d_2$$

$$A_m = \left(\frac{b_1 + b_2}{2} \right) \left(\frac{d_1 + d_2}{2} \right)$$



Q. A road embankment is 4m wide at formation level with a side slope of 2:1. The average height of embankment is 4m with average gradient as 1 in 30, from 210 m to 330m contour. Find length of road and quantity of earthwork.

$$\text{Gradient} = \frac{\text{Difference in RL}}{\text{Horizontal distance.}}$$

$$\frac{1}{30} = \frac{330 - 210}{\text{length of road}} \Rightarrow \text{length of road} = \underline{\underline{3600 \text{ m}}}$$

$$A = (b + nh)h = (12 + 2 \times 4)4 = 80 \text{ m}^2$$

$$\text{Volume of earth work, } V = A \times L$$

$$= 80 \times 3600 = \underline{\underline{288000 \text{ m}^3}}$$

P-75

$$A_1 = 6 \times 4 = 24 \text{ m}^2$$

$$A_2 = 4 \times 2 = 8 \text{ m}^2$$

$$A_m = \left(\frac{6+4}{2} \right) \left(\frac{4+2}{2} \right) = 15 \text{ m}^2$$

$$V = \frac{6}{6} (24 + 4 \times 15 + 8) = \underline{\underline{92 \text{ m}^3}}$$

P-76

$$1. \quad V = 30 \left(\left(\frac{30+105}{2} \right) + 63 \right) = \underline{\underline{3915 \text{ m}^3}}$$

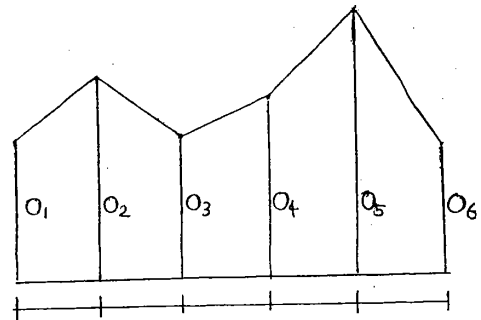
$$2. \quad V = \frac{30}{3} \left((30+105) + 4 \times 63 \right) = \underline{\underline{3870 \text{ m}^3}}$$

$$3. \quad \text{Prismoidal correction, } C_p = 3915 - 3870 = \underline{\underline{45 \text{ m}^3}}$$

$$6. \quad V = \frac{30}{3} \left((20+30) + 4(40+50) + 2 \times 60 \right) = \underline{\underline{5300 \text{ m}^3}}$$

$$7. \quad h = 5$$

$$V = \frac{5}{3} \left((3850+450) + 4(3450+800) + 2(2600) \right) = \underline{\underline{44166.66 \text{ m}^3}}$$



→ Correction for Curvature.

- Prismoidal and trapezoidal formula were derived on the assumption that the end sections are in parallel plane. When the centre line of cutting or the embankment is curved in plan, it is common practice to calculate the volume with the end sections were in parallel planes and then apply correction for curvature.

- No correction for single level sections.

$$C_c = \frac{d}{6R} (w_1^2 - w_2^2) \left(h + \frac{b}{2n} \right).$$

R → radius of curve.

27th nov,
SATURDAY

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MINOR INSTRUMENTS

1. Abney Level.

It is used for measurement of difference in level and tracing contour.

2. Abney Clinometer

It is used for measurement of slopes and setting grade.

3. Tangent Clinometer

To find RL difference by measuring through inclined line of sight. It is suitable for plane table surveying.

4. Ceylon Ghat Tracer

It is used for measuring slopes and especially setting the grades.

5. Pantagraph

It is used for reproducing, enlarging and reducing the maps.

6. Sextant

It is used for measurement of horizontal and vertical angles.

a) Natural Sextant : used in ships for navigation purposes

b) Box Sextant : used in plane & chain survey for measurement of slopes.

7. Mining Dial.

It is the combination of theodolite and prismatic compass and used for mining surveys.

10. Planimeter

used for measurement of areas on the plan.

11. Stream Gauge.

It is used to measure the discharge of a stream.

12. Tellurometer

It is a microwave EDM used for linear measurement.

13. Heliograph.

It is used as a sun signal in triangulation survey.

14. Fathometer

It is used to measure the depth of ocean.

15. Altimeter

Height measuring equipment.

16. Tide Gauge.

To determine water level and its variations.

17. Distomat.

EDM used for accurate linear measurements.

18. Brunton's Compass

Combination of prismatic compass and clinometer used for measurement of bearing and vertical angles.

19. Eideograph.

Improved version of pantagraph.

3rd nov,
SATURDAY

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04. THEODOLITE SURVEY

→ Theodolite.

- It is used for measurement of horizontal and vertical angles directly.

- Indirectly, it can also be used for calculating distances, and elevation of objects.

→ Miscellaneous Operations with Theodolite.

- magnetic bearing of a line.

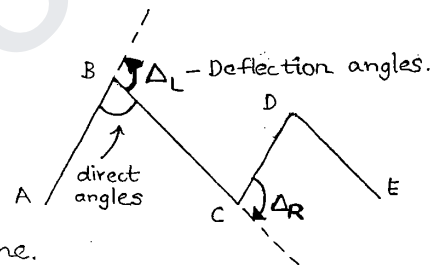
- direct angles

- deflection angles.

- prolongation of a straight line.

- locating the intersection point between any two straight lines.

- to lay off a horizontal angle



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Theodolite.

Transit Vernier

Non-transit vernier

* Transitting / Plunging / Reversing :-

It is the process of rotating the telescope in a vertical plane about horizontal axis

* face left observations:

When vertical circle is to the left side of an observer, observations made are called 'Face Left Observations'.

It is called as 'Telescope in normal condition and bubble is up.'

2. Cross Hair Ring Test

* face right observations:

When the vertical circle is to the right side of an observer, observations made are called as face right observations.

By transitting, face left can be made to face right and vice versa.

* Swinging of Telescope:

Rotation of telescope in the horizontal plane. If telescope is rotated in clockwise direction, it is a 'Right Swing' and if telescope rotated in anti-cw direction, it is a 'Left Swing'.

✓	FL	RS
	FR	RS

FL	LS
FR	LS

X

FL-RS is the best combination.

* Double sighting or Double Centering

It is the process of measuring the angles twice, once the telescope in normal condition and once the telescope in the inverted condition.

→ Main Parts of a Theodolite.

1. Telescope

2. Levelling head

3. Lower plate } vernier

4. Upper plate } plates

5. Plate Levels

6. Altitude bubble

7. Vertical Circle

8. A - Strands

9. T - frames

10. Tripod.

→ Working Operations :

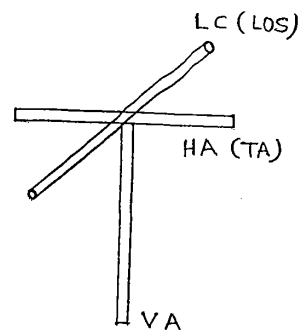
- (i) Both clamp screws are tightened; no movement of telescope.
- (ii) Releasing lower clamp screw, tightening upper clamp screw, object can be bisected without change in the vernier readings.
- (iii) Releasing upper clamp screw, tightening lower clamp screw, object can be bisected with changed vernier readings.

NOTE:

- ① Size of theodolite is the diameter of the main horizontal graduated circle. Generally it is 80 mm to 120 mm.

→ Fundamental Lines & their Relations.

1. Horizontal Axis or Trunion Axis
2. Vertical Axis
3. Line of collimation or LOS
4. Axis of Plate levels.
5. Axis of Attitude bubble.



* Fundamental relations

- (i) Axis of plate level $\perp^{\text{r}}$ Vertical axis.
- (ii) Horizontal axis $\perp^{\text{r}}$ Vertical axis.
- (iii) Line of collimation (LOS) $\perp^{\text{r}}$ horizontal axis.
- (iv) Axis of attitude bubble $\parallel$ LOS (when horizontal & vertical circle reads 0).

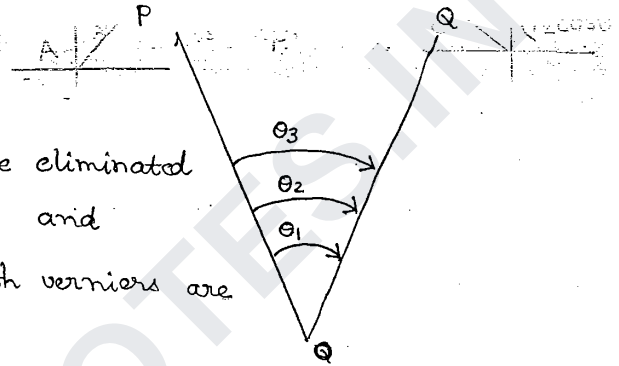
→ Temporary Adjustments.

1. Setting
2. Centering
3. Levelling.
4. Elimination of parallax.

→ Measurement of Horizontal Angle by Repetition method.

It is an accurate method because the degree of precision attained is to a much finer degree than the least count of the vernier.

$$\angle POQ = \frac{\theta_3}{3}$$

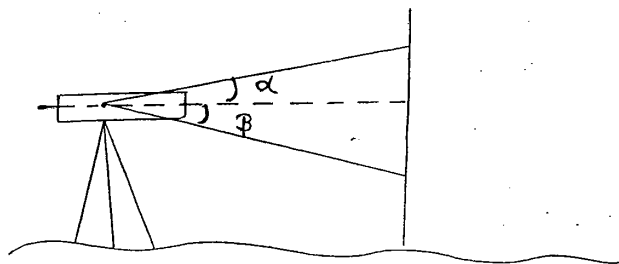


- * The following errors will be eliminated
- due to eccentricity of verniers and centres are eliminated as both verniers are used.
 - errors due to line of sight and trunion axis being out of adjustment are eliminated because the readings on both faces are taken.
 - error due to inaccurate graduations are eliminated.

→ Measurement of Horizontal Angle by Reiteration method.

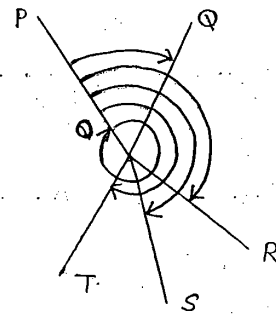
Reiteration method (or) Direction method (or) Method of Series. → Horizontal angles of a group having a common vertex point can be measured.

→ Measurement of Vertical Angles



α → angle of elevation

β → angle of depression.



→ Permanent Adjustments

1. Plate Level Test.

To make the plate bubble central to their run when the vertical axis of theodolite is truly vertical.

2. Cross Hair Ring Test.

To make the vertical cross hair lie in a plane perpendicular to the horizontal axis.

3. Azimuth test.

To make LOS $\perp$ to horizontal axis.

4. Spire test

To make horizontal axis $\perp$ to vertical axis.

5. Vertical Arc test.

To make the vertical circle indicate zero when the LOS is $\perp$ to vertical axis.

Q A theodolite is placed at A and a 3m long vertical staff is held at B. The depression angle made at reading of 2.5 m marking on staff is $6^{\circ}10'$. The horizontal distance b/w A & B is 2200 m. HI of at A is 1.2 m. and RL of point A is 880.88 m. Using combined correction, determine RL of point B in metres.

Correct staff reading on B

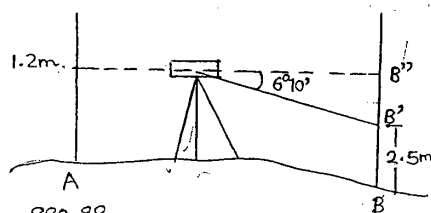
$$= 2.5 - 0.06735 \times (2.2)^2$$

$$= 2.174 \text{ m.}$$

$$\text{RL of B} = (880.88 + 1.2) - 2200 \tan 6^{\circ}10'$$

$$- 2.174$$

$$= \underline{\underline{642.204 \text{ m.}}}$$



8th nov,
SATURDAY

05. THEODOLITE TRAVERSING

1. Loose Needle Method.

Bearings of lines will be measured with theodolite wrt the compass fitted (in-built).

2. Fast Needle Method.

N-direction will be established separately by prismatic compass and bearings are measured. It is more accurate than loose needle method.

3. Method of Included Angles (Backbearing Method).

Used for GALE'S traverse tables.

4. Method of Deflection Angles.

It is suitable for open traverse.

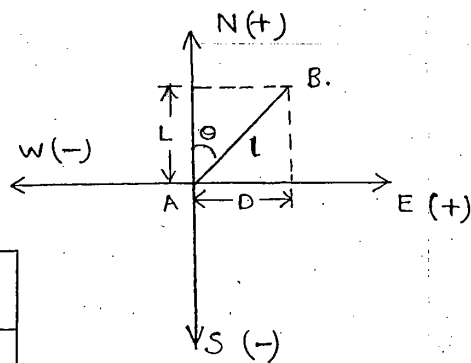
→ Traverse Computations.

Latitude, $L = l \cos \theta$

Departure, $D = l \sin \theta$

⊙ If L & D of a line are given,

length of a line, $l = \sqrt{L^2 + D^2}$
bearing of line, $\theta = \tan^{-1}\left(\frac{D}{L}\right)$



⊙ If Latitude and Departure of a line are equal,

$$\text{Bearing} = 45^\circ$$

→ Consecutive co-ordinates or Dependent co-ordinates.

These are the L & D of a succeeding station wrt preceding line



Co-ordinates of stations will be calculated from assumed origin.

Total co-ordinates of A is given as $(300, 300)$.

Now $\Rightarrow$ $L \begin{pmatrix} - \\ - \end{pmatrix}$ $D \begin{pmatrix} - \\ - \end{pmatrix}$ Now $\Rightarrow$ $L \begin{pmatrix} + \\ - \end{pmatrix}$ $D \begin{pmatrix} - \\ - \end{pmatrix}$

→ Closing Error & Angle of Misclosure

In a closed traverse ABCDEA,

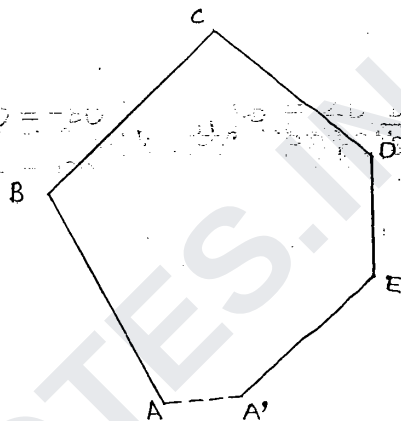
AA' is the closing error.

- In a closed traverse algebraic sum of latitudes of all lines is equal to zero.

- Algebraic sum of departures of all lines is equal to zero.

$$\text{ie } \Sigma L = 0$$

$$\Sigma D = 0$$



∴ Algebraic sum of latitudes of all line except AA' + latitude of AA' = 0.

$$\text{ie } \Sigma L + L' = 0$$

$L' = -\Sigma L$
$D' = -\Sigma D$

Similarly,

- Closing error, $e = \sqrt{(L')^2 + (D')^2}$

$$e = \sqrt{(-\Sigma L)^2 + (-\Sigma D)^2}$$

Bearing of AA', $\theta = \tan^{-1}\left(\frac{D'}{L'}\right) = \tan^{-1}\left(\frac{\Sigma D}{\Sigma L}\right)$

Angle of } $\theta = \tan^{-1}\left(\frac{\Sigma D}{\Sigma L}\right)$
misclosure }

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(10)

Q. Line	L	D
AB	100	-200
Bc	250	-100
CA	-600	500

$$\Sigma L = -250$$

$$\Sigma D = 200$$

$$\therefore L' = 250 \text{ \& } D' = -200$$

$$\text{Closing error, } e = \sqrt{250^2 + 200^2} = 320.16$$

$$\text{Angle of misclosure, } \theta = \tan^{-1}\left(\frac{200}{250}\right) = 38.65^\circ$$

$$\theta = \underline{\underline{N 38.65^\circ W}}$$

9th nov,
SUNDAY

⊙ If $A(L_1, D_1)$ & $B(L_2, D_2)$ are given,

$$\text{Length AB} = \sqrt{(L_2 - L_1)^2 + (D_2 - D_1)^2}$$

$$\tan \theta = \frac{D_2 - D_1}{L_2 - L_1}$$

⊙ If e be the closing error in the bearing of last line of a closed traverse, correction to first bearing of a line is given by:

$$\text{Correction to I} = \frac{1 \cdot e}{N}$$

$$\text{Correction to II} = \frac{2 \cdot e}{N}$$

$$\text{Correction to III} = \frac{3 \cdot e}{N}$$

$$\text{Correction to last line} = \frac{N \cdot e}{N} = e$$

→ Balancing the Traverse.

1. Bowditch's Method.

In this method, linear measurements are directly proportional to $\sqrt{l}$ and angular measurements are ~~inversely~~ ~~directly~~ proportional to $\sqrt{l}$, where l is length of a line in traverse. It is also called as a Compass Rule in which linear measurements are accurate than angular measurements.

$$\text{i.e. } LM \propto \sqrt{l}$$

$$AM \propto \frac{1}{\sqrt{l}}$$

$$\therefore \text{Correction to latitude, } C_L = \Sigma L \frac{l}{\Sigma l}.$$

$$\text{Correction to departure, } C_D = \Sigma D \frac{l}{\Sigma l}.$$

where $\Sigma l \rightarrow$ perimeter of traverse.

2. Transit Rule.

This is most suitable for theodolite traversing in which angular measurements are accurate than the linear measurements.

$$C_L = \Sigma L \cdot \frac{L}{L_s}$$

$$C_D = \Sigma D \cdot \frac{D}{D_s}$$

$L_s \rightarrow$ arithmetic sum of latitudes

$D_s \rightarrow$ arithmetic sum of departures.

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L	D
10	40
-20	-70
40	-20

$$C_L = \sum L \frac{L}{L_s}$$

$$= 30 \times \frac{10}{70} = \underline{\underline{4.286}}$$

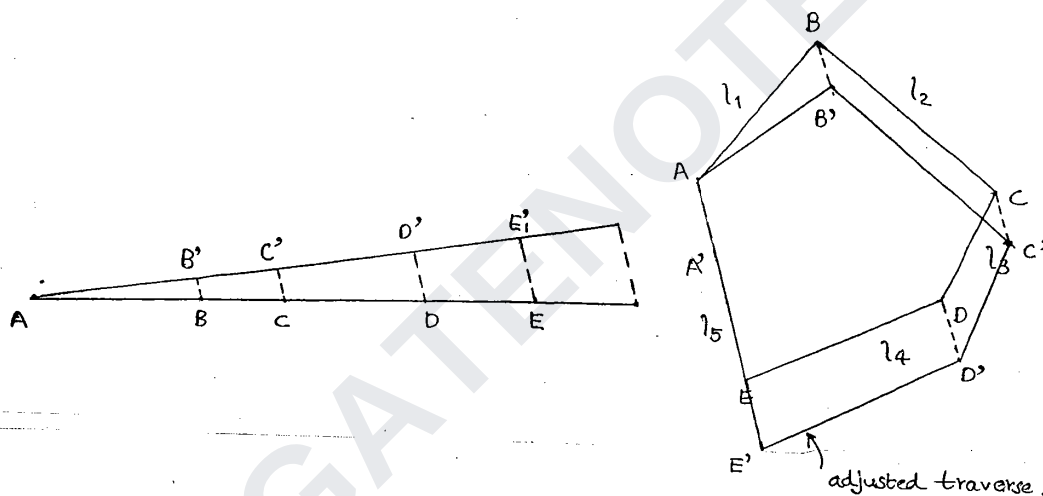
$$\sum L = 30 \quad \sum D = -50$$

$$L_s = 70 \quad D_s = 130$$

$$C_D = \sum D \frac{D}{D_s} = \frac{-50 \times 40}{130}$$

$$= \underline{\underline{15.385}}$$

3. Graphical Method.



4. Axis Method

It is adopted when angles are measured very accurately and corrections to be applied only to the lines.

$$\text{Relative error of closure} = \frac{\text{Closing error}}{\text{Perimeter}}$$

$$= \frac{e}{P} = \frac{1}{P/e}$$

$$\text{Degree of accuracy for linear measurements} = \frac{e}{P}$$

$$\text{Degree of accuracy for angular measurements} = \tan^{-1} \left(\frac{e}{P} \right)$$

9th nov,
SUNDAY

05. OMITTED MEASUREMENTS

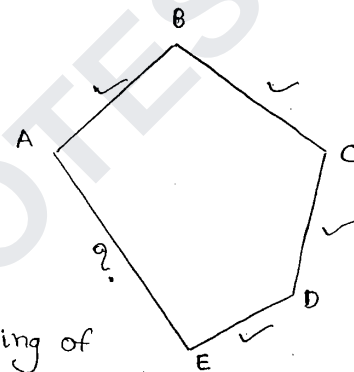
- Maximum number of omitted measurements = 2
as $\Sigma L = 0$ & $\Sigma D = 0$.

Case 1: Length & Bearing of one side is omitted.

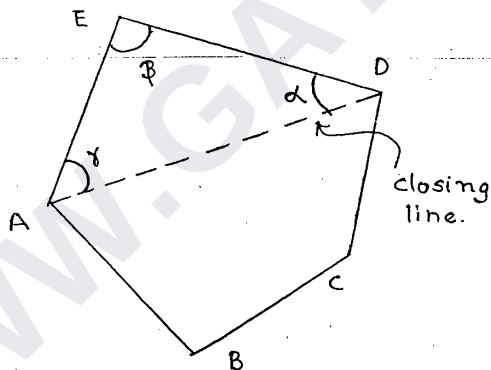
Latitude of EA = $-\Sigma L$

Departure of EA = $-\Sigma D$.

Length of EA = $\sqrt{(-\Sigma L)^2 + (-\Sigma D)^2}$



Case 2: Length of one side & bearing of adjacent side are omitted.



Line	Length	Bearing
AB	✓	✓
BC	✓	✓
CD	✓	✓
DE	?	✓
EA	✓	?

Applying sine rule in $\triangle ADE$,

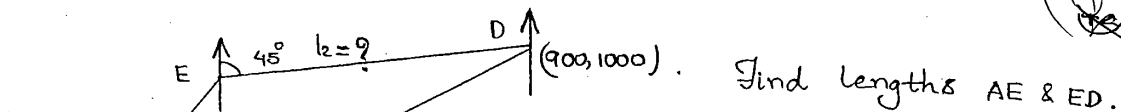
$$\frac{\sin \alpha}{EA} = \frac{\sin \beta}{AD} = \frac{\sin \gamma}{DE}$$

$$\sin \beta = \frac{\sin \alpha \cdot DA}{EA} \Rightarrow \beta \text{ is obtained.}$$

$$\therefore \gamma = 180 - (\alpha + \beta).$$

$$DE = \frac{\sin \gamma \cdot EA}{\sin \alpha}$$

Q.



Method I :

Latitude E wrt A = Latitude of E wrt D.

$$ie \quad 100 + l_1 \cos 30 = 900 + l_2 \cos 45$$

$$0.866 l_1 - 0.707 l_2 = 800$$

Departure of E wrt A = Departure of E wrt D.

$$200 + l_1 \sin 30 = 1000 + l_2 \sin 45$$

$$0.5 l_1 - 0.707 l_2 = 800.$$

7-28

Line	Length (m).	Bearing
PR	200	0°
QR	1000	45°
RS	907	180°
SP	?	?

$$\Sigma L = 0$$

$$\Rightarrow 200 \cos 0 + 1000 \cos 45 + 907 \cos 180 + SP \cos \theta = 0$$

$$\Sigma D = 0$$

$$\Rightarrow 200 \sin 0 + 1000 \sin 45 + 907 \sin 180 + SP \sin \theta = 0.$$

$$SP \cos \theta = -0.107$$

$$SP \sin \theta = -707.107$$

$$\Rightarrow \theta = \tan^{-1} \left(\frac{707.107}{0.107} \right) = 90^\circ$$

Complete Class Note Solutions
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For SP,

$$\text{latitude} = -\Sigma L = -106 \quad (S)$$

$$\text{departure} = -\Sigma D = -707.10. \quad (W).$$

$$\Rightarrow S 90^\circ W = 270^\circ$$

$$SP = \underline{\underline{707.107 \text{ m}}}$$

It is an indirect levelling in which

52. Latitude of Q wrt P = Latitude of Q wrt R.

$$100 + l_1 \cos 30 = 1000 + l_2 \cos 45,$$

$$0.866 l_1 - 0.707 l_2 = 900.$$

Departure of Q wrt P = Departure of Q wrt R.

$$200 + l_1 \sin 30 = 1000 + l_2 \sin 45.$$

$$0.5 l_1 - 0.707 l_2 = 800.$$

$$\Rightarrow l_1 = 273.224 \text{ m}$$

$$l_2 = \underline{\underline{938.314 \text{ m}}}$$

P-42

$$1. \quad \Sigma L = 400.75 + 100.25 - 199 - 300 \\ = 2$$

$$\Sigma D = 99.25 + 399.75 - 300.5 - 200.5 \\ = -2.$$

$$\therefore L' = -\Sigma L = -2 \quad (S)$$

$$D' = -\Sigma D = 2. \quad (E)$$

$$e = \sqrt{-2^2 + 2^2} = \underline{\underline{2.82 \text{ m}}}$$

$$\theta = \tan^{-1} \left(\frac{D'}{L'} \right) = 45^\circ \Rightarrow S 45^\circ E = \underline{\underline{135^\circ}}$$

$$2 \quad \Sigma L = 0$$

$$200 \cos \theta + 98 \cos 178 + 1 \cos 270 + 86.4 \cos 1^\circ = 0.$$

$$200 \cos \theta - 97.94 + 0 + 86.386 = 0$$

$$\cos \theta = 0.0577 \Rightarrow \theta = \underline{\underline{86.68^\circ}}$$

$$\Sigma D = 0$$

$$200 \sin 86.68^\circ + 98 \sin 178 + 1 \sin 270 + 86.4 \sin 1^\circ = 0.$$

$$1 = \underline{\underline{204.59 \text{ m}}}$$

$$3. \quad 150 + l_1 \cos 29.30^\circ = 1500 + l_2 \cos 45^\circ 45'$$

$$200 + l_1 \sin 29.30^\circ = 1300 + l_2 \sin 45^\circ 45'$$

$$l_1 = 712.71 \text{ m.}$$

$$l_2 = 1045.7 \text{ m}$$

$$4. \quad \Sigma L = 0.$$

$$250 \cos 60 + 300 \cos 290 + 350 \cos 190 + l_1 \cos \theta = 0.$$

$$\Rightarrow AD \cos \theta = L' = 117.08 \text{ m (N)}$$

$$\Sigma D = 0$$

$$250 \sin 60 + 300 \sin 290 + 350 \sin 190 + AD \sin \theta = 0$$

$$\Rightarrow AD \sin \theta = D' = 126.18 \text{ m (E)}$$

$$\theta = \tan^{-1} \left(\frac{126.18}{117.08} \right) = 47.14^\circ$$

$$AD = 172.13 \text{ m.}$$

5. In closed traverse ABDEA,

$$200 \sin 121 + 350 \cos 121 + 280 \sin 235 +$$

$$5 \sin 205 + BD \sin \theta = 0$$

$$BD \sin \theta = D' = 60.04$$

$$\Sigma L = 0$$

$$\Rightarrow 200 \cos 121 + 280 \sin 235 + 5 \cos 205 + BD \cos \theta = 0$$

$$BD \cos \theta = L' = 268.14 \text{ m.}$$

$$\theta = 12.62^\circ \quad \& \quad BD = 274.78 \text{ m.}$$

$$\begin{aligned} \angle CDB = \alpha &= (BB)_{CD} - (FB)_{DB} \\ &= 143 - 12.62 = 130.38 \end{aligned}$$

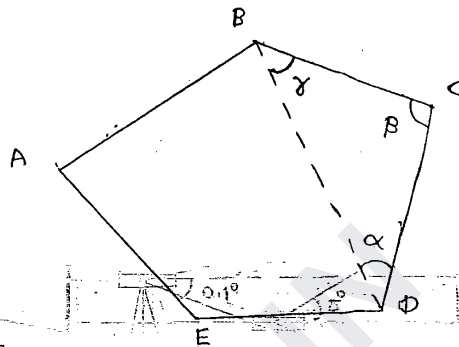
$$\frac{\sin \alpha}{BC} = \frac{\sin \beta}{BD} = \frac{\sin \gamma}{CD}$$

$$\frac{\sin 130.38}{350} = \frac{\sin \beta}{274.78} = \frac{\sin (180 - (130.38 + \beta))}{CD}$$

$$\beta = 36.73^\circ$$

$$CD = 102.5 \text{ m.}$$

$$\beta =$$



4th nov,
SUNDAY

45
44

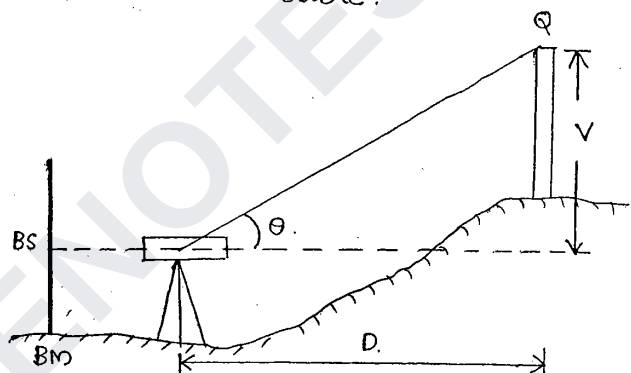
TRIGONOMETRICAL LEVELLING

It is an indirect levelling in which heights and distances can be calculated by measuring vertical angles.

Case 1: When the base of object is accessible.

$$V = D \tan \theta$$

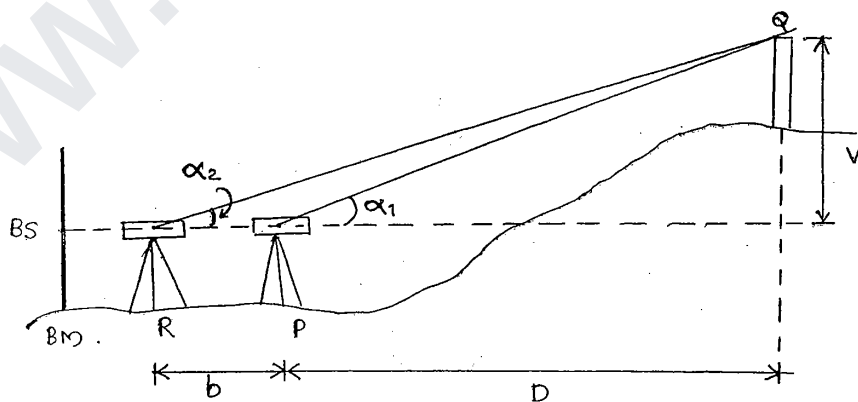
$$\text{RL of } Q = \text{RL of BM} + \text{BS} + v.$$



Case 2: Base of object is inaccessible.

Instrument stations & Object must be in same vertical plane.

a) Instrument asces are at the same level.



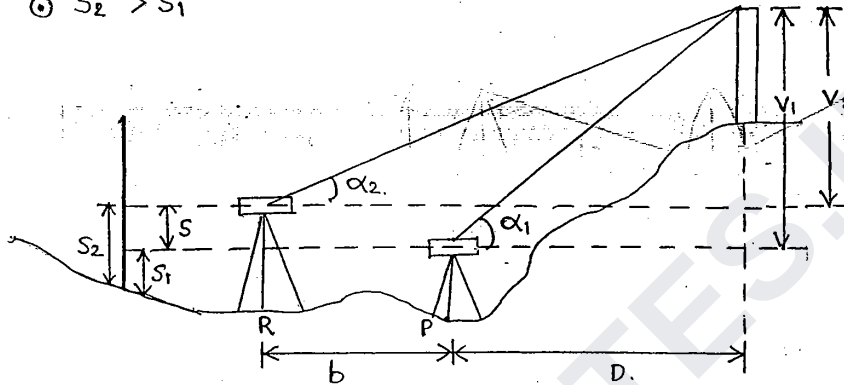
$$V = D \tan \alpha_1 = (D+b) \tan \alpha_2$$

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

$$RL \text{ of } Q = RL \text{ of BM} + BS + V_1$$

b) Instrument axes are at different levels

$$\odot S_2 > S_1$$



$$\begin{aligned} S_2 - S_1 &= S = V_1 - V_2 \\ &= D \tan \alpha_1 - (D+b) \tan \alpha_2 \end{aligned}$$

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

$$RL \text{ of } Q = RL \text{ of BM} + S_1 + V_1 \quad (\text{or})$$

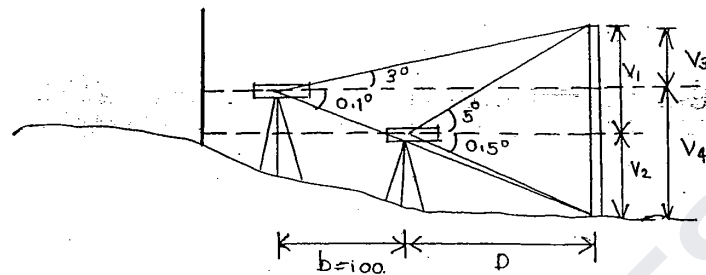
$$RL \text{ of BM} + S_2 + V_2$$

$$\odot S_2 < S_1$$

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

Q Horizontal distance b/w 2 stations P & Q is 100 m. The vertical angles from P & Q to the top of a vertical tower at T are 3° & 5° above horizontal resp'tly. The vertical angle from the P & Q to the base of tower are 0.1° & 0.5° below horizontal resp'tly. Stations P & Q and tower are in the same vertical plane, and also P & Q being on the same side of T.

Neglecting combined correction, height in m of tower is -?



$$h = V_1 + V_2 = V_3 + V_4$$

$$\Rightarrow D \tan 5^\circ + D \tan 0.5^\circ = (100 + D) \tan 3^\circ + (100 + D) \tan 0.1^\circ$$

$$0.0962 D = 5.415 + 0.054 D$$

$$0.042 D = 5.415$$

$$D = \underline{\underline{128.78 \text{ m}}}$$

$$\begin{aligned} \therefore \text{Height of object} &= D (\tan 5^\circ + \tan 0.5^\circ) \\ &= \underline{\underline{12.39 \text{ m}}} \end{aligned}$$

9th Nov,
SUNDAY

08. TACHEOMETRIC SURVEYING

- Tacheometer is a theodolite in which a special diaphragm with top central & bottom cross hairs b/w eye piece and object of a telescope

- Accuracy of tacheometer is 1 in 1000.

→ Systems of Tacheometric Surveying

1. Stadia-hair System.

(i) Fixed Stadia Hair.

(ii) Movable Stadia Hair.

2. Tangential Method.

3. By special instruments.

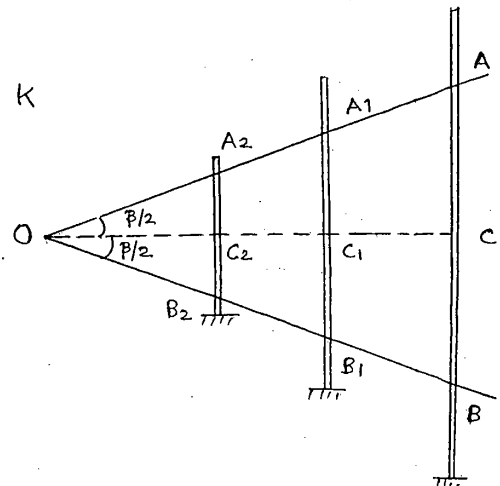
* Stadia hair method. (Fixed).

Principle: The ratio of perpendicular to the base is constant

$$\frac{OC}{AB} = \frac{OC_1}{A_1B_1} = \frac{OC_2}{A_2B_2} = \text{const} = K$$

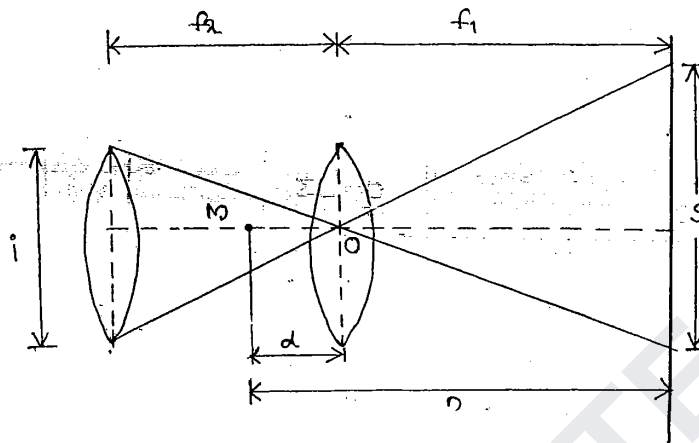
$$\begin{aligned} \tan\left(\frac{\beta}{2}\right) &= \frac{A_2C_2}{OC_2} \\ &= \frac{\frac{1}{2} A_2B_2}{K A_2B_2} \end{aligned}$$

$$\Rightarrow K = \frac{1}{2} \cot\left(\frac{\beta}{2}\right)$$



∴ If $\beta = 34' 22.64'' \approx 34' 22''$, $K = 100$

* Distance formula when LOS is horizontal.



M → centre of instrument

O → optical centre of objective & eyepiece.

s → staff intercept

i → stadia intercept.

d → distance b/w optical centre & centre of instrument

D → horizontal distance b/w object & instrument

f_1 & f_2 → focal lengths of objective & eyepiece resp.

$$\frac{f_1}{f_2} = \frac{s}{i} \rightarrow \text{①} \quad \left\{ \text{similar } \Delta s \right\}$$

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2}$$

Multiplying with $f f_1$,

$$f_1 = f + f \left(\frac{f_1}{f_2} \right) = f + f \left(\frac{s}{i} \right)$$

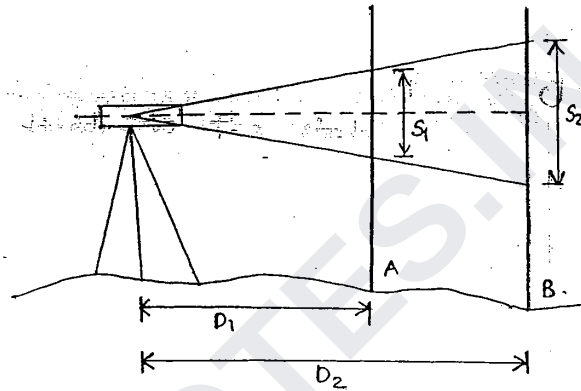
Horizontal distance, $D = f_1 + d$.

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

$$D = Ks + C$$

$$K = \frac{f}{i} \quad (\text{multiplicative const.})$$

$$C = f + d \quad (\text{additive const.})$$



→ Distance & Elevation formula.

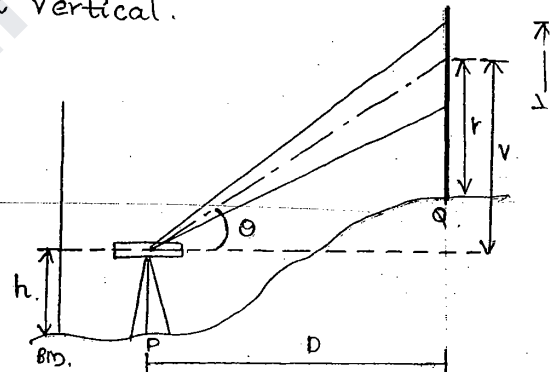
1. LOS inclined & Staff held vertical.

r → central hair reading
or axial hair reading.

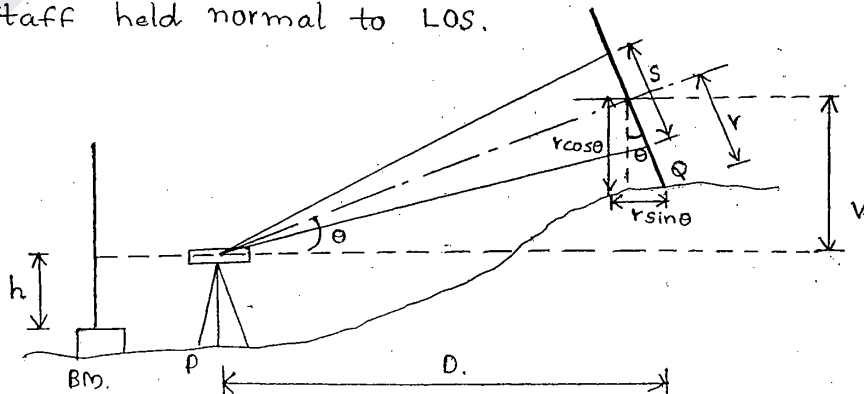
$$D = KS \cos^2 \theta + C \cos \theta$$

$$V = \frac{KS}{2} \sin 2\theta + C \sin \theta$$

$$RL \text{ of } Q = RL \text{ of } BM + h + v - r$$



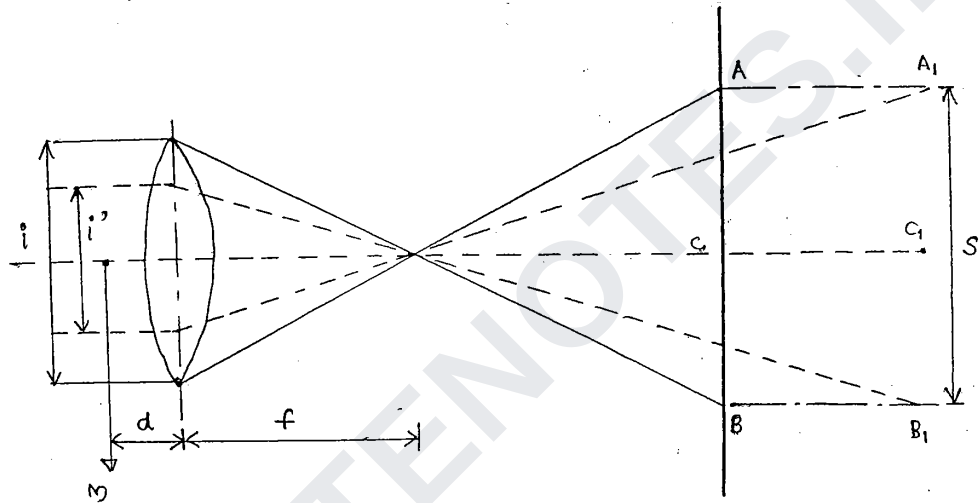
2. Staff held normal to LOS.



$$D = (Ks + C) \cos \theta + r \sin \theta$$

$$V = (Ks + C) \sin \theta$$

* Movable Hair Method.



$p \rightarrow$ pitch of micrometer screw.

$m \rightarrow$ no. of rotations

$$i = mp.$$

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

$$= \frac{f}{mp} s + f + d.$$

$$= \left(\frac{f}{p} \right) \frac{1}{m} s + (f + d).$$

$$D = \frac{K}{m} s + C.$$

- In this method, stadia interval is variable whereas the staff intercept is kept constant.

- Staff intercept is called as base.

- If base is horizontal, method is called as 'horizontal subtense bar method'

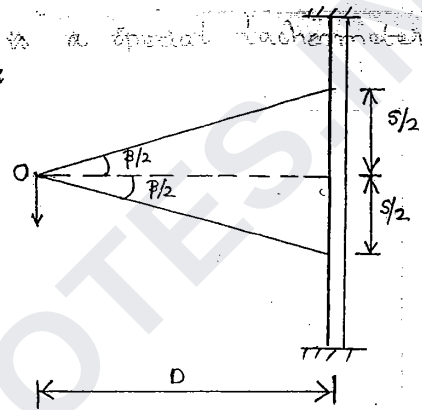
* Horizontal Subtense Bar Method:

$$\tan\left(\frac{\beta}{2}\right) = \frac{\left(\frac{S}{2}\right)}{D}$$

$$\therefore D = \frac{S}{2 \tan\left(\frac{\beta}{2}\right)}$$

when β is taken in ($^{\circ}$, '')

$$D = \frac{S}{2 \times \left(\frac{\beta}{2}\right)} = \frac{S}{\beta} \times 206265 ; \beta \text{ in seconds}$$



It is used for measurement of distances & elevations on sloping grounds. It is accurate for long sights.

→ Effect of Angular Error

$$D = \frac{S}{\beta}$$

$$\partial D = -\frac{S}{\beta^2} \cdot \partial \beta$$

$$= -\frac{1}{\beta} \left(\frac{S}{\beta}\right) \partial \beta$$

$$\Rightarrow \partial D = -D \frac{\partial \beta}{\beta}$$

NOTE:

⊙ Positive error in $\partial \beta$ will produce a negative error in D and vice versa

→ Tangential Method

- In this method readings corresponding to cross hairs are not required.

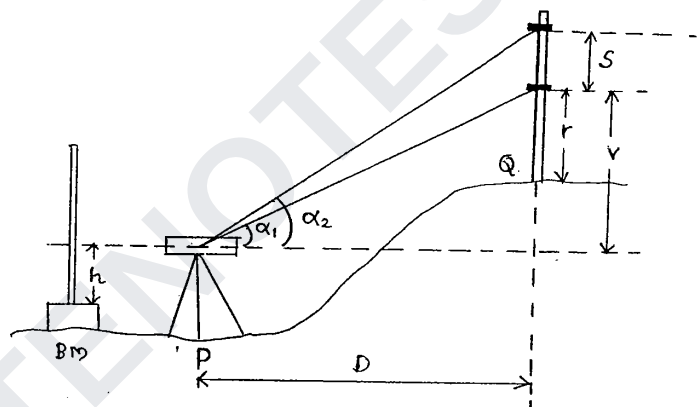
- Target staff is used for taking the vertical angles on the vanes fixed to the staff.

Case 1: Both are angle of elevation.

$$V = D \tan \alpha_1$$

$$V + S = D \tan \alpha_2$$

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



$$RL \text{ of } Q = RL \text{ of } B.M. + h + V - r.$$

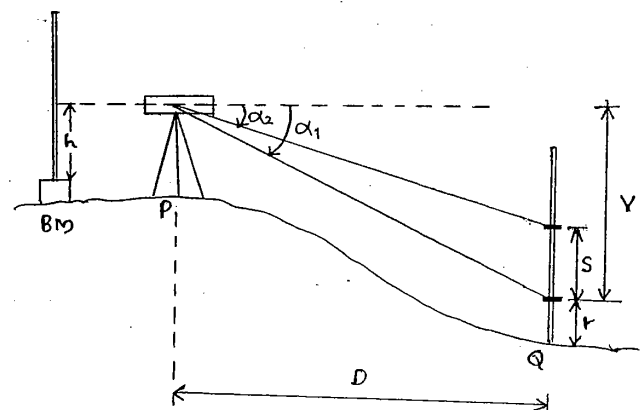
Case 2: Both are angles of depression.

$$V = D \tan \alpha_2$$

$$V - S = D \tan \alpha_1$$

$$S = D(\tan \alpha_2 - \tan \alpha_1)$$

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



$$RL \text{ of } Q = RL \text{ of } B.M. + h - V - r.$$

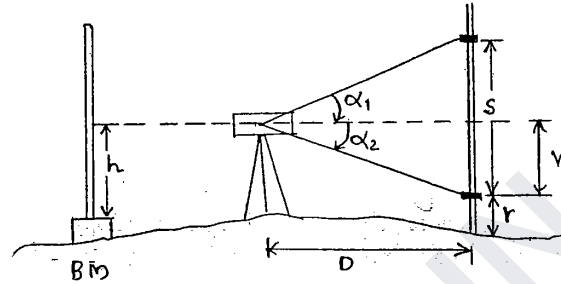
Case 3: One angle of elevation & One angle of depression

$$V = D \tan \alpha_2$$

$$S - V = D \tan \alpha_1$$

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

$$D = \frac{S}{\tan \alpha_1 + \tan \alpha_2}$$



$$RL \text{ of } Q = RL \text{ of } BM + h - V - r$$

→ Special Instruments

1. Beamann's Stadia Arc
2. Jeff Cott Direct Reading Tacheometer.

In this instrument, a special diaphragm is fixed with fixed pointer (red colour) and movable pointers (blue and black)

Horizontal distance = (difference in the readings of blue and red pointers) $\times 100$.

Vertical distance = (difference in the readings of black and red pointers) $\times 100$

3. Range Finder

It is suitable for inaccessible points below which the distance can be measured. It is more suitable for targetting the objects at a height of 500 km

4. Gradiometer

It is used in setting out grades in tangential method of tachymetry.

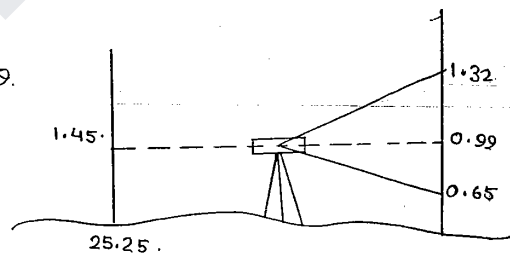
5. Omnimeter

It is a special tachymeter invented by Eckohold and used for tangential method of tachymetry.

P-63,

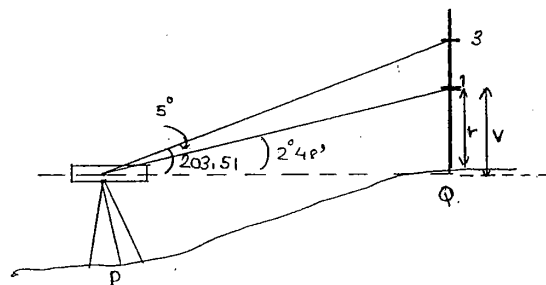
$$\begin{aligned} Q.1. \quad D &= \left(\frac{f}{i}\right) s + (f+d) \\ &= \frac{200}{4} (3-1) + (200+100) \times 10^{-3} \\ &= \underline{\underline{1003 \text{ m}}} \end{aligned}$$

$$\begin{aligned} Q.2. \quad \text{RL of SS} &= 25.25 + 1.45 - 0.99 \\ &= \underline{\underline{25.71 \text{ m}}} \end{aligned}$$



$$\begin{aligned} Q.3. \quad D &= Ks \cos^2 \theta + C \cos \theta \quad \left\{ \begin{array}{l} \text{if not given, assume } K=100 \\ C=0 \end{array} \right\} \\ &= 100 (2.985 - 2.225) \cos^2 (7^\circ 54') = \underline{\underline{74.564 \text{ m}}} \end{aligned}$$

$$\begin{aligned} Q.7. \quad V &= D \tan 2^\circ 48' \\ V+2 &= D \tan 5^\circ \\ \Rightarrow D &= 51.84 \text{ m.} \\ V &= 2.535 \text{ m.} \end{aligned}$$



$$\text{RL of Q} = 203.51 + 2.535 - 1 = \underline{\underline{205.045 \text{ m}}}$$

09

$$V = D \tan 1^{\circ}30'$$

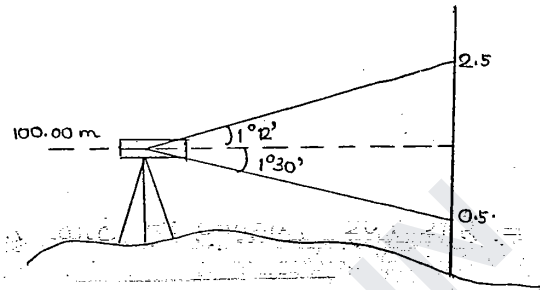
$$2-V = D \tan 1^{\circ}12'$$

$$D = 42.43 \text{ m.}$$

$$V = 1.11 \text{ m.}$$

$$\text{RL of } Q = 100 - V - 0.5$$

$$= \underline{\underline{98.41 \text{ m}}}$$



10.

$$D = 100 (2.265 - 1.545) \cos^2 4^{\circ}25'20''$$

$$= \underline{\underline{71.572 \text{ m.}}}$$

$$V = D \tan 4^{\circ}25'20'' = 5.535 \text{ m.}$$

$$\text{RL of } Q = 123.265 + V - 1.7905 + 1.28$$

$$= \underline{\underline{128.17 \text{ m}}}$$

6th nov,
SUNDAY

(S)

07. CURVES

Curves are used to change the direction of highways or railways

Curves are classified as follows:

1. Horizontal Curves

- (i) Simple Curves
- (ii) Compound Curves
- (iii) Reverse curves.
- (iv) Transition curves.

2. Vertical Curves

- (i) Summit curves
- (ii) Sag curves.

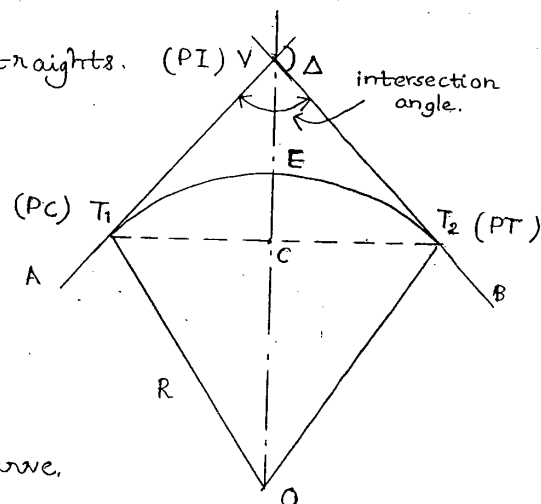
→ Simple Curves

- Arc connecting two straight. (PI) V Δ intersection angle.

T_1 → point of curve.

V → point of intersection.

T_2 → point of tangency.



- Point of Curve is the one where straight line becomes a curve.

- Point of tangency is where a curve becomes a tangent.

- Point of intersection is the intersection of two tangents VT_1 and VT_2

$AT_1 \rightarrow$ back tangent

$BT_2 \rightarrow$ forward tangent.

$\rightarrow$ intersection angle.

It is an angle b/w two tangents VT_1 and VT_2 .

$\rightarrow$ deflection angle (or) deviation angle (Δ)

$$\Delta = 180 - \text{intersection angle.}$$

$\rightarrow$ normal chord

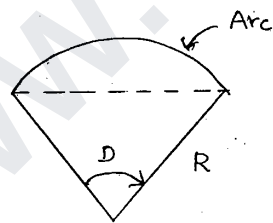
A chord b/w two successive regular stations on a curve. Generally it is equal to 'peg interval'.

$\rightarrow$ sub chord

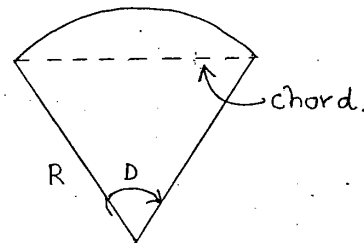
It is any chord shorter than the normal chord

$\rightarrow$ Designation of a Curve.

The sharpness of the curve is designated either by its radius or by its degree of curvature.



■ Arc definition



■ Chord definition.

* Degree of a Curve:

It is a central angle of the curve, i.e., subtended by an arc or chord of fixed length.

Curve is designated by radius in UK, by degree in USA, Canada, India.

For 30 m arc, $30 : 2\pi R = D : 360^\circ$

$$R = \frac{1718.9}{D}$$

For 20 m arc or chord, $20 : 2\pi R = D : 360$

$$R = \frac{1145.9}{D}$$

→ Elements of Simple Curve.

1. Tangent length

$$VT_1 = VT_2 = R \tan\left(\frac{\Delta}{2}\right)$$

2. External distance or Apex distance.

$$VE = R \left(\sec \frac{\Delta}{2} - 1 \right)$$

3. Mid ordinate or Versed sine.

$$CE = R \left(1 - \cos \frac{\Delta}{2} \right)$$

4. Length of Long chord.

$$T_1 C_2 T_2 = 2R \sin \frac{\Delta}{2}$$

5. Length of curve

$$l = R \Delta \quad (\Delta \text{ in radians})$$

$$= \frac{\pi R \Delta}{180} \quad (\Delta \text{ in degrees})$$

$$l = \frac{\pi \Delta}{180} \left(\frac{1718.9}{D} \right) = \frac{30 \Delta}{D} \quad (\text{for 30 m arc}).$$

$$\text{Similarly, } l = \frac{20 \Delta}{D} \quad (\text{For } 20 \text{ m arc}).$$

→ Methods for setting Simple Curve.

(i) Linear method.

(ii) Angular method.

Q. Calculate the necessary data for setting a simple curve for the following data

a) Chainage of PI = 55+60 (55 chains, 60 links).

b) Radius of curve = 300 m.

c) Deflection angle = 30°

d) Peg interval = 20 m. (length of chain)

$$\text{Length of tangent} = R \tan \frac{\Delta}{2}$$

$$= 300 \tan 15 = 80.38 \text{ m}$$

$$\text{Length of curve} = \frac{\pi R \Delta}{180} = \frac{\pi \times 300 \times 30}{180} = 157.07 \text{ m}$$

$$\text{Chainage of point of intersection} = 55 + 60$$

$$= 55 \times 20 + 60 \times 0.2$$

$$= 1112 \text{ m}$$

$$\text{Chainage of } T_1 = 1112 - 80.38 = 1031.62 \text{ m}$$

$$\text{Chainage of } T_2 = 1031.62 + \text{length of a curve}$$

$$= 1031.62 + 157.07 = 1188.69 \text{ m}$$

$$\text{First subchord length, } C_1 = 1040 - 1031.62$$

$$= \underline{\underline{8.38 \text{ m}}}$$

53

Last subchord length, $C_1 = 1188.69 - 1180$
 $= 8.69 \text{ m.}$

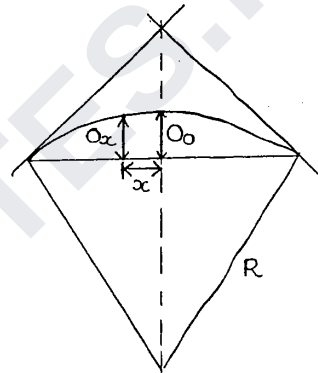
No. of normal chords $= \frac{1180 - 1040}{2} = 7 \text{ no.s}$

→ Linear method

* Offset from Long chord.

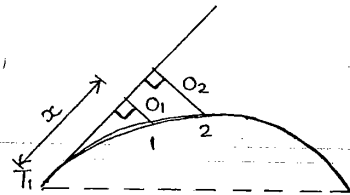
$$O_0 = R - \sqrt{R^2 - (L/2)^2}$$

$$O_x = O_0 - \frac{x^2}{2R}$$



* Perpendicular offset from tangent.

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



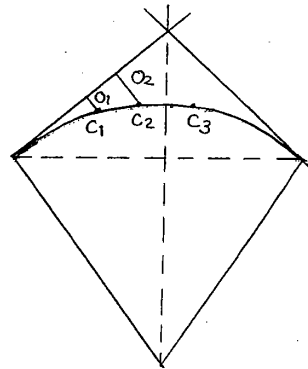
* By offsets from the chords
 Produced.

$$O_1 = \frac{C_1^2}{2R}$$

$$O_2 = \frac{C_2}{2R} (C_1 + C_2)$$

$$O_3 = \frac{C_3}{2R} (C_2 + C_3)$$

$$O_n = \frac{C_n}{2R} (C_{n-1} + C_n)$$



$$C_1 = 8.38 \text{ m}$$

$$C_2 = C_3 = C_4 = C_5 = C_6 = C_7 = C_8 = 20 \text{ m.}$$

$$C_9 = 8.69 \text{ m}$$

→ Angular method.

* Rankine's method of Deflection Angles (or)
Method of tangential Angles

Tangential angle, $\delta = \frac{1718.9 \times C}{R}$ minutes

$$\delta_1 = 1718.9 \times \frac{8.38}{300} = \underline{\underline{48.015 \text{ minutes}}}$$

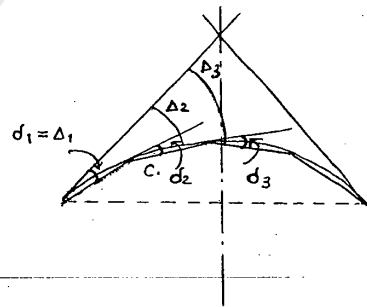
$$\delta_2 = \delta_3 = \delta_4 = \delta_5 = \delta_6 = \delta_7 = \delta_8 = \frac{1718.9 \times 20}{300} \\ = \underline{\underline{114.59'}}$$

$$\delta_9 = 1718.9 \times \frac{8.69}{300} = \underline{\underline{49.79'}}$$

Deflection angle, $\Delta_1 = \delta_1$

$$\Delta_2 = \Delta_1 + \delta_2$$

$$\Delta_3 = \Delta_2 + \delta_3$$



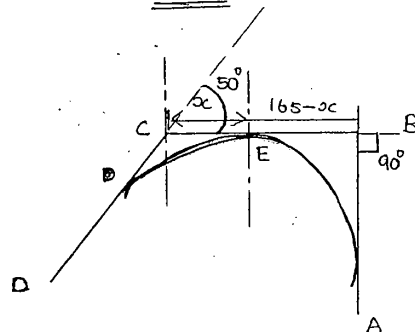
P-58.

01. Length of middle ordinate, $O_0 = R - \sqrt{R^2 - (L/2)^2}$
 $= 180 - \sqrt{180^2 - 30^2}$
 $= \underline{\underline{2.52 \text{ m}}}$

02. $R \tan 45 = 165 - x$

$$R \tan \frac{50}{2} = x$$

$$R = \underline{\underline{112.56 \text{ m}}}$$

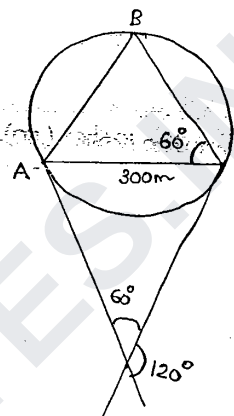


$$\begin{aligned}
 03. \quad O_0 &= 50 - \sqrt{50^2 - 30^2} \\
 &= \underline{\underline{10 \text{ m}}}
 \end{aligned}$$

$$07. \quad \Delta = 120^\circ$$

$$300 = R \tan\left(\frac{\Delta}{2}\right)$$

$$R = \frac{300}{\tan 60} = \underline{\underline{173.2 \text{ m}}}$$



$$\begin{aligned}
 08. \quad L &= 2R \sin \frac{\Delta}{2} \\
 &= 2 \times 600 \times \sin 30 = \underline{\underline{600 \text{ m}}}
 \end{aligned}$$

$$\begin{aligned}
 \text{Mid ordinate} &= R \left(1 - \cos \frac{\Delta}{2}\right) \\
 &= 600 (1 - \cos 30) = \underline{\underline{80.38 \text{ m}}}
 \end{aligned}$$

13.

$$09. \quad 2R \sin \frac{\Delta}{2} = R \tan \frac{\Delta}{2}$$

$$2 \sin \frac{\Delta}{2} = \tan \frac{\Delta}{2}$$

$$\Delta = \underline{\underline{120^\circ}}$$

→ Vertical Curves

— It is used to connect two different grade lines of railways or highways to smooth out the changes in vertical motion. It contributes safety, comfort and appearance.

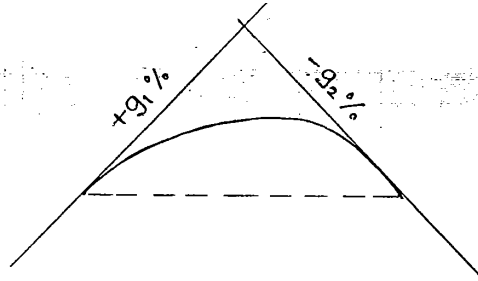
— Parabola or circular arc is the best suited vertical curve because the rate of change of grade is

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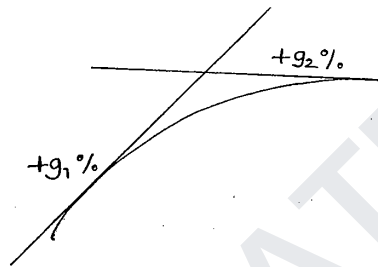
uniform.

* Summit Curves

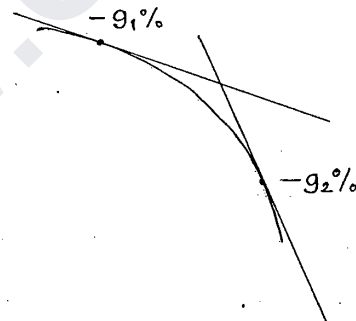
(i) $+G_1\%$ followed by $-G_2\%$



(ii) $+G_1\%$ followed by $+G_2\%$

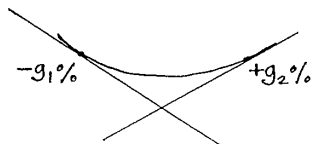


(iii) $-G_1\%$ followed by $-G_2\%$

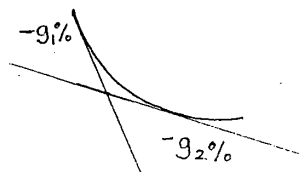


* Sag Curves

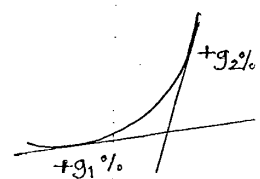
(i) $-g_1\%$, $+g_2\%$



(ii) $-g_1\%$, $-g_2\%$



(iii) $+g_1\%$, $+g_2\%$



55

→ Length of Vertical Curve.

$$L = \frac{g_1 - g_2}{r}$$

$r \rightarrow$ rate of change of grade

$r = 0.06\%$ per 20 m for 1st class railways (150-300 kmph)

$r = 0.03\%$ per 20 m for IInd class railways (< 150 kmph)

Q. If two grades of +1.2% and -0.9% meet to form a vertical curve, rate of change of grade is 0.1% per 30m the length of vertical curve is — ?

$$L = \frac{g_1 - g_2}{r} = \frac{1.2 - (-0.9)}{0.1/30} = \underline{\underline{630 \text{ m}}}$$

→ Transition Curves.

A curve of varying radius and varying curvature introduced b/w the tangent length and a circular curve or b/w two branches of compound curve or a reverse curve is known as 'transition or easement curve'.

* Advantages

(i) It allows a gradual transition of curvature from tangent to the circular curve or from circular curve to the tangent.

(ii) The radius of curvature increases or decreases gradually.

(iii) It is provided for the gradual change in super elevation in a convenient manner.

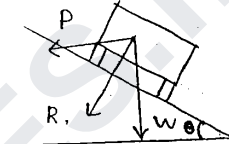
(iv) It eliminates danger of derailments overturning, slide slipping of vehicles and discomfort to the passengers.

* Super elevation (or) cant

$$\tan \theta = \frac{P}{W} = \frac{V^2}{Rg} = \frac{Bh}{hB}$$

$$\therefore \boxed{h = \frac{Bv^2}{Rg}} \Rightarrow \text{Roads.}$$

$$\boxed{h = \frac{Gv^2}{Rg}} \Rightarrow \text{Railways.}$$



$$\begin{aligned} \text{* Centrifugal ratio} &= \frac{P}{W} = \frac{1}{4} \quad (\text{for roads}) \\ &= \frac{1}{8} \quad (\text{for railways}). \end{aligned}$$

* Hands off Velocity (Design Speed).

Speed of vehicle in the absence of frictional force

$$\tan(\theta + \phi) = \frac{v^2}{Rg}$$

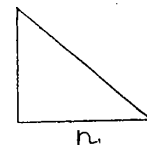
$$\Rightarrow \boxed{v = \sqrt{\tan \theta \times Rg}}$$

→ Length of transition curve

(i) By rate of superelevation.

$$\boxed{L = nh}$$

$$\left\{ \frac{1}{n} = \frac{h}{L} \right\}$$



$$= n \frac{Bv^2}{Rg} \Rightarrow \text{roads}$$

$$= n \frac{Gv^2}{Rg} \Rightarrow \text{railways ; } v \text{ in } \underline{\underline{m/sec}}$$

(ii) By time rate.

Let the time rate of application be α cm/sec.

Let superelevation be h cm

Let speed be V m/s.

$$L = V \times t = V \times \frac{h}{\alpha}$$

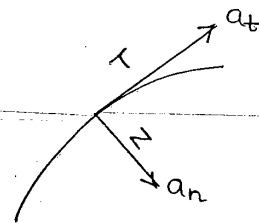
$$L = \frac{Bv^3}{Rg\alpha} ; \text{ for highways}$$
$$= \frac{Gv^3}{Rg\alpha} ; \text{ for railways.}$$

(iii) By rate of change of radial acceleration.

- It is the most efficient method

$$\text{Radial acceleration } \left. \begin{array}{l} \\ \end{array} \right\} a_n = \frac{v^2}{R}$$

Rate of change of radial acceleration, $\alpha = \frac{a_n}{t} = \frac{v^2}{Rt}$



$a_t \rightarrow$ tangential accl.
 $a_n \rightarrow$ radial accl.

$$\text{Length of transition curve, } L = vt$$

$$= \frac{v^3}{R\alpha}$$

$$\boxed{L = \frac{v^3}{R\alpha}} ; v \text{ in m/s}$$

NOTE:

• $\alpha = 0.0003 \text{ m/sec}^2/\text{sec}$ experienced by Mr. Shortt for comfort to the passengers.

$$\therefore \text{ length of transition curve, } L = \frac{v^3}{14R} ; v \text{ in kmph.}$$

⊙ Ideal transition curve is CLOTHOID (cubic spiral)

Q What is the length of transition curve, if speed of vehicle is 60 kmph, $\alpha = 0.3 \text{ m/sec}^2/\text{sec}$. and radius of curvature is 300 m.

$$L = \frac{v^3}{14R} = \frac{60^3}{14 \times 300} = 51.43 \text{ m}$$

