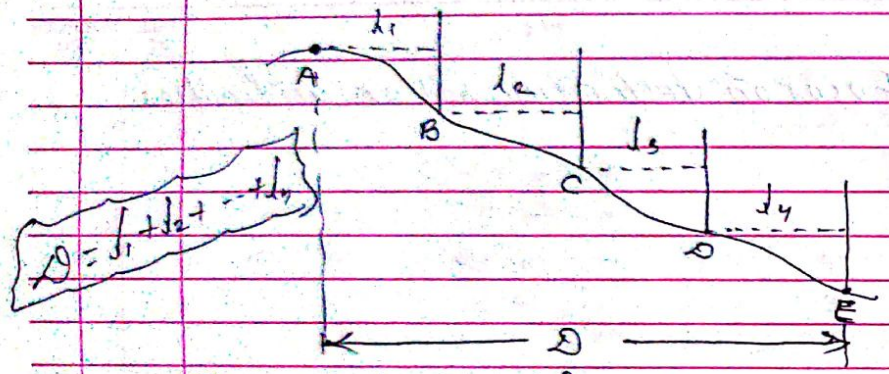
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Distance measurement in sloping ground.

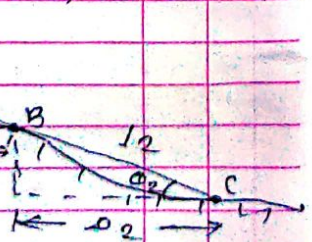
① Direct Method



The ⊥ distance from any point A or B or --- to ranging rod say l_1, l_2 --- l_n are measured. The sum of these lengths gives the horizontal distance bet^n starting & ending points.

② Indirect Method

Inclined distance bet^n A & B, B & C & --- say l_1, l_2 --- l_n are measured.



Then D_1, D_2 can be calculated as,

$D_1 = l_1 \cos\theta_1$ $D_2 = l_2 \cos\theta_2$

& soon.

total req. horizontal distance $= \Sigma l \cos\theta$.

The slope of line can be measured with the help of clinometer.

Page No.: Date: YOUVA

✻✻ Surveying:-

The science & art of determining the relative positi of various points above, on or below the surface of earth by means of measuring distances, direction and elevations using different instrumen The objective of surveying is the preparation of plans & maps of the areas. The development of surveying instruments & metho was occured with the development of human civilizati The art of surveying & preparation of maps has been practiced from the ancient times. Surveying is the first step for the execution of any project.

✻✻ Objects of Surveying:-

→ To determine the relative position or x, y & z of points on earth's surface.
→ To layout or to mark the position of proposed structure on ground.
→ To show the natural features like terrain, rivers, mountain, depression, lakes, forests etc, political features like town, village, districts, zones, country boundaries etc as well as different engineering works like roads, railways, canals, bridges, dams etc.
→ To determine areas, volumes & other related quantities.
→ preparation of plans & maps of the areas.

Classification of Surveying:-

a) Primary division
- plane survey
- Geodetic survey

1) plane Survey:-
→ plane survey is that type of surveying in which the earth surface is assumed as a plane & curvature of the earth is neglected.
→ The level line is considered as straight & all plumb lines are considered parallel.
→ All triangles formed by survey lines are considered as plane triangles.
→ Surveying covering area upto 260 KM^2 may be treated as plane survey because the difference in length betn the arc & its subtended curve chord on the earth surface for a distance of 18.2 KM is only 10cm
→ Less degree of accuracy in observation.

2) Geodetic Survey:-
→ The surveys in which the curvature of earth is taken into account & higher degree of accuracy in linear & angular observation is acheived are known as geodetic surveying.
→ These surveys extend over large areas more than 260 KM^2 & where the degree of accuracy required is high.
→ The objective of geodetic survey is to determine the precise position on the surface of earth.
→ Knowledge of spherical trigonometry is necessary for making measurement for geodetic surveys.

b) According to nature of field survey:-
1) Land Survey
The land survey may be further divided into following cases.

a) Topographical survey:-
This survey is carried out for determining the natural features of the country such as hills, valleys, rivers, lakes etc. It is also used for artificial objects such as canals, railways, roads, towns & villages.

b) Cadastral Survey:-
This survey is performed to determine the additional details such as boundaries of fields, houses & other property.

c) City Survey:-
This survey is performed in connection with town planning schemes such as drainage, water supply & for laying outplots, roads, streets etc.

d) Engineering Survey:-
This survey is performed for determining & collecting data for the design of engineering works such as roads, railways, reservoirs or water supply, sewage etc. The engineering survey may be further subdivided into the following classes

- Reconnaissance Survey:- for determing the feasibility & rough cost of scheme.
- Preliminary Survey:- for collecting more precise data to choose the best location for the work.
- Location Surveys: for setting out the work on the ground.

2) Hydrographic or Marine Survey:-
It deals with the mapping of large water bodies for the purpose of navigation, construction of harbor, prediction of tides & determination of Mean sea level.

3) Astronomical Survey:-
These are carried out for determing absolute locations of any points on the surface of earth. This consists in observation to the heavenly bodies such as sun or any fixed star.

c) According to purpose of surveying:-
1) Mine Survey: This is used for exploring mineral wealth.
2) Geological Survey: This is used for determing different strata in earth crust.
3) Archeological Survey:- This is carried out to prepare maps of ancient culture.
4) Military Survey:- This is carried out for Preparation of maps of area of military

d) According to the instrument used:-
1) Chain survey
2) Compass Survey
3) plane table survey
4) Levelling
5) Theodolite / Total station
6) photogrammetric -
7) GPS
8) Drone
9) Traverse survey
10) Aerial Survey -

* Principle of Surveying:-
There are two principles of surveying. They are

a) Working from whole to part
b) Location of new station by at least two measurements from fixed refrence points.

a) Working from whole to part:-
The working from whole to the part is done in surveying in order to ensure that errors & mistakes of one portion donot affect the remaining portion. It is very essential to establish control points. The main objective of working from whole to the part is to prevent the accumulation of errors and to control localize minor errors.

b) Location of a point / new station by at least two measurements from fixed refrence points.

→ The relative positions of the points to be surveyed should be located by measurement from at least two points of refrence, the position of which have already been fixed The location of any point (say) A from x & y can be verified by the measurement as shown below.

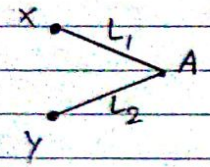


Fig: Locating point using distance from two points.

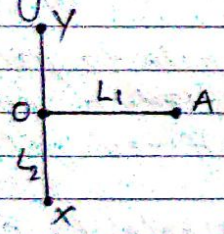


Fig: Locating point using perpendicular distance.

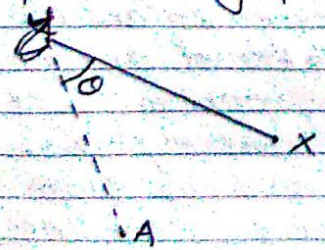


Fig: Locating point using distance & refrence angle.

The representative fraction of a map if scale is small & plan if scale is large

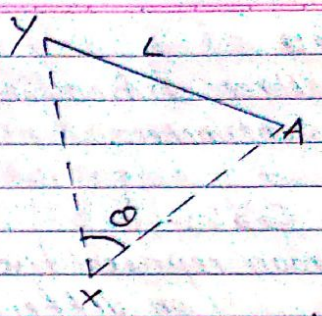


Fig: Locating point using distance & angle.

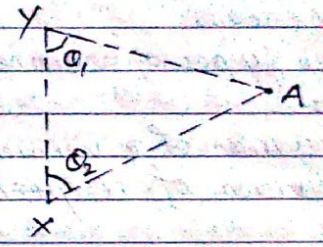


Fig: Locating point using angle from two different points.

✱ Scale:-

The proportion by which we either reduce or increase the actual size of the object on a drawing is known as scale. It is also defined as the ratio of distance on a map or plan to the distance on ground.

* Types of Scale:

① plain scale

↳ used to read only two dimensions such as meters & decimeters.

$R.F = \left[\frac{\text{area of map (a)}}{\text{Area of actual field (A)}}\right]^{1/2}$

② Diagonal Scale:-
- used to read three dimensions such as meters, decimeters & centimeters.

③ Comparative Scale:-
- It has a common representitive fraction but read in different measures.

④ Vernier Scale:-
- Used for measuring fractional parts of smallest division of main scale.
- There are two types of vernier scale.

ⓐ Direct vernier:-
The smallest division of a vernier is shorter than the smallest division of its ~~primary~~ main scale.

main (n−1) = n × vernier



ⓑ Retrograde Vernier:-
The smallest divison of a vernier is longer than the smallest division of its primary scale.

(n+1)

⑤ Scale of chords:- It is used to measure angles.

※ Representative Fraction (R.F)
The ratio of distance on the map to the distance on the ground is called R.F.
Eg:- When 1 cm on a map represents 50m on ground, then the R.F of scale is,

$$R.F = \frac{1}{50 \times 100} = \frac{1}{5000}$$

* plan:- plan is a graphical representation of the feature which is made after surveying of the area. It is made on horizontal plane. plan is nothing, it is the top view of surveying area.

※ Measurement in Nepal:-

In hilly region
Ropani - Anna - paisa - dam
1 Ropani = 16 Anna
1 Anna = 4 paisa
1 paisa = 4 dam

In Terai:-
1 Bigha = 20 katha
1 katha = 20 dhur

Interconversion,
1 Bigha = 13 Ropani
Again,
1 Ropani = 74 × 74 sq.feet
1 m = 3.281 feet

Nm A plot of land acquired for a factory site measures $25 \times 20\ cm^2$ on a village map drawn on a scale 1cm = 100m. What will be its area on a Topo sheet on 1:50,000 scale? Convert the field area in Ropani-Anna-paisa-dam system & Bigha-katha-dhur System what is its area is hectares?

→ Map scale ⇒ 1cm = 100m ⇒ $\frac{1}{10000}$

Map area ⇒ 25cm × 20cm
Field area ⇒ (25 × 100) × (20 × 100) = 50 00000 m^2

Area in Topo sheet = $\frac{50,00,000}{50,000 \times 50,000} = \frac{1}{500}\ m^2$

1 Ropani = 74 × 74 Sq feet = 508.69 sq.m

508.69 sq.m ⟶ 1 Ropani
1 sq.m ⟶ $\frac{1}{508.69}$ Ropani
50 00000 sq.m ⟶ $\frac{50\,00000}{508.69}$ Ropani
= 9829.169 Ropani

In R-A-P-D System.

Ropani	Anna	paisa	Dam
9829	2	2	3.
	[0.169 x 16 = 2.704]	[0.704 x 4 = 2.816]	[0.816 x 4 = 3.264]

18 Ropani = 1 Bigha

1 Ropani = $\frac{1}{18}$ Bigha

9829.169 Ropani = $\frac{9829.169}{18}$ Bigha

= 756.089 Bigha

In B-K-D system

Bigha	katha	Dhur
756	17	16
	[0.89 x 20 = 17.8]	[0.8 x 20 = 16]

* Field area in hectares:

1 hectares = 10,000 m^2

1 m^2 = $\frac{1}{10,000}$ hectares

50,00000 m^2 = $\frac{1}{10,000}$ x 50,00000 hectares.

= 500 hectares.

** Levelling:-

Levelling is the process of determining relative elevations of points with respect to a datum line. It deals with measurements in vertical plane.

* Basic terms used in Levelling

a) Level surface:-
- Surface parallel to mean spherical surface of earth.
- e.g: surface of still water in lake.
- A level surface may be regarded as curved surface if every point on it are equidistance from centre of earth.

b) Level Line:-
- Any line lies in level surface is called level line.

c) Horizontal surface:-
- Surface tangent to the ~~horizontal~~ level surface.
- Any line lying in horizontal surface is called Horizontal line

d) Datum:-
- Datum is any arbitary surface or line from which elevation of points are measured.

e) Elevation:- / Reduced Level (R.L)
- It is the vertical distance above or below the datum

f) Bench Mark:-
- It is a fixed refrence point of known elevation.

g) Back sight:-
- It is a staff reading taken on a point of known

elevation as on a bench mark or a change point.
- It is the first staff reading taken after the level is setup.
- It is also called a plus sight.

h) Fore Sight (FS)
- It is a staff reading taken on a point whose elevation (RL) is to be determined as on a change point.
- It is the last staff reading taken before shifting of level to another position.
- It is also called minus sight.

i) Intermediate Sight (IS)
- It is any other staff reading taken on a point of unknown elevation from the same setup of the level.
- All the sights taken betn back sight & fore sight are intermediate sight.

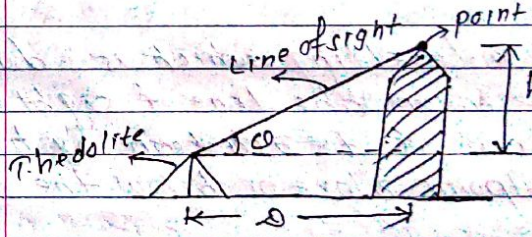
j) Change point / Turning point:
→ It is the point denoting the shifting of level.
→ It is the point on which the fore & back sights are taken.

k) Station:-
→ It is a point whose ~~station~~ elevation is to be determined.
→ It is the point where the staff is held & the reading taken during process of levelling.

l) Height of Instrument (H.I)
It is the elevation (RL) of plane of collimation (or plane of sight) when the instrument is correctly ~~levelled~~ leveled. It is equal to reduced level of B.M + back sight (BS)

※ Methods of Levelling:-

a) Trigonometric levelling:-
→ In this method of levelling, elevation of points are computed by measuring horizontal distances & vertical angles.
→ It is also called stadia levelling or indirect levelling.



h = D tanθ
θ ⇒ vertical angle
D ⇒ horizontal distance

b) Barometric levelling:-
→ It is based on the difference of atmospheric pressure between the points.
→ According to this method, atmospheric pressure is inversely proportional to the altitude.
→ This method is used for rough & exploratory works which can give an accuracy of ± 1m to ± 3 m.

c) Spirit levelling (direct / optical levelling)
→ It is the branch of levelling in which the vertical distances wrt horizontal line may be used to determine the relative difference in elevation betn two adjacent points. A horizontal plane of sight is established by means of spirit level. In spirit levelling, a sprit level & sighting devices (telescope) are combined & vertical distances are measured by observing on graduated rods placed on the points.
→ It is the most precise method of determining elevations & commonly used by engineers.

d) Fly levelling:-
→ Fly levelling is a levelling which is done to find by how much the line of sight is above the benchmark & to ascertain by how much is the nextpoint above or below the line of sight.

e) Reciprocal levelling:-
→ Levelling betn two widely separated points in which observations are made in both directions to eliminate the effects of atmospheric fraction & the curvature of earth.

f) Differential levelling:-
→ The establishment of differences in elevation betn two or more points wrt datum.

g) Profile levelling:-
The process of determining the elevations of a series of points at measured intervals along a line such as centreline of road or natural feature such as stream bed etc.

h) Precise levelling
→ Highest order of levelling work.
→ In this type of levelling, reading observed & recorded to decimals of a millimeter.

※ Principle of levelling:-
The principle of levelling is furnishing a horizontal line of sight & finding the vertical distances of the points above or below the line of sight. The line of sight is provided with a sprit level & a graduated level staff is used for measuring the height of line of sight above staff station

※ Booking Method & their reductions:-
There are two main Methods of booking levels
a) Rise & fall method
b) Height of collimation Method.

a) Rise and fall Method:-
In this method, the difference of level between two consecutive points for each setting of the instrument, is obtained by comparing their staff readings. The difference betn their staff reading indicates a rise if the back staff

reading is more than the foresight & a fall if it is less than foresight.

Arithmetic check

$\Sigma B.S - \Sigma F.S = \Sigma Rise - \Sigma Fall$ = RL of last point − RL of first point.

Eg

B.S (m)	F.S (m)	Rise (m)	Fall (m)	RL (m)	Remarks
1.350		–	–	1325.325	BM 10
0.503	1.200	0.15	–	1325.475	A
0.150	2.100	–	1.597	1323.870	B
3.800	0.250	–	0.1	1323.770	C
	0.450	3.35	–	1327.128	BM 11

$\Sigma B.S = 5.803m$ $\Sigma F.S = 4m$ $\Sigma Rise = 3.5m$ $\Sigma Fall = 1.697m$

Check

$\Sigma B.S - \Sigma F.S = 5.803 - 4 = 1.803m$

$\Sigma Rise - \Sigma Fall = 3.5 - 1.697 = 1.803m$

$(RL)_{last\ point} - (RL)_{first\ point} = 1327.128 - 1325.325 = 1.803m$

(b) Height of collimation Method:-

In this method, height of instrument is calculated for each setting of the instrument by adding backsight to the elevation of B.M. The elevation of reduced level of the turning point is then calculating by subtracting fore sight from H.I. If there are some intermediate points, the R.L of those points is calculated by subtracting the intermediate sight from the height of instrument for that setting.

Eg:- (Put - 16)

Arithmetic check

$\Sigma B.S - \Sigma F.S = (RL)_{last\ point} - (RL)_{first\ point}$

B.S	F.S	H.I	RL	Remarks
2.543		1294.153m	1291.61m	B.M
2.094	1.645	1294.602m	1292.508m	A
1.916	1.436	1295.082m	1293.166m	B
2.725	0.956	1296.851m	1294.126m	C
	1.855		1294.996m	D

$\Sigma B.S = 9.278$ m $\Sigma F.S = 5.892$

$\Sigma BS - \Sigma FS = 9.278 - 5.892 = 3.386m$

$(RL)_{last} - (RL)_{first} = 1294.996 - 1291.61 = 3.386m.$

※ The following consecutive reading were taken with a dumpy level and a 4m levelling staff on a continuously sloping ground at 30m intervals:

0.680, 1.455, 1.855, 2.330, 2.885, 3.380, 1.055, 1.860, 2.265, 3.540, 0.835, 0.945, 1.530 & 2.250

The RL of starting point was 80.750m.

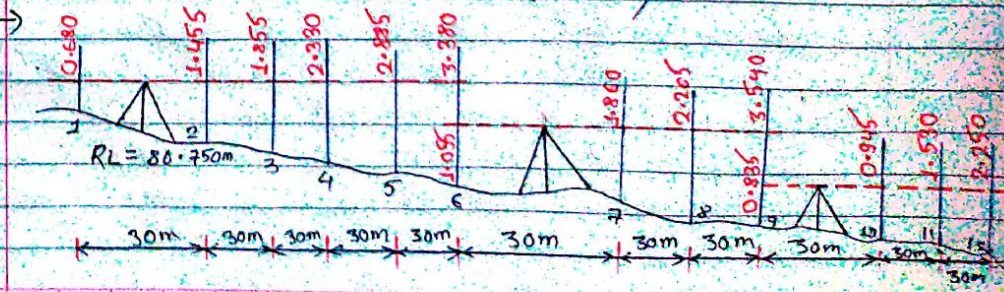
a) Rule out a page of level book & enter the above reading.

b) Carryout reduction of height by both collimation as well as Rise & Fall Method.

c) Apply the arithemetic checks.

d) Determine the gradient of the line joining the first & last point.

→



SN	chainage (m)	BS (m)	IS (m)	FS (m)	Rise (m)	Fall (m)	H.I (m)	RL (m)	Remarks
1	0	0.680			–	–	81.430	80.750	BM
2	30		1.455		–	0.775	81.430	79.975	
3	60		1.855		–	0.400	81.430	79.575	
4	90		2.330		–	0.475	81.430	79.100	
5	120		2.885		–	0.555	81.430	78.545	
6	150	1.055		3.880	–	0.495	79.105	78.050	C.P
7	180		1.860		–	0.805	79.105	77.245	
8	210		2.265		–	0.405	79.105	76.840	
9	240	0.835		3.540	–	1.275	76.400	75.565	C.P
10	270		0.945		–	0.110	76.400	75.455	
11	300		1.530		–	0.585	76.400	74.870	
12	330			2.250	–	0.720	76.400	74.150	
Sum		2.570		9.170		6.600			

Arithmetic check

$\Sigma BS - \Sigma FS = 2.570 - 9.170 = -6.600$

$\Sigma Rise - \Sigma Fall = 0 - 6.600 = -6.600$

$(RL)_{last} - (RL)_{first} = 74.150 - 80.750 = -6.600$

Gradient = $\frac{\text{difference in level}}{\text{horizontal distance}}$

$= \frac{6.6}{330}$

$= \frac{1}{50}$ ✱.

✱ Sources of error in Levelling:-

1) Personal error:-
- The instrument may not be leveled perfectly.
- The focusing of the eyepiece & object glass may not be perfect & parallax may not be eliminated.
- The reading of stadia hair rather than the central collimation hair may be taken by mistake.
- A wrong entry may be made in level book.
- The staff may not be properly & fully extended.

2) Instrumental Error:-
- The line of collimation may not be parallel to the axis of the bubble tube.
- The internal arrangement of the focusing tube is not perfect.
- The graduation of the levelling staff may not be perfect.

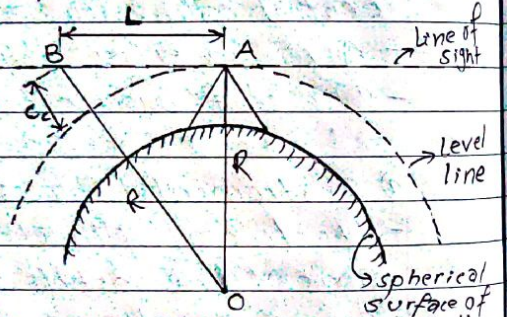
3) Natural Error:-
- When the distance of sight is long, the curvature of the earth may affect the staff reading.
- The effect of refraction may cause a wrong staff reading.
- The effect of high winds & a shining sun may result in wrong staff reading.

* Effect of curvature & Refraction:-

① Effect of curvature:-

→ For long sights, the curvature of the earth affects staff reading. The line of sight is horizontal but the level line is curved & parallel to mean spheroidal surface of the earth.

→ The vertical distance between the line of sight & level line at a particular place is called curvature correction.



→ Due to curvature, object appears lower than they really are.

Let, L = Horizontal distance, (KM)
R = Radius of earth
C_c = curvature correction.

Then, In $\triangle OAB$

$OB^2 = OA^2 + AB^2$

$\Rightarrow (R + C_c)^2 = R^2 + L^2$

$\Rightarrow R^2 + 2RC_c + C_c^2 = R^2 + L^2$

$\Rightarrow 2RC_c = L^2$ [$\because C_c^2$ can be neglected in comparison of radius of earth]

$\Rightarrow C_c = \frac{L^2}{2R}$

put $R = 6370$ KM, $C_c = \frac{1000 \times 1000 (m^2) \times L^2}{2 \times 6370 \times 1000 (m)}$

$$C_c = 0.0785\, L^2 (m)$$

→ Curvature correction (C_c) is always taken as negative.
Hence,
True staff reading = Observed staff reading − C_c

② Effect of refraction:-

→ Rays of light are refracted when they pass through layers of ~~earth~~ air varying density. so, when long sights are taken, the line of sight is refracted towards the surface of earth in curved path.

→ Due to the effects of refraction, object appear higher than they really are.

→ However, on the average the refraction correction (C_r) is taken as one seventh of curvature correction (C_c)

i.e, $C_r = \frac{1}{7} C_c$

$\Rightarrow C_r = \frac{1}{7} \times \frac{L^2}{2R}$

$\Rightarrow C_r = \frac{1}{7} \times 0.0785\, L^2$

$\Rightarrow C_r = 0.0112\, L^2$ (meter)

→ Refraction correction (C_r) is taken as positive.
Hence,
True staff reading = Observed staff reading + C_r

* Temporary adjustment of level
  - ↳ Setting up the level
  - ↳ levelling
  - ↳ Focusing the eypiece
  - ↳ Focusing the objective

Types of level
- Dumpy level
- wye level
- Reciprocal level
- Tilting

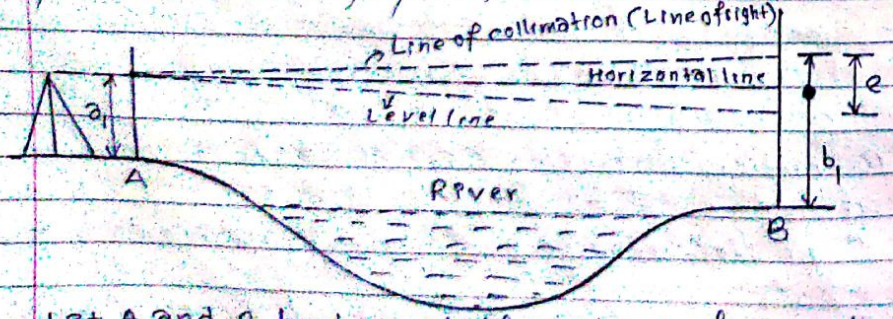
* Reciprocal levelling:-

→ Levelling betⁿ two widely separated points in which observation are made in both directions to eliminate the effect of refraction & curvature of earth.

→ When it is not possible to set up the level midway between two points such as levelling across large water bodies, reciprocal levelling is employed.

→ Forexample, if the two points are located on opposite banks of river, it wouldnot be possible to set up the instrument in river bed. The difference of elevation betⁿ two points is determined by reciprocal levelling by first setting up the instrument at one bank & holding the staff at other bank & then interchanging the position of staff & instrument.

→ It is used to measure level difference betⁿ two points in case of pond, lake, rivers etc.

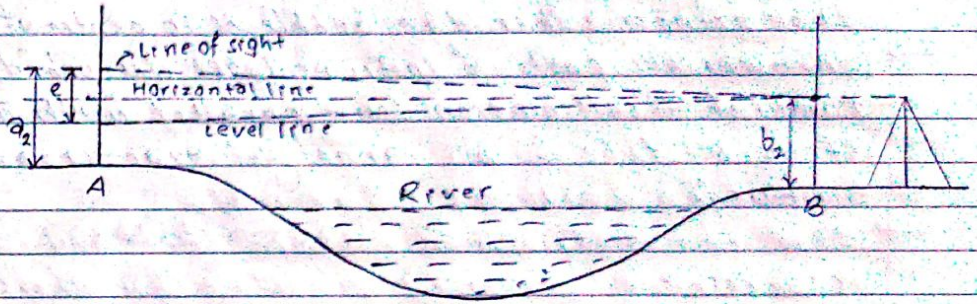


Let A and B be two staff position of two bank of river. Firstofall, an instrument is setup near to A. The reading on staff A and B be a_1 & b_1 respectively. Let, h = true difference of level betⁿ A & B.

Then, Assuming staff A is at higher position than B,

$$h = (b_1 - e) - a_1$$

$$h = (b_1 - a_1) - e \quad \text{-----} \quad (1)$$



Now, the instrument is setup near to B. Let the reading on staff A & B be a_2 & b_2 respectively. Then,

$$h = b_2 - (a_2 - e)$$

$$h = (b_2 - a_2) + e \quad \text{---} \quad (2)$$

Adding equation (1) & (2),

$$\boxed{2h = (b_1 - a_1) + (b_2 - a_2)}$$

Also,

$$h = \frac{(b_1 - a_1) + (b_2 - a_2)}{2}$$

The true difference of level betⁿ A & B is equal to the mean of the two apparent differences of level.

Also, equating (1) & (2),

$$(b_1 - a_1) - e = (b_2 - a_2) + e$$

$$e = \frac{(b_1 - a_1) - (b_2 - a_2)}{2}$$

The combined error is equal to half of diffⁿ of apparent differences of level.

✻ Two peg test:- (permanent adjustment of level)

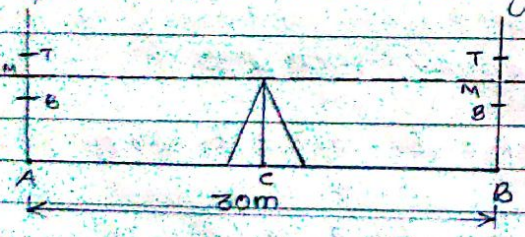
→ This test is performed to check whether the line of collimation is parallel with axis of bubble tube or not when the bubble is in centre of its run.

→ When various parts of instrument get disturbed, the line of collimation isnot parallel with axis of bubble tube & we cannot get exact reading on staff.

Procedure:-

→ Select two points A & B at a distance 2m apart on a firmly level ground. Set the pegs at both point.

→ setup the instrument at mid C of AB. Take the staff reading A & B, keeping the bubble central of its run. Let the reading be a_1 & b_1 resp.



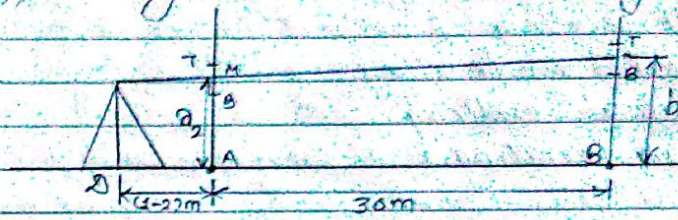
Now, staff reading at A, $(a_1) = \left(\frac{T+B+M}{3}\right)_A$ (Mean of top, middle & bottom reading)

Staff reading at B, $(b_1) = \left(\frac{T+B+M}{3}\right)_B$

Let, A is at higher side, then level difference betⁿ A & B is,

$$h_1 = |b_1 - a_1|$$

→ Now, shift the level to a point D near to A. Take the staff reading A & B. Let the reading be a_2 & b_2 resp.



Staff reading at A, $(a_2) = \left(\frac{T+B+M}{3}\right)_A$

Staff reading at B, $(b_2) = \left(\frac{T+B+M}{3}\right)_B$

The level difference betⁿ A & B when instrument is near to A,

$$h_2 = |b_2 - a_2|$$

Then, Error = $|h_2 - h_1|$

$$\text{Precision} = \frac{1}{10,000}$$

$$\text{Accuracy} = \frac{1}{(\text{Length}/\text{Error})}$$

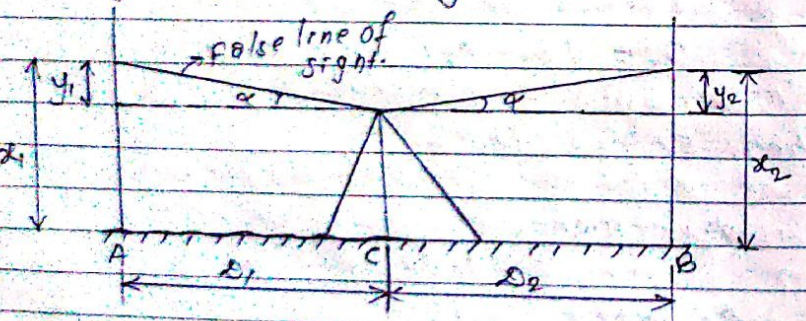
If the accuracy is greater the precision, the instrument is in adjustment otherwise, the line of sight is not parallel to axis of bubble tube & it is necessary to repair the instrument for correction.

※ Types of spirit levelling:-

- Differential levelling
- profile levelling
- Cross-sectional levelling
- Reciprocal levelling
- Precise levelling.

※ Balancing backsight & foresight:-

When the difference in elevation bet^n any two points is determined from a single setup by backsighting on one point & foresighting on other, the error due to non-parallelism, curvature & refraction occur. Such error can be eliminating by making length of two sights equal.



Let A & B be two point on ground. C be the midpoint of AB. The instrument is setup at point 'C'

Let,

Staff reading at A = x_1

Staff reading at B = x_2

Now,

True staff reading at A = $x_1 - y_1$

True staff reading at B = $x_2 - y_2$

Also, from figure,

$y_1 = D_1 \tan\alpha$

$y_2 = D_2 \tan\alpha$

The correct difference in level bet^n A & B is,

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

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

$= (x_1 - x_2) + [D_2 \tan\alpha - D_1 \tan\alpha]$

Since, $D_1 = D_2$,

so,

correct diff^n in level = $x_1 - x_2$

∴ If the instrument is setup in the middle of two points, the difference in elevation can be directly calculated by taking difference of two reading (x_1 & x_2) & no correction is required for inclination of line of sight.

※ Simple levelling:-

→ When the difference of level between two points is determined by setting the levelling instrument midway bet^n the points, the process is called simple levelling.

→ No intermediate sight.

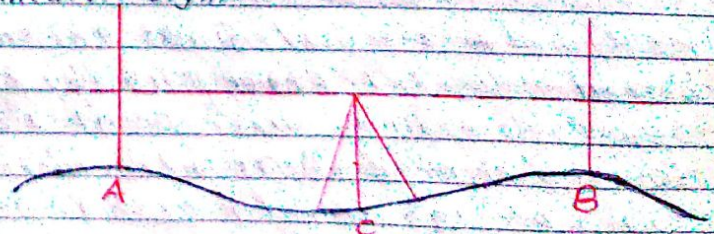


Fig:- Simple levelling

→ In this levelling, we calculate the elevation difference bet^n only two points.

Differential levelling:-

→ The establishment of differences in elevation betn two or more points with respect to datum.

→ continuous/compound levelling

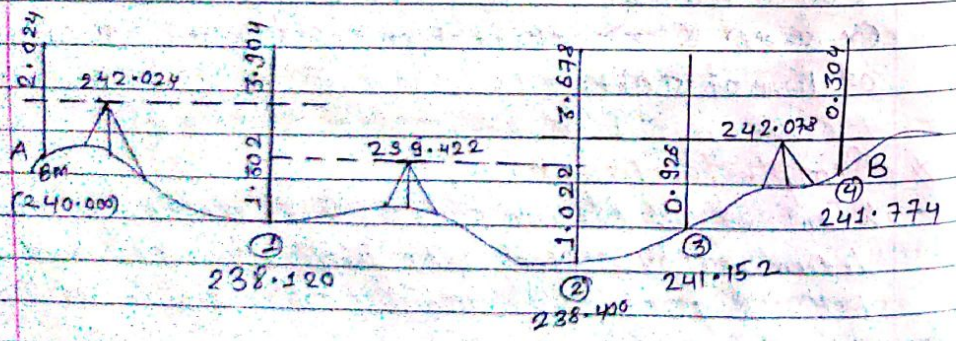
→ It is adopted when

i) The points are a great distance apart

ii) The difference of elevation betn the points is large.

iii) There is obstacles betn the points.

→ In this process, number of interstation are located & instrument is shifted to each station & observed the elevation of interstation points. Finally difference betn original two points elevation is determined.



→ Here, A & B are two points. The distance AB has been divided into three parts by choosing three additional points. Here point 1 & 2 serve as turning point & ③ as intermediate point. Then, we can transfer RL from point A to B either by Rise & fall method or by height of collimation method.

Profile Levelling:-

→ The process of determining the elevation of series of points ~~with respect to datum~~ at measured intervals along a line such as centerline of road, natural feature i.e. stream bed, railway, etc is called profile levelling.

Field procedure:-

→ Suppose AB, BC, CD and DE are the directions of the centre of the road alignment. The levelling instrument is placed at suitable positions (say L_1, L_2, L_3 - - -) and after temporary adjustments, the staff reading are taken. The first staff reading of any set up is entered in the BS column, and the last in the FS column. The other reading are entered in the IS column. The fore bearing and back bearing of each line should be measured and entered in the level book.

→ Temporary bench-marks should be placed at some chainage intervals (say of 200m) on the roots of trees or some permanent points. At the end of day's work, a temporary bench mark must be placed at a suitable point. Fly levelling should be done to connect this TBM to the starting point of the day's work. At the time of fly levelling, all the TBMs (which were placed previously) are also connected in order to detect the place of error. The next day's work is started from TBM placed on the previous day.

→ After taking data, profile is plotted on special paper, on which vertical scale is much larger than horizontal.



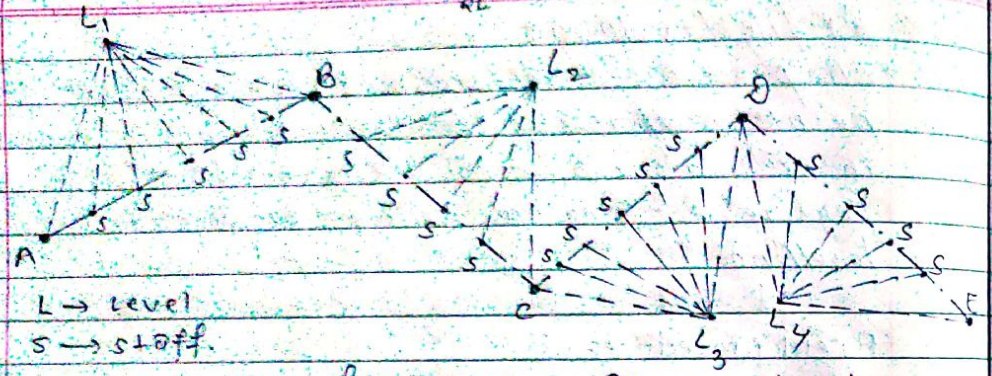
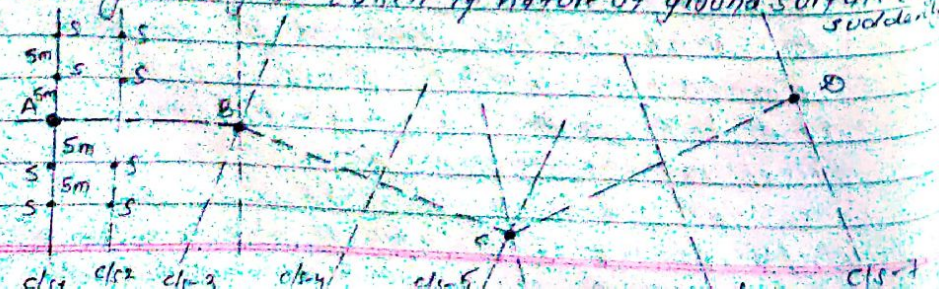


Fig: profile levelling (Longitudinal sectioning)

✱ Cross Section levelling:-

while longitudinal levelling is in process, cross-sectional levelling should also be done. The cross-sections are taken perpendicular to the centre line of the alignment at some regular intervals (say 20m, 40m etc). The purpose of X-section levelling is to know the undulation of ground surface transverse to the centre of the road. The length of X-section depends upon the nature of work. In case of ordinary work the length may be 20m or 40m on each side of centre line. The levels are taken at an interval of 5m on each side. Additional readings may be taken if nature of ground surface changes suddenly.



✱ Trigonometrical levelling:-

→ The branch of levelling in which difference of elevations is determined from the observed vertical angles and known horizontal distance is called trigonometric levelling.

→ The vertical angles are generally observed by theodolite & horizontal distance are either measured directly or computed trigonometrically.

→ Depending upon the horizontal distance betⁿ stations trigonometrical levelling, may be classified into
- Observation of heights & distance as plane survey
- Observation of heights & distance as geodetic survey

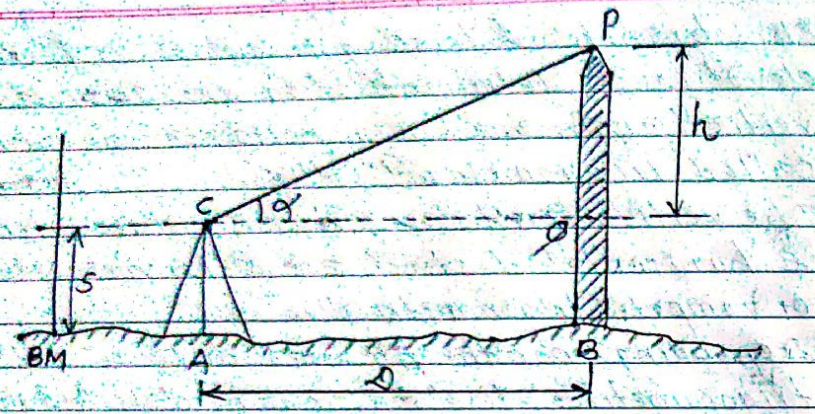
→ In trigonometrical levelling in plane survey, following three cases involved:

case 1: Base of object is accessible
case 2: Base of object is inaccessible and instrument station and the elevated objects are in same vertical plane.
case 3: Base of object is inaccessible and instrument station & elevated objects are not in same vertical plane.

✱ Base of object is accessible:-

In this case, the horizontal distance between the instrument & the object can be measured directly.

Let, an instrument is placed at 'A' & 'B' be the base of pole. 'D' be the horizontal distance betⁿ A & B. Let, α be angle of elevation from C to P. 's' be the staff reading at BM shown in figure



→ The reduced level (RL) of benchmark is already known. The RL/elevation of 'p' can be calculated as,

RL of p = R.L of BM + s + h

Also,

In △ pcQ, $\tan\alpha = \frac{h}{D}$

$h = D\tan\alpha$

Then,

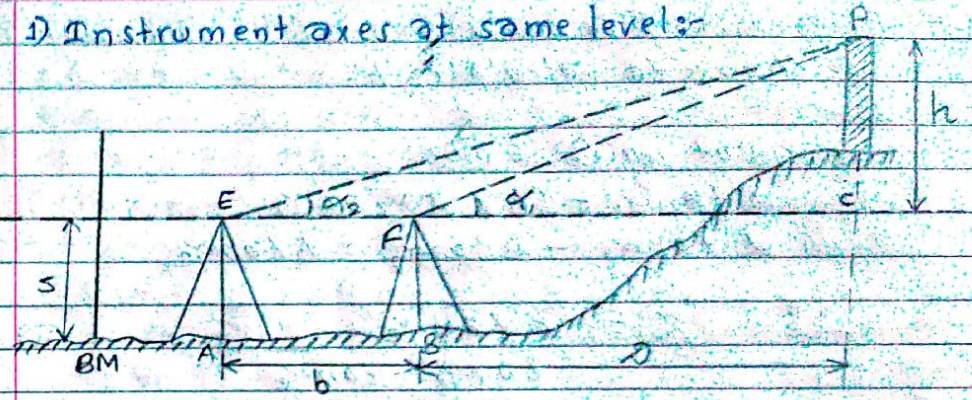
$$\boxed{\text{RL of } p = \text{R.L of BM} + s + D\tan\alpha} \quad \text{—(I)}$$

Using eqn (I), we can easily calculate the RL of point 'p' when base of the object is accessible

※ Base of object is inaccessible & instrument station & elevated object are in same vertical plane.

1) Instrument axes at same level
2) Instrument axes at different level
   a) Height of instrument near to the object is lower
   b) Height of instrument near to the [illegible]

1) Instrument axes at same level:-



→ When the horizontal distance betn instrument & object is difficult to measure, two instrument station are used so that they are in the same vertical planes as the elevated object.

→ Firstafall, instrument is setup at B & read vertical angle α_1.

→ Transit the telescope so that the line of sight is reversed. Mark the second instrument station 'A' on the ground.

→ Let, 'b' be the distance betn two instruments.

→ The staff is kept at benchmark

→ The instrument is setup at 'A' & vertical angle α_2 is measured.

From figure,

RL of p = RL of BM + s + h ----(I)

In △ PCE,

$\tan\alpha_1 = \frac{h}{D} \Rightarrow h = D\tan\alpha_1$ ----(II)

In △ PEC,

$\tan\alpha_2 = \frac{h}{b+D} \Rightarrow h = (b+D)\tan\alpha_2$ ----(III)

Here, b, α_1 & α_2 are known. D is unknown & we have to calculate 'h'.

So,

equating (ii) & (iii)

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

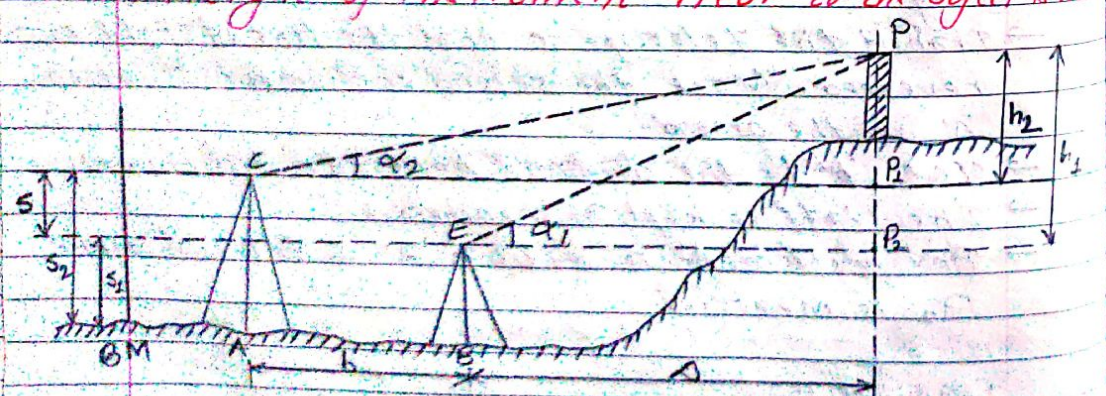
$$D\tan\alpha_1 - D\tan\alpha_2 = b\tan\alpha_2$$

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

put the value of D in either eqn (i) or (ii) to calculate h.

2) Instrument axes at different level:-

a) Height of instrument near to the object is lower



→ In this case, the instrument near to the object (i.e at B) is lower.

→ Firstofall, instrument is setup at B & measure vertical angle α_1. Then, transit the theodolite telescope so that line of sight is reversed. Locate point A & measure distance AB equal to b

→ The staff reading be S_1 at benchmark when the instrument is at position B. Then, again setup the instrument at A & measured vertical angle α_2. The staff reading be S_2 at benchmark (BM) when instrument is at A.

From figure,

In $\triangle PP_2E$, $\tan\alpha_1 = \frac{h_1}{D} \Rightarrow h_1 = D\tan\alpha_1$ ---(i)

In $\triangle PP_1C$, $\tan\alpha_2 = \frac{h_2}{b+D} \Rightarrow h_2 = (b+D)\tan\alpha_2$ ---(ii)

Also,

$$S_2 - S_1 = S = h_1 - h_2$$

Then, eqn (i) - eqn (ii)

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

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

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

$$D = \frac{S + b\tan\alpha_2}{\tan\alpha_1 - \tan\alpha_2} \quad ---(iii)$$

Substituting value of D in either eqn (i) or (ii) Then, we can calculate h_1 & h_2.

Therefore,

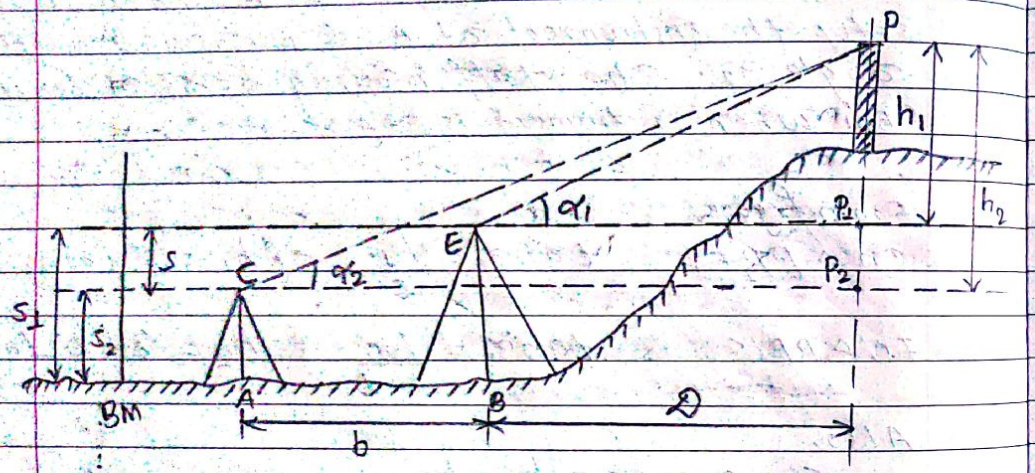
RL of P = RL of BM + S_1 + h_1

or

RL of P = RL of BM + S_2 + h_2

In this way, we can calculate the RL of 'P' when height of instrument near to the object is lower.

b) Height of instrument near to the object is higher



→ In this case, the instrument near to object is higher in position than instrument at A.

→ The process is similar as case 2(a).

Then,

From figure,

In △ PP1E, $\tan\alpha_1 = \frac{h_1}{D} \Rightarrow h_1 = D\tan\alpha_1$ --(i)

In △ PP2C, $\tan\alpha_2 = \frac{h_2}{b+D} \Rightarrow h_2 = (b+D)\tan\alpha_2$ --(ii)

Also,

$S_1 - S_2 = S = h_2 - h_1$

Then, eqn(ii) − eqn(i),

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

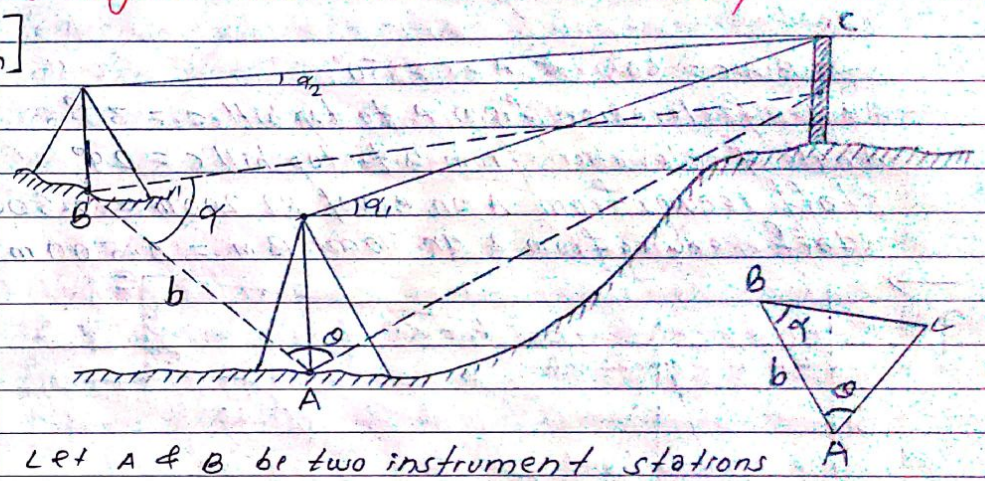
$S - b\tan\alpha_2 = D(\tan\alpha_2 - \tan\alpha_1)$

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

Then, RL of P = RL of BM + S_1 + h_1 (h_1 & h_2 are calculated

= RL of BM + S_2 + h_2 (by putting value of

** Base of object is inaccessible & instrument station & object are not in same vertical plane:-



Let A & B be two instrument stations not in same vertical plane. Firstofall, set the instrument at A & measure α_1. Shift the instrument to B & measure α_2.

Let, In △ ABC,

$\angle ABC = \alpha$, $\angle BAC = \theta$

Using sine law,

$$\frac{BC}{\sin\theta} = \frac{AC}{\sin\alpha} = \frac{AB}{\sin(180^\circ - \alpha - \theta)}$$

Then, $BC = \frac{b\sin\theta}{\sin(180^\circ - \alpha - \theta)}$

$AC = \frac{b\sin\alpha}{\sin(180^\circ - \alpha - \theta)}$

Then $h_1 = AC\tan\alpha_1$

$h_2 = BC\tan\alpha_2$

T8.

Find the RL of top of Hill 'c' from the following observations taken from two stations A & B, 50m apart.

$\angle BAC = 60°$, $\angle ABC = 50°$

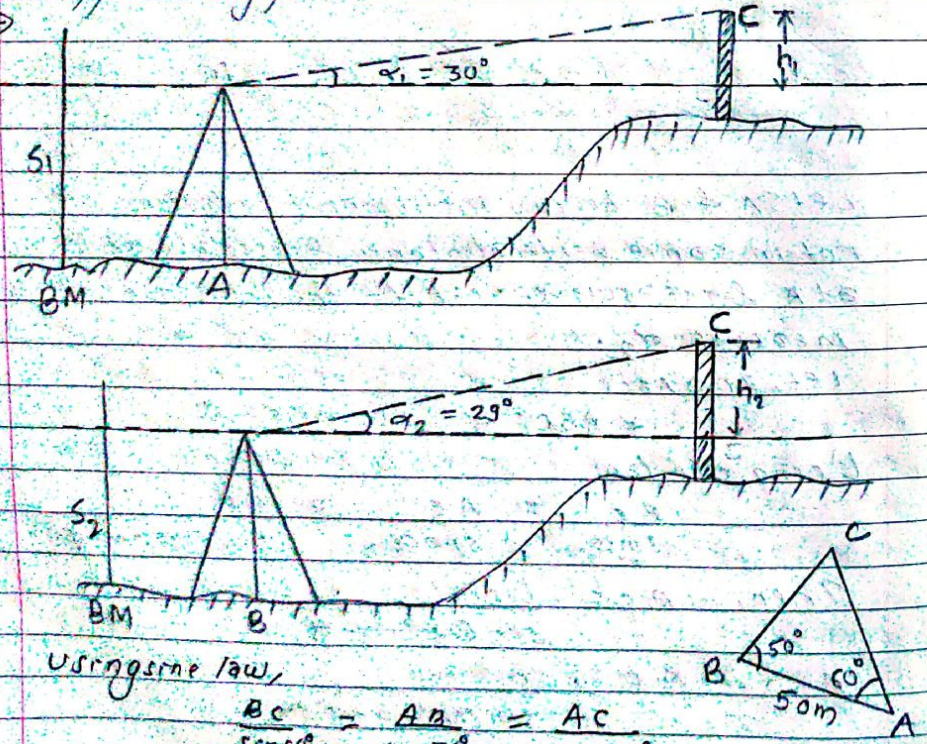
Angle of elevation from A to top hill C = 30°

Angle of elevation from B to top hill C = 29°

Staff reading from A on BM of RL 200m = 2.600m

Staff reading from B on same BM = 0.500m.

→



Using sine law,

$$\frac{BC}{\sin 60°} = \frac{AB}{\sin 70°} = \frac{AC}{\sin 50°}$$

$BC = 46.08m$ $\quad AC = 40.76m$

$h_1 = AC \tan 30° = 23.53m$ $\quad h_2 = BC \tan 29° = 25.542m$

RL of C = 200 + 2.5 + 23.53 = 226.03m

RL of C = 200 + 25.542 + 0.5 = 226.04m

Compass Surveying:-

→ The branch of surveying in which ~~of~~ direction of survey lines are determined by a compass and their lengths by chaining or taping directly on the surface of earth is called compass surveying.

→ Before recommending compass survey for any area, it must be ascertained that area is not magnetically disturbed.

Traverse:-

→ A series of straight lines connected together to form an open or closed polygons.

→ There are two types of traverse

a) closed traverse

b) Open traverse

a) Closed traverse:-

→ A closed traverse starts from one transverse station and closed either on same station or another transverse station whose location is already known.

→ Both starting and end point known.

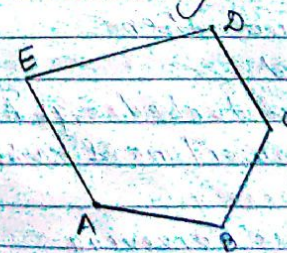


Fig (a): Loop traverse

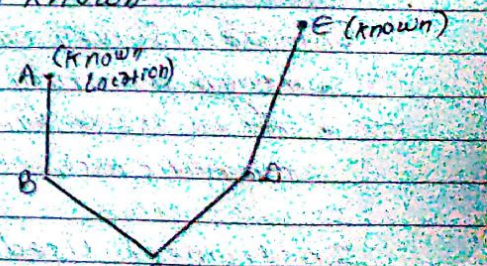
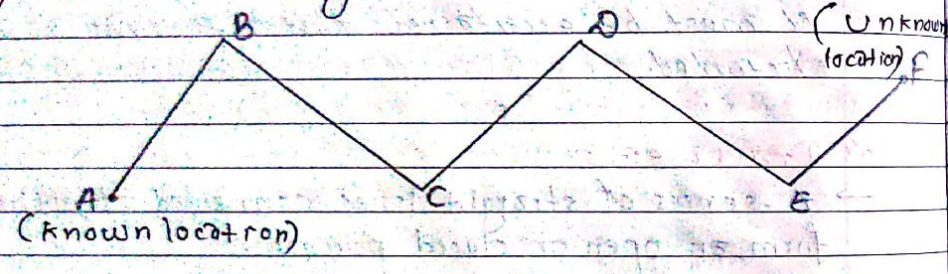


Fig: Link traverse

b) Open traverse:-

→ In open traverse, the location of starting traverse station is known but the location of ending traverse station is unknown.

→ Eg: Road, Railway line, canals etc.



Some important points:-

Compass survey is preferred when

- The area to be surveyed is large
- Time is limit (short time provided)
- High accuracy is not required.
- High crowded area which creates difficulty in surveying
- Site is undulated.

Compass Survey is not preferred when

- High accuracy is required
- Area is magnetically disturbed. Eg: Area having magnetic rock, Iron core, power line & substations etc.

Instrument used in compass Surveying

- prismatic compass
- Tape
- Ranging rod
- Tripod stand
- Arrows & pegs.

Methods of Traversing:-

a) chain traverse:- direction of traverse measured by chain survey
b) Compass traverse:- direction of traverse determine by magnetic compass
c) plane table traverse:- plane table used to plot traverse line.
d) Stadia traverse:- Angle bet^n traverse measured by tacheometer
e) Theodolite traverse:- Angle measured with theodolite.

Meridian:-

→ The fixed line of refrence about which bearing is measured in clockwise direction is called meridian.

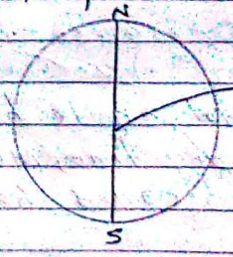
→ There are four types of Meridian

- True meridian
- Magnetic meridian
- Grid meridian
- Arbitary meridian

a) True Meridian:-

→ The line joining North and south pole formed by intersection of earth surface and ~~the~~ joining plane containing northpole & South pole is called true meridian.

→ It is also called geographical Meridian.



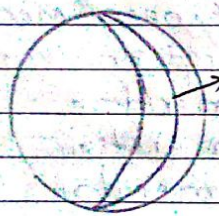
b) Magnetic Meridian:-

→ Magnetic meridian is the direction indicated by a freely suspended magnetic needle at that point.



The suspended needle should be free from magnetic force other than that of earth.

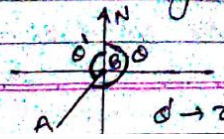
c) Grid meridian:-
Sometime, for preparing a map some state/country assume ~~some~~ vertical line parallel to true meridian for a particular zone. These lines are termed as Grid Meridian.

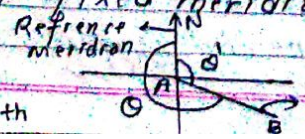


d) Arbitary Meridian:-
The meridian taken in any direction for surveying of small area is called arbitary meridian.

※ Bearing:-
→ The horizontal angle between refrence meridian and the survey line, measured in clockwise direction is called bearing.

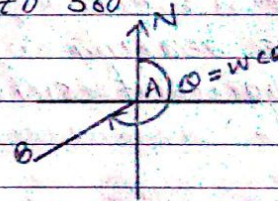
※ Azimuth:-
Smaller angle beth fixed meridian & survey line.





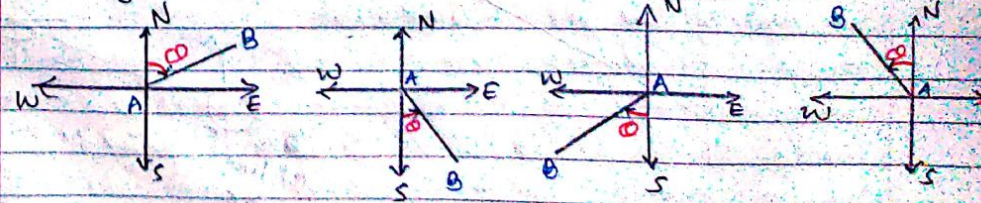
※ Designation of Bearing:-
The common system of notation of bearing are
a) Whole Circle Bearing
b) Quadrantal Bearing (Reduced bearing)

a) Whole Circle Bearing (WCB)
→ In WCB, the bearing of a line is measured in clockwise direction from north end of refrence meridian.
→ Its value ranges from 0° to 360°
→ It is denoted by θ.



b) Quadrantal Bearing / Reduced Bearing:-
→ In quadrantal bearing, the bearing of line is measured eastward or westward from north or south whichever is nearer. North and south are used as refrence meridian & direction can be either clockwise or anticlockwise. It varies from 0-90°

First letter N or S
Then angle of bearing
Lastly the letter E or W.
} step



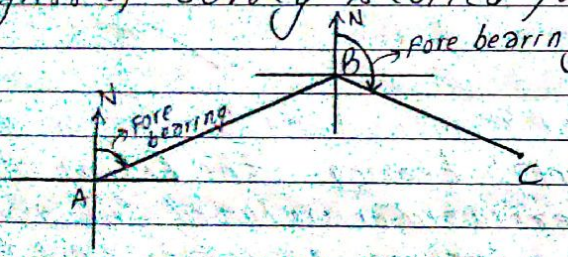
Here, AB → survey line.
θ → Bearing.

* Conversion of WCB to RB

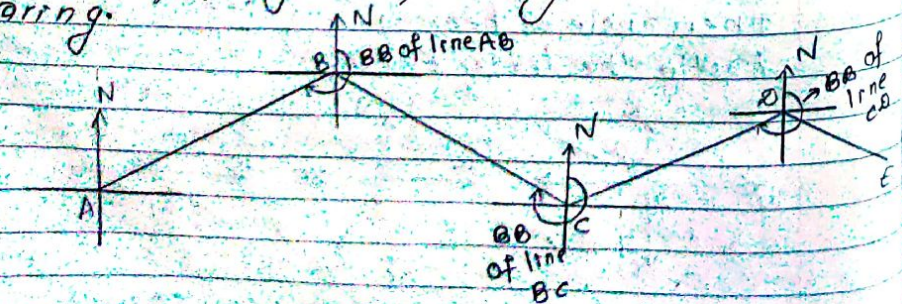
W.C.B	Quadrantal	Conversion
1) 0 – 90°	NOE	∅ = WCB
2) 90° – 180°	SOE	∅ = 180 – WCB
3) 180° – 270°	SOW	∅ = WCB – 180
4) 270° – 360°	NOW	∅ = 360 – WCB

* Fore Bearing (FB) & Back Bearing (BB)

a) Fore Bearing:-
The bearing of a line in the direction of progress of survey is called fore bearing.



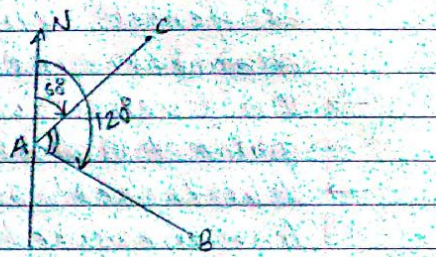
b) Back Bearing:-
The bearing of a line in the opposite direction of progress of survey is called back bearing.



→ The difference of backbearing & forebearing is exactly 180°

→ If BB > 180°, FB = BB – 180°
BB < 180°, FB = BB + 180°
→ If FB > 180°, BB = FB – 180°
FB < 180°, BB = FB + 180°

* Included Angle:-
Let, AB and AC are survey lines.



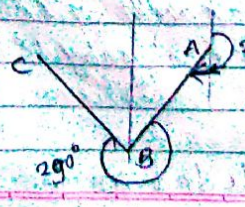
* BAC = FB of AB – FB of AC
= 120° – 60°
= 60° ⇒ Included Angle.

* Calculation of Bearing from included Angles:-
→ If bearing of previous line and included angle (clockwise angle from previous line to next line) is known, bearing of next line can be calculated. The calculation is done as follows.

Bearing of next line = Bearing of previous line + Included angle ± 180° – 540°

Let, sum = Bearing of previous line + Included angle.
The above line include three cases.
a) If sum > 180°, bearing of next line = sum – 180°
b) If sum < 180°, bearing of next line = sum + 180°
c) If sum > 540°, bearing of next line = sum – 540°

Eg



Sum = 220° + 290°
= 510°
Sum > 180°, so,
Bearing of BC = 510° – 180°
= 330°

Local Attraction:-

→ The direction taken by a compass needle is affected by magnetic attractions other than that of earths magnetic field such as fences, underground piers, reinforcing bars, magnetic rock, iron cores, electric poles etc.

→ In such situation, Magnetic needle doesnot point towards north south.

→ Due to local attraction, the direction of survey line measured by compass needle doesnot give true bearing. so, to calculate exact/true value of bearing, the error due to local attraction should be considered.

Detection of Local attraction:-

The presence of local attraction at any station may be detected by observing the fore bearing & back bearing of the line. If the difference betn fore bearing & back bearing is 180°, both end stations are free from local attraction.

Elimination of Local Attraction:-

1) Included Angle Method
2) Local Attraction Method
   - a) one line differ exactly by 180°
   - b) No line differ exactly by 180°

Included Angle Method.

Step1: calculation of included angle

Step2: Sum of included angle → findout Theoretical sum (2n-4)×90°

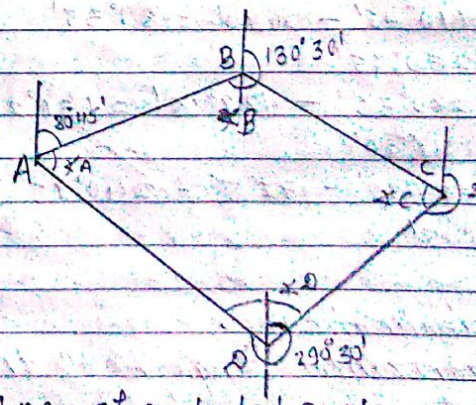
Step3: Find error & divide error equally.

Step4: corrected Bearing.

Q7 The following bearing were observed in a compass traverse. Make correction for local attraction, find corrected bearings & find interior angles of the traverse.

→

Line	FB	BB	Angle	corrected Angle	corrected FB	corrected BB
AB	80°45'	260°00'	∠A = 30°25'	∠A = 30°10'	80°35'	260°35'
BC	130°30'	311°35'	∠B = 129°30'	∠B = 129°15'	131°20'	311°20'
CD	240°15'	60°15'	∠C = 71°20'	∠C = 71°5'	240°15'	60°15'
DA	290°30'	111°10'	∠D = 129°45'	∠D = 129°30'	290°45'	110°45'



Calculation of included angle.

∠A = BB of DA − FB of AB = 111°10' − 80°45' = 30°25'

∠B = BB of AB − FB of BC = 260° − 130°30' = 129°30'

∠C = BB of BC − FB of CD = 311°35' − 240°15' = 71°20'

∠D = 360° − FB of DA + BB of CD = 360° − 290°30' + 60°15' = 129°45'

sum of included angle.

$= \angle A + \angle B + \angle C + \angle D$

$= 30°25' + 129°30' + 71°20' + 129°45'$

$= 361°$

But,

Sum of total interior angle $= (2n-4) \times 90°$

$= (2 \times 4 - 4) \times 90° = 360°$

Error $= 361° - 360° = +1°$

Correction $= -15'$ $[1°/4]$

The corrected included angle are written in table.

Now,

The difference of FB & BB of line CD is exactly 180°. so, station C & D are free from local attraction.

Then,

BB of CD = 60°15'

FB of DA = 290°45'

BB of DA = 290°45' − 180° = 110°45'

FB of AB = 110°45' − 30°10' = 80°35'

BB of AB = 260°35'

FB of BC = 260°35' − 129°15' = 131°20'

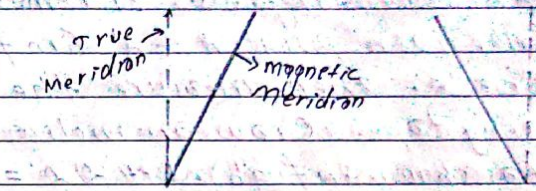
BB of BC = 311°20'

FB of CD = 311°20' − 71°5' = 240°15'

BB of CD = 60°15'

✻ Magnetic declination:-

It is the horizontal angle bet^n true meridian & magnetic meridian. If the magnetic meridian is on eastern side of true meridian, the angle of declination is said to be eastern declination or positive declination. Similarly, if the magnetic meridian is on western side of true meridian, the angle of declination is said to be western declination or negative declination.



∴ True bearing = Magnetic bearing ± Magnetic declination

✻ Difference bet^n prismatic Compass & Surveyor compass:-

→

parameters	prismatic Compass	Surveyor Compass
① Compass needle	Broad type needle	Edge bar type needle.
② Graduated Ring	Attached with needle.	Attached with box
③ Sight vane	eye vane & object vane are ~~different~~ same.	~~diff~~ eye & object vane.
④ Reading system	WCB	QB
⑤ Tripod	May be needed.	Must use tripod.

✻ Graphical method for correction error in traverse for compass survey:-

Sometimes, when a closed traverse is plotted, the finishing & starting points may not coincide due to error in measurement of length and angle. Such error can be corrected using graphical methods

Let ABCDA' be the above traverse. point A & A' are not coincide due to error in measurement. The starting & finishing points are not coincides in case of compass surveying. So, to remove the error, first of all, join point A & A' (in fig a) let length of AA' be 'e'. Then, measure the length L_1, L_2, L_3 & L_4 using tape. Draw an incline line joining E & A. (as shown in fig b). Let $DD' = e'$, $CC' = e''$, $BB' = e'''$ be perpn length from horizontal line to inclined line. The values of e', e'' & e''' represents the amount by which respective station are to be shifted. Then, A', B', C', D' be the new station of A, B, C & D. After this, the traverse joining A', B', C' & D' is drawn. So, the error can be eliminated graphically.

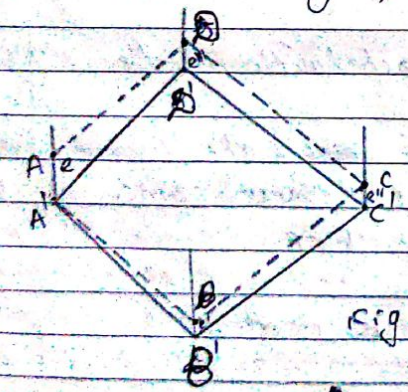


Fig (a): Traverse line

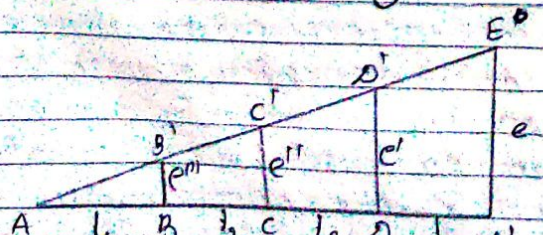


Fig (b).

* Theodolite:-

→ Theodolite is an important & most precise instrument for measurement of horizontal & vertical angles.

→ Least count of theodolite → 20 sec

Least count of compass → 30 sec

→ Theodolite is used for following purpose

i) Measuring horizontal, vertical & deflection angles

ii) Measuring magnetic bearing.

iii) Measuring horizontal distance between two points.

iv) Determining difference in elevation.

v) Prolonging survey lines

vi) Finding vertical height of the object.

* Types of Theodolite:-

a) According to telescope:-

1) Transit Theodolite:-

The theodolite in which line of sight can be reversed by revolving the telescope through 180° in vertical plane

2) Non-transit theodolite:-

The theodolite in which line of sight cannot be reversed by revolving the telescope through 180° in vertical plane.

b) According to device used for reading graduated circle

1) Vernier theodolites:

→ For reading graduated circle, verniers are used.

2) Micrometer theodolites:

→ A micrometer is used for reading graduated circle.

c) According to display

a) Analog
→ Measurement is not accurate & output is not displayed in digital form.
→ The value changes continuously

b) Digital:-
→ measurement is accurate & output is displayed in digital form.
→ sensitivity of digital theodolite is more & widely used in surveying.

d) According to angle measurements:-

a) Degree:-
→ The angle measured in degree in clockwise dirn

b) Grade:-
→ Angle measured in grade.

→ can measure coordinate directly.

Theodolite	Total station	EDM
① specialized for measuring horizontal & vertical angle	① Can measure distance, angles & elevations.	Electronic distance measuring instrument is a surveying instrument for measuring distance electronically betn two points through electromagnetic waves.
② Low level of precision	② High level of precision	
③ not based on EDM	③ Based on EDM	
④ Maximum range < 1KM	④ Range: 3-5 KM	
⑤ Data cannot be transfer into computer.	⑤ Data can be easily transfer into computer using USB port	
⑥ Less expensive.	⑥ More expensive.	
⑦ Knowledge of trigonometry is required.	⑦ Knowledge of trigonometry is not required.	accuracy → $1/10^5$
⑧ At least two surveyor req.	⑧ Only one surveyor can perform	Range → 100 KM

DDM → Direct Distance Measurement → by chaining or taping
ODM → Optical Distance Measurement → conducted by tacheometry with the help of optical way
EDM → Electronic Distance Measurement

* Basic terms Used in Thedolite Survey:-

i) Centering:-
process of setting up the thedolite exactly over the ground station point.

ii) Transiting:-
process of turning the telescope in vertical plane.

iii) Face left:-
Vertcle circle is on the left at the time of observation

iv) Face Right:-
Vertical circle is on the right at time of observation

v) Changing face:-
Operation of bringing the vertical circle from one side of observer to other side

vi) Swinging:-
Turning of telescope in horizontal plane.
when telescope is turned clockwise, it is called right swing
when telescope is turned anticlockwise, it is called left swing

vii) Vertical axis:-
It is the axis about which thedolite rotate in horizontal plane.

viii) Horizontal axis:-
It is the axis of rotation of telescope in vertical plane.
It is known as trunion axis.

ix) Line of collimination (Line of Sight)

→ It is the imaginary line passing through the intersection of the cross hairs at the diaphragm & optical centre of the object glass.

x) Axis of the telescope:-

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

Parts & their function [Theodolite]

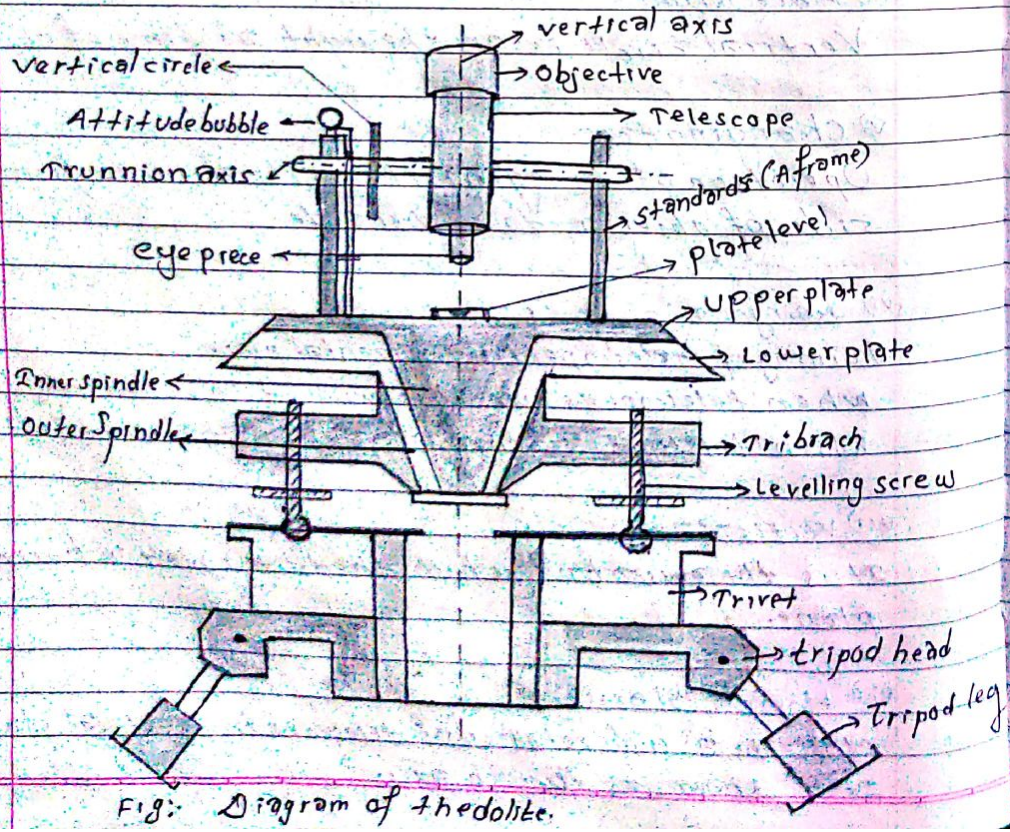


Fig: Diagram of theodolite.

1) Telescope:-

→ It is very important parts of theodolite which is mounted in trunnion axis. It is used for both internal & external focusing of the theodolite & transit, it can be rotated about its horizontal axis in vertical plane.

2) Vertical Circle:-

→ It is a circular graduated arc attached to the trunnion axis of telescope. It is divided into four quardants & each quardant is graduated from 0° to 90°. Face left or face right during observation depends on position of vertical circle.

3) Index frame / I-frame:-

→ The index bar is provided on the standard in front of vertical circle. It carries two verniers at the two ends of horizontal arm. The vertical leg of index bar is provided with a clip screw at the lower end by means of which attitude bubble can be brought to the centre.

4) Standards (A frame):-

Two frames are provided on the upper plate to support the telescope, the vertical circle & vernier scales.

5) Levelling head:-

The trivet, levelling screws & tribach constitute the body which is known as levelling head. It attach theodolite to the tripod.

6) Trivet:-

→ It is a circular plate having a central, threaded hole for fixing the thedolite on the tripod stand. It is also called base plate. Three foot screws are attached on this plate by means of ball shaket arrangement.

7) Foot Screws:-

→ These are meant for levelling the instrument. The lower part of screw are attached to trivet & upper part is attached to tribrach.

8) Tribrach:-

→ It is a triangular plate carrying three foot screws at its ends.

9) Lower plate:-

The lower plate is attached to outer spindle. It is also known as scale plate because it carries a horizontal circle on it. It carries lower clamp screw & tangential screw.

10) Upper plate:-

It is attached to inner spindle & contains vernier scale. It carries upper clamp screw & tangential screw.

11) plate levels:-

There are two plate bubbles which are mounted at right angles to each other on upper plate.

The bubbles are meant for levelling the instrument at the time of measuring horizontal angles.

12) Tripod:-

Tripod consists of three solid & flanged legs over which thedolite is kept while surveying. The lower ends of each leg are provided with pointed iron shoes.

13) plumb Bob:-

A plumb bob is suspended from the spindle for centering the instrument exactly over the station Mark.

14) Compass:-

Some thedolites are provided with compass which can be used to measure bearing of the line.

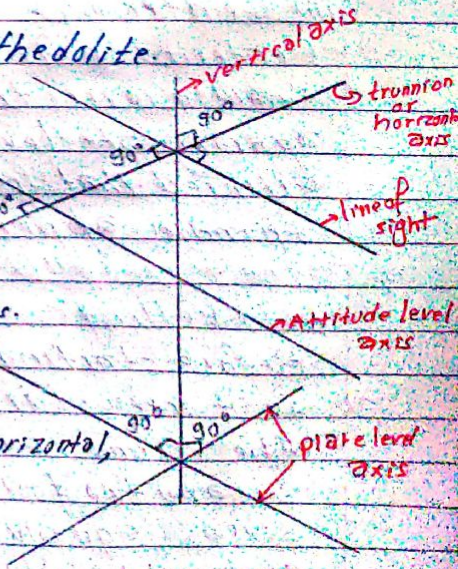
✻ Construction Principle of thedolite.

i) The axes of plate level is perpn to vertical axis.

ii) The trunnion axis is perpn to vertical axis.

iii) Line of sight is at right angles to the horizontal axis.

iv) The axis of the altitude level is parallel to the line of collimation when it is horizontal, & the vertical circle reads zero.



※ Temporary adjustment of the dolite:-

Temporary adjustment are the adjustment which are required to be made at each setting of instrument before taking observations.

i) Setting the thedolite over the station:-

The tripod stand is placed over the station. Then, thedolite is lifted from box & fixed on top of stand by means of wing nut.

ii) Approximate levelling by tripod stand:-

Two legs of the tripod stand are placed well apart & fixed on ground. Then, approximate levelling is done by moving third leg in or out such that bubble is approximately at the centre of its run.

iii) Centering:-

Centering is the process of setting the instrument exactly over the station. At the time of approximate levelling by tripod stand, it is ensured that plumb bob suspended from the spindle approximately lies over the station peg.

iv) Levelling:-

Before starting the levelling operation, all the foot screws are brought to the centre of their run. Firstly all plate bubble is placed parallel to any two foot screw. By turning both these screws equally inwards or outwards the bubble is brought to the centre. Then, the plate bubble is turned through 90° so that it is perpendicular to the line joining the first & second screws. Then, by turning the third foot screws, the bubble is brought to the centre.

v) Focusing the eye piece:-

The eye-piece is focused so that the cross hairs can be seen clearly. The eye-piece is moved in or out by turning it clockwise or anticlockwise until the cross hairs appear distinct & sharp.

vi) Focusing the object glass:-

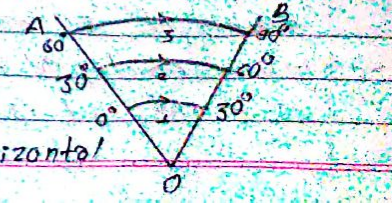
The telescope is directed towards the object to be sighted & focusing screw is turned till the image appears clear & sharp. The image so formed is in plane of cross hairs.

※ Measurement of horizontal & vertical angles:-

There are two methods of measuring horizontal & vertical angles in thedolite surveying.

a) Repetition Method:-

In this method, angle is added a number of times. The total is divided by the number of readings to get the angle. The angle should be measured clockwise in face left & face right positions, with three repetition at each face. The average of the all reading divided by number of reading gives exact horizontal angle beth A & B.



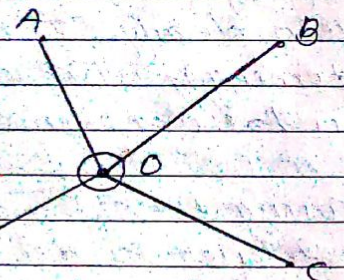
b) Reiteration method:-

It is also known as direction method. This method is suitable when several angles are measured from a single station. In this method all the angles are measured successively & finally the horizon is closed. So, that the total sum of angle must be equal to 360°.

Check

$\angle AOB + \angle BOC + \angle COD + \angle AOD = 360°$

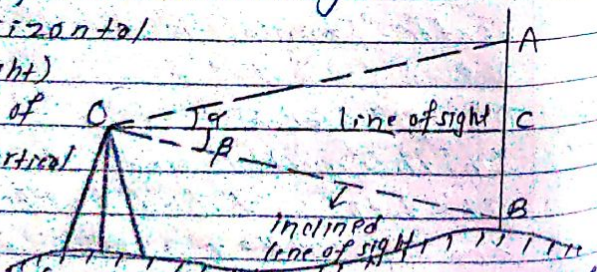
Here, instrument is set up at O & all four angle are measured by turning the telescope.



→ We measure horizontal angle (generally) by repetition & reiteration method.

* Measurement of vertical Angles.

→ The angle betn horizontal line (line of sight) & inclined line of sight is called vertical angle.



→ When inclined line of sight is below the line of sight, it is called angle of depression (β).

→ When inclined line of sight is above the line of sight, it is called angle of elevation (α).

* Errors in theodolite

a) Instrumental Errors:-

→ Axis of plate bubble may not be perpⁿ to vertical axis.
→ Line of collimation not perpⁿ to horizontal axis.
→ Horizonal axis not perpⁿ to vertical axis
→ The axis of altitude level is not parallel to line of collimation
→ Eccentricity of inner & outer axes.

b) Personal Errors:-

→ Centering & levelling may not be done properly.
→ clamp screws may not fixed properly.
→ Focusing is not done perfectly.
→ The object is not bisected accurately
→ Little knowledge in operating the theodolite also leads to error in observation.

c) Natural Errors:-

→ High temp^r causes error due to irregular refraction.
→ High wind causes vibration in the instrument & this may lead to wrong readings.

* Calculation of average direction:-

a) If FL > FR, avg dirⁿ / Mean = $\frac{FL + FR + 180°}{2}$

b) If FL < FR, avg dirⁿ / Mean = $\frac{FL + FR - 180°}{2}$

Q During the survey of the suspension bridge, the following observation were made in $\triangle ABC$. AB is bridge axis & LC of instrument is 1'00".

Inststation	sighted to	Horizontal circle Reading Face left	Face right
A	B	0°00'00"	180°00'20"
	C	54°38'	234°38'20"
B	C	0°	179°59'50"
	A	89°20'40"	269°21'00"
C	A	0°	180°
	B	36°01'	215°58'20"

Compute the angle by mean direction method & correct them if necessary. If the length of line BC is 58.232m. Find span of bridge axis AB.

→

Instrument station	sighted to	Face	Horizontal circle reading	Mean	Angle
A	B	L	0°00'00"	10"	
		R	180°00'20"		∠BAC =
	C	L	54°38'	54°38'10"	54°38'
		R	234°38'20"		
B	C	L	0°	-0°0'5"	
		R	179°59'50"		∠CBA =
	A	L	89°20'40"	89°20'50"	89°20'55"
		R	269°21'		
C	A	L	0°	0°	∠ACB
		R	180°		= 35°59'40"
	B	L	36°01'	35°59'40"	
		R	215°58'20"		
Total					179°58'35"

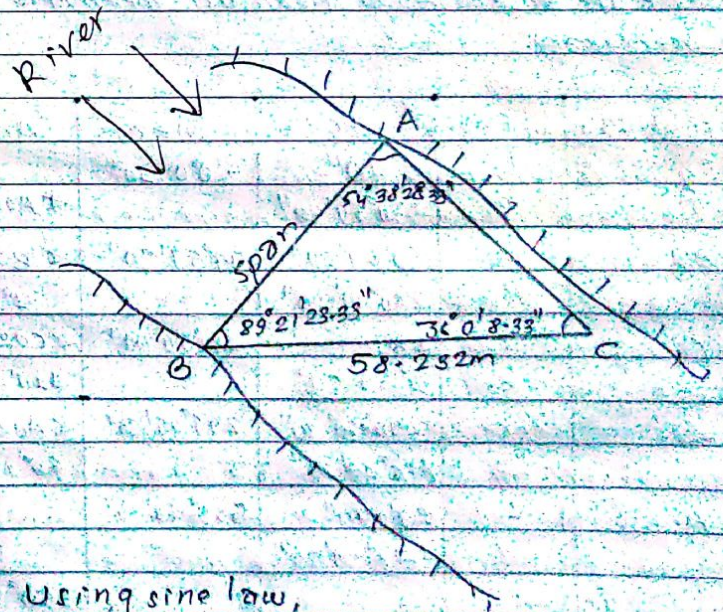
The total sum of internal angle = (2n-4) × 90°
= 2 × 90° = 180°

But the calculated angle = 179°58'35"

Error = 179°58'35" - 180° = -0°1'25"

correction = $\frac{+0°1'25"}{3}$ = 28.33"

∴ Corrected angle, ∠A = 54°38' + 28.33" = 54°38'28.33"
∠B = 89°20'55' + 28.33" = 89°21'23.33"
∠C = 35°59'40" + 28.33" = 36°0'8.33"
sum = 180°



Using sine law,

$\frac{BC}{\sin(54°38'28.33")} = \frac{AB}{\sin(36°0'8.33")}$

⇒ AB = 41.972 m

Span of bridge axis AB = 41.972 m

Reiteration method.

Q. A direction theodolite is set over station 'O' to measure directions to stations A, B, C, D & A. The observed dirⁿ from that station are tabulated below.

Inst stn	Sighted stn	HCR observation	
		FL	FR
O	A	10"	179°59'30"
	B	90°54'20"	280°00'40"
	C	181°	0°59'20"
	D	295°20'30"	116°10'30"
	A	29'40"	179°29'20"

Calculate the mean angles by mean dirⁿ method & adjust them if necessary.

→

Inst stn	sighted stn	Face	HCR	Mean	Angle
O	A	L	10"	- 8°00'10"	
		R	179°59'30"		∠AOB=99°57'40"
O	B	L	90°54'20"	99°57'30"	∠BOC=
		R	280°00'40"		81°02'10"
O	C	L	181°	180°59'40"	∠COD=
		R	0°59'20"		114°45'50"
O	D	L	295°20'30"	295°45'30"	∠AOD=
		R	116°10'30"		64°14'00"
O	A	L	29'40"	359°59'30"	
		R	179°29'20"		
Total					359°59'40"

Since, horizon is closed, then total angle of a complete circle is 360°.

∴ Error = 359°59'40" - 360° = -0°0'20"

correction = $+\frac{0°0'20"}{4}$ = +0°0'05"

If horizon is not closed, we cannot calculate error.

∴ Corrected angles are,

∠AOB = 99°57'40" + 5" = 99°57'45"

∠BOC = 81°02'10" + 5" = 81°02'15"

∠COD = 114°45'50" + 5" = 114°45'55"

∠DOA = 64°14' + 5" = 64°14'05"

sum = 360°

Repetition method

Q. Calculate the mean angle AOB by the mean direction method from the following data in a standard booking format.

Inst station	Target Station	Face	HCR Set I	HCR Set II
'O'	A	L	0	90°
	B	L	121°	211°00'40"
	B	R	301°00'20"	31°00'20"
	A	R	179°59'30"	269°59'40"

→ Solution

Inst station	set	Target station	Face	HCR	Mean	Angle
O	I	A	L	0°	-00°00'15"	
			R	~~90°~~ 179°59'30"		121°00'2
		B	L	121°	121°00'10"	
			R	301°20"		
		A	L	90°	89°59'55"	
			R	269°59'40"		121°00'3
	II	B	L	211°00'40"	211°00'30"	
			R	31°00'20"		
Total						

Here,

In set I,

$\angle AOB = 121°00'10'' - (-00°00'15'') = 121°00'25''$

In set II,

$\angle AOB = 211°00'30'' - 89°59'55'' = 121°00'35''$

∴ Average angle of $\angle AOB$ is, $= \frac{121°00'25'' + 121°00'35''}{2}$

$= 121°00'30''$

∴ $\angle AOB = 121°00'30''$.

Degree of precision / permissible error in levelling:-

degree of precision depends on type of instrument, skill of observa, survey area & atmospheric condition.

permissible error in levelling is,

$E = C\sqrt{K}$ (mm)

where, E = permissible error in mm

C = constant

K = distance in KM

a) For rough levelling, $E = (\pm 100\sqrt{K})$ mm

b) For ordinary levelling $E = \pm 24\sqrt{K}$ mm

c) For accurate levelling, $E = \pm 12\sqrt{K}$ mm

d) For precise levelling, $E = \pm 4\sqrt{K}$ mm

Ch-5

Theodolite Traversing:-

Traverse:- [Follow notes on Compass Survey]

Checks in traverse:-

a) closed traverse:-

Linear measurement: check by two way measurement

precision

1 : 2000 → plane ground

1 : 1000 → sloping ground.



Angular measurement

For closed loop traverse,

sum of interior angles = $(2n-4)\times 90°$

sum of exterior angles = $(2n+4)\times 90°$

For link traverse,

↳ observed & known bearing of line is compared.

Error = $\pm\sqrt{n}\times$ L.C

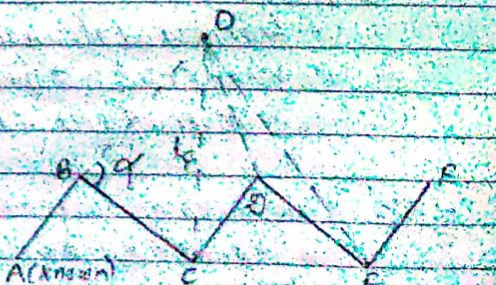
b) Open traverse:-

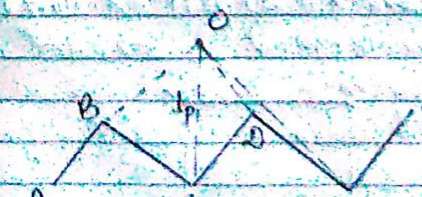
Linear measurement:-

Two way measurement

Angular Measurement:-

↳ By establishing auxiliary points.





$l_p = l_f$

※ Field procedure of thedolite traversing:

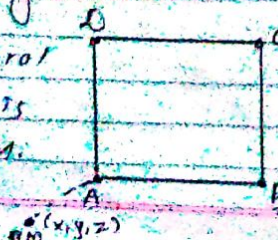
① Reconnaissance surveys
- ↳ For predicting feasibility & rough cost
- ↳ Number of surveyor required
- ↳ Time of surveying
- ↳ Other points to be considered before traversing

② Establishment of control points & three point refrence criteria
1) Interconnected control points should be intervisible.
2) Leg: ratio 2:3 maintain
3) Establish at elevated area

③ Two way linear measurement
- ↳ Ranging & stepping method.

④ Angular measurement
- ↳ Horizontal angles for x & y
- ↳ vertical angles for z
- ↳ Number of sets for accuracy

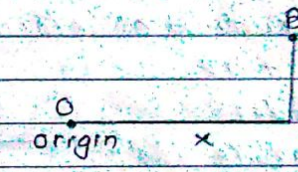
Note: The main objective of thedolite traversing is to find the (x, y, z) coordinate of control points wrt bench mark whose (x, y, z) coordinate is already located.



Here, A, B, C & D are control points whose (x, y, z) is to be found out wrt B.M.

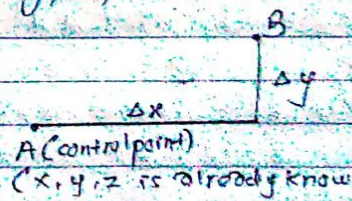
※ Types of coordinate:

a) Independent Coordinate:
If the coordinate of any point is taken/calculated by taking origin as datum/refrence is known as independent coordinate.



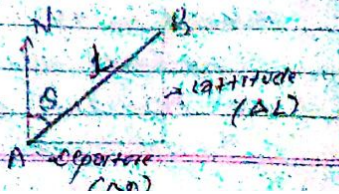
Here, the coordinate of BM (x, y) is independent coordinate

b) Consecutive Coordinates:
If the coordinate of any point is calculated by taking control point/Bench Mark or points of known x, y, z as refrence, such type of coordinate is consecutive coordinate.



※ Latitude & departure:

The latitude (northing or southing) of a survey line is defined as its coordinates measured parallel to assumed meridians. The departure (easting or westing) of a survey line is defined as its coordinate measured at right angle to the assume meridians.



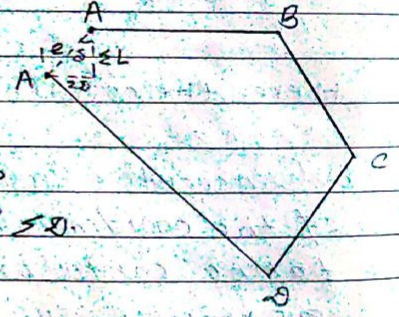
Coordinate of B,
$(x_B, y_B) = [(x_A + l\sin\theta), (y_A + l\cos\theta)]$

* Closing error in closed traverse:-

Sometimes, when a closed traverse is plotted according to the field measurements, the finishing & starting points maynot coincides due to error in measurement of length or angle. Such type of error is known as closing error.

→ In a closed traverse, the algebric sum of lattitudes ΣL should be zero & algebric sum of departure should be zero.

→ The error of closure can be findout by finding ΣL & ΣD.



Then,

from figure,

closing error (e) = AA'

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

& dirn of closing error is,

$$\tan\theta = \frac{\Sigma D}{\Sigma L} \quad \text{---(ii)}$$

* Balancing the traverse:-

The term 'balancing the traverse' is generally applied to the operation of applying corrections to lattitudes & departures so that $\Sigma L = 0$ & $\Sigma D = 0$. This applied only when the survey forms a closed polygon. The common method of adjusting the traverse is

a) Bowditch's method b) Transit method
c) Graphical Method

a) Bowditch's Method:

→ According to this method, the correction to lattitude or departure is distributed in proportion to length of the sides.

→ This method is used when both linear & angular measurements are of equal precision.

Corrected to lattitude of any side = Total error in lattitude × length of that side / Perimeter of traverse

$$\Rightarrow C_L = \Sigma L \times \frac{l}{\Sigma l} \quad \& \quad C_D = \Sigma D \times \frac{l}{\Sigma l}$$

b) Transit rule:-

→ According to this rule, the total correction to lattitude or departure is distributed in proportion to lattitude & departure of that side.

→ This Method is used when angular measurement are more precise than linear measurement.

correction to lattitude = Total error in lattitude × Lattitude of that line / Arithmetic sum of lattitude

$$C_L = \Sigma L \times \frac{L}{L_r} \quad \& \quad C_D = \Sigma D \times \frac{D}{D_r}$$

Where,

L_r = Arithmetic sum of lattitude

D_r = Arithmetic sum of departure

c) Graphical Method:-

In the figure, ABCDA' represent an unbalanced traverse having a closing error equal to A'A = e, since first point A & last point A' are not coinciding. The total closing error is distributed linearly to all sides in proportion shown in fig (b). The ordinate of aA' made equal to (A'A = e). The constructing similar Δ, the errs at B, C & D are found. Then, Bb', Cc', Dd' are drawn parallel to line of closing error (AA') & made equal to e''', e'' & e' respectively. The traverse AB'C'D'A' obtained by joining A', B', C' & D' & A' is the adjusted traverse.

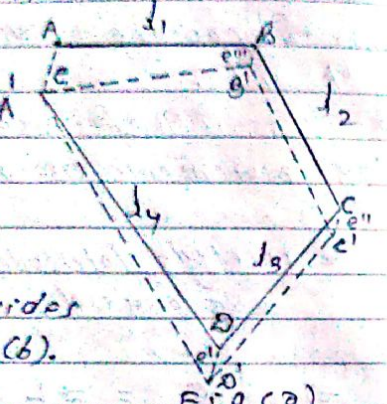


Fig (a)

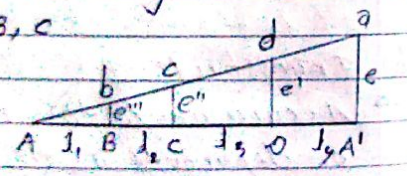


Fig (b)

* Gales traverse table:-

The tabular form which is widely used for calculating independent coordinate in traversing. The steps to be followed are

1) Adjust interior angle
2) Calculate bearing of line
3) Calculate consecutive coordinate
4) Calculate Σl & ΣD.
5) Apply necessary correction
6) Calculate independent coordinates.

* Degree of accuracy in traversing:-

The degree of accuracy in traversing depends upon the errors which may be instrumental or personal or natural made during traversing. Less the error, more will be the degree of accuracy in traversing. For higher degree of accuracy in traversing, the effect of error in linear measurement must be equal to error in angular measurements. To get a relationship beth precision of angular & linear measurements, consider 'D' be the corrected position of point wrt 'A' such that,

AD = L & ∠BAC = θ.

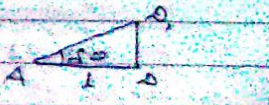
In the field measurement, let δθ be the error in angular measurement & 'e' be the error in linear measurement so that D_2 is faulty position of point D as obtained from field measurements. Then,



$$\tan \delta\theta = \frac{DD_1}{L}$$

$$DD_1 = L \tan \delta\theta$$

Displacement of D due to angular error = $DD_1 = L \tan \delta\theta$



$$\tan \delta\theta = \frac{DD_1}{AD}$$

Displacement of D due to linear error, $D_1D_2 = e$.

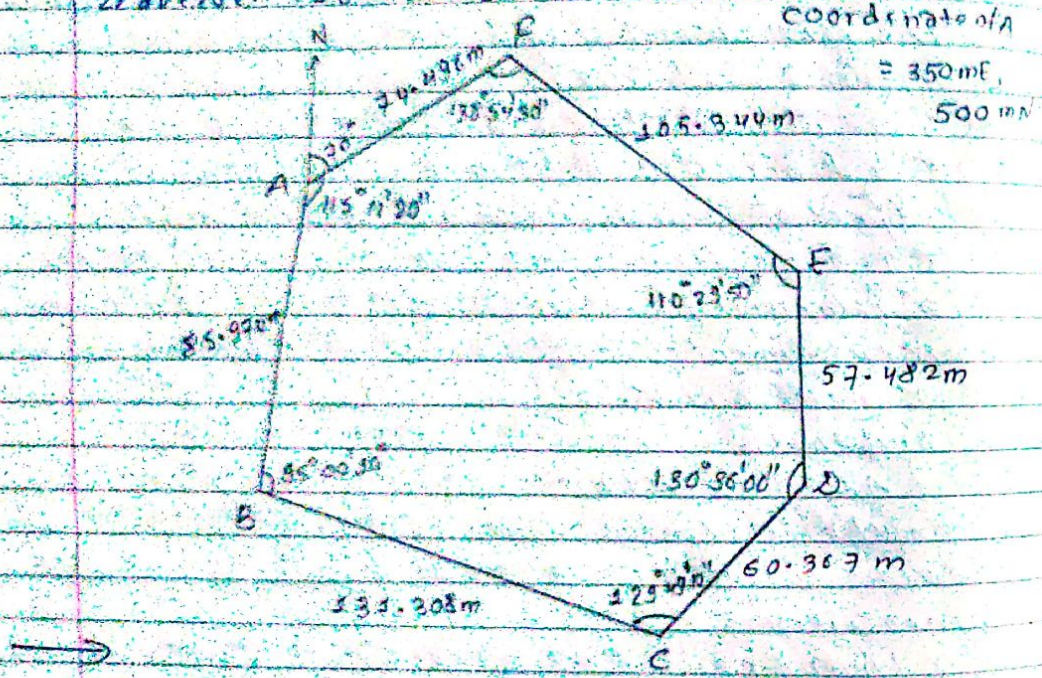
Then, for higher degree of precision,

$$L \tan \delta\theta = e$$

$$\delta\theta = \tan^{-1}\left(\frac{e}{L}\right)$$

If precision of linear measurement is $\left(\frac{e}{L}\right) = \frac{1}{5000}$, the allowable angular error is $\delta\theta = 41''$.

* Find the independent coordinates in following traverse. (20" total station used)



coordinate of A = 350mE, 500mN

Total sum of internal angle = 720°1'

Error but, actual sum = (2n − 4) × 90° = (2×6 − 4) × 90° = 720°

Error = 720°1' − 720° = 1' = 60"

correction = −60"/6 = −10"

Station	Traverse leg	Distance (m)	Angle D M S	correction	corrected Angle	Bearing	Consecutive coordinate: Lat (+) / Lat (−)	Consecutive coordinate: Dep (+) / Dep (−)	Correction: Δl	Correction: Δd	Corrected: l	Corrected: d	Independent coordinate: X (m)	Independent coordinate: Y (m)
													350	500
A	AB	74.492	115 11 20	−10"	115 11 10	250°	−70 / −25.49	−3.77 / −85.62	0.0046	0.0188	−25.49	−85.62	324.51	414.38
B	BC	85.922	85 00 30	−10"	85 00 20	180° 11' 10"	−85.62 /	−3.77 /	0.0053	0.0217	−85.62	−3.77	238.89	328.76
C	CD	60.367	130 56 00	−10"	130 55 50	95° 00' 00"	−5.24 /	60.14 /	0.0037	0.0153	−5.24	60.13	233.27	388.89
D	DE	57.482	110 23 50	−10"	110 23 40	45° 55' 50"	39.80 /	41.47 /	0.0035	0.0145	39.80	41.47	273.07	450.03
E	EF	105.844	138 54 30	−10"	138 54 20	276° 19' 30"	23.49 /	−102.64 /	0.0065	0.0268	23.49	−102.64	296.56	347.39
F	FA	74.492	115 11 20	−10"	115 11 10	232° 14' 50"	−47.01 /	−57.84 /	0.0046	0.0188	−47.01	−57.84	249.55	289.55
A		474.506	720 1 0	−60"	720 0 0	180° 00' 00"	−0.131	0.032						

Total correction in latitude = ΣL * (length of side / total length) × correction

$C_B = \frac{l_{BC}}{\text{total length}} \times \text{correction} = \frac{85.922}{474.506} \times 60'' = 10.86''$

$C_C = \frac{l_{CD}}{\text{total length}} \times \text{correction} = \frac{60.367}{474.506} \times 60'' = 7.63''$

$C_D = \frac{l_{DE}}{\text{total length}} \times \text{correction} = \frac{57.482}{474.506} \times 60'' = 7.27''$

$C_E = \frac{l_{EF}}{\text{total length}} \times \text{correction} = \frac{105.844}{474.506} \times 60'' = 13.39''$

$C_F = \frac{l_{FA}}{\text{total length}} \times \text{correction} = \frac{74.492}{474.506} \times 60'' = 9.42''$

Error, $e = \sqrt{(\Sigma l)^2 + (\Sigma d)^2} = \sqrt{(0.131)^2 + (0.032)^2} = 0.135$

Accuracy = $\frac{\text{length/error}}{\text{length}} = \frac{474.506/0.135}{1} = \frac{3514.86}{1} = \frac{3514.86}{3000}$

* Calculate consecutive as well as independent coordinate including closing error for the following traverse

Line	AB	Bc	cD	DE	EA
length(m)	89.31	219.76	151.18	159.10	232.26
W.C.B	45°30'	72°05'	161°52'	228°03'	300°42'

Assume coordinate of A (1000m, 1000m)

By using Bowditch's method

Line	length (m)	W.C.B	Consecutive coordinate: Dlong $l\sin\theta$	Dlat $l\cos\theta$	Correction C_x	Correction C_y	Corrected Dlong	Corrected Dlat	Independent coordinate: X (m)	Y (m)
									1000	1000
AB	89.31	45°30'	63.595	62.754	0.0332	0.0328	63.627	62.787	1063.627	1062.787
Bc	219.76	72°05'	209.103	67.605	0.0817	0.0264	209.185	67.631	1272.812	1130.418
cD	151.18	161°52'	47.855	-143.91	0.0562	-0.0905	47.911	-144.00	1320.723	986.418
DE	159.10	228°03'	-118.552	-106.412	0.0591	0.0952	-118.493	-106.317	1202.230	880.101
EA	232.26	300°42'	-199.878	114.978	0.0863	-0.1390	-199.792	114.839	1002.438	994.940
Total	850.61		0.523	0.508						

Closing error $(E) = \sqrt{\Sigma x^2 + \Sigma y^2} = \sqrt{0.523^2 + 0.508^2} = 0.654$

Error $= \pm 1.151 \times \sqrt{n} = \pm 1.151 \times \sqrt{5} = \pm 2.98$

Precision $= \frac{1}{\text{Length/e}} = \frac{1}{8500/0.654} = \frac{1}{13000}$

Actual Precision $= \frac{1}{\text{Length/e}} = \frac{1}{8500/2.98} = \frac{1}{2852}$

Transit rule,

$C_L = \frac{\Sigma L \times \text{Latitude where } L}{\Sigma L}$ = Arithmetic sum of latitude

$C_D = \frac{\Sigma D \times \text{Departure where } D}{\Sigma D}$ = Arithmetic sum of departure.

* Omitted Measurements

→ Sometimes, it is impossible to take all measurements due to obstacle or some quantity is missing to measured. In this condition, the missing data can be calculated by providing latitudes & departures.

Since, for a closed traverse, ΣL and ΣD are zero, we have

→ $\Sigma L = l_1\cos\theta_1 + l_2\cos\theta_2 + l_3\cos\theta_3 + \dots = 0$

$\Sigma D = l_1\sin\theta_1 + l_2\sin\theta_2 + l_3\sin\theta_3 + \dots = 0$

→ There are four general cases of omitted measurements

1) a) When the bearing of one side is omitted.
   b) When the length of one side is omitted.
   c) When the bearing and length of one side is omitted.
2) When the length of one side and bearing of another side are omitted.
3) When the lengths of two sides are omitted.
4) When the bearing of two sides are omitted.

1) Bearing or length or both of one side omitted:-

→ In this condition, the bearing or length or both bearing & length of one side is missing to measure.

Q. In a closed traverse running anticlockwise, calculate the following missing data.

Line	Length (m)	W.C.B
AB	343.56	245°18'
Bc	375.08	113°37'
cD	?	?
DE	417.66	37°25'
EA	457.26	323°42'

→

Line	length (m)	Bearing	Latitude (Lcosθ)	departure (Lsinθ)
AB	343.56	245°18'	-143.562	-312.127
BC	371.08	113°37'	-148.660	340
CD	?	?	Lcosθ	Lsinθ
DE	417.66	37°26'	331.721	253.773
EA	457.25	321°42'	358.838	-283.393

$\Sigma L = -143.562 - 148.660 + L\cos\theta + 331.721 + 358.838$

$= 398.337 + L\cos\theta$

Similarly,

$\Sigma D = -1.747 + L\sin\theta$

But for close traverse,

$\Sigma L = 0 \Rightarrow L\cos\theta = -398.337$ —— (i)

$\Sigma D = 0, \Rightarrow L\sin\theta = +1.747$ —— (ii)

eqn (i) & eqn (ii)

$\frac{\cos\theta}{\sin\theta} = \frac{-398.337}{1.747}$

$\Rightarrow \tan\theta = \frac{-1.747}{398.337}$

$\Rightarrow \theta = -0.25°$

Length of line,

$L = \sqrt{(L\sin\theta)^2 + (L\cos\theta)^2}$

$= \sqrt{398.337^2 + 1.747^2}$

$L = 398.34\,m$

3) Length of one side & bearing of another adjacent side is omitted.

Q) In a closed traverse ABCDE running anticlockwise, calculate the missing data.

Line	Length (m)	W.C.B
AB	343.56	245°18'
BC	371.08	?
CD	?	113°37'
DE	417.66	37°25'
EA	457.25	321°42'

→ Join B & D, To calculate bearing & length of line BD

Consider a closed traverse ABDEA,

Line	Length	Bearing	L = Lcosθ	D = Lsinθ
AB	343.56	245°18'	-143.562	-312.127
BD	L	θ	Lcosθ	Lsinθ
DE	417.66	37°25'	331.721	253.773
EA	457.25	321°42'	358.838	-283.393



For closed traverse,

$\Sigma L = 0$ & $\Sigma D = 0$.

$\Rightarrow 546.997 + L\cos\theta = 0$ & $-341.747 + L\sin\theta = 0$

$\Rightarrow L\cos\theta = -546.997$ —— (i) $L\sin\theta = 341.747$ —— (ii)

eqn (ii) divided by (i)

$\tan\theta = -0.6247$ & $L = 644.97\,m$

$\theta = -31.995°$

Find α & apply sine law.

3) Bearing of two adjacent sides are omitted.

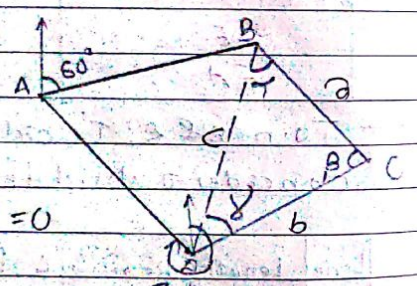
Line	Length	WCB
AB	550	60°
BC	1200	?
CD	880	?
DA	1050	310°

→ To find bearing & length of BD,

$\Sigma L = 0$

$\Rightarrow 550\cos 60° + L\cos\theta + 1050\cos 310° = 0$

$\Rightarrow L\cos\theta = -949.926$ — (i)



$\Sigma D = 0$

$\Rightarrow 550\sin 60° + L\sin\theta + 1050\sin 310° = 0$

$\Rightarrow L\sin\theta = 328.032$ — (ii)

eqn (ii) / eqn (i)

$\tan\theta = -0.3453$

$\theta = -19.051° = -19° 3' 3''$

Length (L) = 1005 m.

Since, ΣL is positive & ΣD is negative. So line BD falls in fourth quadrant.

$\therefore \theta_{BD} = 270 + 19° 3' 3'' = 289° 3' 3''$

$s = \frac{a+b+c}{2} = \frac{1200 + 880 + 1005}{2} = 1542.5$ m

Also,

$\Delta = \sqrt{s(s-a)(s-b)(s-c)} = 433735.58$ — (i)

As we know,

$\Delta = \frac{1}{2}ac\sin\alpha = \frac{1}{2}ab\sin\beta = \frac{1}{2}bc\sin\gamma$ — (ii)

Using (i) & (ii),

$\alpha = 46° 00'$ $\beta = 55° 13' 55''$ & $\gamma = 78° 46' 17''$

Bearing of DC = Bearing of BD + $\gamma \pm 180°$

$= 289° 3' 3'' + 78° 46' 17'' - 180°$

$= 187° 49' 20''$

Bearing of CD = $187° 49' 20'' - 180° = 7° 49' 20''$

Bearing of CB = Bearing of DC + $\beta \pm 180°$

$= 187° 49' 20'' + 55° 13' 55'' - 180°$

$= 63° 3' 15''$

Bearing of BC = $243° 3' 15''$

4) Length of two sides are omitted.

Side	Length	Bearing (WCB)
AB	500	98° 30'
BC	620	30° 20'
CD	468	298° 30'
DE	L_1	230° 0'
EA	L_2	150° 30'

→

Side	length	WCB	Latitude	departure
AB	500	98° 30'	−73.9047	494.507
BC	620	30° 20'	535.123	313.118
CD	468	298° 30'	223.310	−411.286
DE	L_1	230° 0'	$-0.6427L_1$	$-0.7664L_1$
EA	L_2	150° 30'	$-0.8674L_2$	$0.4974L_2$

$\Sigma L = 0$

$\Rightarrow 0.6427L_1 + 0.8674L_2 = 684.528$ — (i)

$\Sigma D = 0$

$0.7664L_1 - 0.4974L_2 = 396.339$ — (ii)

On solving (i) & (ii)

$L_1 = 695.32$

$L_2 = 273.976$

* Tacheometric Surveying:-

→ It is the method of surveying in which horizontal distance & relative vertical elevations are determined from subtended interval & vertical angles observed with an instrument.

→ Tacheometer is used for measuring horizontal distance & relative vertical elevations in Tacheometry.

Tacheometer is a transit theodolite having stadia telescope filled with horizontal hairs called stadia hairs in addition to usual cross hair.

Application of Tacheometric Surveying:-

- preparation of topomap
- Survey work in difficult terrain
- Establishing secondary control points (fly point)
- Hydrographic survey
- Detail filling
- Reconnaissance surveys for highways, railways etc
- checking of already measured distances.

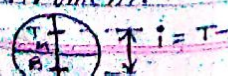
* System of Tacheometric Surveying:-

① Stadia method → Fixed hair (stadia)
② Non-stadia method → Movable hair (subtense)

a) Fixed hair Method (Stadia method)

In fixed hair method, the stadia hairs are fixed or constant distance apart. The reading on the staff corresponding to all three wires are taken. The staff intercept (T-B) depends on the distance of the staff from the instrument.

(P) Stadia interval is constant.

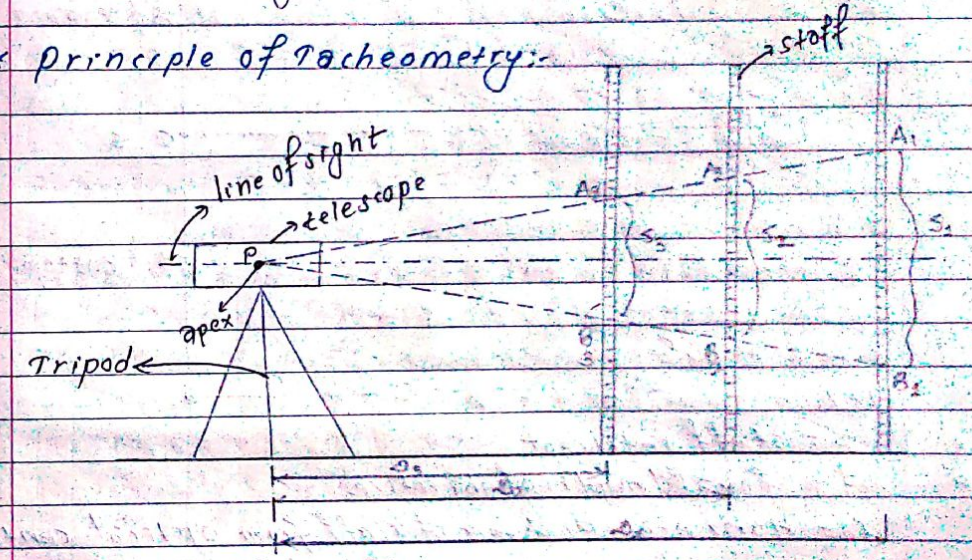


System of tacheometric surveying.
① stadia system
② Tangential system
③ Measurement by means of special instrument.

b) Movable hair method (subtense method)

This method is similar to fixed hair method except that the stadia interval is variable. Suitable arrangement is made to vary the distance betⁿ stadia hairs.

* Principle of Tacheometry:-



Isoceles triangle: PA_1B_1, PA_2B_2 & PA_3B_3.

In an isosceles triangle, the ratio of perpendicular distance from apex to base & base width is always constant.

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

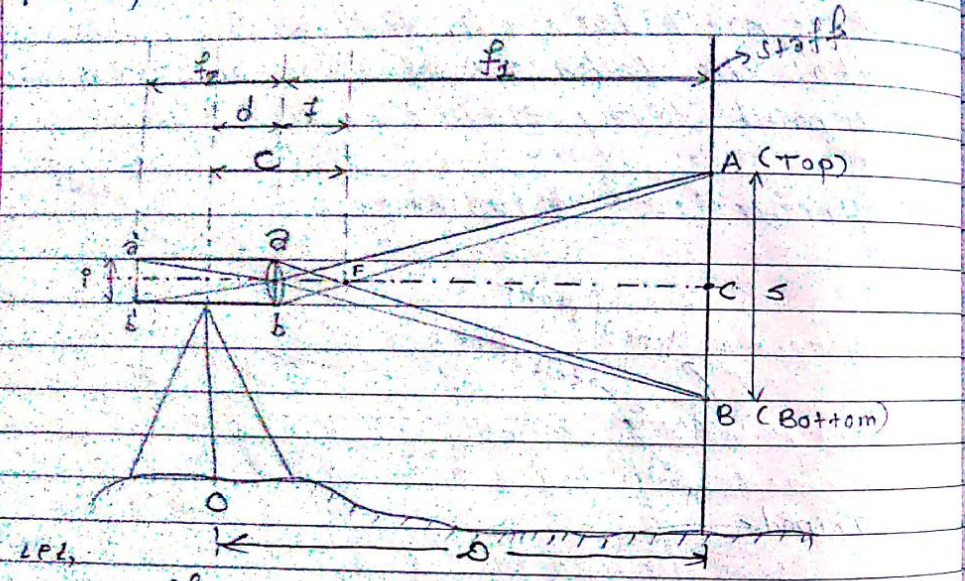
Here, S_1, S_2 & S_3 are staff intercept

f → Focal length of objective.

i → stadia interval.

f/i → multiplying constant [Generally taken 100]

* Principle of stadia method:-



Let,

S = staff intercept

f = focal length of objective.

f_1 = Horizontal distance of staff from optical centre of the objective.

f_2 = Horizontal distance of the cross wires from 'b'

d = distance of vertical axis of instrument from 'a'.

D = Horizontal distance of staff from vertical axis of instrument.

By laws of optics, the images of A & B of the staff will appear along the stadia hair at 'a' b respectively.

From similar Δ, abF & ABF

$$\frac{ab}{f} = \frac{AB}{Fc}$$

$$\Rightarrow Fc = f * \frac{AB}{ab} = f * \frac{S}{i}$$

Then, distance from vertical axis of instrument to staff is given by,

$$D = d + f + Fc$$

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

$$D = KS + C$$

where,

$K = \frac{f}{i}$ = multiplying constant (100)

$C = (d+f)$ = Additive constant of instrument.

The point 'F' in figure is called anallactic point.

Elevation of staff station = Elevation of instrument axis − central hair reading.

* Distance & Elevation formula for staff vertical & normal but line of sight is inclined:-

1) Vertical staff:-

Let,

P = Instrument station

M = position of instrument axis

Q = staff station

O = Optical centre of the objective.

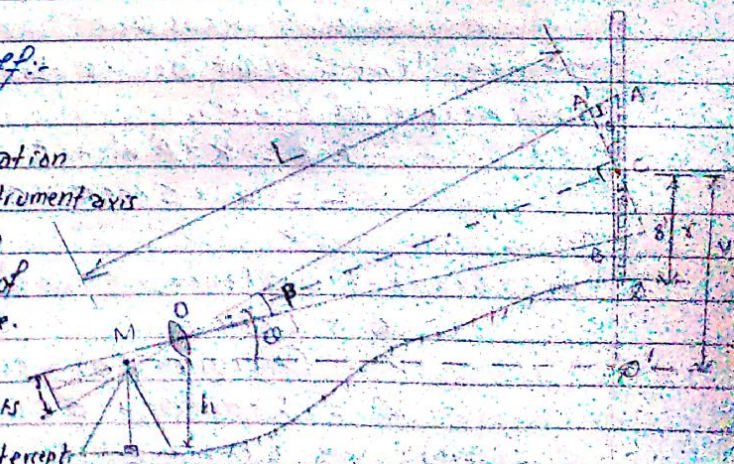
A, C, B → three hairs reading points

S = AB = staff intercept

i = stadia interval

Φ = Inclination of line of sight with horizontal

L = Length MC measured along line of sight.



D = Horizontal distance bet^n instrument & staff
V = vertical diff bet^n line of sight & horizontal line.
h = Height of instrument
r = central hair reading
β = Angle bet^n two extreme rays corresponding to stadia hairs.

From figure,

$\angle AA'c = 90° + \beta/2$

$\angle BB'c = 90° - \beta/2$

$\beta/2$ is very small. so, we can neglect it, then,

$\angle AA'c = \angle BB'c \approx 90°$

From $\triangle ACA'$

$$\cos\theta = \frac{A'c}{AC}$$

$A'c = AC\cos\theta$

$A'B' = AB\cos\theta$

$A'B' = S\cos\theta$.

Since, line of sight 'oc' is ⊥^{r} to line A'B'. so, From principle of stadia,

$Mc = K * A'B' + C$

$L = K * S\cos\theta + C$

Then,

Horizontal distance,

$D = L\cos\theta$

$D = (KS\cos\theta + C) * \cos\theta$

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

Similarly, $V = L\sin\theta$

$V = (K * S\cos\theta + C)\sin\theta$

$V = KS\sin\theta\cos\theta + C\sin\theta$

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

Since, 'C' is very small, so, if we neglect 'C',

$D = KS\cos^2\theta$ &

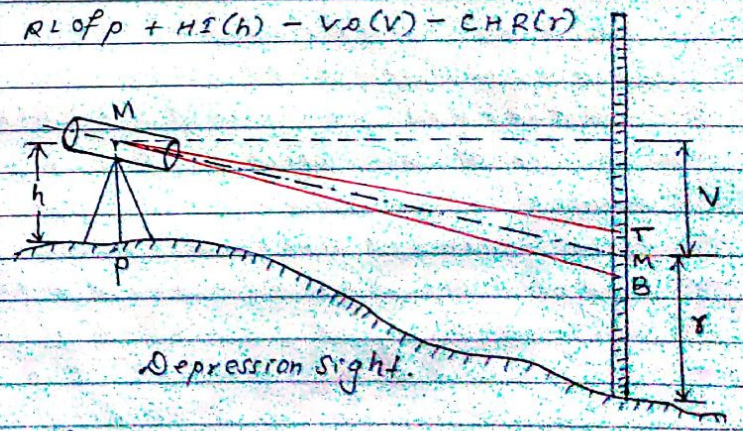
$$V = \frac{KS\sin 2\theta}{2}$$

* Elevation of staff station for angle of elevation:

Elevation of staff station = RL of P + HI + vertical diff(V) − central hair reading (r)

* Elevation of staff station for angle of depression,

Elevation of staff station = RL of P + HI(h) − V.D(V) − CHR(r)



2) Normal staff:

Case(a): Line of sight at an angle of elevation 'θ'

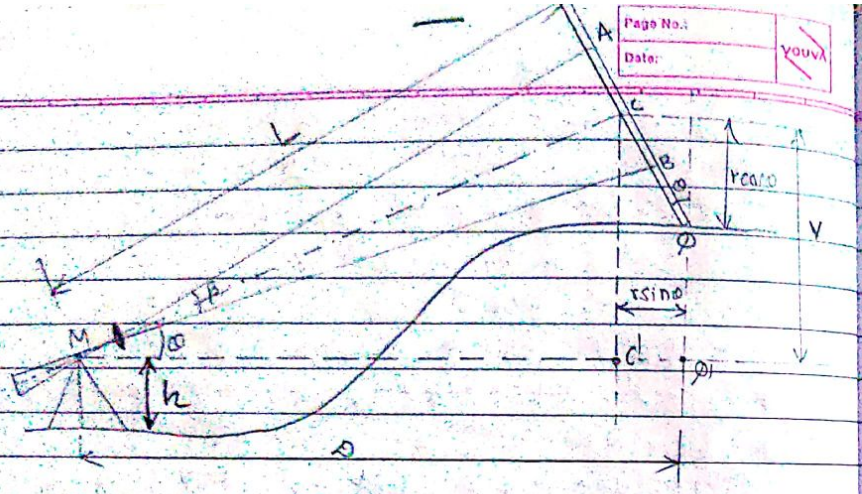
Let,

AB = S = staff intercept.

CP = r = Axial hair reading.

With the same notation as in previous case,

$M.c = KS + C$



The horizontal distance beth p & ϕ is given by

$D = MC' + C'\phi'$

$D = L\cos\theta + r\sin\theta$

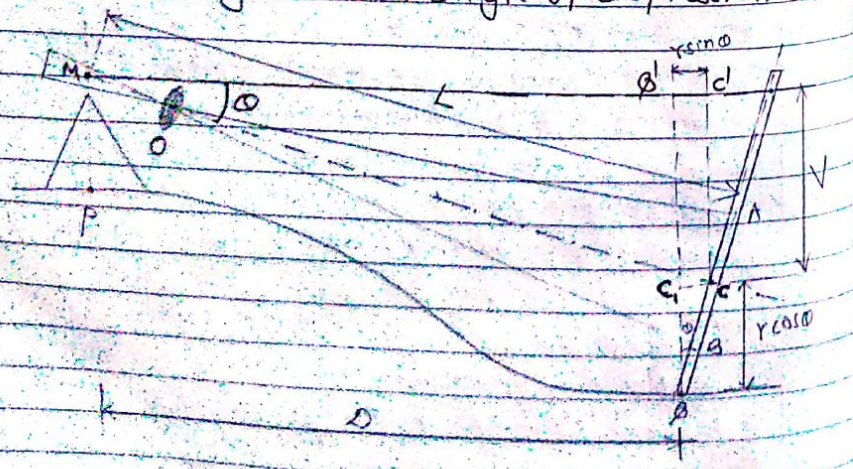
$D = (Ks+C)\cos\theta + r\sin\theta$.

Similarly,

$V = (Ks+C)\sin\theta$. $\quad [V = L\sin\theta]$

Elevation of ϕ = Elevation of $p + h + V - r\cos\theta$.

Case 'b': Line of sight at an angle of depression 'θ'.



Here, $MC = L = KS + C$

$D = MC' - C'\phi'$

$= L\cos\theta - r\sin\theta$

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

Also,

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

Then, Elevation of ϕ = RL of $p + h - V - r\cos\theta$.

Q. The elevation of point 'p' is to be determined by observation from two adjacent station of tacheometric survey. The staff was held vertically upon the point & the instrument is fitted within an anallactic lens, the constant of the instrument being 100. Compute elevation of point 'p' from following data.

Inst station	Height of axis	staff point	Vertical Angle	staff reading	Elev. of station
A	1.42	P	$+2°24'$	1.230, 2.055, 2.880	77.750m
B	1.40	P	$-3°36'$	0.785, 1.800, 2.815	97.135m

a) observation from A to P,

$S = 2.880 - 1.230 = 1.650$m

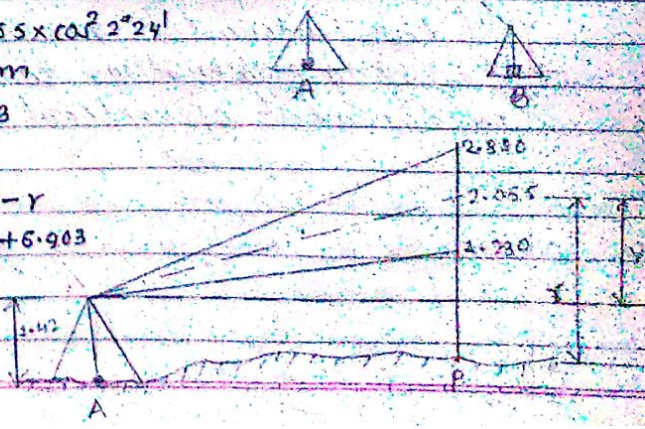
$D = KS\cos^2\theta = 100 \times 1.65 \times \cos^2 2°24' = 164.7$m

$V = \frac{KS\sin 2\theta}{2} = 6.903$

RL of p = RL of A $+ h + V - r$

$= 77.750 + 1.420 + 6.903 - 2.055$

$= 84.018$m

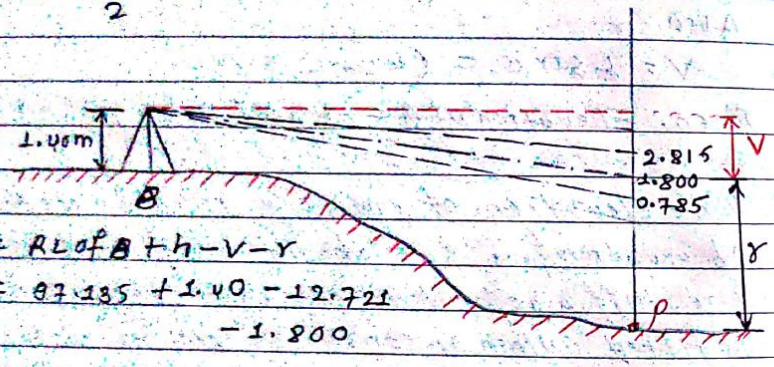


b) Observation from B to P.

$S = 2.835 - 0.785 = 2.03\ m$

$D = K S \cos^2\theta = 100 \times 2.03 \times \cos^2 3°26' = 202.2\ m$

$V = \frac{K S \times \sin 2\theta}{2} = 12.721\ m$



RL of P = RL of B + h − V − r

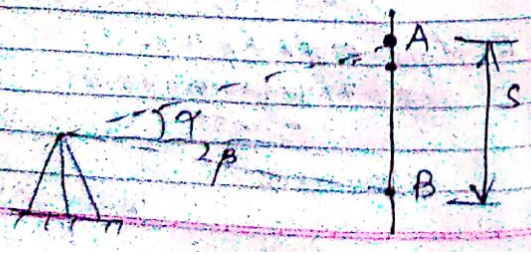
= 97.135 + 1.40 − 12.721 − 1.800

= 84.014 m.

Average elevation of P = $\frac{1}{2}(84.018 + 84.014) = 84.016$ m

* Tangential Method:-

→ In this method staff hair are not used.
→ The reading being taken against the horizontal cross hair.
→ To measure staff intercept, two pointings of the instrument are necessary.
→ For single observation, there is necessity of measurement of vertical angles twice.



* Contouring:-

* Contour:-

→ A contour is an imaginary line on the ground joining the points of equal elevation.
→ A contour line is the line on the map representing a contour.
→ Contouring is the science of representing the vertical dimension of terrain on two dimensional map.

* Contour interval:-

→ The vertical distance betⁿ any two consecutive contours is called contour interval.
→ Contour interval is constant for a particular map
→ Contour intervals for flat country are generally small, eg:- 0.25, 0.5, 0.75 m etc. CI for steep slope in hilly area is generally greater. eg: 5m, 10m, 50 m etc.
→ The horizontal distance betⁿ two points on two consecutive contours is known as horizontal equivalent
→ The contour interval depends upon

1) Nature of ground:-
For flat ground → small contour interval is necessary
For steep slope ground → Large CI (30m, 40m etc) is required

2) Scale of the map:-
CI is inversely proportional to scale of map.

3) The purpose & extent of survey:-
For detailed & accurate design work, small contour interval is used & for minor purpose, large contour interval is used.

4) Time & expense of field & of fice work:-
less time, less money → more contour interval & vice versa.

7) The depression between two summit is called saddle. It is denoted by four set of contour lines.





Characteristics of contours:

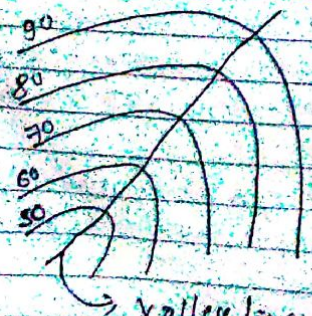
1) All points in a contour line have the same elevation.
2) Two contour line never cross with each other except in case of overhanging cliff.
3) Contour line close together indicate steep slope
   Contour line far apart indicate gentle slope
   Equally spaced contour line indicate → uniform slope
   A series of straight parallel & equally spaced contours represent a plane surface.
4) A series of closed contour lines on the map represent a hill, if the higher values are inside.

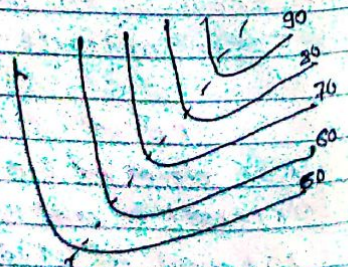


5) A series of closed contour lines on the map represent a depression if higher values are outside.



6) The contour lines cross a ridge or valley lines at right angles. If the higher values are inside the bend or loop in the contour, it indicates a ridge & if the higher values are outside the bend, it indicates a valley.





Methods of Contouring

① Direct method ② Indirect method
- ↳ square method
- ↳ cross section method
- ↳ Tacheometeric survey method.

1) Direct Method:-

In this method, the contour to be plotted is actually traced on the ground. Only those points are surveyed which happen to be plotted. After having surveyed those points, they are plotted & contours are drawn through them. The method is slow & tedious & is used for small areas & where great accuracy is required. In direct method field work is divided into two parts.

a) vertical control:- location of points on the contour (using staff)
b) Horizontal control:- Survey of these points.





contour line.

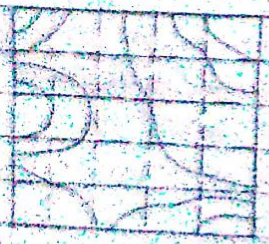
Large number of trials should be done to find survey points on ground of some elevation.

a) Indirect method of contouring:

In this method, some suitable guide points are selected & surveyed. the guide points need not necessarily be on the contours. These guide points having been plotted, serve as basis for interpolation of contours. This is the method most commonly used in engineering surveys.

a) Square method:

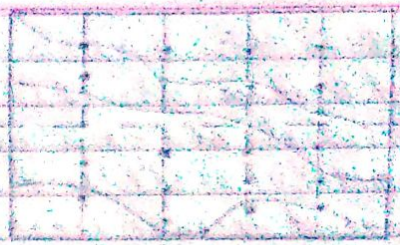
This method is used when area to be surveyed is small & ground is not much undulating. The area to be surveyed is divided into a number of squares. The size of squares may vary from 5 to 20m depending upon nature of contour & contour interval. The elevation of corners of squares are then determined by means of level & staff. The contour line may then be drawn by interpolation. This method is also known as spot levelling



Square method of contouring

b) Cross-sections method:

The cross-section method is most suitable for survey of long narrow strips such as a road, railway, canal etc. In this method, points are taken almost at regular intervals across cross section. The contour points can be located directly on the cross section as



The points are then plotted & contours are drawn by interpolation.

Fig. Cross section method

c) Tacheometric Method:

In case of hilly terrain, tacheometric method is used. Since, the tacheometer provides both horizontal as well as vertical control. the tachometer may be set on a point from which greater control can be obtained. Radial lines can be set making different angles with magnetic meridian as first radial line. On each radial line, readings may be taken on levelling staff kept at different points. The point must be so chosen that approximate vertical difference in elevation between two consecutive points is less than the contour interval. The elevation of each point is calculated by tacheometric calculation & Contours can be interpolated as usual.

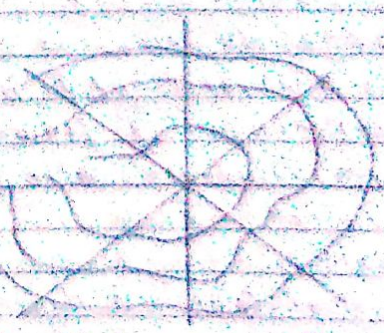


Fig. Tacheometric Method of contouring

* Uses of contour line:

① Drawing of sections:
② Determination of intervisibility betⁿ two points
③ Tracing of contour gradients & location of route.
④ Measurement of drainage areas
⑤ Calculation of Reservoir capacity.
⑥ Intersection of surfaces & measurement of Earth work.

* Interpolation of contours:

Interpolation of contours is the process of spacing the contours proportionately between the plotted ground points established by indirect methods. The main method of interpolation are
① By estimation
② By arithmetic Calculation
③ By graphical Method.

① By Estimation:

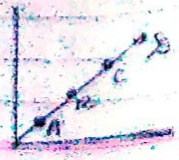
This method is extremely rough & is used for small scale work only. The positions of contour points between guide points are located by estimation.

② By arithmetic calculation:

This method is accurate but time consuming. The positions of contour points between guide points are located by arithmetic calculation.

③ By Graphical method.

In this method, interpolation is done with the help of tracing paper sheet.



Triangulation & Trilateration:

→ Triangulation is the process of measuring the angles of a chain or network of triangles formed by stations marked on the surface of earth. The system consists of number of interconnected triangle in which length of only one base line & the angles of triangle are measured very precisely which are used to calculate the coordinate of vertices.

→ Trilateration is the method of surveying in which length of all all sides of triangle are measured & few angles are measured to check our survey work. The length of side of triangle can be measured by using E.D.M.

Difference between Triangulation & Trilateration:

Triangulation	Trilateration
① All angles are measured in triangulation	① All sides are measured in trilateration
② A distance of baseline is measured	② Azimuth of initial line is measured
③ Some check base lines are also measured to control scale error.	③ So are check angles are measured to control azimuth error.
④ Intervisibility between stations essential	④ Intervisibility between stations not essential
⑤ More internal checks	⑤ Less internal checks
⑥ Side length calculated using angles	⑥ Angles can be calculated using sides
⑦ Less accurate as compared trilateration	⑦ More accurate
⑧ More economic	⑧ Less economic
⑨ Scale control is not possible	⑨ Scale control is provided
⑩ More number of surveyors required	⑩ Less number of surveyor involved in trilateration.

* Principle of triangulation & trilateration

→ The principle of triangulation is that, if all the three angles & length of one side of triangle are known, then the length of remaining side can be calculated by using trigonometry. (sine law).

→ The principle of triangulation is that, if length of all sides of triangle & angles only one angle is known, the remaining angles can be calculated using trigonometry (sine law).

* Classification of triangulation system:
On the basis of extent, purpose & accuracy required in survey work, triangulation system can be classified into three types.
1) Primary / First order Triangulation
2) Secondary / Second-order Triangulation
3) Tertiary / Third-order Triangulation.

1) Primary / First order triangulation:
→ Highest grade of triangulation.
→ Used to determine shape or size of earth or to cover a vast area i.e. a whole country.
→ Station of first order triangulation are generally selected 25 to 150 KM apart.

2) Secondary / Second order Triangulation:
→ The secondary triangulation consists of a number of points fixed within the framework of primary triangulation.
→ It provide points closer together than that of primary
→ Station of second order triangulation are generally 10-25 KM apart.
→ used to survey the small area such as province district, zone etc.

3) Tertiary / Third order triangulation:
→ The tertiary triangulation consists of a number of points fixed within the framework of secondary & primary triangulation
→ Station of second order triangulation are generally 2-10 km apart.
→ Used for purpose of detailed engineering & other surveys.
→ The accuracy of measurement decreased when we come from primary to tertiary triangulation.

* Strength of figure:
→ The strength of figure is a factor to be considered in establishing a triangulation system to maintain the computation within a desired system.
→ It plays also an important role in deciding the layout of a triangulation system.
→ The accuracy in triangulation depends on
i) Magnitude of angles in individual triangle
ii) Arrangement of triangle (Shape of triangle)
→ Strength of figure suggests that the triangles must be well conditioned (almost equilateral) & range of angle must be (30° to 120°).

* Purpose of triangulation & trilateration
- Establishment of accurate control points for different types of survey work.
- Accurate location of engineering work:
a) Fixing centerline, terminal points for tunnel
b) Fixing centerline & abutment of big bridge.
c) To refer the control point across wide sea channel.

Introduction to Field application

The field application of triangulation survey is divided into two parts:

a) Field work

b) Computation & Calculations.

A) Field Work:-

Field work of triangulation consists of following items

i) Reconnaissance:-

- Rough survey / preliminary field inspection of entire area covered by triangulation
- Collection of relevant data.

ii) Erection of signal & tower:-

The tower is constructed over a station to the support the instrument & the instrument consists of a signal which is detected by the wave passes through EDM. By using this, triangulation stations are selected.

iii) Measurement of baseline:-

The accuracy of entire triangulation system depends on measurement of baselines. For short distance, it can be measured by using tape & for long distance EDM is used.

iv) Measurement of horizontal angles:-

The horizontal angles are measured using electronic theodolite.

v) Measurement of vertical angles:-

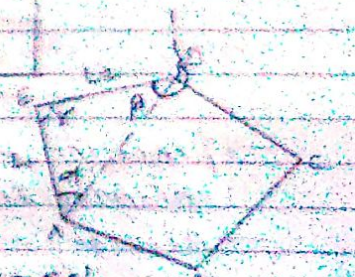
Measurement of vertical angles is required to compute the elevation of triangulation stations.

B) Computation & Calculation:-

The data are measured in field is computed & all required calculation is done using principle of triangulation.

Q Find missing length

Side	Length	WCB
AB	500	68° 30'
BC	620	30° 20'
CD	468	238°30'
DE	?	248°
EA	?	158° 10'



In closed traverse ABCDA,

$\Sigma L = 500 \cos(68°30') + 620 \cos(30°20') + 468 \cos(238°30')$

$= 684.528 m$

$\Sigma D = 396.34 m$ $\quad \Sigma L = -\Sigma L$ & $\Sigma D = -\Sigma D$

Since both ΣL & ΣD are negative so, line AD lies in third quadrant.

$\tan\theta = \frac{\Sigma L}{\Sigma D}$

$\theta = \tan^{-1}\left(\frac{\Sigma L}{\Sigma D}\right) = 30°4'$

Bearing of DA = $180° + 30°4' = 210°4'$

& Length of DA = $\sqrt{684.528^2 + 396.34^2} = 790.99 m$.

β = bearing of DE - bearing of DA = $228° - 210°4' = 17°55'$

γ = bearing of EA - bearing of ED = $158°10' - 50° = 108°10'$

$\alpha + \beta + \gamma = 180^\circ$

$\alpha = 180^\circ - \beta - \gamma$

$= 180^\circ - 19^\circ 55' - 100^\circ 10'$

$= 59^\circ 55'$

Using sine law

$$\frac{L_1}{\sin\beta} = \frac{L_2}{\sin\alpha} = \frac{L}{\sin\gamma}$$

$$L_1 = \frac{\sin\beta \times L}{\sin\gamma} = \frac{\sin(19^\circ 55') \times 791}{\sin(100^\circ 10')} = 273.75m$$

$$L_2 = \frac{\sin\alpha \times L}{\sin\gamma} = \frac{\sin(59^\circ 55') \times 791}{\sin(100^\circ 10')} = 695.368m$$

→ Precision:- degree of perfection used in instruments, methods and the observations.

→ Accuracy:- Degree of perfection obtained.

NOTES

Accuracy:-

The closeness of the average value to the target or standard value is known as accuracy.

Foreg:- Target diameter = 10 mm

Average of sample = 10.079 mm

Difference = $|10.079 - 10.00| = 0.079$ mm

The difference gives whether data are accurate or not.

Precision

The closeness of individual values to each other.

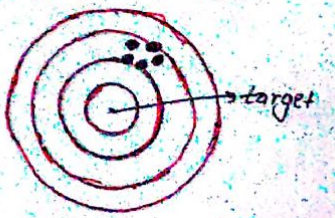
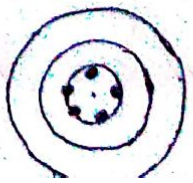
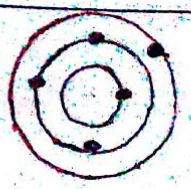
Foreg:- Largest drameter of sample (L) = 10.42 mm

Smallest drameter of sample (S) = 9.83 mm

Difference betⁿ maxᵐ & minᵐ value = 0.59 mm

The diffⁿ betⁿ maxᵐ & minᵐ value shows the closeness of individual value to the target value.

✻ Archer's Bow target technology:-

	High Accuracy	Low accuracy.
High precision		
Low precision		

NOTES

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Principle of EDM:-

An electronic wave is transmitted from instrument (i.e at A) towards the reflector (i.e at B) and reflector reflects the wave back towards the transmitting unit. The equipment at transmitting unit received the reflected wave since it also acts as receiver. Now, by recording the transit time & knowing the velocity of wave, the instrument is able to compute and display the required distance betn the stations. The wave travel double distance, instrument to reflector & reflector to instrument. Hence,

$$2D = Vt \Rightarrow D = \frac{Vt}{2}$$

where,

D = Distance beth two points

V = velocity of wave = 3×10^8 m/s.

t = total transit time

{ frequency of electromagnetic wave = 1000 kHz = 1000 cycles/sec



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a = b
α = β



