LEARNING MATERIAL OF

LAND SURVEY – I

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Surveying :-

Definition :-

⇒ Surveying is the art of determining the relative positions of different objects on the surface of the earth by measuring the horizontal distances between them & by preparing a map to any suitable scale.

→ Measurements are only taken in horizontal plane.

Object of Surveying :-

⇒ The aim of surveying is to prepare a map to show the relative positions of the object on the surface of the earth.

⇒ The map is drawn to some suitable scale. It shows the natural features of the country, such as towns, villages, road, railways, rivers, irrigation canals.

Uses of Surveying :-

(1) To prepare a Topographical map :-

It shows the hills, rivers, villages, towns of a country.

(2) To prepare a cadastral map :-

It shows the boundaries of fields, houses & other properties

(3) To prepare an engineering map :- It shows the details of engineering work such as roads

railways, Irrigation canals, Town Bail.

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(4) To prepare a military map :-

which shows the road & railway communications with different parts of a country. Such a map also shows the different important points for the defence of the country.

(5) To prepare a contour Map :-

It is used to determine the capacity of a reservoir & to find the best possible routes for roads & railways.

(6) To prepare a geological Map :-

which shows the areas excluding underground resources.

(7) To prepare a archaeological map :-

The map excluding places where ancient relics exist.

Classification of Surveying

- Primary Surveying
  - plane Surveying
  - Geodetic Surveying
- Secondary Surveying

1. M. P

Plane surveying

- The shape of earth is spheroidal. Thus the surface of the earth is curved.
- In plane surveying the curvature of the earth is neglected & it is assumed to be flat surface.
- In this surveying a line joining two points is considered to be straight.
- plane surveying is done on an area of less than 250 sq.m.
- plane surveying conducted by state agencies like Irriga-tion department & railway department.

Geodetic Surveying

- In Geodetic surveying the curvature of the earth is considered.
- The line joining any two points is considered as curved line.
- It is used for large area i.e. greater than 250 sq.m.
- This survey is conducted by the department of great trigonometrical survey (G.T.S) of India.

Secondary classification

- Based on Instrument
  - Chain surveying
  - Compass surveying
  - Plane table
  - Theodolite
- Based on methods
  - Triangu-lation
  - Traverse
- Based on object
  - Geological surveying
  - Archaeologi-cal surveying
  - Mine surveying
  - Military surveying
- Based on nature of field
  - Land surveying
  - marine surveying
  - Astronomical surveying

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→ Tacheometric
→ photographic

Based on methods:-

(a) Triangulation surveying :- The lines form a system of triangles.

(b) Traverse surveying:- The various stations forms polygon.

Based on object:-

(a) Geological surveying:- The information about different strata of the earth's surface.

(b) Mine surveying:- To find out the positions and volume of material in mines, bore holes for underground works.

(c) Archaeological surveying:- To bring out the relics of antiquity.

(d) Military surveying:- To determine the routes and points of strategic importance.

Based on nature of field:-

(a) Land surveying:- Topographical survey:- It is defined as the shape or configuration of the earth's surface it is used to locate the cities, hills, valleys, rivers, railway lines, roads, pipe line etc.

(b) Cadastral survey:- To locate property boundaries of land.

(c) City survey:- To locate layout of streets, building, sewers, pipes etc within city limit.

(d) Marine survey:- Survey conducted on or near the body of water such as bay, lake, harbour, river etc. to estimate water flow and to determine shape of areas below the water surface.

(e) Astronomical survey:- survey conducted to determine the latitudes, longitudes, azimuths, local time etc for various places on the earth by observing sun or stars.

1-M-P Principles of surveying :-

- The principle of surveying are :-
(a) To work from whole to the part.
(b) To locate a new station by at least to measurements (linear or Angular) from fixed reference point.

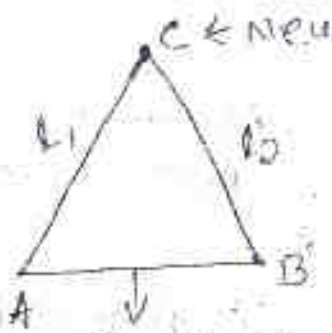
(a) To work from whole to the part :-

> The whole area is first enclosed by main stations & main survey line.
> The area is then divided into a number of well conditioned or equi-lateral triangle.
> The main survey line is measured very accurately. Then the side of triangle are measured.
> During this procedure, if there is any error in measurement of any side of triangle, then it will not affect the whole work. The error can always be detected and eliminated.
> But if the reverse process (from part to the whole) is followed, then the minor error in measurement will get accumulated, magnified & become uncontrollable at the end of survey work.

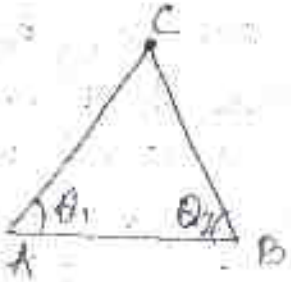
(b) To locate a new station :-

- The new station should always be fixed by at least two measurements from fixed reference point.
- The station can be located by taking
  (a) Linear measurements
  (b) Angular measurements
  (c) Both Linear & Angular measurements

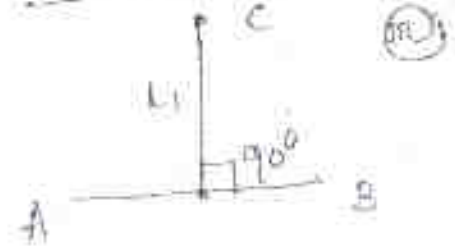
(a) Linear measurements



(b) Angular measurement



(c) Linear & Angular



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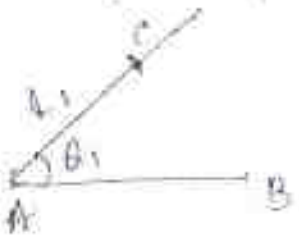

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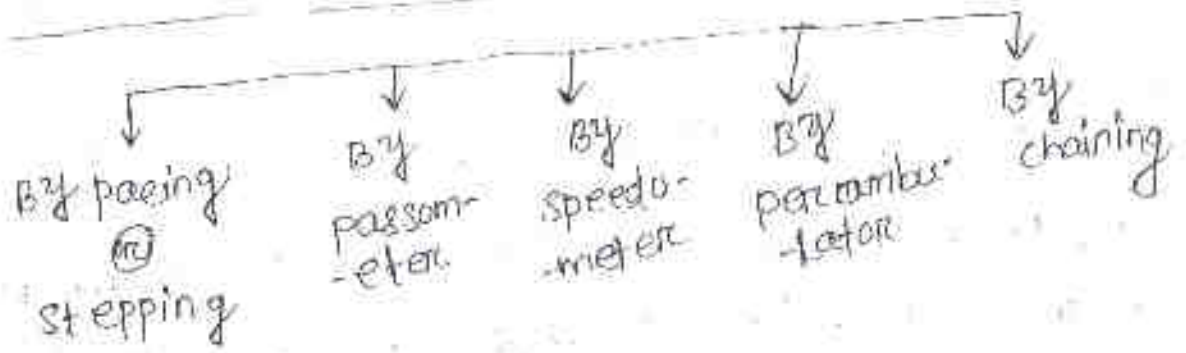
Methods of linear measurement :-



By paeing (or) stepping :-

⇒ For rough & speedy work, distances are measured by paeing, i.e by counting the number of walking steps of a man.

⇒ The walking step of a man is considered 2.5ft (or) 80 c.m.

⇒ This method is generally employed in the reconnaissance survey of any project.

By passometer :-

⇒ A small instrument just like a stop watch i.e passometer which is used for counting the number of steps automatically which ~~by~~ some is a mechanical device.

⇒ It offers an improvement over the normal paeing method when a very long distance is to be measured & when it becomes very tedious to count & extremely difficult to remember to number of steps.

(iii) By speedometer :-

This is used in automobiles for recording distances.

(iv) By perambulation :-

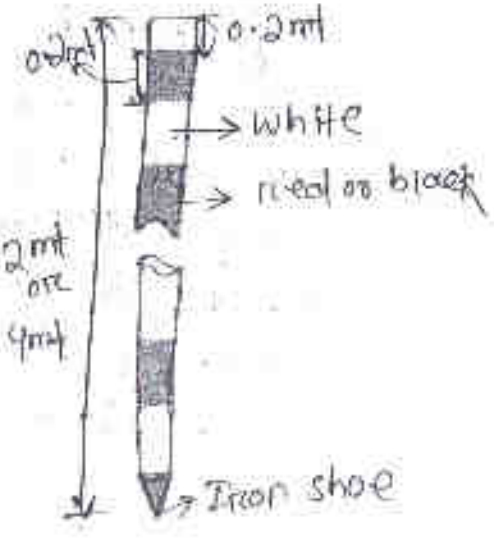
> It is a wheel fitted with a fork & handle.
> The wheel is graduated & shows a distance per revolution.
> There is a dial which records the no of revolution. Thus the distance can be measured.

(v) By chaining

> This is the accurate & common method for measuring the distance.
> In this method for measurement chain (or) Tape is used.

Instruments used for Linear measurements :-

(i) Ranging rod :-



> Rods which are used for ranging (i.e the process of making a line straight) a line are known as ranging rods.
> These are also used for making the main stations in chain surveying.

⇒ These rods are made of seasoned timber (or) seasoned bamboo.

⇒ Sometimes G.I pipes (Galvanised Iron) of 25 m.m diameter are used as ranging rods.

⇒ They are generally circular in section & having length 2mt & 4mt

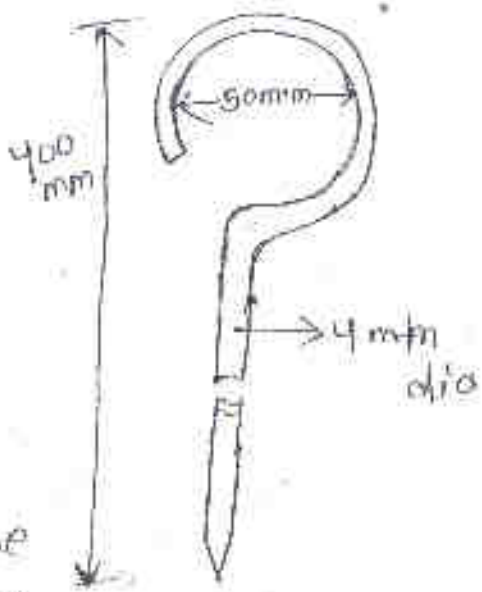
⇒ The rod is divided into equal parts of 20 c.m each & the divisions are painted black & white (or) red & white so that the rod is visible from long distance. The lower end of the rod is pointed & it is called the iron shoe.

(2) Arrow :-

⇒ Arrows are made of steel wire of 4 m.m diameter.

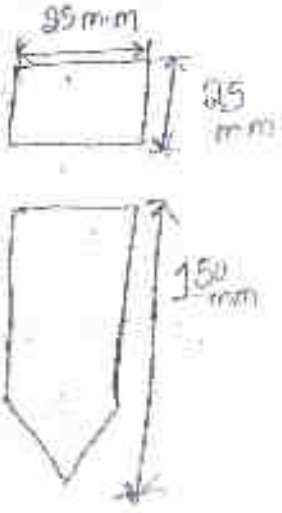
⇒ One end of the arrow is bent into a ring of 50 m.m diameter & the other end is pointed.

⇒ It is used for counting the number of chains while doing chain surveying.



(3) peg :-

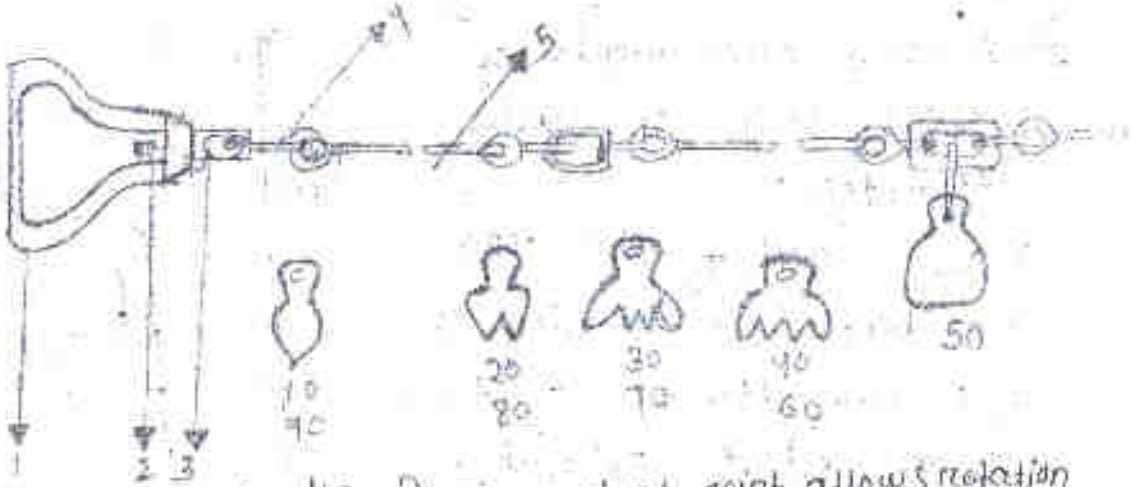
⇒ These are normally made of wood which having length 15 c.m & 2.5 c.m square at the top.



> These are used for marking the substation in surveying in survey line.

(4) Chain

A chain is prepared with 100 or 150 pieces of galvanised mild steel wire of 4 mm diameter. The ends of the pieces are bent to form loops. Then the pieces are connected together with the help of three oval rings, which make the chain flexible. To brass handles are provided at the two ends of the chain. Tallies are provided at every 10 or 25 links for facility of counting. 'one link' means the distance between the centres of adjacent middle rings.



1. Brass handle
2. Collar
3. Eye bolt
4. Circular ring
5. End link

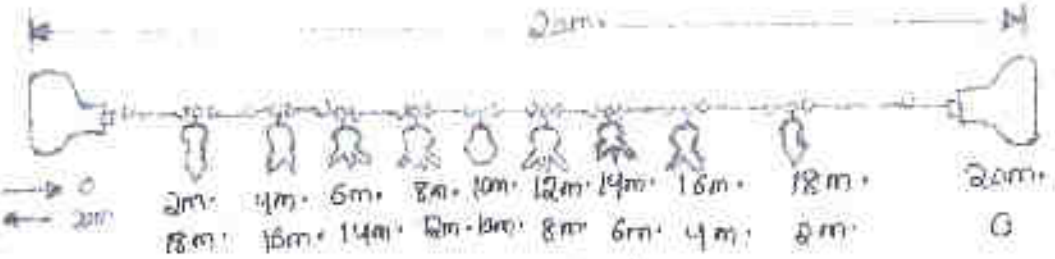
} The swivel joint allows rotation of the handle preventing the deformation due to twist in the end link.

The following are the different types of chain.

(a) Metric chain
(b) Steel band
(c) Engineers chain
(d) Gunter's chain &
(e) Revenue chain

(a) Metric chain :-

Metric chains are available in lengths of 20m. and 30m. The 20m. chain is divided into 100 links, each of 0.2m. talies are provided at every 10 links (2m). This chain is suitable for measuring distances along fairly level ground. The arrangement of tallies is shown :



20M chain

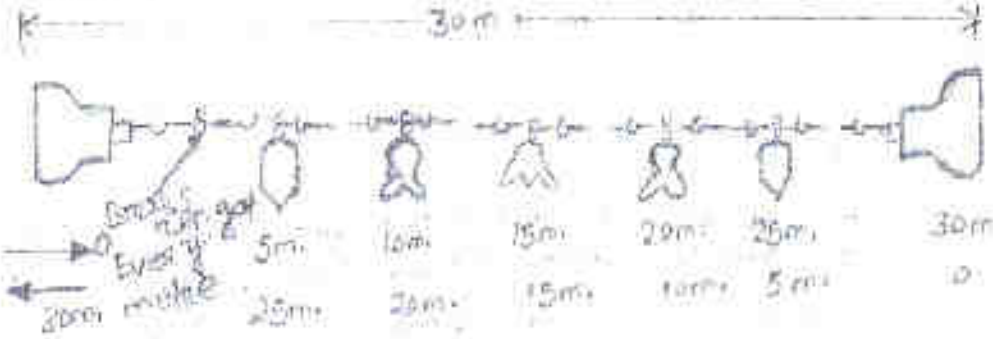
(20m. chain)

You may see from the arrangement of tallies that the central tally is round and that the other tallies have one, two, three or four teeth. So, each tooth may correspond to two different readings when considered from opposite ends. Therefore, during the measurement, the surveyor should bear in mind the position of the central tally.

As per IST recommendations, tallies should be provided after every 5m and brass rings after every 1m. the central tally has two teeth and the

tallies on opposite sides of it have one tooth each.

The 30m. chain is divided into 150 links. So, each link is of 0.2m. The tallies are provided after every 25 links. A round brass ring is fixed after every metre. This chain is heavy and is also suitable for measuring distances along fairly level ground. Here the central tally has three teeth.



(30m. chain)

(b) Steel band :- It consists of a ribbon of steel of 16 m.m. width and of 30 m. length. It has a brass handle at each end. It is graduated in metres, decametres and centimetres on one side and has 0.2m link on the other. The steel band is used in projects where more accuracy is required.

(c) Engineer's chain :- The engineer's chain is 100 ft long and is divided into 100 links. So, each link is of 1 ft. Tallies are provided at every 10 links, the central tally being round. Such chains were previously

used for all engineering works.

(d) Gunter's Chain:- It is 66 ft long and divided into 100 links. So each link is of 0.66 ft. It was previously used for measuring distances in miles and furlongs.

(e) Revenue chain The revenue chain is 33 ft long and divided into 16 links It is mainly used in cadastral survey.

Chains have the following advantages:-

(i) They can be read easily and quickly.
(ii) They can withstand wear and tear.
(iii) They can be easily repaired or rectified in the field.

They have the following disadvantages:-

(i) They are heavy and take too much time to open or fold.
(ii) They become longer or shorter due to continuous use, the chain sags excessively.
(iii) When the measurement is taken in suspension, the chain sags excessively.

steel bands following advantages:-

(i) They are very light and easy to open or fold.
(ii) They maintain their standard length even after continuous use.
(iii) When the measurement is taken in suspensions, they sag slightly.

Disadvantages :-

(i) If handled carelessly, they break easily.
(ii) They cannot be repaired in the field.
(iii) They cannot be read easily.

Tapes :-

The following are the different types of tapes :
(a) cloth or linen tape.
(b) metallic tape.
(c) steel tape &
(d) Invar tape.

(a) cloth or linen tape :-

Such a tape is made of closely woven linen and is varnished to resist moisture. It is 15 m.m wide and available in lengths of 10 and 15 m.m This tape is generally used for measuring offsets and for ordinary works.

(b) Metallic Tape :- When linen tape is reinforced with brass or copper wires to make it durable then it is called a metalic tape. This tape is available in lengths of 15, 20 and 30 m. It is wound on a leather case with a brass handle at the end It is commonly used for all survey works.

(c) Steel Tape :- The steel tape is made of steel ribbon of width varying from 6 to 16 m.m. The commonly available lengths are 10, 15, 20, 30 and 50 m. It is graduated in metres, decimetres and centimetres. It is not used in the field, but chiefly for standardising chains and for measurements in constructional works.

(d) Invar Tape :- Invar tape is made of an alloy of steel (64%) and nickel (36%). Its thermal coefficient is very low. Therefore, it is not affected by change of temperature. It is made in the form of a ribbon of 6 m.m. width and is available in length of 30, 50 and 100 m. It is used at places, where maximum precision is required. It is generally used in the triangulation Survey conducted by the Survey of india department.

RANGING :- The process of establishing intermediate points on a straight line between two end points is known as ranging. Ranging must be done before a survey line is chained. Ranging must be done by direct observation by the naked eye or by the ranger or by theodolite. Generally, ranging is done by the naked eye with the help of three ranging rods.

NOTE :- Line ranger is also used for ranging.

Ranging may be of two kinds:-

(1) Direct, and

(2) Indirect or reciprocal.

Direct Ranging :- When intermediate ranging rods are fixed on a straight line by direct observation from end stations, the process is known as direct ranging. Direct ranging is possible when the end stations are intervisible. The following procedure is adopted for direct ranging.

Assume that A and B are two end station of a chain line. Where two ranging rods are already fixed. Suppose it is required to fix a ranging rod at the intermediate point P on the chain line AB. The surveyor stands about 2 m behind the ranging rod at A by looking towards the line AB. The assistant holds a ranging rod at P vertically at arm's length. The rod should be held lightly by the thumb and forefinger. Now, the surveyor directs the assistant to move the ranging rod to the left or right until the three ranging rods, ~~the~~ Come exactly in the same straight line. To check the non-verticality of the rods, the surveyor bends down and looks through the bottom of the rods. The ranging will be perfect when the three ranging rods coincide and appear as a single rod when the surveyor is satisfied that the ranging is perfect, he signals the assistant to fix the single

ranging rod on the ground by waving both his hands up and down following the same procedure, the other ranging rods may be fixed on the line.

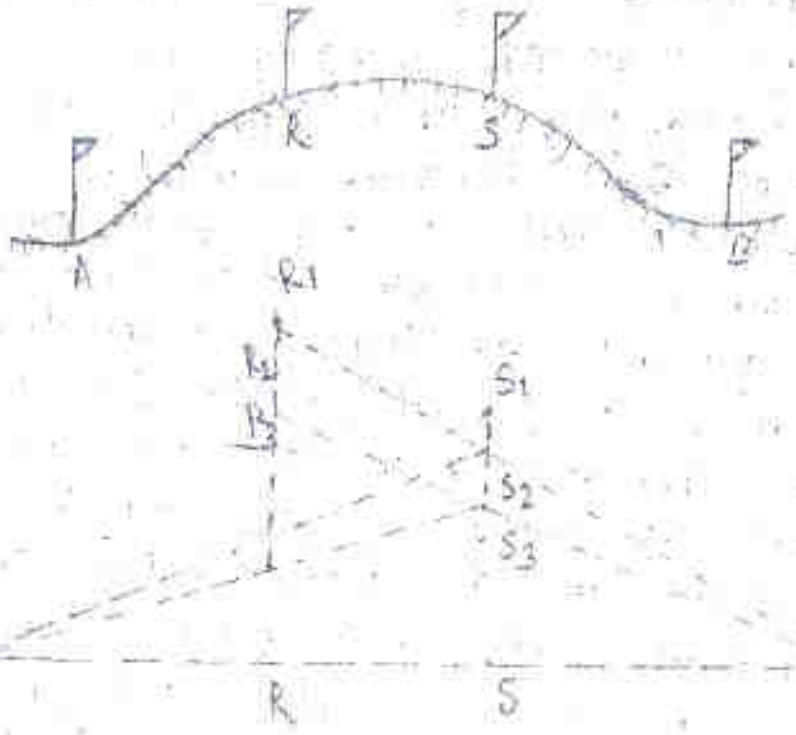
(Direct Ranging)

Indirect or Reciprocal Ranging :-

When the end stations are not intervisible due to there being high ground between them, intermediate ranging rods are fixed on the line in an indirect way. This method is known as indirect ranging or reciprocal ranging. The following procedure is adopted for indirect ranging.

Suppose A and B are two end stations which are not intervisible due to high ground existing between them. Suppose it is required to fix intermediate points between A and B. Two chainmen take up positions at R_1 and S_1 with ranging rods in their hands. The chainman at R_1 stands with his face towards B so that he can see the ranging rods at S_1 and B. Again the chainman at S_1 stands with his face towards A so that he can see the ranging rods at R_1 and A. Then the chainmen proceed to range the line by directing

each other alternately. The chainman at R_1 directs the chainman at S_1 to come to the position S_2 so that R_1, S_2 and B are in the same straight line. Again the chainman at S_2 direct the chainman at R_1 to move to the position of R_2 so that S_2, R_2 and A are in the same straight line. By directing each other alternately in this manner, they change their positions every time until they finally come to the positions R and S, which are in the straight line AB, this means the points A, R, S and B are in the same straight line.



(Reciprocal Ranging)

UNFOLDING AND FOLDING A CHAIN

(1) unfolding :- To open a chain, the strap is unfastened and the two brass handles are held in the left hand and the bunch is thrown forward with the right hand. Then one chainman stands at the starting station by holding one handle and another moves forward by holding the other handle until the chain is completely extended.

(2) folding :- To fold the chain, a chainman should move forward by pulling the chain at the middle. Then the two halves of the chain will come side by side. After this commencing from the central position of the chain, two pairs of links are taken at a time with the right hand and placed on the left hand alternately in both directions finally, the two brass handles will appear at the top. The bunch should be then fastened by the strap.

OBSTACLE IN CHAINING :- 5 may 2021

Conversion table for units :-

Length

1 inch = 2.54 cm

12 inch = 1 Foot

1 Foot = 0.3048 mt

3 foot = 1 yard

5 ½ yard = 1 rod (or) pole

4 pole (66 ft) = 1 chain

10 chain = 1 furlong

8 furlong = 1 mile

6 feet = 1 fathom

120 fathoms = 1 cable length

1 nautical mile → 6,080 feet
1.152 mile
1.852 km

1 mile = 1,760 yard (or)
5,280 feet (or)
1·609 km.

1 hedametre = 10 decametre.

1 kilometre = 1000 metre

Area

1 acre = 4,840 square yards
(or)
→ 3·025 bighas

$1\ km^2$ = 100 hectare

1 hectare = $10,000\ m^2$

1 bigha = 1600 square yard

20 kathas = 1 bigha

LEADER AND FOLLOWER

The chainman at the forward end of the chain, who drags the chain forward, is known as the leader. The duties of the leader are as follows:—

(i) To drag the chain forward with some arrows and a ranging rod,
(ii) To fix arrows on the ground at the end of every chain, and
(iii) To obey the instructions of the follower.

The chainman at the rear end of the chain, who holds the zero end of the chain at the station, is known as the

station, is known as the follower. The duties of the follower are as follows.

(1) To direct the leader at the time of ranging.

(2) To carry the rear handle of the chain

(3) To pick up the arrows inserted by the leader.

METHOD OF CHAINING ON LEVEL GROUND

Before starting the chaining operation, two ranging rods should be fixed on the chain line, at the end stations. The other ranging rod, should be fixed near the end of each chain length, during the ranging operation.

To chain the line, the leader moves forewards by dragging the chain and by taking with him a ranging rod and ten arrows. The follower stands at the starting station by holding the other end of the chain. When the chain is fully extended, the leader holds the ranging rod vertically at arm's length. The follower directs the leader to move his rod to the left or right until the ranging rod is exactly in line. Then the follower holds the zero end of the chain by touching the station peg. The leader stretches the chain by moving it up and down with both hands. and finally places it on the line. He then inserts an arrow on the ground at the end of the chain and marks with a cross('x').

Again, the leader moves forward by dragging the chain with nine arrows and the ranging rod. At the end of the chain he fixes another arrow as before. As the leader moves further, the follower picks up the arrows which where inserted by the leader. During chaining, the surveyor or an assistant should conduct the ranging operation.

In this way, chaining is continued. when all the arrows have been inserted and the leader has none left with him, the follower hands them over to the leader, this should be noted by the surveyor. To measure the remaining fractional length, the leader should hold the zero end of the chain at the last arrow. Then the odd links should be counted.

OBSTACLE IN CHAINING

A chain line may be interrupted in the following situations:

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

(ii) when chaining is obstructed, but vision is free.

(iii) when chaining and vision are both obstructed.

chaining free but vision obstructed

Such a problem arises when a rising ground or a jungle area interrupts the chain line. Here, the end

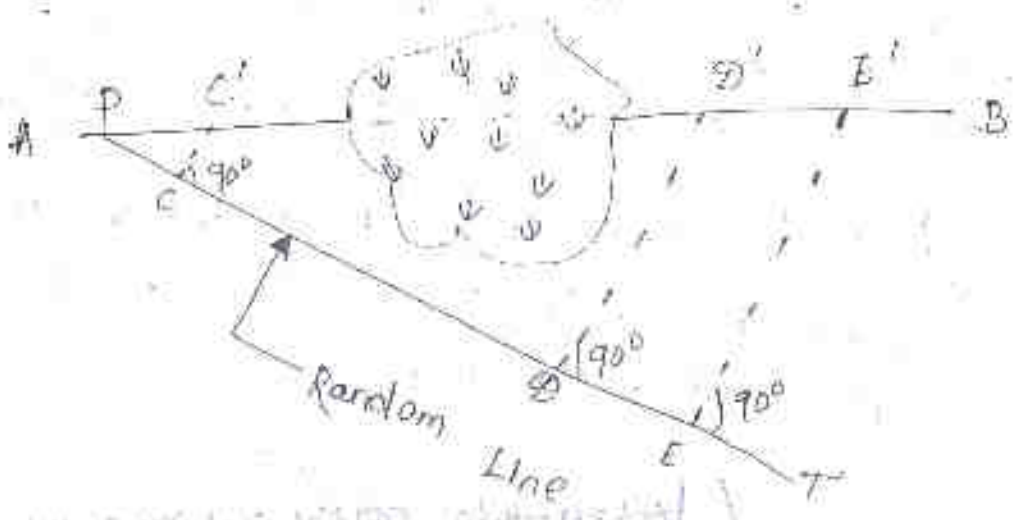
stations are not intervisible. There may be two cases.

Case-1 The end stations may be visible from some intermediate points on the ~~resing~~ ground. In this case, reciprocal ranging is resorted to, and the chaining is done by the stepping method.

Case-2 The end stations are not visible from intermediate points when a jungle area comes across the chain line. In this case the obstacle may be crossed over using a random line as explained below.

Let AB the actual chain line which cannot be ranged and extended because of interception by a jungle. Let the chain line be extended up to R. A point P is selected on the chain line and a random line, and perpendiculars are projected from them. The perpendicular at C meets the chain line at C1.

Theoretically, the perpendicular at D and E will meet the chain line at D1 and E1 now, the distances PC, PD, PE and CC, are measured.



From triangles pQQ, and pCC,

$$\frac{QQ'}{pQ} = \frac{CC'}{pC}$$

$$QQ' = \frac{CC'}{pC} \times pQ \quad \text{---} \quad (1)$$

Again from triangles PEE, and pCC'

$$\frac{EE'}{PE} = \frac{Cc'}{pC}$$

$$EE' = \frac{Cc'}{pC} \times PE \quad \text{---} \quad (11)$$

From equation (1) and (11) the lengths QQ, and EE' are calculated. These calculated distances are measured along the perpendiculars at Q and E. points Q' and E' should lie in the chain line AB. which can be extended accordingly.

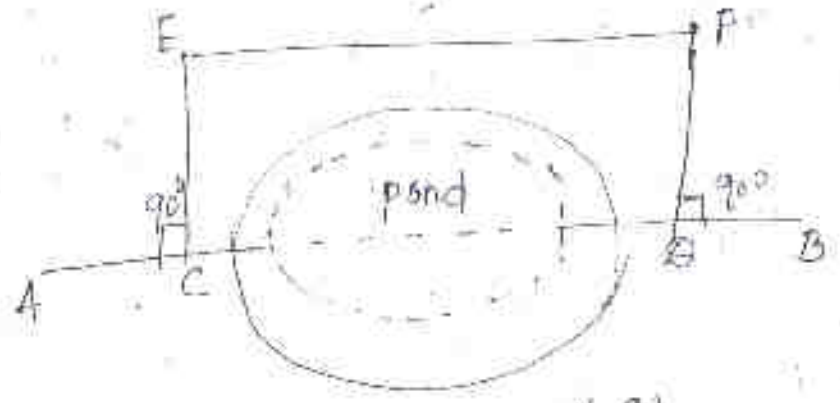
Distance $PE_1 = \sqrt{PE^2 + EE'^2}$

$AB = AP + PE_1 + BE_1$

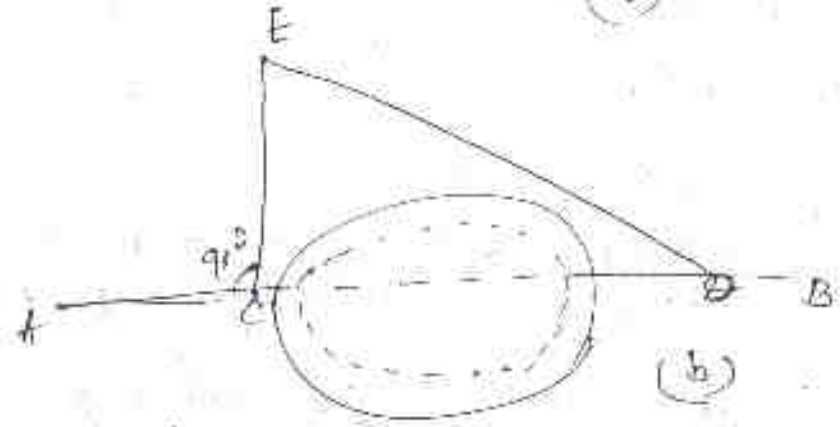
Chaining obstructed but vision free:

Such a problem arises when a pond or a river comes across the chain line. The situations may be tackled in the following ways.

Case 1 when a pond interrupts the chain line, it is possible to go around the obstruction.



(a)



(b)

(Chaining obstructed but vision free - pond)

Suppose AB is the chain line Two points C and D are selected on it on opposite banks of the pond. Equal perpendicular CE and DF are erected at C and D The distance EF is measured.

Here CD = EF

The pond may be crossed by forming a triangle as shown. A point C is selected on the chain line. The perpendicular CE is set out at C and a line ED is suitably taken. The distances CE and ED are measured.

So $CD = \sqrt{ED^2 - CE^2}$

In fig (a) AB = AC + CD + BD

Case - 2 Sometimes it is not possible to go around the obstruction.

(a) Imagine a small river comes across the chain line. Suppose AB is

the chain line. Two points C and D are selected on this line on opposite banks of the river. At C a perpendicular CE is erected and bisected at F. A perpendicular is set out at E and a point G is so selected on it that D, F and G are in the same straight line.

From triangle DCF and GEF

$$GE = CD$$

The distance GE is measured, and thus the distance CD is obtained indirectly.

(b) Consider the case when a large river interrupts the chain line.

Let AB be the chain line. points C, D and E are selected on this line such that D and E are on opposite banks of the river. The perpendiculars DF and CG are erected on the chain line in such a way that E, F and G are on the same straight line. The line FH is taken parallel to CD.

Now, from triangles DEF and HFG.

$$\frac{ED}{DE} = \frac{FH}{HG}$$

where $FH = CD$

$$ED = \frac{FH}{HG} \times DF$$

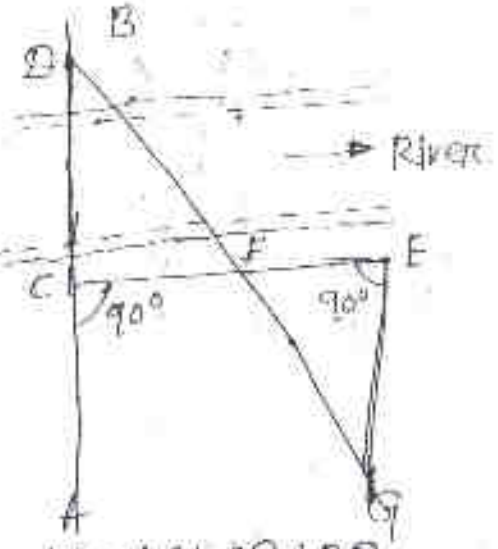
$$CH = DF$$

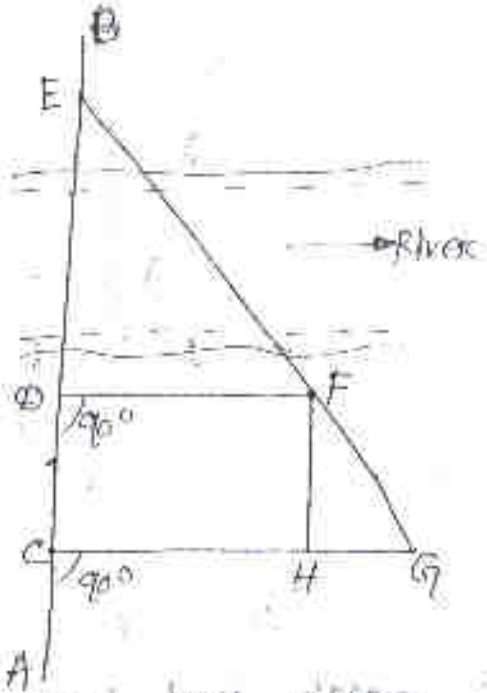
$$HG = CG - CH$$

$$= \frac{CD}{CG - DF} \times DF$$

$$\therefore HG = CG - DF$$

The distances CD, DF and CG are measured. Thus, the required distance ED can be calculated.





In Fig (a) = AB = AC + CD + BD

In Fig (b) = AB = AC + CD + DE + EB

(chaining obstructed but vision free- River)

Chaining and vision both obstructed:-

Such a problem arises when a building comes across the chain line. It is solved in the following manner.

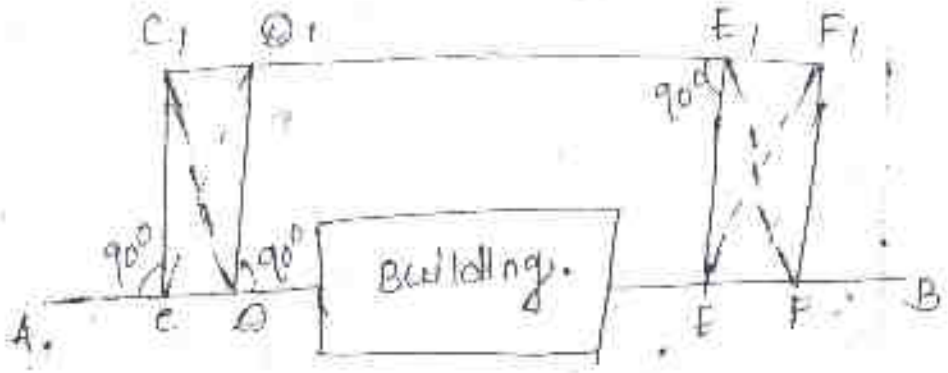
Suppose AB is the chain line, two points C and D are selected on it on one side of the building. Equal perpendiculars CC1 and DD1 are erected. The line C1D1 is extended until the building is crossed on the extended line, two points E1 and F1 are selected. Then perpendiculars E1E and F1F are so erected that.

E1E = F1F = D1D = C1C

Thus, the points C, D, E and F will lie on the same straight line AB

DE = D1E1

Here The distance D1E1 is measured, and is equal to the required distance



(chaining and vision both obstructed)

$AB = AC + CQ + QE + EF + FB$

8 May 2021

Errors & Mistakes in chaining :-

- Errors
  - Compensating Error
  - Cumulative Error
    - positive Error
    - Negative Error

Compensating Error :-

Errors which may occur in both directions (i.e both positive & negative) and which finally tend to compensate are known as compensating errors. These errors do not affect survey work seriously. They are proportional to $\sqrt{L}$, where L is the length of the line. such errors may be caused by

(a) Incorrect holding of the chain,
(b) Horizontality and verticality of steps not being properly maintained during the stepping operation.
(c) Fractional parts of the chain or tape not being uniform throughout its length
d) Inaccurate measurement of right angles with chain and tape.

Cumulative Errors :-

Errors which may occur in the same direction and which finally tend to accumulate are said to be cumulative. The seriously affect the accuracy of the work, and are proportional to the length of the line(L). The errors may be positive or negative.

positive Errors :- when the measured length is more than the actual length (i.e when the chain is too short) the error is said to be positive. Such errors occur due to

(a) The length of chain or tape being shorter than the standard length.
(b) Slope correction not being applied
(c) Correction for sag not being made.
d) Measurement being taken with faulty alignment.
(e) Measurement being taken in high winds with the tape in suspension.

Negative Errors :- When the measured length of the line is less than the actual length (i.e when the chain is too long) the error is said to be negative. These errors occure when the length of the chain or tape is greater than the standard length due to the following reasons.

(a) The opening of ring joints.
(b) The applied pull being much greater than the standard pull.
(c) The temperature during measurement being much higher than the standard temperature.
(d) Wearing of connective rings,
e) Elongation of the links due to heavy pull.

Mistakes Errors occuring due to the Carelessness of the Chainman are called mistakes. The following are a few common mistakes.

(a) Displacement of arrows : once an arrows is withdrawn from the ground during chaining, it may not be replaced in proper position. If required due to some reason.
(b) A full chain length may be omitted or added. This happens when arrows are lost or wrongly counted.
(c) A reading may be taken from the end of the chain.

This happens when the bath of the tape is noted without observing the central tally (i.e when the bath is noted from the wrong end)

(d) The numbers may be read from the wrong direction: for instance, a '6' may be read as a 'g'

(e) Some number may be called wrongly. For example, 50.2 may be called "fifty-two" without the decimal point being mentioned.

(f) While making entries in the field book, the figures may be interchanged due to carelessness: for instance, 245 may be entered instead of 254.

PRECAUTIONS AGAINST ERRORS AND MISTAKES:-

The following precautions should be taken to guard against errors and mistakes.

(1) The point where the arrows is fixed on the ground should be marked with a cross (x).

(2) The zero end of the chain or tape should be properly held.

(3) During chaining, the number of arrows carried by the follower and leader should always tally with the total numbers of arrows taken.

(4) while noting the measurement from the chain, the teeth of the tally should be verified with respect to the correct end.

(5) The chainman should call the measurement loudly and distinctly and the surveyor should repeat them while booking.

(6) Measurements should not be taken with the tape in suspension during high winds.

(7) In stepping operations, horizontality and verticality should be properly maintained.

(8) Ranging should be done accurately.

(9) No measurement should be taken with the chain in suspension.

(10) Care should be taken so that the chain is properly extended.

Chain & Tape Corrections :- Tape

(1) Temperature Correction (Ct) :-

$$\boxed{Ct = \alpha (Tm - To) L}$$

where α = Co-efficient of thermal expansion

Ct = in mt.

Tm = temperature during measurement in degree centigrade (28) celcius.

To = Temperature at which the tape was standardised in degree centigrade (00) celcius.

L = Length of tape in mt.

NOTE

* C_t → (+ve) → $T_m > T_o$
  → (-ve) → $T_m < T_o$

* α (steel tape) → 11×10^{-6} per degree centigrade (or) celcius.

(2) Pull correction (C_p) — (it may add)

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

where C_p = pull correction in mt

P_m = pull applied during measure. in kg.

P_o = pull at which the tape was standerdised, in kg

L = Length of tape, in mts

A = cross sectional area of tape, in cm^2

E = Modulus of elasticity (young's modulus)

NOTE :-

* C_p → +ve $P_m > P_o$
  → -ve $P_m < P_o$

* $E = 2.1 \times 10^6$ kg/cm^2

(3) Slope correction (C_h) :-

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

NOTE :- This type of correction is always negative.

(*) $$C_h = l\,(1 - \cos\theta)$$

(or)

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

(4) Sag Correction (C_s) :-

$$C_s = \frac{L\,(wL)^2}{24n^2 P_m^2} \quad \text{and} \quad C_s = \frac{L w^2}{24 n^2 P_m^2}$$

Where C_s = Sag correction in mt

L = Length of the tape (or) chain in mt.

n = number of span

P_m = pull applied during measurement

w = weight of tape per unit length in Kg per mt.

W = Total weight of tape in kg.

NOTE:- This correction is always negative.

(5) Normal tension (P_n) :-

⇒ The tension at which the effect of pull is neutralised by the effect of sag is known as normal tension.

P_n → $\frac{(P_n - P_0)L}{AE} = \frac{L(wL)^2}{24P_n^2}$ (Assume)

P_n → $\frac{(P_n - P_0)L}{AE} = \frac{Lw^2}{24P_n^2}$

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(1) Chain :-

(1) correction applied to incorrect length :-

True length of the line (TL) = $\left(\frac{L'}{L}\right) \times ML$

where L = true length of chain

L' = True length ± error

= L ± e

ML = measured length

NOTE :-

* use (+ve) sign when the chain (or) tape is too long.
* use (-ve) sign when it is too short.

(2) Correction of Incorrect Area :-

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

(3) Hypotenusal Allowance :-

$$\text{Correction per tape length} = L(\sec\theta - 1)$$

L = Length of tape
θ = Slope of the ground

NOTE :-

It is always added to the chain length

16. The distance between two points, measured with a 20 mt chain was recorded as 327 mt. It was afterwards found that the chain was 30 cm too long. What was the true distance between the points ?

Data given :-

True length $= \left(\frac{L'}{L}\right) \times ML$

$e = 3c.m$

$L = 20mt$

$ML = 327 mt$

$L' = 20 + 0.03 = 20.03 mt$

$$TL = \left(\frac{20.03}{20}\right) \times 327$$

$$= 327.49 mt$$

(or) $20mt = 327mt$

$20.03 = \frac{327}{20} \times (20.03)$

$= \left(\frac{20.03}{20}\right) \times 327$

$= 327.49 mt$

26. The distance between two stations was 1200 mt when measured with a 20 mt. chain was 0.05 mt too long. The same distance when measured with a 30 mt chain the measured length was found to be 1195 mt. what was the error in 30mt chain ?

20m chain

$e = +0.05 mt$

$L' = 20 + 0.05 = 20.05 mt$

$TL = \frac{L'}{L} \times ML$

$= \frac{20.05}{20} \times 1200$

$= 1203 mt$

TL = 1203 mt

L = 30 mt

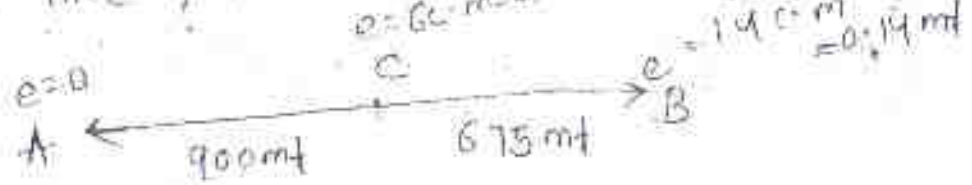
ML_2 = 1195 mt L' = 30 + e

$TL = \frac{L'}{L} \times ML$

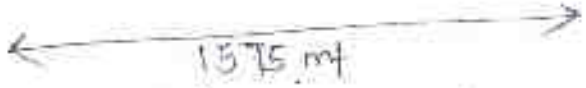
$\Rightarrow L' = \frac{TL \times L}{ML} = \frac{1203}{1195} \times 30 = 30.20$ mt

$e = L' - L = 30.20 - 30 = 0.20$ mt

3 Q. A line was measured by a 20 mt chain which was accurate before starting the day's work. After chaining 900 mt the chain was found to be 6 cm too long. After chaining a total distance of 1575 mt the chain was found to be 14 cm too long find the true distance of the line?



20 mt



20mt chain

AB (TL) = AC(TL) + CB(TL)

AC(TL) :-

L = 20 mt, ML = 900mt, L' = 20+e = 20+0.03 = 20.03mt

$e = \frac{e+0.06}{}$ = 0.03mt.

$$AC(TL) = \frac{20.03}{20} \times 900$$

$$= 901.35 \text{ mt}$$

CB(TL) :-

L = 20 mt , ML = 675 mt L' = 20 + e = 20.1 mt.

= 20 + 0.1 = 20.1 mt

$$e = \frac{0.06 + 0.14}{2} = 0.1 \text{ mt}$$

$$CB(TL) = \frac{20.1}{20} \times 675 = 678.375 \text{ mt}$$

$$AB(TL) = 901.35 + 678.375 = \underline{1579.72 \text{ mt}}$$

(4) Q A 20 mt. steel tape was standardised on flat ground, at a temperature of 20° C & under a pull of 15 kg. The tape was used at a temperature of 30° C & under a pull of 10 kg. The cross-sectional area of the tape is 0.22 cm² & its total weight is 400 gm. The young's modulus of elasticity is 2.1×10^6 kg/cm² & $\alpha = 11 \times 10^{-6}$ per °C find the correct horizontal distance?

Soln L = 20 mt ; E = 2.1×10^6 kg/cm²

T_0 = 20° C $\alpha = 11 \times 10^{-6}$ per °C

P_0 = 15 kg A = 0.22 cm²

T_m = 30° C W = 400 gm = 0.4 kg

P_m = 10 kg

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

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

$$= 0.00220 \text{ mt} \quad (+ve)$$

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

$$= \frac{(10-15)\,20}{0.22 \times 2.1 \times 10^6 \text{ kg/cm}^2}$$

$$= (-0.00238 \text{ mt})$$

$$C_s = \frac{Lw^2}{24n^2 p m^2} = \frac{20 \times (0.04)^2}{24 \times 12 \times 10^2}$$

$$= 0.00133 \text{ mt (-ve)}$$

Total correction $= +0.00220 - 0.00238 - 0.00133$

$= -1.5$ mt.

Correct horizontal distance $= 20 - 0.00151$

$= 19.99849$ mt.

50 A 30 mt steel tape was standardised at a temperature of 20°C & under a pull of 5 kg. The tape was used at a temperature 25°C & under a pull of P kg the cross-sectional area of tape is 0.02 cm² & its weight per unit length is 22 g/mt. $E = 2 \times 10^6$ kg/cm² & $\alpha = 11 \times 10^{-6}$ per °C find the correct distance if P is (a) 5 kg (b) 11 kg ?

Soln Given data:-

$L = 30$ mt $\quad T_m = 25°C$ $\quad E = 2 \times 10^6$ kg/cm²

$T_o = 20°C$ $\quad A = 0.02$ cm² $\quad \alpha = 11 \times 10^{-6}$ per °C

$P_o = 5$ kg $\quad P_{m_1} = 5$ kg, $P_{m_2} = 11$ kg $\quad n = 1$

$= 22$ g/mt $= 0.022$ kg/mt

(a) $P = 5\,kg$

(i) $C_T = 11 \times 10^{-6} (25 - 20) \times 30$

$= 0.00165$ mt (tve)

(ii) $C_P = \frac{(5-5) \times 30}{0.02 \times 2 \times 10^6} = 0$

(iii) $C_S = \frac{L(\omega L)^2}{24 n^2 p^2 n} = \frac{30 \times (0.022 \times 30)^2}{24 \times 5^2}$

$= 0.02178$ (-ve)

$T_C = 0.00165 - 0.02178 = -0.02013$

Correct horizontal distance $= 3.0 - 0.0201$

$= 29.979$ mt

(b) $P = 11\,kg$

(i) $C_T = 0.00165$ mt (tve)

(ii) $C_P = \frac{(11-5) \times 30}{0.02 \times 2 \times 10^6} = 0.0045$ mt (tve)

(iii) $C_S = \frac{L(\omega L)^2}{24 n^2 p^2 n} = \frac{30 \times (0.022 \times 30)^2}{24 \times 11^2}$

$= 0.0045$ (-ve)

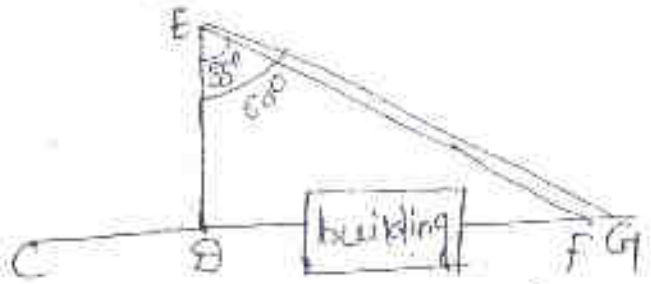
$T_C = +0.00165 + 0.0045 - 0.0045$

$= 0.00165$

Correct HE $= 30 + 0.00165 = 30.00165$ mt.

18 may 2021

Q A survey line CD intersects a building. To overcome the obstacle a perpendicular DE, 85mt long is set out at D from E, two lines EF & EG are set out at angles of 55° & 60° respectively with ED find the lengths EF & EG such that points F & G falls on the prolongation of CD Also find the obstructed distance DF?



EF, EG, DF = ?

Soln $\triangle DEF$, $\cos\theta = \frac{b}{h}$

$$\cos 55^\circ = \frac{DE}{EF}$$

$$\Rightarrow EF = \frac{85}{\cos 55^\circ} = 148.19 \text{ mt}$$

$\triangle DEG$, $\cos\theta = \frac{b}{h}$

$$\cos 60^\circ = \frac{DE}{EG}$$

$$EG = \frac{85}{\cos 60^\circ} = 170 \text{ mt}$$

$\triangle DEF$, $\tan\theta = \frac{p}{b}$

$$\Rightarrow \tan 55^\circ = \frac{DF}{DE}$$

$$\Rightarrow DF = DE \times \tan 55^\circ$$

$$= 85 \times \tan(55^\circ) = 121.39 \text{ mt}$$

$=$

26 An old map was plotted to a scale of 40mt = to 1cm over the years, this map has been shrinking & a line originally 20 cm long is only 19.5 cm at present. Again the 20mt chain was 5cm too long. If the present area of the map measured is 125.50 cm2 find the true area of the land surveyed?

Sol^n

19.5cm on the map originally 20 cm

1 cm on the map originally $= \frac{20}{19.5} = 1.025$ cm

1 cm^2 " " " $= (1.025)^2$ cm^2

125.50 cm^2 " " " $= 125.50 \times 1.050625$

$= 131.853$ cm^2

Scale on map 1 cm = 40 mt ground

1 cm^2 $= (40)^2$ mt^2 ground

$= 1600$ mt^2

131.853 cm^2 $= 1600 \times 131.853$ on ground

$= 210964.8$ m^2

Correction for Area $= \left(\frac{L'}{L}\right)^2 \times M$

$= \left(\frac{20 + 0.05}{20}\right)^2 \times 210964.8$

$= 212020.94$ mt^2

$= 21.2020$ hectare

(1 hectare $= 10000$ m^2)

Chapter - 2

PRINCIPLE OF CHAIN SURVEYING

The principle of chain surveying is triangulation. This means that the area to be surveyed is divided into a number of small triangles which should be well conditioned. In chain surveying the sides of the triangles are measured directly on the field by chain or tape, and no angular measurements are taken. Here the lines and check lines control the accuracy of work.

It should be noted that plotting triangs requires no angular measurements to be made, if the three sides are known.

Chain surveying is recommended when

(1) The ground surface is more or less level.
(2) A small area is to be surveyed.
(3) A small-scale map is to be prepared
(4) The formation of well-conditioned triangles is easy.

Chain surveying is unsuitable when

(1) The area is crowded with many details.
(2) The area consists of too many undulations.
(3) The area is very large
(4) The formation of well-conditioned triangles becomes difficult due to obstacles.

A Large - scale and small - scale maps :-

When 1 cm of a map represents a small distance, it is said to be a large - scale map for example,

1 cm = 1 m. i.e RF = $\frac{1}{100}$

When 1 cm of the map represents a large distance, it is called a small - scale map.

Ex:- 1 cm = 100 m. i.e 1 RF = $\frac{1}{10,000}$

A map having an RF of less than 1/500 is considered to be large - scale. A map of RF more than 1/500 is said to be small - scale.

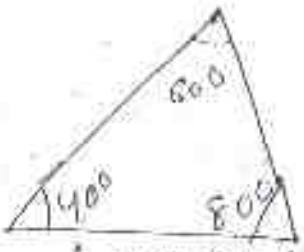
WELL - CONDITIONED AND ILL - CONDITIONED TRIANGLES

A triangle is said to be well conditioned when no angle in it is less than 30° or greater than 120°. An equilateral triangle is considered to be the best - condition or ideal triangle.

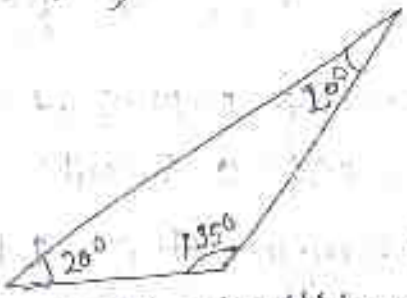
well - conditioned triangles are preferred because their apex point are very sharp and can be located by a single 'dot'. In such a case, there is no possibility of relative displacement of the plotted point.

A triangle in which an angle is less than 30° or more than 120° is said to be ill conditioned.

(Ideal triangle)



(well-conditioned triangle)



(ill-conditioned Triangle)

ill-conditioned triangles are not used in chain surveying. This is because their apex points are not sharp and well defined, which is why a slight displacement of these points may cause considerable error in plotting.

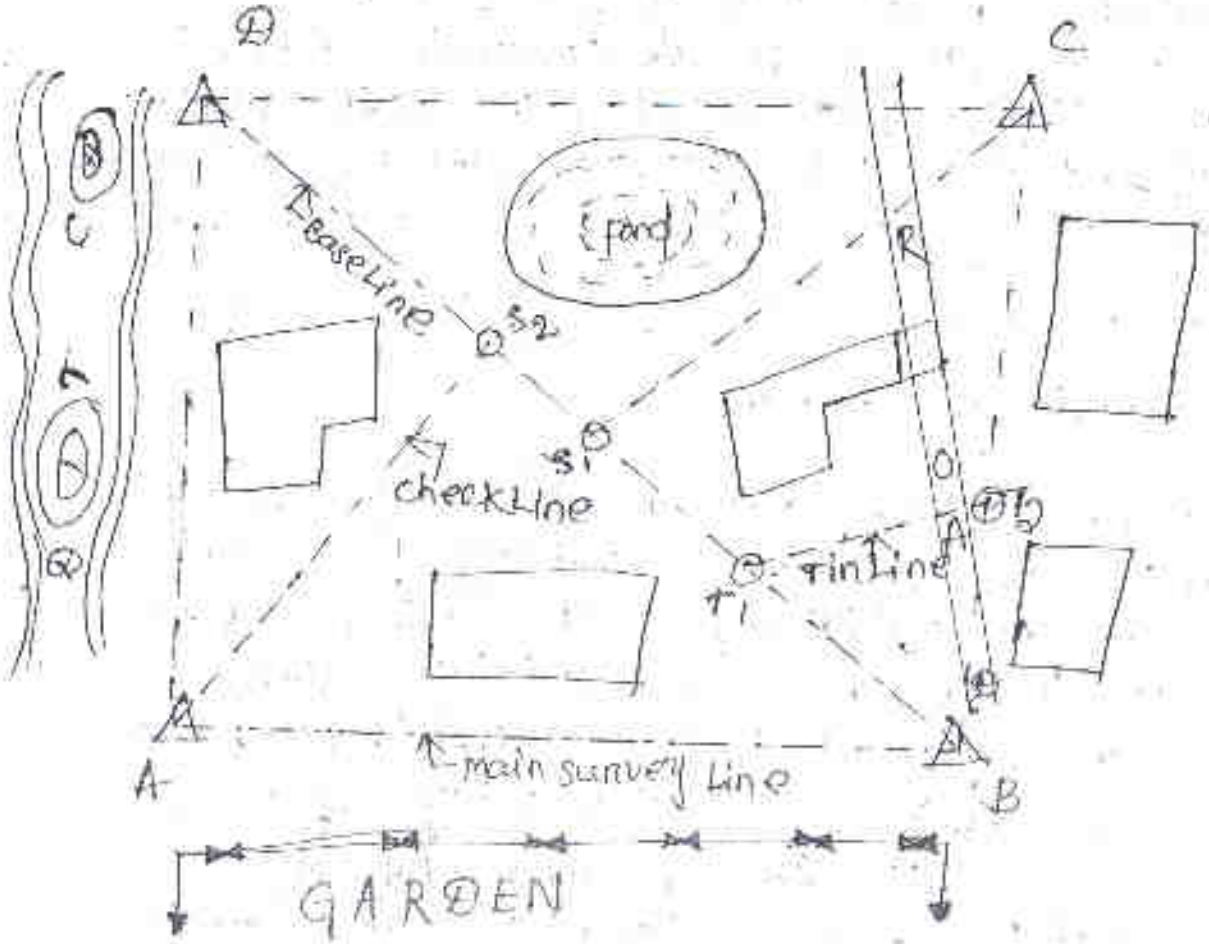
RECONNAISSANCE SURVEY AND INDEX SKETCH

Before the commencement of any survey work, the area to be surveyed is thoroughly examined by the surveyor who then thinks about the possible arrangement of the framework of survey. This primary investigation of the area is termed as reconnaissance survey or reconnaitre.

During reconnaissance survey work the surveyor should walk over the area and note the various obstacles and whether or not the selected stations are intervisible. The main stations should be so selected that they enclose whole area. The surveyor

should also take care the triangles formed are went not the vertices objects which are to be located.

The neat hand sketch of the area which is prepared during reconnaissance survey is known as the index sketch or key plan. The index sketch shows the skeleton of the survey work. It indicates the main survey stations, sub-stations, tie stations, base line arrangement for framework of triangles and the approximate positions of different objects. This sketch is an important document for the surveyor and for the person who will plot the map. It should be attached to the starting page of the field book.



(Index sketch)

Definitions and Illustrations

① Survey stations

Survey stations are the points of the beginning and the end of a chain line. They may also occur at any convenient points on the chain line. Such stations may be (i) main stations,
(ii) subsidiary stations,
(iii) tie stations.

Main stations :- Stations taken along the boundary of an area as controlling points are known as main stations. The lines joining the main stations are called main survey lines. The main survey lines should cover the whole area to be surveyed. The main stations are denoted by 'A' with letters A, B, C, D etc. The chain lines are denoted by

Subsidiary stations :-

Stations which are on the main survey lines or any other survey lines are known as subsidiary stations. These stations are taken to run subsidiary lines for dividing the area into triangles, for checking the accuracy of triangles and for locating important details. These stations are denoted by ⊙ with letters S_1, S_2, S_3 etc.

Tie stations

These are also subsidiary stations taken on the main survey lines. Lines joining the tie stations are known as tie lines. Tie lines are mainly taken to fix the directions of adjacent sides of the chain survey map. These are also taken to form chain angles in chain traversing. When triangulation is not possible. (chain angles are described afn sometimes tie lines are taken to locate interior details. Tie stations are denoted by '@' with letters T1, T2, T3 etc.

(2) Base line:- The line on which the framework of the survey is built is known as the base line. It is the most important line of the survey. Generally, the longest of the main survey lines is considered the base line. This line should be taken through fairly level ground, and should be measured very carefully and accurately. The magnetic bearings of the base lene are taken to fix the north line of the map.

3. Check line:- The lene joining the apex point of a triangle to some fixed point on its base is known as the check line. It is taken to check the accuracy of the triangle. Sometimes this line helps to locate interior details.

offset :- The lateral measurement taken from an object to the chain line is known as offset - offsets are taken to locate objects with reference to the chain line. They may be of two kinds perpendicular and oblique.

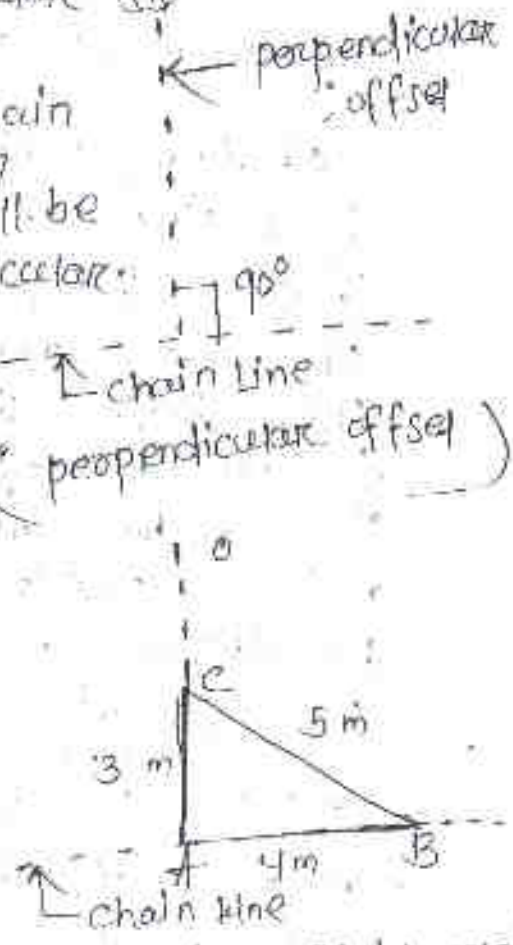
perpendicular offsets

When the lateral measurements are taken perpendicular to the chain line, they are known as perpendicular offsets. perpendicular offsets may be taken in the following ways:

a) By setting a perpendicular by swinging a tape from the object to the chain line the point of minimum reading on the tape will be the base of the perpendicular.

b) By setting a right angle in the ratio 3:4:5

(setting a perpendicular in the ratio 3:4:5)



(setting a Right angle)

(setting a Right angle by a Builder's square)

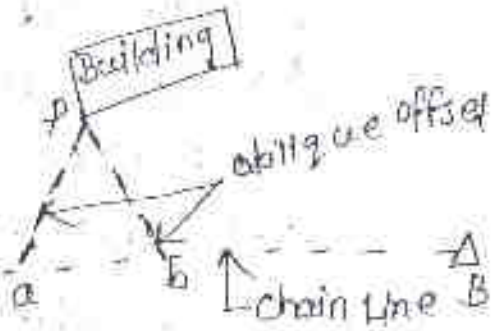
(c) By setting a right angle with help of builders square or tri-square.

(d) By setting a right angle by cross-staff or optical square.

Oblique offsets

Any offset not perpendicular to the chain line is said to be oblique. oblique offsets are taken when the objects are at a long distance from the chain line or when it is not possible to set up a right angle due to some difficulties. Such offsets are taken in the following manner.

Suppose AB is a chain line and P is the corner of a building. Two points a and b are taken on the chain line. The chainages of a and b are noted. The chainage ap and bp are measured and noted in the field book. Then ap and bp are the oblique offsets. (oblique offset) when the triangle abp is plotted, the apex point p will represent the position of the corner of the building.



perpendicular offsets are preferred for the following reasons :—

(a) They can be taken very quickly.

(b) The progress of survey is not hampered.

(c) The entry in the field book becomes easy.

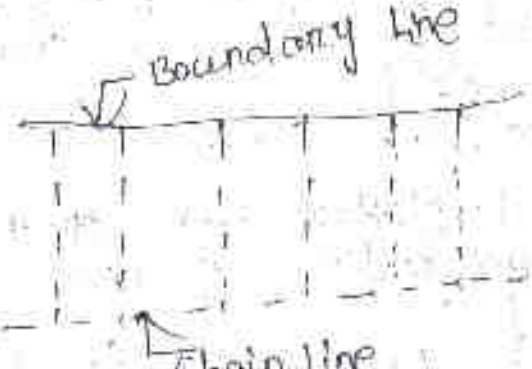
(d) The plotting of the offsets also becomes easy.

number of offsets :-

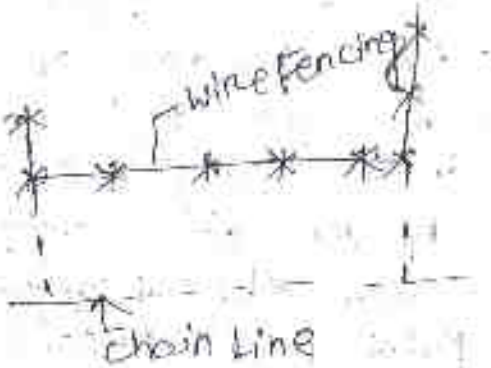
The offsets should be taken according to the nature of the object, so there is no hard-and-fast rule regarding the number of offsets. It should be remembered that the objects are to be correctly represented and hence the number of offsets should be decided on the field. Some guidelines are given below.

(a) When a boundary of the object is approximately parallel to the chain line, perpendicular offsets are taken at regular intervals.

(b) When the boundary is straight, perpendicular offsets are taken at both ends of it.



(Boundary parallel to chain line)

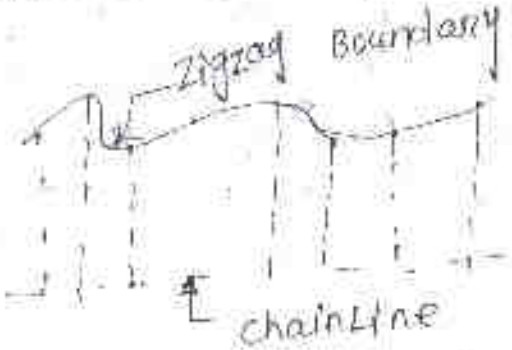


(Straight Boundary)

(c) When the boundary line is zigzag, perpendicular offsets are taken at every point of bend to represent the shape of the boundary accurately. In such a case, the interval of the offsets may be irregular.

(d) When a road crosses the chain line, the chainage of the

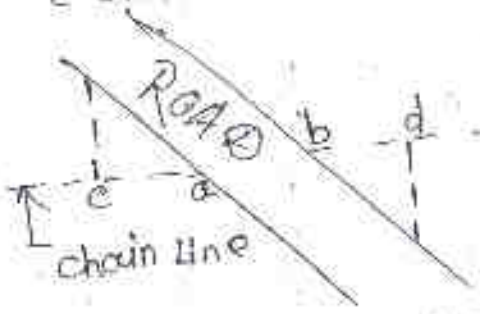
intersection point is to be noted.



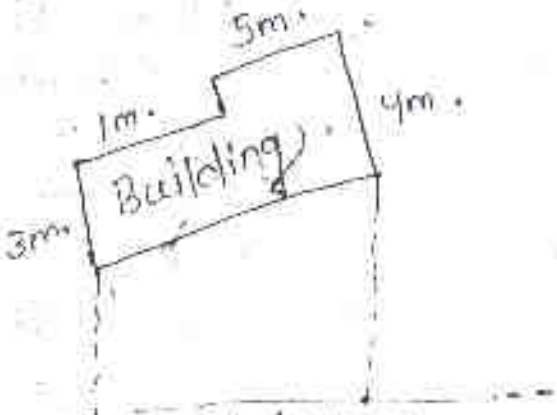
(Zigzag Boundary)

(Road crossing perpendicularly)

(e) when a road crosses a chain line obliquely, the chainages of intersection points a and b are noted. Then at least one offset is taken on both sides of the intersection points. More offsets is taken on both sides of the intersection points. More offsets may be taken depending on the nature of the road. Here perpendicular offsets are taken at c and d



(Road crossing obliquely)

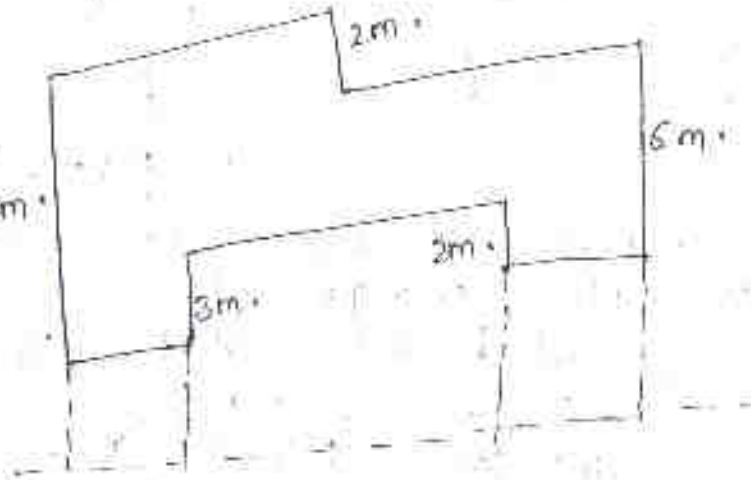


(For small Building)

(f) when the building is small. Its corners are fixed by perpendicular or oblique offsets and the other dimensions are taken directly on the field and noted in the field book.

(g) when the building is large, zigzag in shape and oblique to the chain line

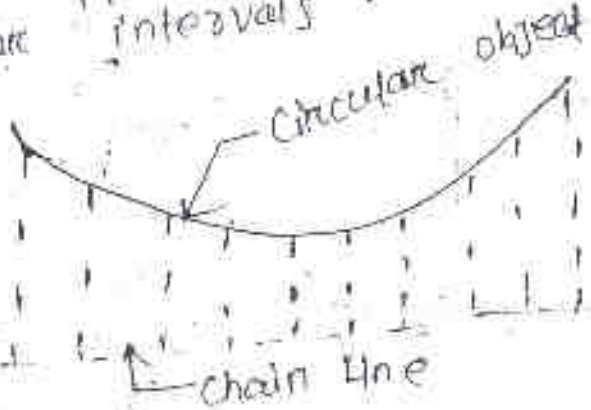
then the corners are fixed by perpendicular or oblique offsets. Then the full plan of the building is drawn on a separate page along with all the dimensions. This page should be attached with the field book at the proper place.



Chain line

(Large Building)

(b) When the object is circular, perpendicular offsets are taken at short and regular intervals.



(Circular object)

(c) Limiting length of offset

The maximum length of the offset should not be more than the length of the tape used in the survey. Generally, the maximum length of

offset is limited to 15 m. However, this length also depends upon the following factors:

(i) The desired accuracy of the map
(ii) The scale of the map
(iii) The maximum allowable deflection of the offset from its true direction
(iv) The nature of the ground.

SELECTION OF SURVEY STATIONS

The following points should be remembered during the selection of survey stations.

(1) The stations should be so selected that the general principle of surveying may be strictly followed.
(2) The stations should be intervisible.
(3) The stations should be selected in such a way that well-conditioned triangles may be formed.
(4) The base line should be the longest of the main survey lines.
(5) The main survey lines should pass close to the boundary line of the area to be surveyed.
(6) The survey lines should be taken through fairly level ground, as far as practicable.
(7) The survey lines should be taken close to the objects so that they can be located by short offsets.
(8) The tie stations should be suitably selected for taking check lines in the directions of adjacent sides.
(9) The subsidiary stations should be suitably selected for taking check lines.

(10) Stations should be so selected that obstacles to chaining are avoided as far as possible.

(11) The survey lines should not be very close to main roads, as survey work may then be interrupted by traffic.

THE FIELD BOOK :—

The notebook in which field measurement are noted is known as the 'field book'. The size of the field book is 20cm x 10cm and it opens lengthwise. Field books may be of two types.

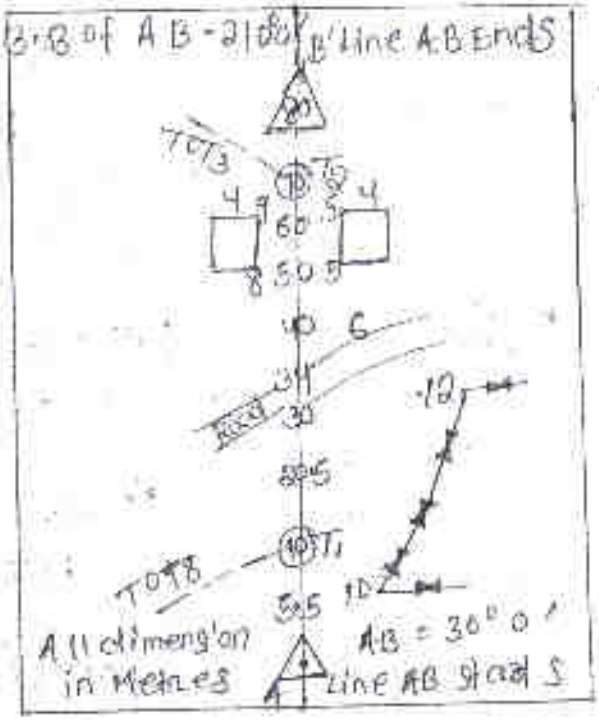
① Single-line

② Double-line

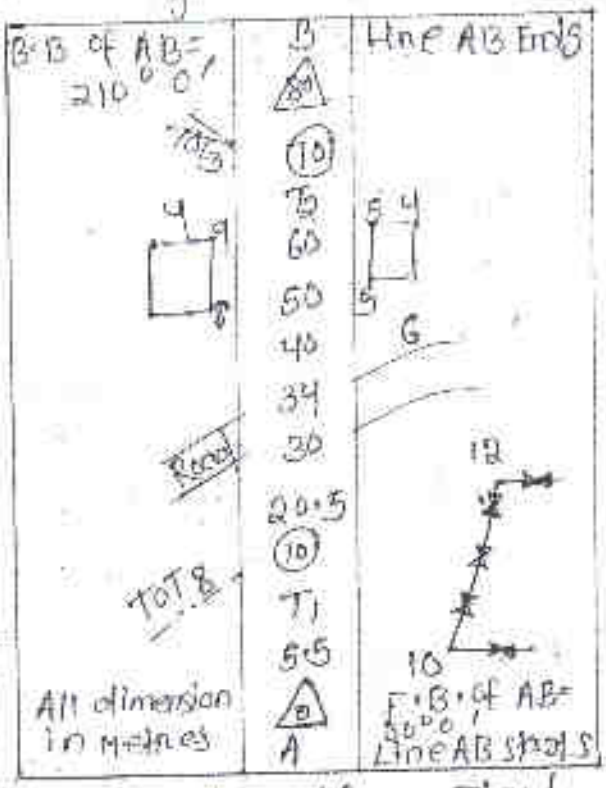
Single-line field book — In this type of field book, a single red line is drawn through the middle of each page. This line represents the chain line, and the changes are written on it. The offsets are recorded with sketch to the left or right of the chain line. The recording of the field book is started from the last page and continued towards the first page. The main stations are marked by 'A' and subsidiary stations as the station are marked by '⊙'.

Double-line field book — In this type of field book, two red lines, 1.5 cm apart, are drawn through the middle of each page. This column represents the chain line, and the

chainages are written in it. The offset are recorded, with sketches to the left or right of this column. The recording is begun from the last page and continued towards the first. The main stations are marked by 'Δ' and substations by ⊙. This type of field book is commonly used.



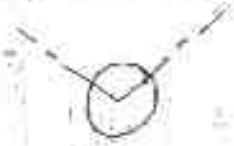
(Single-Line Field Book)



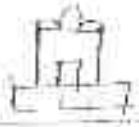
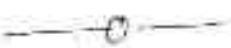
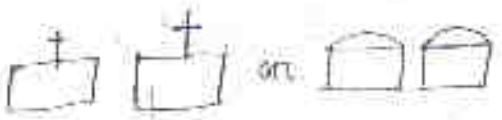
(Double-Line Field Book)

CONVENTIONAL SYMBOLS

In a map the objects are shown by symbols and not by names. So the surveyor should known the following conventional symbols for standard some common objects.

S.no	object	Symbol	Colour.
1	North line		Black
2	Main stations or triangulation station		Red or crimson lake
3	Traverse stations or substations		Red or crimson lake
4	chain line		Red or crimson Lake
5	River		prussian blue
6	Canal		Prussian blue
7	Lake or pond		prussian blue
8	open well		prussian blue
9	Tube well		Black
10	Foot path		Black
11	Metalled road		Burnt sienna
12	unmetalled road		Burnt sienna

S.NO	object	Symbol	Colour
13	Railway line (single)		Black
14	Railway line (double)		Black
15	Road bridge or culvert		Black
16	Railway bridge or culvert		Black
17	Level crossing		Black and burnt sienna
18	Wall with gate		Black
19	Boundary line		Black
20	Hedge		Green
21	Wire fencing		Black
22	Pipe fencing		Prussian blue
23	Wood fencing		Yellow
24	Building (pukka)		Crimson lake
25	Building (kaccha)		Umber
26	HHS		Yellow
27	Temple		Crimson Lake
28	church		Crimson lake

Sl no	object	symbol	colour
29	Mosque		Crimson lake
30	Benchmark		Black
31	Tree		Green
32	Jungle		Green
33	Orchard		Green
34	Cultivated land		Black & Green
35	Barren land		Black
36	Rough pasture		Black
37	marsh or swamp		Black
38	Embankment		Black
39	Cutting		Black
40	(a) Telegraph line		Black
	(b) Telegraph post		Black
41	(a) Electric line		Black
	(b) Electric post		
42	Burial ground or Cemetery		Crimson lake

NEW CHAPTER 21 May 2021

COMPASS TRAVERSING

⇒ The chain surveying the area to be surveyed is divided into a number of triangles

⇒ This method is suitable for fairly level ground covering small areas.

⇒ But when the area is large undulating & crowded with many details, triangulation is not possible, in such an area traversing is adopted

⇒ In traversing, the framework consist of a number of connected line. (or)

⇒ The length are measured by chain & the direction identified by angle measuring instrument.

⇒ In one of the method, the angle measuring instrument is compass Hence the process is compass traversing

Definitions

1) True Meridian The line or plane passing through the geographical north pole, geographical south pole and any point on the surface of the earth is known as the 'true' meridian or 'geographical meridian'. The true meridian of a station is constant, The true meridians passing through different points on the earth's surface are not parallel but converge

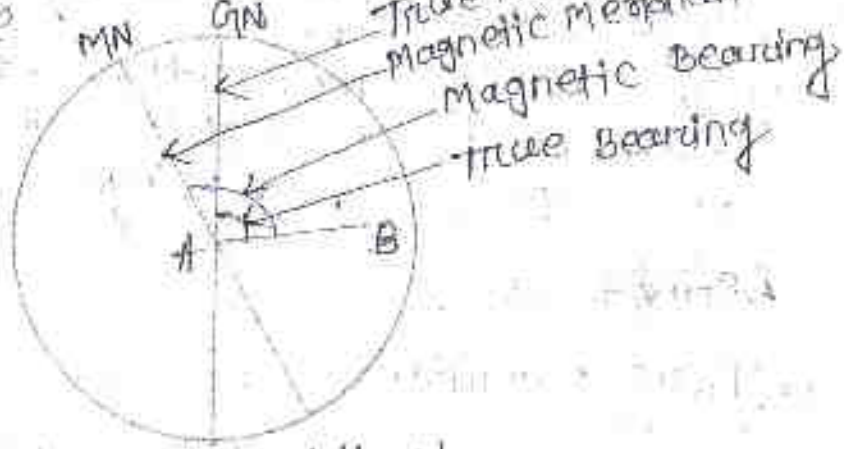
towards the poles. But for surveys in small areas, the true meridians passing through different points are assumed parallel.

The angle between the true meridian and a line is known as 'true bearing' of the line. It is also known as the 'azimuth'.

Magnetic Meridian :-

When a magnetic needle is suspended freely and balanced properly unaffected by magnetic substances, it indicates a direction. This direction is known as the 'magnetic meridian'.

The angle between the magnetic meridian and a line is known as the 'magnetic bearing' or simply the 'bearing' of the line.



(Meridians)

(3) **Arbitrary meridian** :- Sometimes for preparing a map, some state agencies assume several lines parallel to the true meridian for a particular zone. These lines are termed grid lines and the central line the 'grid meridian'.

The bearing of a line are termed with grid lines and respect to the grid meridian is known as the grid bearing of the line.

(4) Grid Meridian

Some times for preparing a map, some state agencies assume several lines parallel to the true meridian for a particular zone. These lines are termed grid lines and the central line the grid meridian. The bearing of a line with respect to the grid meridian is known as the grid bearing of the line.

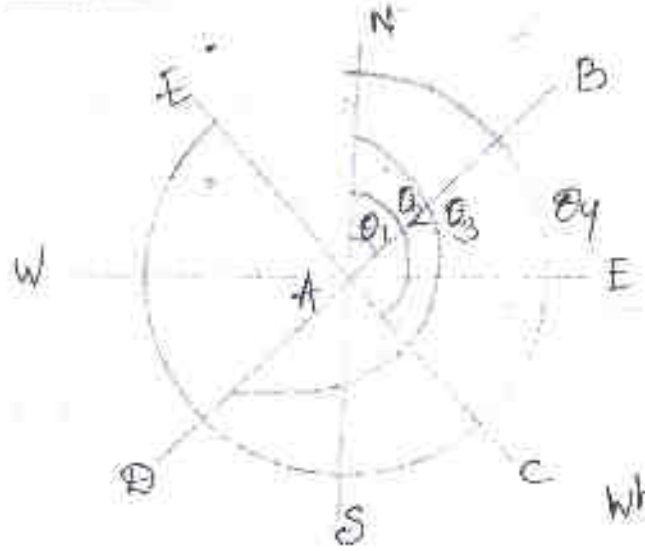
(5) Designation of Magnetic Bearings

Magnetic bearings are designated by two systems.

(a) whole circle bearing
(b) Quadrantal bearing (QB)

(a) Whole circle Bearing (WCB)

The magnetic bearing of a line measured clockwise from the north pole towards the line, is known as the whole circle bearing of that line. such a bearing may have any value between 0° and 360°. The whole circle bearing of a line is obtained by prismatic compass.



Whole circle Bearing (W.C.B)

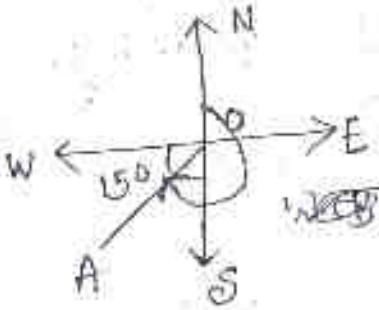
Ex:- WCB of AB = θ_1

WCB of AC = θ_2

WCB of AD = θ_3

WCB of AE = θ_4

Q1



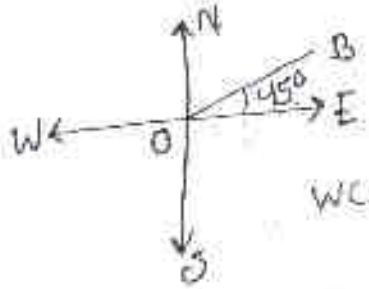
W.C.B of OA = ?

W.C.B = 255°

QB of OA = S 75° W

Ans

Q2

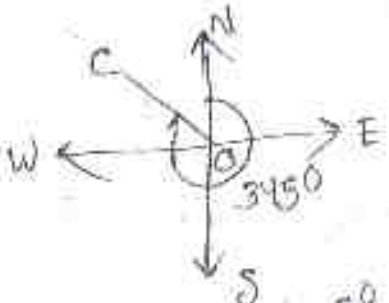


W.C.B of OB = ?

WCB = 45°

Ans -

Q3

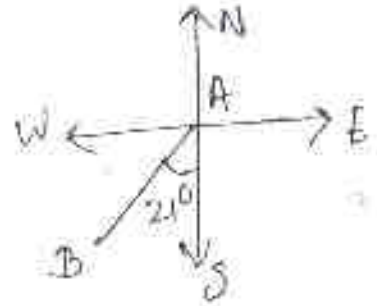


W.C.B of OC = ?

Ans W.C.B = 345°

QB = N 15° W

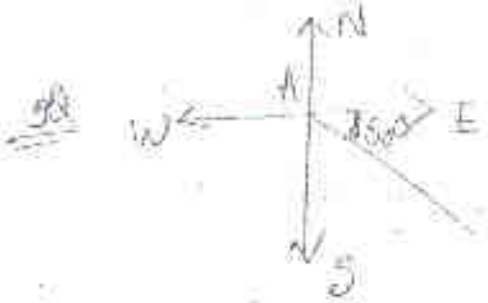
Q



~~W.C.B of OA =~~

RB of AB = S 21° W

Ans



RB of AB = ?

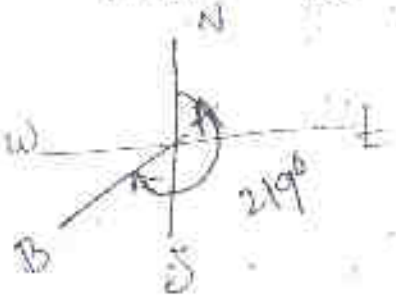
∴ W.C.B of AB = ?

RB of AB = 90° − 60° = S 30° E

W.C.B of AB = 150°

Ans

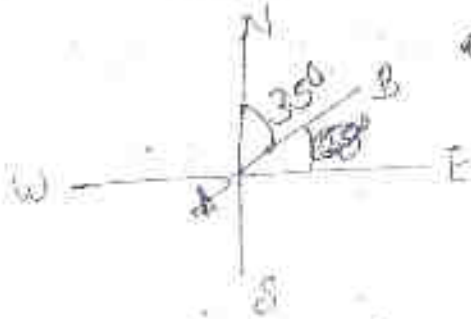
Q If the W.C.B of a line AB = 219° then find RB of AB = ?



RB of AB = 39° (219 − 180°)

Q If W.C.B of AB = 35°

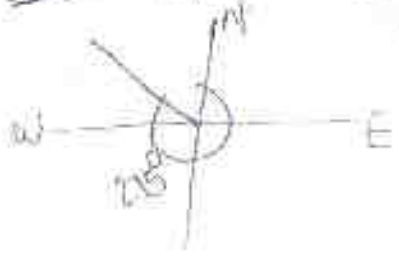
RB of AB = ?



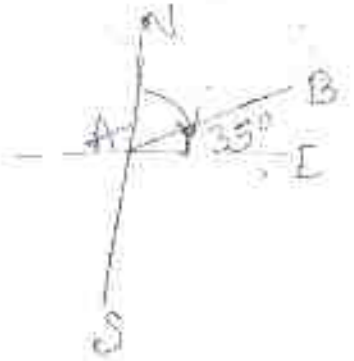
~~RB of AB = 90° − 35° = S 55°~~

RB of AB = N 35° E

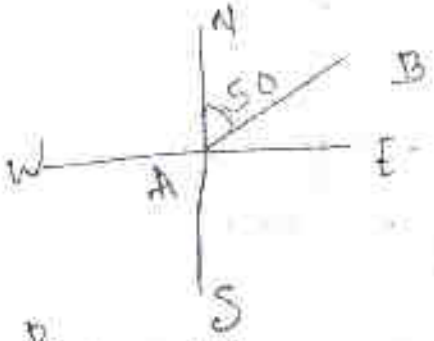
Q If W.C.B of AB = 275° RB of AB = ?



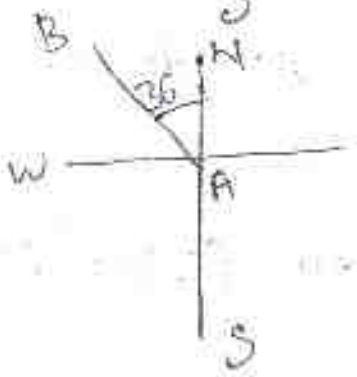
RB of AB = N 85 W

9Q 

Ans W.C.B of AB = 55°

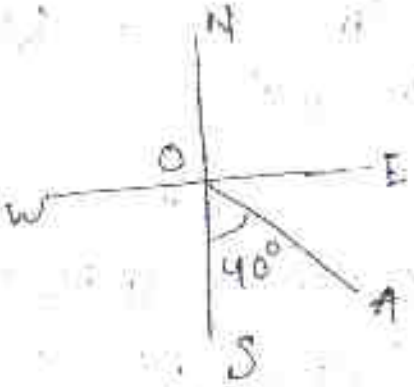
10Q 

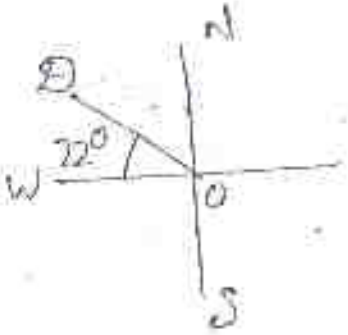
W C B of AB = 50°

11Q 

W CB of AB = 360° - 36°
= 324°

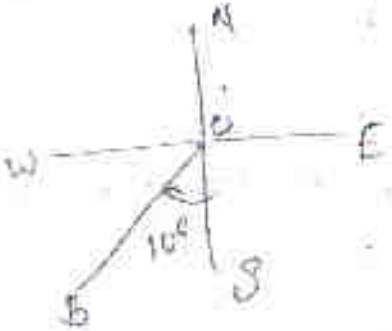
12 W CB of OA = 180 - 40
= 140°



13 

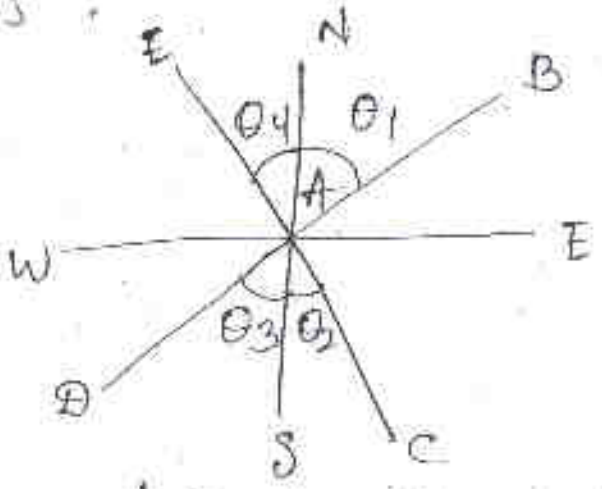
W CB of OD = 270 + 22
= 292°

14 W CB of OB = 180 + 10
= 190°



(b) Quadrantal Bearing (QB) :-

The magnetic bearing of a line measured clock wise or counter clockwise from the North pole or South pole (whichever is nearer the line) towards the East or west, is known as the 'quadrantal bearing' of the line. This system consists of four quadrants - NE, SE, SW and NW. The value of a quadrantal bearing lies between 0° and 90°, but the quadrants should always be mentioned. Quadrantal bearings are obtained by the surveyor's Compass.



(Quadrantal Bearing 'QB')

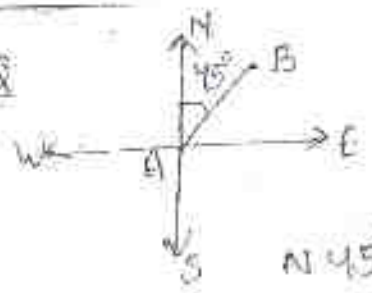
Ex:- QB of AB = N θ1 E

QB of AC = S θ2 E

QB of AD = S θ3 W

QB of AE = N θ4 W

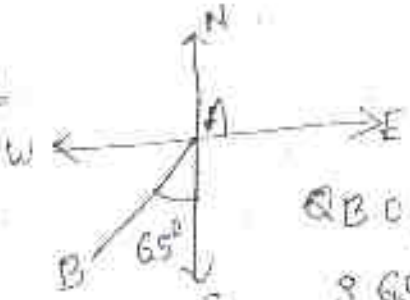
1Q



QB of AB:

N 45° E

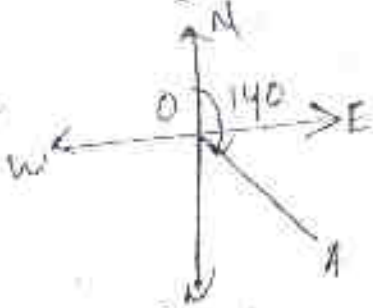
2Q



QB of AB:

S 65° W

3Q

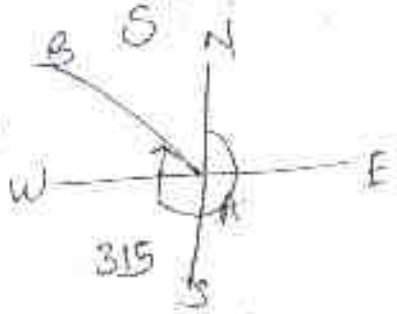


QB of OA =

(180° − 140°) = 40°

S 40° E

4Q

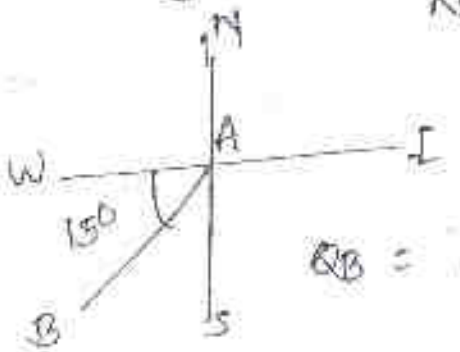


QB of AB

(360° − 315°) = 45°

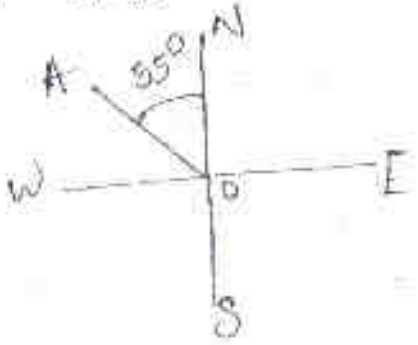
N 45° W

5Q

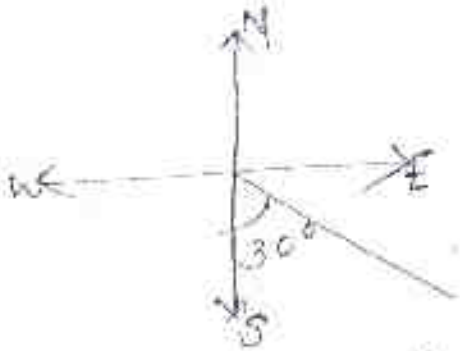


QB = S 75° W

6Q N 55° W



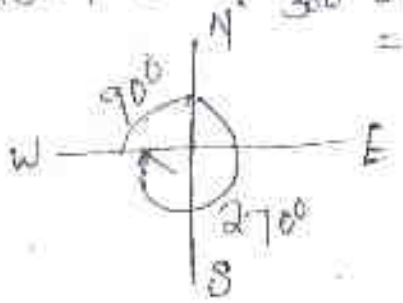
7Q S 30° E



8Q W.C.B of AB = 270°

QB of AB: ?

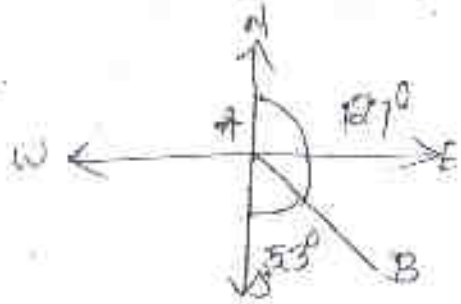
360 − 270 = 90°



N 90° W

9Q W.C.B of AB = 127°

QB of AB = ?



S (180° − 127°) E

(b) Reduced Bearing (RB) :-

When the whole circle bearing of a line is converted to quadrantal bearing, it is termed the reduced bearing. Thus the reduced bearing is similar to the quadrantal bearing. Its value lies between 0° and 90° but the quadrants should be mentioned for proper designation.

The following table should be remembered for conversion of WCB to RB.

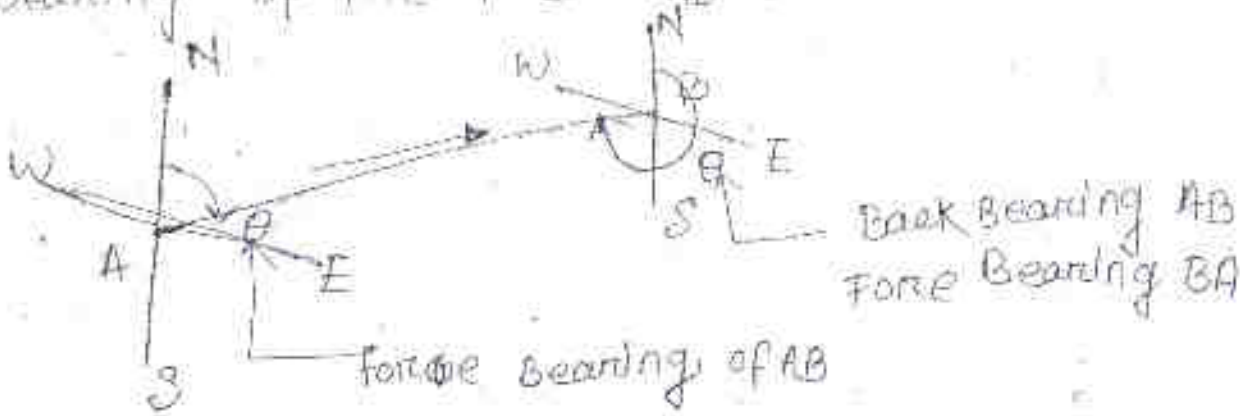
WCB between	Corresponding RB	Quadrant
0° - 90°	RB = WCB	NE
90° - 180°	RB = 180° - WCB	SE
180° - 270°	RB = WCB - 180°	SW
270° - 360°	RB = 360° - WCB	NW

Fore and Back Bearing

Every line has two bearings one is observed along the progress of the survey or forward direction, and is called 'fore bearing' and the second is observed in the reverse or opposite direction and is called 'back bearing'.

Consider the line AB shown. Here, we consider the direction of meridian as upward and the bearing is measured clockwise from the meridian.

The bearing as measured at a along the progress of survey A to B is θ, so the angle or bearing θ is the fore bearing of the line $\overline{AB}$.



(Fore Bearing and Back Bearing)

Similarly, the bearing as measured at B in the opposite direction of the progress of the survey A to B, along clockwise direction is B. The bearing B is the back bearing of the line $\overline{AB}$.

It is clear that the fore bearing and back bearing of a line differ exactly by 180°.

i.e Back bearing = Fore Bearing ± 180°

use positive sign(+ve) when fore bearing is less than 180° and (-ve) sign when it is more than 180°.

In case of quadrantal bearing system the numerical value of fore bearing and back bearing is equal but the quadrants are opposite.

ex:- If the fore bearing is N30° E
back bearing S 30° W.
than the fore bearing of the line
AB is equal to back bearing the
line BA. i.e the opposite direction
of the progress of survey.

1Q F.B of a line AB = 310°

B.B = ?

Ans F.B = 310°

B.B = FB - 180°

= 310° - 180°

= 130°

2Q F.B = 95°

B.B = 95° + 180° = 275°

3Q F.B = 245°

B.B = 245 - 180°

= 65°

4Q FB = 169°

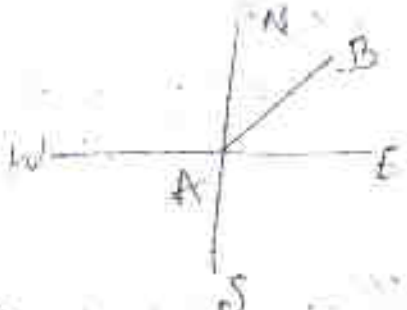
B.B = 169 + 180 = 349°

5Q F.B = 181°

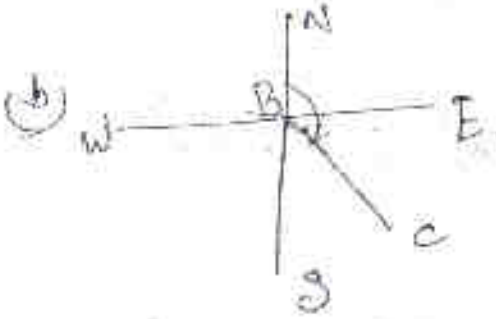
B.B = 181° - 180° = 1°

16 (a) WCB of AB = 45° 30'

QB of AB = N 45° 30' E



(b)



WCB of BC = 125° 45'

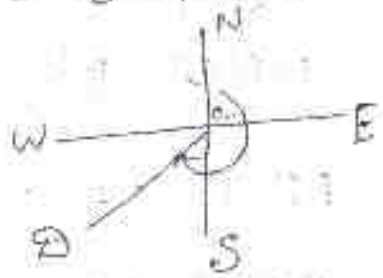
QB of BC = 180° - 125° 45'

= S 54° 15' E

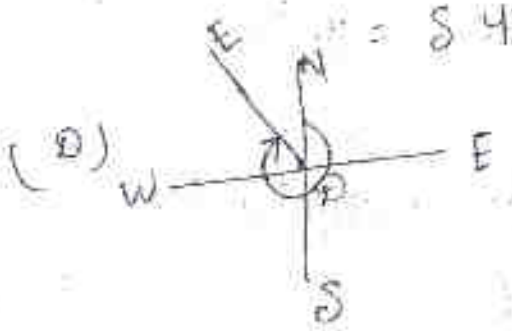
(c) WCB of CD = 222° 15'

QB of CD = 222° 15' - 180°

= S 42° 15' W



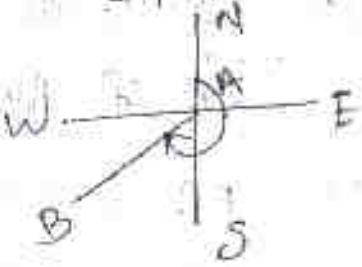
(d)



WCB of DE = 320° 30'

QB of DE = 360° - 320° 30'

= N 39° 30' W

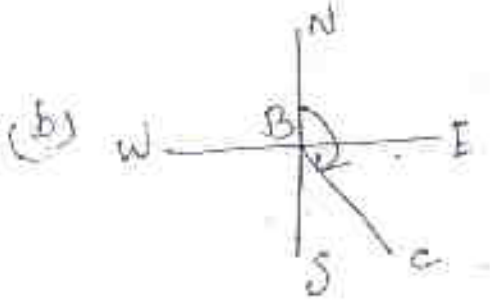


20 (a) QB of AB = S 36° 30' W

WCB of AB = 180° + 36° 30'

= 216° 30'

(b)



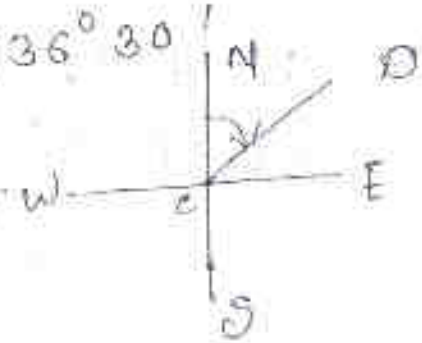
QB of BC = S 43° 30' E

WCB = 180° - 43° 30'

= 136° 30'

(c) QB of CD = N 26° 45' E

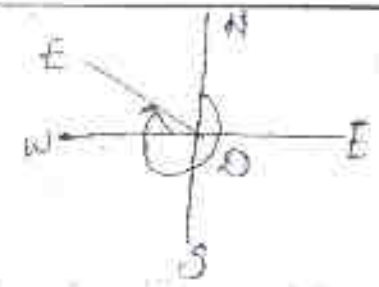
WCB = 26° 45'



(d) RB of DE = N 40° 15' W

WCB = 360° − 40° 15'

= 319° 45'



3Q (a) FB of AB = 310° 30'

B.B of AB = 310° 30' − 180° = 130° 30'

(b) FB of BC = 145° 15'

BB of BC = 145° 15' + 180° = 325° 15'

(c) FB of CD = 210° 30'

BB of CD = 210° 30' − 180° = 30° 30'

(d) FB of DE = 60° 45'

BB of DE = 60° 45' + 180° = 240° 45'

4Q (a) FB of AB = S 30° 30' E

BB of AB = N 30° 30' W

(b) FB of BC = N 40° 30' W

BB of BC = S 40° 30' W

(c) FB of CD = S 60° 15' W

BB of CD = N 60° 15' E

(d) FB of DE = N 45° 30' E

BB of DE = S 45° 30' W

5Q (a) BB of AB = [illegible]

FB of AB = $40^\circ 30' + 180^\circ = 220^\circ 30'$

(b) BB of ABC = $310^\circ 45'$

FB of ABC = $310^\circ 45' - 180^\circ = 130^\circ 45'$

(c) BB of CD = $145^\circ 45'$

FB of CD = $145^\circ 45' + 180^\circ = 325^\circ 45'$

(d) BB of DE = $215^\circ 30'$

FB of DE = $215^\circ 30' - 180^\circ = 35^\circ 30'$

5B (a) BB of AB = N $30^\circ 30'$ W

FB of AB = S $30^\circ 30'$ E

(b) BB of BC = S $40^\circ 15'$ E

FB of BC = N $40^\circ 15'$ W

(c) BB of CD = N $60^\circ 45$ E

FB of CD = S $60^\circ 45'$ W

(d) BB of DE = S $45^\circ 30'$ W

FB of DE = N $45^\circ 30'$ E

Dip of the magnetic needle

If a needle is perfectly balanced before magnetisation, it does not remain in the balanced position after it is magnetised. This is due to the magnetic influence of the earth. The needle is found to be inclined towards the pole. This inclination of the needle with the horizontal is known as the dip of magnetic needle.

It is found that the north end of the needle is deflected downwards in the northern hemisphere and that its South end is deflected downwards in the Southern hemisphere. The needle is just horizontal at the equator. To balance the dip of the needle, a rider (brass or silver coil) is provided along with it. The rider is placed over the needle at a suitable position to make it horizontal.

Local Attraction :-

A magnetic needle indicates the north direction when freely suspended or pivoted. But if the needle comes near some magnetic substances, such as iron ore, steel structures, electric cables conveying current, etc. it is found to be deflected from its true direction, and does not show the actual north. This disturbing influence of magnetic substances is known as local attraction.

To detect the presence of local attraction the fore and back bearings of a line should be taken. If the difference of the fore and back bearings of the line is exactly 180° then there is no local attraction.

If the FB and BB of a line do not differ by 180° then the needle is said to be affected by local attraction, provided there is no instrumental error.

To compensate for the effect of local attraction, the amount of error is found out and is equally distributed between the fore and back bearings of the line for example, I consider the case when,

observed FB of AB = 60°30'

observed BB of AB = 240°B'

Calculated BB of AB = 60°30' + 180°0' = 240°30'

Corrected BB of AB = 1/2 (240°0' + 240°30')

= 240°15'

Hence Corrected FB of AB = 240°15' - 180°0' = 60°15'

Method of Application of Correction

(a) First Method :- The interior angles of a traverse are calculated from the observed bearings. Then an angular check is applied. The sum of the interior angles should be equal to $(2n-4) \times 90^\circ$ (n being the number of sides of the traverse). If it is not so, the total error is equally distributed among all the angles of the traverse.

Then, starting from the unaffected line, the bearings of all the lines may be corrected by using the corrected interior angles. This method is very laborious and is not generally employed.

(b) Second Method :- In this method, the interior angles are not calculated from the given table, the unaffected line is first detected. Then, commencing from the unaffected line, the bearings of the other affected lines are corrected by finding the amount of correction at each station.

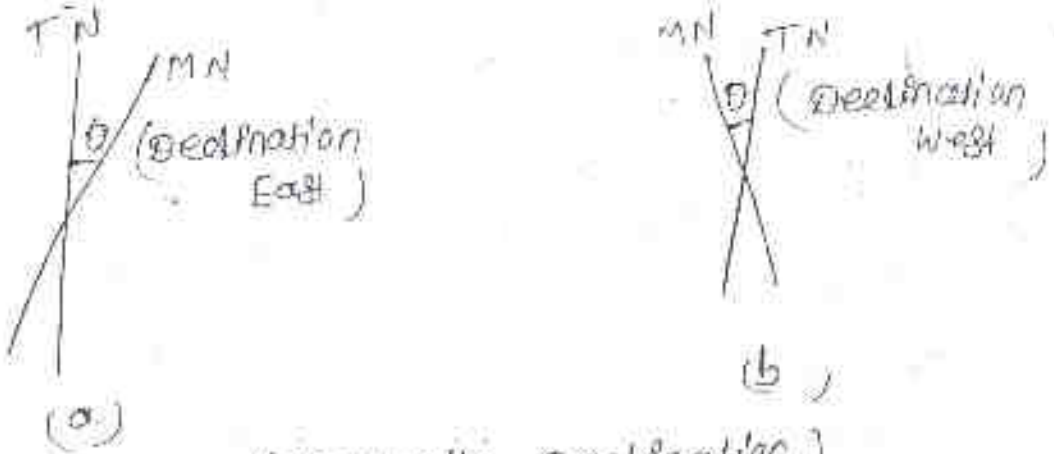
This is an easy method, and one which is generally employed.

Magnetic Declination :-

The horizontal angle between the magnetic meridian and true meridian is known as magnetic declination.

When the north end of the magnetic needle is pointed towards the west side of the true meridian, the position is termed declination west (θW)

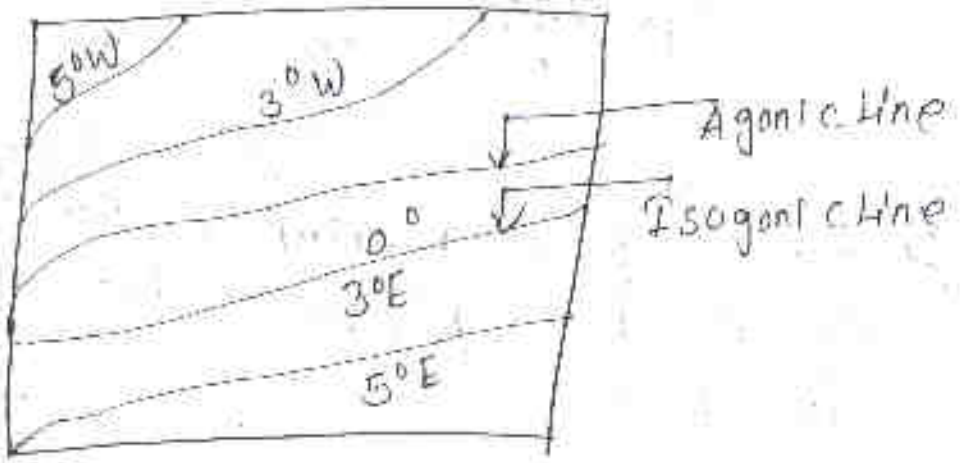
When the north end of the magnetic needle is pointed towards the east side of the true meridian, the position is termed declination East. (θE)



(Magnetic Declination)

Isogonic and Agonic lines

Lines passing through points of equal declination are known as 'isogonic' lines. The line passing through points of zero declination is said to be the 'agonic' line.



(Isogonic and Agonic lines)

1 June 2021

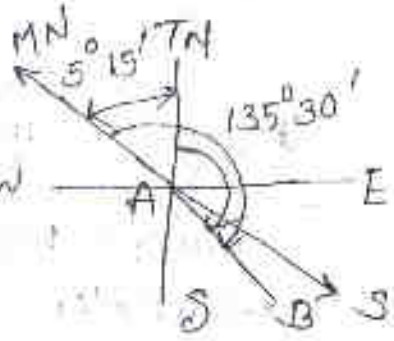
10 (a) The magnetic bearing of a line AB is 135°30' what will be the true bearing, if the declination is 5° 15' W ?

(b) The true bearing of a line CD is 210°45' what will be its magnetic bearing if the declination is 8° 15' W ?

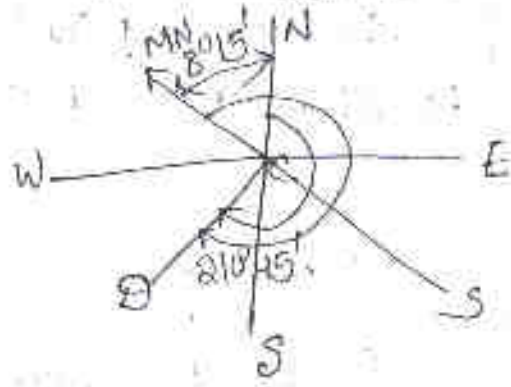
Sol^n (a) True Bearing = M.B ± M.D

= 135°30' − 5°15'

= 130°15'



(b)



Magnetic Bearing

= True Bearing ± Magnetic Bearing

= 210°45' + 8°15'

= 219°00'

20 The magnetic bearing of a line CD is S 30°15'W if the declination is find HS true bearing 10°15'E ?

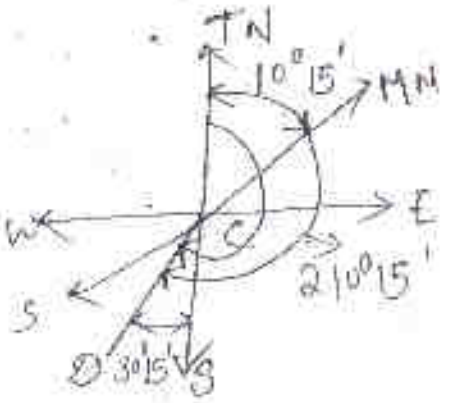
Sol^n QB = 30°15'

W.C.B = 180° + 30°15' = 210°15'

True Bearing = M.B ± M.D

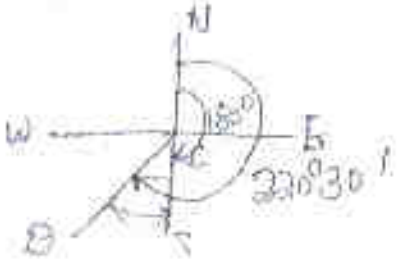
= 210°15' + 10°15'

= 220°30'



Required TB = 220°30' − 180°

= S 40°30' W



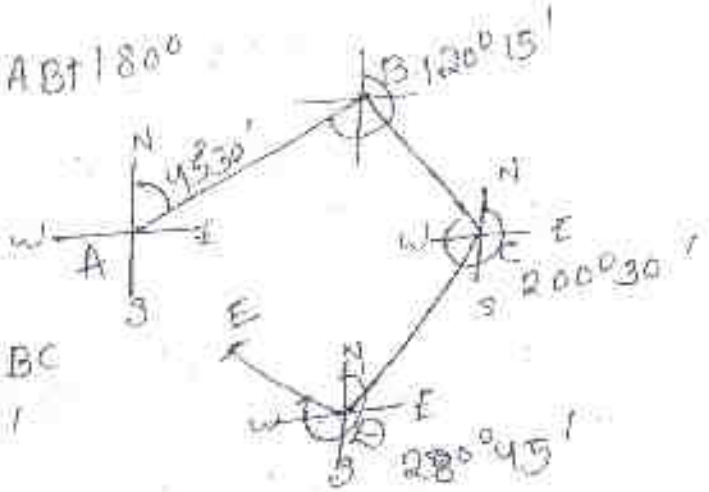
3 June 2021

Q The FB of the lines AB, BC, CD & DE are 45° 30', 120°15', 200°30' & 200°45' respectively. Find angle ∠B, ∠C & ∠D ?

Soln (i) BB of AB = FB of AB ± 180°

= 45°30' + 180°

= 225°30'



∠B = BB of AB − FB of BC

= 225°30' − 120°15'

= 105°15'

(ii) BB of BC = 120°15' ± 180° = 300°15'

∠C = BB of BC − FB of CD

= 300°15' − 200°30'

= 99°45'

(iii) BB of CD = FB of CD − 180°

= 200°30' − 180° = 20°30'

Exterior ∠D = FB of DE − BB of CD

Interior ∠D

= 280°45' − 20°30'

= 260°15'

Interior angle D = 360° − exterior ∠D

= 360° − 260°15'

= 99°45'

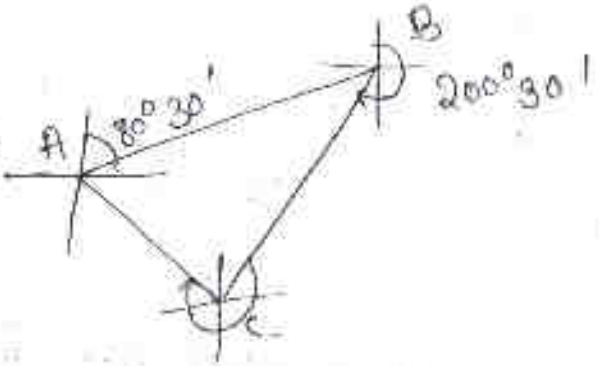
Q A traverse is above done by three stations A, B, C in clock wise order in the form of an equilateral tolangle. If the fore bearings of other sides?

Sol^n

(i) FB of BC = BB of AB - $\angle B$

$= (80°30' + 180°) - 60°$

$= 206°30'$



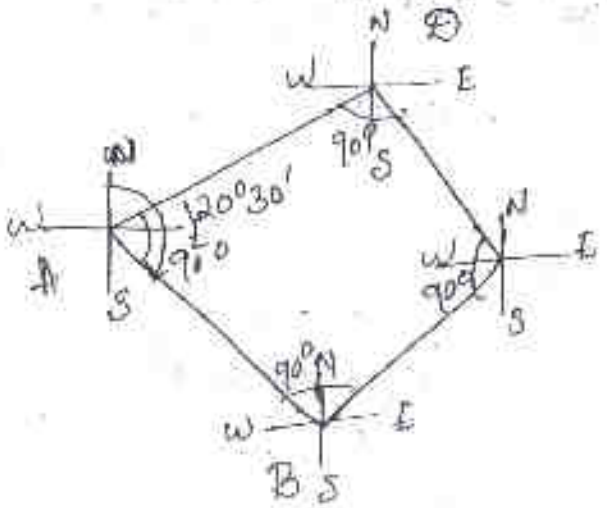
(ii) FB of AC = BB of BC + external angle at C

$= (200°30' - 180°) + (360° - 60°)$

$= 20°30' + 300°$

$= 320°30'$

Q A traverse is made in the form of a square taking in clockwise order. If bearing of AB is 120°30', find the bearing other side?



Sol^n FB of AB = 120°30'

(i) external angle (B) = 360° - 90° = 270°

BB : $120°30' + 180° = 300°30'$

FB of BC = BB of AB - external $\angle B$

$= 300°30' - 270°$

$= 30°30'$

(ii) BB of BC : $180° + 30°30'$

$= 210°30'$

FB of CD = BB of BC + Internal angle $\angle C$

$= 210°30' + 90° = 300°30'$

(iii) FB of DA = BB of CD + Internal $\angle D$

$= (300°30' - 180°) + 90°$

$= 120°30' + 90° = 210°30'$

Checked:

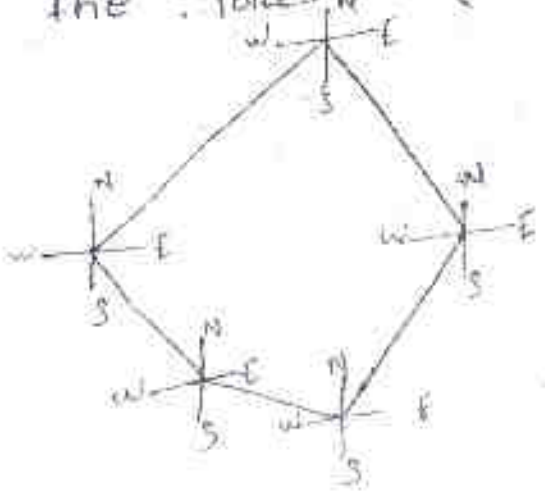
FB of AB = BB of DA + Internal $\angle A$.

$= (210°30' - 180°) + 90°$

$= 30°30' + 90°$

$= 120°30'$

5. A closed traverse is conducted with five station, AB, BC, CD & DE taken in anticlockwise direction in the form of regular pentagon of the FB of AB is $30°0'$. Find the forebearing of other side.



Sol^n (i) BB of AB $= 30^\circ + 180^\circ$

$= 210^\circ$

FB of BC = BB of AB + Internal $\angle B$

$= 210^\circ + 108^\circ = 318^\circ$

(ii) FB of CD = BB of BC + Internal $\angle C$

$= (318^\circ - 180^\circ) + 108^\circ$

$= 246^\circ$

(iii) FB of DE = BB of CD + Internal $\angle D$

$= (246^\circ - 180^\circ) + 108^\circ$

$= 174^\circ$

FB of EA = BB of DE + External $\angle B$

$= (174^\circ + 180^\circ) - (360^\circ - 108^\circ)$

$= 354^\circ - 252^\circ = 102^\circ$

Internal angle
$= (2 \times 5 - 4) \times 90^\circ$
$= 540^\circ$
5 angle $= 540^\circ$
1 angle $= 108^\circ$

Checked

BB of EA $= 102^\circ + 180^\circ = 282^\circ$

FB of AB = BB of EA - external $\angle E$

$= 282^\circ - (360^\circ - 108^\circ)$

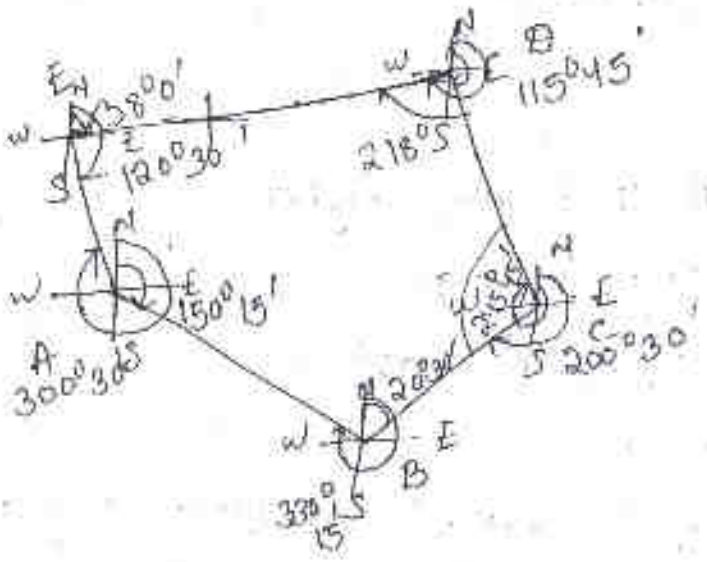
$= 282^\circ - 252^\circ = 30^\circ$

5 The following are the FB & BB of a closed traverse.

side	FB	BB
AB	$150^{\circ}15'$	$330^{\circ}15'$
BC	$20^{\circ}30'$	$200^{\circ}30'$
CD	$295^{\circ}45'$	$115^{\circ}45'$
DE	$218^{\circ}0'$	$38^{\circ}0'$
EA	$120^{\circ}30'$	$300^{\circ}30'$

a) calculation the interior angle of the traverse?

Soln



Internal Angle

(i) Exterior $\angle A$ = (BB of EA - FB of AB)

$= (300^{\circ}30' - 150^{\circ}15')$

$= 150^{\circ}15'$

Interior $\angle A$ = $360^{\circ} - 150^{\circ}15' = 259^{\circ}45'$

(ii) Exterior $\angle B$ = BB of AB - FB of BC

$= 330^{\circ}15' - 20^{\circ}30'$

$= 309^{\circ}45'$

Interior $\angle B = 360^0 - 309^0 45' = 50^0 15'$

(iii) Interior $\angle C$ = FB of CD − BB of BC

$= 295^0 45' - 200^0 30'$

$= 95^0 15'$

(iv) Interior $\angle D$ = FB of DE − BB of CD

$= 218^0 - 115^0 45'$

$= 102^0 15'$

(v) Interior $\angle E$ = FB of EA − BB of DE

$= 120^0 30' - 38^0 0'$

$= 82^0 30'$

Checked

Σ of all Interior angle

$= (2n-4) \times 90^0$

$= (2\times5-4) \times 90^0$

$= 540^0$

Sum of Calculate Interior angle :-

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

$= 209^0 45' + 50^0 45' + 95^0 15' + 102^0 15' + 82^0 30'$

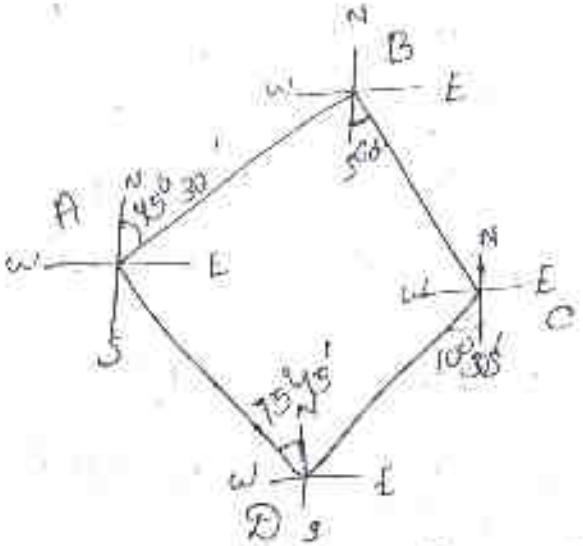
$= 540^0$

5. The following are the bearing of a closed traverse

Side	FB	BB
AB	N $45^0 30'$ E	S $45^0 30'$ N
BC	S $60^0 0'$ E	N $60^0 0'$ W
CD	S $10^0 30'$ E	N $10^0 30'$ W

DA N 75°45'W S 75°45'E

Calculate the interior angle of the traverse.



(i) Interior angle $\angle A = 180° - (45°30' + 75°45')$

$= 58°45'$

(ii) Interior angle $\angle B$ = BB of AB + FB of AB, $\angle B$

$= 45°30' + 60°0'$

$= 105°30'$

(iii) Interior angle $\angle D$ = BB of CD + FB of DA

$= 75°45' + 10°30' = 86°15'$

(iv) Interior angle $\angle C = 180° - (60°0' + 10°30')$

$= 109°30'$

∴ Sum of Calculate interior angle :-

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

$= 58°15' + 105°30' + 109°30' + 86°15'$

$= 360°0'$

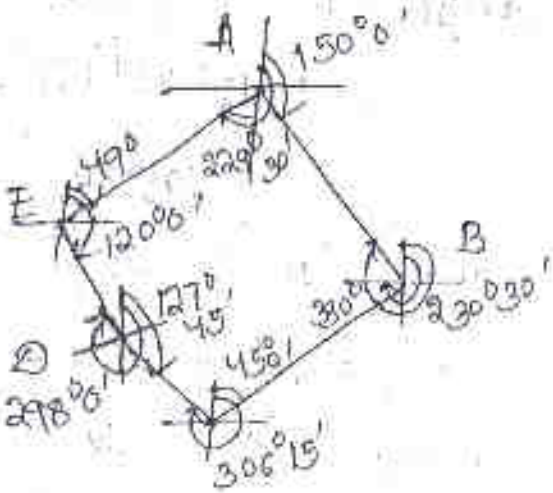
Checked Σ of all interior angle $= (2 \times 4 - 4) \times 90°$

$= 360°0'$

14 Jun 2021

Q The following are the bearings observed in traversing with a compass, an area where local attraction was suspected. calculate the interior angles & corrected than if necessary.

Line	FB	BB
AB	150°0'	330°0'
BC	230°30'	48°0'
CD	306°15'	127°45'
DE	298°0'	120°0'
EA	49°30'	229°30'



Sol^n

Interior angle ∠A = BB of EA - FB of AB

= 229° 30' - 150°0'

= 79°30'

Interior angle ∠B = BB of AB - FB of BC

= 330°0' - 230°30'

= 99°30'

Exterior angle LC = FB of CD - BB of CB

$= 306^0 - 48^0 0' \quad = 258^0 15'$

Interior angle LC $= 360^0 - 258^0 15' = 101^0 45'$

(10) Exterior angle LD = FB of DE - BB of DC

$= 298^0 0' - 127^0 45' = 170^0 15'$

Interior angle $= 360^0 - 170^0 15' = 189^0 45'$

Interior angle LI = BB of DE - FB of EA

$= 120^0 0' - 49^0 0' = 71^0 0'$

Σ of all interior angle

$= (2n-4) \times 90^0 \quad = 540^0$.

Σ of all calculate interior angle

LA + LB + LC + LD + LE

$= 79^0 30' + 99^0 30' + 101^0 45' + 189^0 45' + 71^0 0'$

$= 541^0 0'$

Error $= 541^0 - 540^0 = +1^0$

Correction for angle $= \frac{-16}{5} = \frac{-60^0}{5} = -12'$

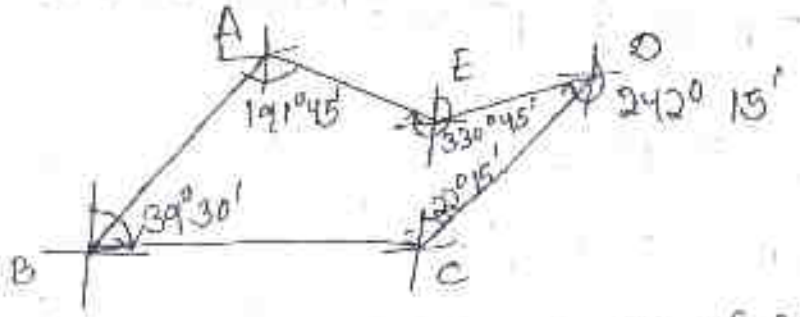
Angle	Calculate value	Correction	Corrected value
LA	$79^0 30'$	$-12'$	$79^0 18'$
LB	$99^0 30'$	$-12'$	$99^0 18'$
LC	$101^0 45'$	$-12'$	$101^0 38'$
LD	$189^0 45'$	$-12'$	$189^0 33'$

LE	$70^\circ 30'$	-12°	$70^\circ 18'$
		Total =	$540^\circ 0'$

Q The following are the observed bearing of a traverse ABCDEA with a compass in a place where local attraction was suspected.

Line	FB	BB
AB	$191^\circ 45'$	$13^\circ 0'$
BC	$39^\circ 30'$	$222^\circ 30'$
CD	$22^\circ 15'$	$200^\circ 30'$
DE	$242^\circ 45'$	$62^\circ 45'$
EA	$330^\circ 45'$	$147^\circ 45'$

Find the correct bearings of the line?



(i) Interior angle LA = FB of AB − BB of EA
$= 191^\circ 45' - 147^\circ 45' = 44^\circ 0'$

(ii) Interior angle LB = FB of BC − BB of AB
$= 39^\circ 30' - 13^\circ 0' = 26^\circ 30'$

(iii) Exterior angle LC = BB of BC − FB of CD

$= 222^\circ 30' - 22^\circ 15'$

$= 200^\circ 15'$

Interior angle $= 360^\circ - 200^\circ 15'$

$= 159^\circ 45'$

(iv) Interior angle D $=$ FB of DE $-$ BB of CD

$= 242^\circ 45' - 200^\circ 30'$

$= 42^\circ 15'$

(v) Interior angle E $=$ FB of EA $-$ BB of DE

$= 330^\circ 15' - 62^\circ 45'$

$= 267^\circ 30'$

Calculation for Corrected bearings :-

The line DE is free from local attraction

FB of DE $= 242^\circ 45'$ (Correct)

FB of EA $= 330^\circ 15'$ (Correct)

Corrected :-

FB of AB $=$ BB of EA $+$ Internal $\angle A$

$= (330^\circ 15' - 180^\circ) + 44^\circ$

$= 150^\circ 15' + 44^\circ = 194^\circ 15'$

FB of BC $=$ BB of AB $+$ Internal $\angle B$

$= (194^\circ 15' - 180^\circ) + 26^\circ 30'$

$= 14^\circ 15' + 26^\circ 30' = 40^\circ 45'$

FB of CD $=$ BB of BC $-$ External $\angle C$

$= 220^\circ 45' - 200^\circ 15'$

$= 20^\circ 30'$

FB of DE = BB of CD + Internal ∠D

$= (180^\circ + 20^\circ 30') + 42^\circ 15'$

$= 200^\circ 30' + 42^\circ 15'$

$= 242^\circ 45'$

	Correct	
Line	FB	BB
AB	$194^\circ 15'$	$14^\circ 15'$
BC	$40^\circ 45'$	$200^\circ 45'$
CD	$20^\circ 30'$	$200^\circ 30'$
DE	$242^\circ 45'$	$62^\circ 45'$
EA	$330^\circ 45'$	$147^\circ 45'$

Type of Compass

There are two types of compasses :-

(i) Prismatic compass

(ii) Surveyor's compass

Prismatic Compass

In this compass the reading are taken with the help of a prism. The following are the essential parts of this compass.

(a) Compass Box :- The compass box is a circular metalic box (the metal should be non-magnetic) of 8 to 10 cm diameter. A pivot with a sharp point is provided at the centre of the box.

Magnetic Needle and Graduated Ring :-

The magnetic needle is made of a broad magnetised iron bar. The bar is pointed at both ends. The magnetic needle is attached to a graduated aluminium ring.

The ring is graduated from 0° to 360° clockwise, and the graduations begin from the South end of the needle. Thus 0° is marked at the South 90° at the west 180° at the north and 270° at the east. the degrees are again subdivided into half degrees. The figures are written upside down. The arrangement of the needle and ring contains an agate cap pivoted on the Central pivot point. A rider of brass or silver coil is

its dip.

(c) Sight vane and prism :- The sight vane and the reflecting prism are fixed diametrically opposite to the box. The sight vane is hinged with the metal box and consists of a horseshoe at the centre. The prism consists of a sighting slit at the top and two small circular holes, one at the bottom of the prism and the other at the side of the observer's eye.

(d) Dark glasses :- Two dark glasses are provided with the prism. The red glass is ment for sighting luminous objects at night and the blue glass for reducing the strain on the observer's eye in bright daylight.

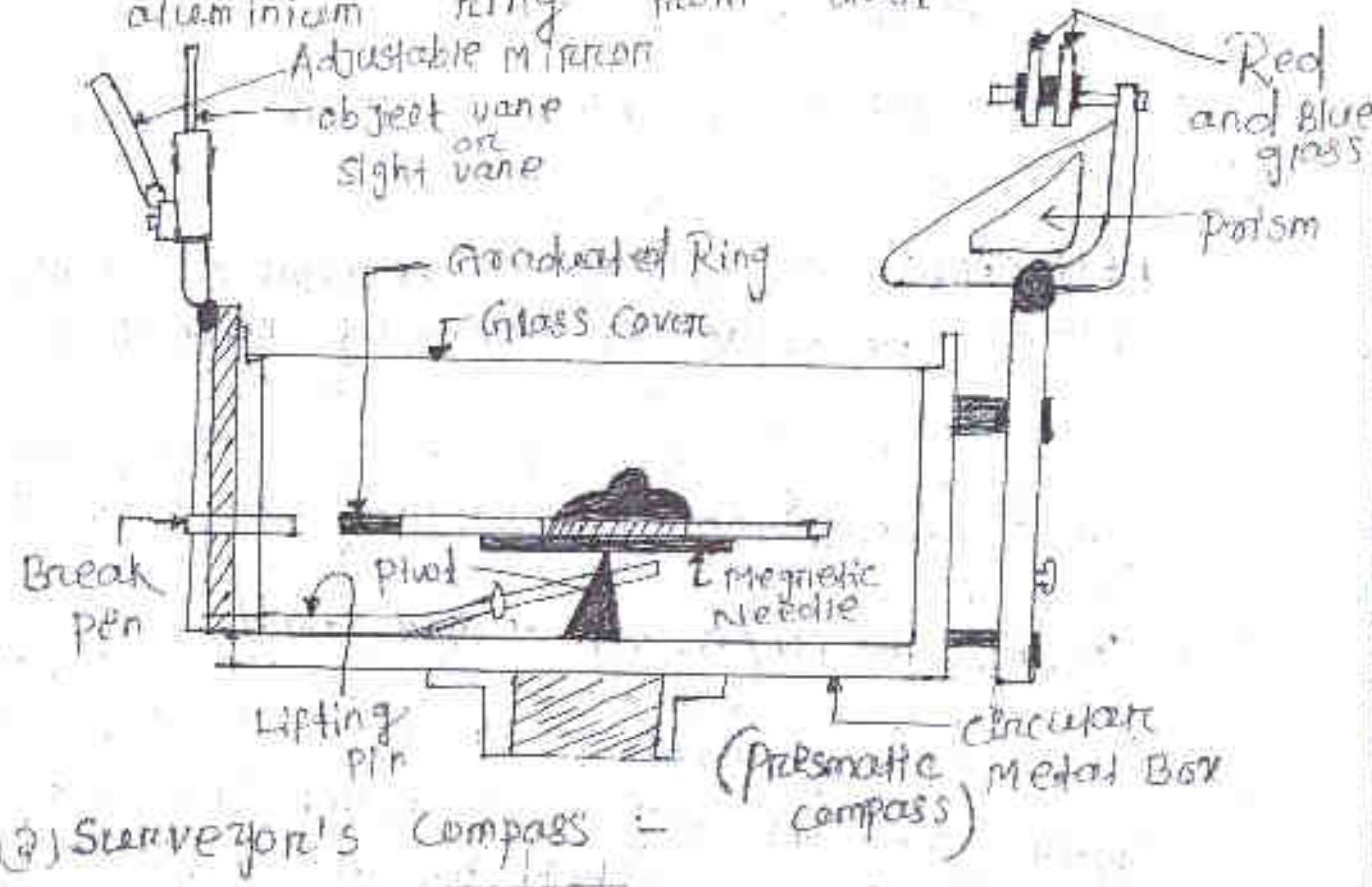
(e) Adjustable Mirror :- A mirror is provided with the sight vane. The mirror can be lowered or raised, and can also be inclined. If any object is too low or too high with respect to the line of sight, the mirror can be adjusted to observe it through reflection.

(f) Brake pin :- A brake pin is provided just at the base of the sight vane. If pressed gently, it stops the oscillations of the ring.

(g) Lifting pin :- A lifting pin is provided just below the sight vane. When the sight vane is folded, it presses the lifting pin.

The lifting pin then lifts the magnetic needle out of the pivot point to prevent damage to the pivot head.

Glass cover :- A glass cover is provided on top of the box to protect the aluminium ring from dust.



(Prismatic compass)

(2) Surveyor's Compass :-

The surveyor's compass is similar to the prismatic compass except for the following points.

(i) There is no prism on it. Readings are taken with the naked eye.

(ii) It consists of an eye-vane (in place of prism) with a fine sight-slit.

(iii) The graduated aluminium ring is attached to the circular box. It is not fixed to the magnetic needle.

(iv) The magnetic needle moves freely over the pivot. The needle shows the reading on the graduated ring.

(v) The ring is graduated from 0° to 90° in four quadrants 0° is marked at the north and south, and 90° at the east and west. The Letters E and W are interchanged from their true positions. The figures are written the right way up.

(f) The mirror is attached to the object vane.

TEMPORARY ADJUSTMENT OF PRISMATIC COMPASS (Field procedure of observing bearing)

The following procedure should be adopted while measuring the bearing by prismatic compass.

(1) Fixing the compass with tripod stand:

The tripod stand is placed at the required station with its legs well apart. Then the prismatic compass is held by the left hand and placed over the threaded top of the stand. After this, the compass box is fixed with the threaded top of the stand.

(2) Centring: Normally, the compass is centred by dropping a piece of stone from the bottom of the compass box. Centring may also be done with the aid of a plumb bob held centrally below the compass box.

(3) Levelling: Levelling is done with the help of a ball-and-socket arrangement provided on top of the tripod stand.

This arrangement is loosend and the box is placed in such a way that the graduated ring rotates freely without touching either the bottom of the box or the glass cover on top.

(4) Adjustment of prism:-

The prism is moved up and down till the figures on the graduated ring are seen sharp and clear.

(5) observation of Bearing:-

After centring and levelling the Compass box over the station, the ranging rod at the required station is bisected perfectly by sighting through the slit of the prism and horsehair at the sight vane.

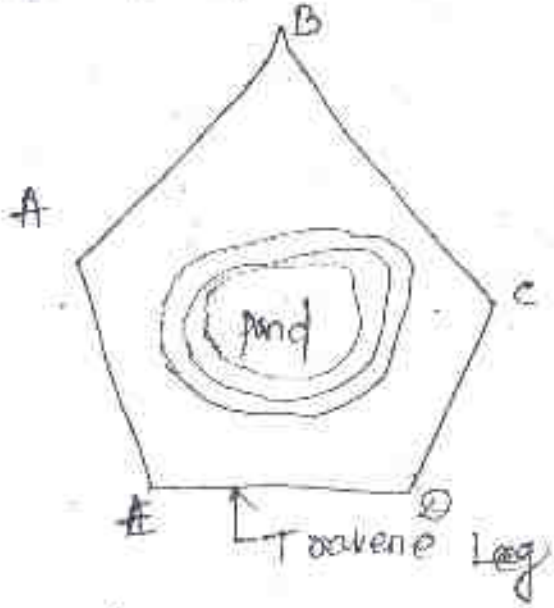
At this time, the graduated ring may rotate rapidly. The brake pin is pressed very gently to stop this rotation when the ring Compass to rest, the box is struck very lightly to verify the horizontality of the ring and the frictional effect on the pivot point. Then the reading is taken from the graduated ring through the hole in the prism. This reading will be the magnetic bearing of the line.

Traversing :- As already stated in the last Section surveying which involves a series of connected lines is known as traversing. The sides of the traverse are known as traverse legs.

In traversing the lengths of the lines ~~forms a closed~~ circuit i.e when the finishing point coincides with the starting point of a survey. It is called a 'closed traverse'. Here ABCDEA represents are measured by chain and the directions are fixed by compass or theodolite or by forming angles with chain and tape.

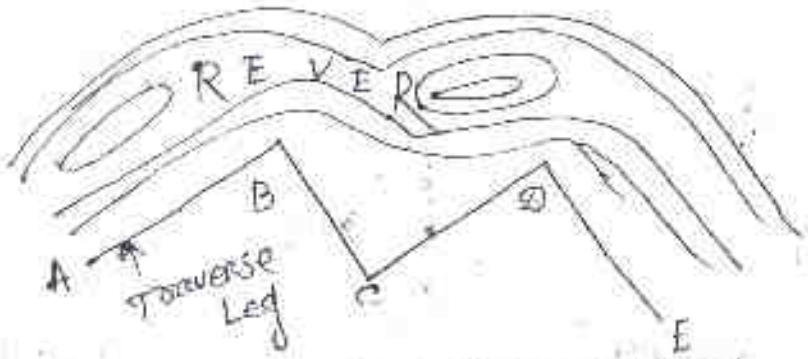
A traverse may be of two types - closed and open.

(i) Closed traverse when a series of a connected lines forms a closed circuit. i.e when the finishing point coincides with the starting point of a survey, it is called a 'closed traverse'. Here ABCDEA represents a closed traverse. closed traverse is suitable for the survey of boundaries of ponds, forests, estates, etc



(closed traverse)

open traverse when a sequence of connected lines extends along a general direction and does not return to the starting point. It is known as 'open traverse' or 'unclosed traverse' Here ABCDE represents an open traverse.



(open traverse)

open traverses is suitable for the survey of roads, rivers, coast lines, etc.

METHODS OF TRAVERSING

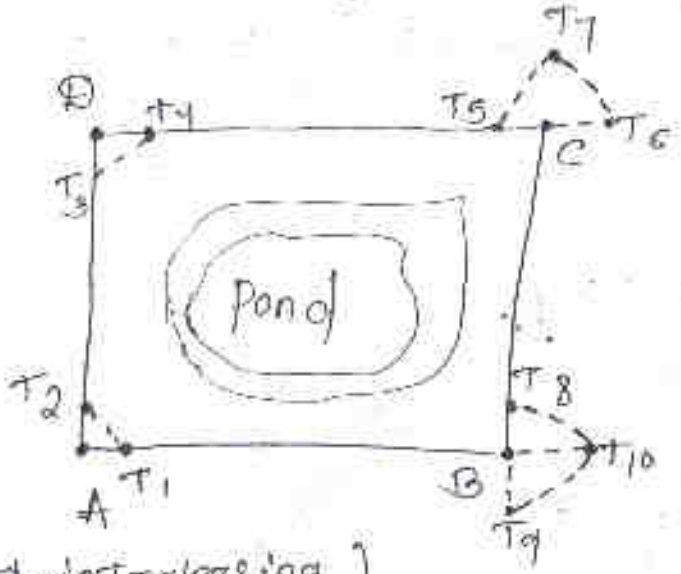
Traverse survey may be conducted by the following methods.

1. chain traversing by (chain angle)
2. Compass traversing (by free needle)
3. theodolite traversing (by fast needle)
4. plane table traversing (by plane table)

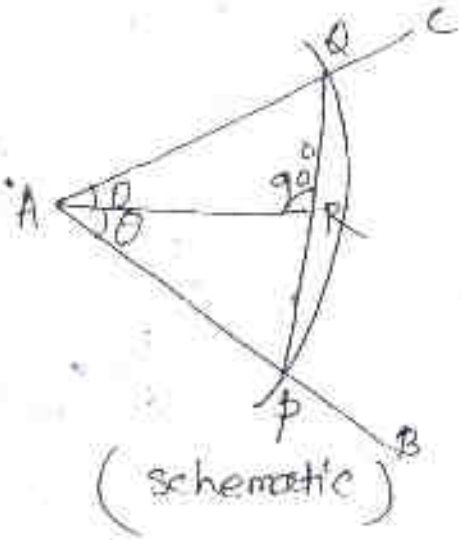
Chain Traversing :-

chain traversing is mainly conducted when it is not possible to adopt triangulation. In this method, the angle between adjacent sides are fixed by chain angles. The entire survey is conducted by chain and

tape only and no angular measurements are taken. When it is not possible to form triangles, as for example in a pond, chain traversing is conducted.



(chain traversing)



(schematic)

The formation of chain angles is explained below.

First Method :- Suppose a chain angle is to be formed to fix the direction of side AB and AD. Tie stations T_1 and T_2 are fixed on fix the directions lines AB and AD. The distances AT_1, AT_2 and T_1T_2 are measured. Then the angle $\angle T_1AT_2$ is said to be the chain angle. So the chain angle is fixed by the line T_1T_2.

Second method :- Sometimes the chain angle is fixed by a chord. Suppose the angle between the lines AB and AC is to be fixed. Taking A as the centre and a radius equal to one tape

length (15m) an arc intersecting the lines AB and AC at points P and Q, respectively is drawn. The chord PQ is measured and bisected at R.

Let $\angle PAR = \theta$

$\angle BAC = 2\theta$

$AP = AQ = 15m$.

In triangle PAR $\sin\theta = \frac{PR}{AP} = \frac{\frac{1}{2}PQ}{\frac{1}{2}AP} = \frac{PQ}{30}$

$\therefore \theta = \sin^{-1}\frac{PQ}{30}$

The angle θ can be calculated from the above equation, and the chain angle $\angle BAC$ can be determined accordingly.

(2) Compass Traversing :- In this method, the fore and back bearing of the traverse legs are measured by a prismatic compass and the sides of the traverse by chain or tape. Then the observed bearings are verified and necessary correction for local attraction are applied. In this method, closing error may occure when the traverse is plotted. This error is adjusted graphically by using Bowditch's rule.

(3) Theodolite Traversing

In such traversing, the horizontal angles between the traverse legs are measured by a theodolite. The length of the legs are measured by chain [illegible] the stadia method

The magnetic bearings of the starting leg is measured by a theodolite. Then the magnetic bearings of the other sides are calculated. The independent coordinates of all the traverse stations are then found out. This method is very accurate.

(4) Plane Table Traversing

In this method, a plane table is set at every traverse station in the clockwise or anticlockwise direction, and the circuit is finally closed. During traversing, the sides of the traverse are plotted according to any suitable scale. At the end of the work, any closing error which may occure is adjusted graphically.

CHECK ON CLOSED TRAVERSE

Check on Angular measurements

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

(b) The sum of the measured exterior angles should be equal to $(2N+4)\times 90^\circ$

(c) The algebraic sum of the deflection angles should be equal to 360°.

Right hand deflection is considered positive and left-hand deflection negative.

check on Linear Measurement

(a) The lines should be measured once each on two different days (along opposite directions) Both measurements should tally.

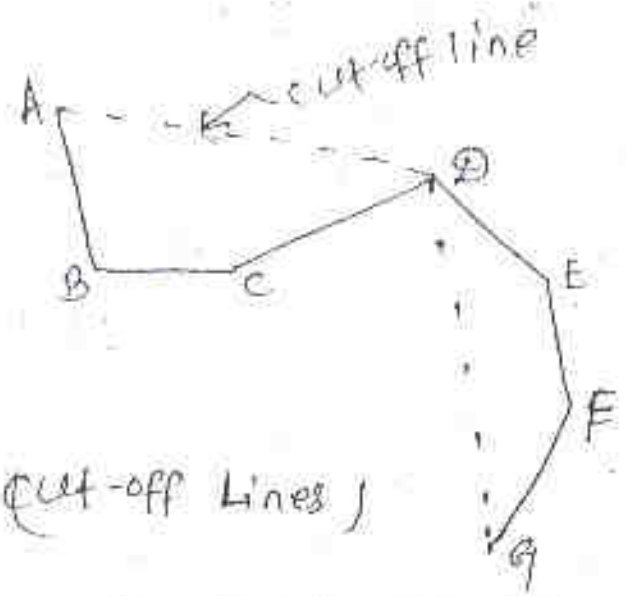
(b) Linear measurements should also be taken by the stadia method. The measurements by chaining and by the Stadia method should tally.

CHECK ON OPEN TRAVERSE

In open traverse, the measurements cannot be checked directly. But sum field measurements can be taken to check the accuracy of the work. The methods are discussed below.

Taking cut-off lines

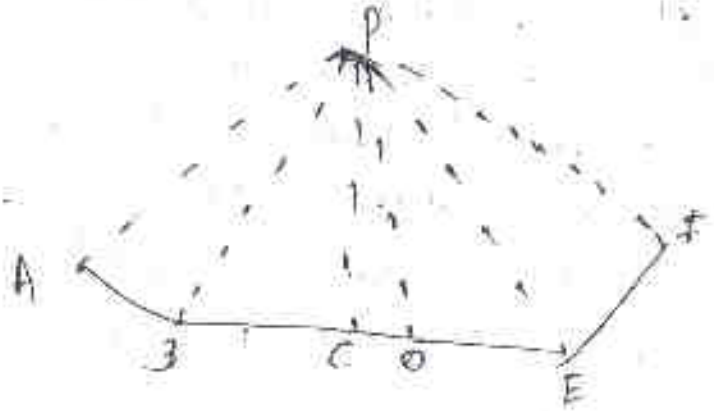
Cut-off lines are taken between some intermediate stations of the open traverse suppose ABCDEFG represents an open traverse. Let AG and DG be the cut-off lines The lengths and magnetic bearings of the cut-off lines are measured accurately After plotting the traverse, the distance and bearing are noted from the map. These distance and bearings should tally with the actual records obtained from the field.



(Cut-off Lines)

Taking an Auxiliary point :-

Suppose ABCDEF is an open traverse. A permanent point P is selected on one side of it. The magnetic bearings of this point are taken from the traverse stations - A, B, C, D etc. If the survey is carried out accurately and so is the ~~prismatic~~ plotting all the measured bearings of P when plotted should meet at the point P. The permanent point P is known as the auxiliary ~~point~~ point.



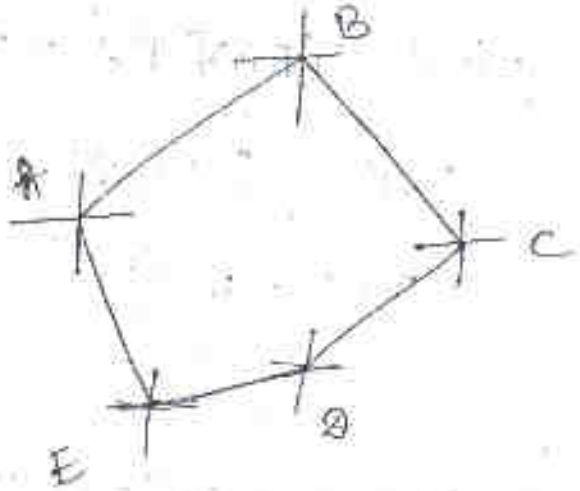
(Auxiliary point)

19 Jun 2021

Q The following bearings were observed in traversing with a compass, an area where local attraction was suspected. Find the amount of local attraction at different stations, the corrected bearings of lines & the included angles.

Line	FB	BB
AB	68° 15′	248° 15′
BC	148° 45′	32° 15′
CD	224° 30′	46° 00′
DE	217° 13′	38° 15′
EA	327° 45′	147° 45′

Sol:



Internal $\angle A$ = BB of EA − FB of AB

= 147° 45′ − 68° 15′

= 79° 30′

Internal $\angle B$ = BB of AB − FB of BC

= 248° 15′ − 148° 45′

= 99° 30′

Internal ∠C = BB of BC − FB of CD

= 325°15′ − 224°30′

= 101°45′

Internal ∠D = 360° − external angle ∠D

= 360° − (FB of DE − BB of CD)

= 360° − (217°15′ − 46°00′)

= 360° − 171°15′ = 188°45′

Internal angle ∠E = 360° − external ∠E

= 360° − (FB of EA − BB of DE)

= 360° − (327°45′ − 38°15′)

= 360° − 289°30′ = 70°30′

CHECKED

= (2n − 4) × 90° = (2×5−4) × 90° = 540

∠A + ∠B + ∠C + ∠D + ∠E

= 79°30′ + 99°30′ + 101°45′ + 188°45′ + 70°30′

= 540°

Calculation for corrected bearings :-

The line EA & AB is free from local attraction.

FB of EA = 327°45' (Correct)

FB of AB = 68°15' (correct)

FB of BC = 148°45' (correct)

FB of CD = BB of BC - Internal ∠C

= (148°45' + 180°) - 101°45'

= 328°45' - 101°45'

= 227°0'

FB of DE = BB of CD + external angle ∠D

= (FB of CD ~~+~~ 180°) + (360° - Internal ∠D)

= (227° - 180°) + (360° - 188°15')

= 47° + 171°0'

= 218°15'

FB of EA = BB of DE + external ∠E

= (218°15' - 180°) + (360° - 70°30')

= 38°15' + 289°30'

= 327°45'

FB of AB = BB of EA - Internal ∠A

= (327°45' - 180°) - 79°30'

= (147°45') - 79°30' = 68°15'

FB of BC = BB of AB - Internal ∠B

= (FB of AB ± 180°) - 99°30'

= (68°15' + 180°) - 99°30'

= 248°15' - 99°30'

= 148°45'

Line	Corrected	
	F B	B B
AB	68°15'	248°15'
BC	148°15'	328°45'
CD	227°0'	47°0'
DE	218°15'	38°15'
EA	327°45'	147°45'

2nd method

(i) The line AB & EA is free form local attraction. So station A, B & E are free from local attraction.

(ii) The FB of BC is also correct.

(iii) FB of BC = 148°45'

BB of BC = 148°45' ± 180° = 328°45'

observed BB of BC = 328°15'

Correction = 328°45' - 328°15'

= +2°30' applied at station 'C'

(iv) FB of CD = observed FB of CD + 2°30'

= 224°30' + 2°30'

= 227°

Corrected BB of CD = 227° − 180° = 47°

observed BB of CD = 46°

Correction = 47° − 46° = 1° applied at station 'D'

(v) FB of DE = observed FB of DE + 1°

= 217°15' + 1°

= 218°15'

Corrected BB of DE = 218°15' − 180°

= 38°15'

observed BB of DE = 38°15'

Line	observed FB	observed BB	Correction	Corrected FB	Corrected BB	Remark
AB	68°15'	248°15'	0° at A	68°15'	248°15'	Station A is free from local attraction.
BC	148°45'	326°15'	0° at B	148°45'	328°45'	station B is free from local attraction.
CD	224°30'	46°00'	+2°30' at C	227°	47°	
DE	217°15'	38°15'	+1° at D	218°15'	38°15'	
EA	327°45'	147°45'	0° at E	327°45'	147°45'	station E is free from local attraction

22 Jun 2021

36

Following are the bearings observed while traversing with a compass an area where local attraction was suspected. Find the correct bearings of the lines & also true bearings if the magnetic declination is 10° W.

Line	FB	BB
AB	59°00'	239°00'
BC	139°30'	317°00'
CD	215°15'	36°30'
DE	208°00'	29°00'
EA	318°30'	138°45'

(i) The line AB is free local Attraction station A & B free Local attraction.

(ii) FB of BC is correct.

BB of BC = 139°30' ± 180° = 319°30'.

Observed BB of BC = 317°00'

Correction = 319°30' − 317°00' = + 2°30' at 'C' station.

(iii) FB of CD = 215°15' + 2°30' = 217°45'

Corrected BB of CD = 217°45' − 180° = 37°45'

observed BB of CD = 36°30'

= 37°45' − 36°30' = +1°15' station 'D'

(iv) FB of DE = 208° + 1°15' = 209°15'

observed BB of DE = 29°15'

Correct = 29°15' − 29°0'

= +0°15' applied station 'E'

(v) FB of EA = 318°30' + 0°15' = 318°45'

observed = 138°45'

Correct bearing of EA = 138°45'

observed			Correction	Corrected		Remarks
Line	FB	BB		FB	BB	
AB	59°0'	239°0'	0° at 'A'	59°0'	239°0'	station A is free from local attraction
BC	139°30'	319°0'	0° at 'B'	139°30'	319°30'	Station B is free from local attraction
CD	215°15'	36°30'	+2°30' at 'C'	217°45'	37°45'	
DE	208°0'	29°0'	+1°30' at 'D'	209°15'	29°15'	
EA	318°30'	138°45'	+0°15' at 'E'	318°45'	138°45'	

True Bearing

Line	Corrected bearing		declination -10°W	True Bearing	
	FB	BB		FB	BB
AB	59° 0'	239°0'	-10°W	49° 0'	229° 0'
BC	139°30'	319°30'	-10° W	129°30'	309°30'
CD	217°45'	37°45'	- 10°W	207°45'	27° 45'
DE	209°15'	29°15'	- 10°W	199°15'	19° 15'
EA	318°45'	138°45'	- 10°W	308°45'	128°45'

SOURCES OF ERROR IN A COMPASS :-

The following are the kinds of error which may occur while taking reading with a compass.

(i) Instrumental Errors :-

(a) The needle may not be perfectly straight and might not be balanced properly.

(b) The pivot point may be eccentric

(c) The graduations of the ring may not be uniform.

(d) The ring may not rotate freely on account of the pivot point being blunt. This may occur due to the head of the pivot being broken because of careless handling.

(e) The sight vane may not be vertical.
(f) The horse hair may not be straight and vertical.

(2) Personal Errors

(a) The centring may not be done perfectly over the station.
(b) The graduated ring may not be levelled.
(c) The object might not be bisected properly.
(d) The readings may be taken or entered carelessly.
(e) The observer may be carrying magnetic substances.

(3) Other Sources of Error

a) There may be local attraction due to the presence of magnetic substances near the station.
(b) The magnetic field could vary on account of some natural causes.
(c) The magnetic declination might vary.

PRECAUTIONS TO BE TAKEN IN COMPASS SURVEYING

The following precautions should be taken while conducting a compass traverse:
i) The centring should be done perfectly.
ii) To stop the rotation of the graduated ring, the breake pin should be pressed very gently and not suddenly.

(5) Readings should be taken along the line of sight and not from any side.

(4) When the compass has to be shifted from one station to another, the sight vane should be folded over the glass cover. This is done to lift the ring out of the pivot to avoid unnecessary wear of the pivot head.

(5) The compass box should be tapped gently before taking the reading. This is don to find out whether the needle rotates freely.

(6) The stations should not be selected near magnetic substances.

(7) The observer should not ~~be~~ seete carry magnetic substances.

(8) The glass cover should not be dusted with a handkerchief, because the glass may be charged with electricity and the needle may be deflected from its true direction. The glass cover should be cleaned with a moist finger.

PLOTTING of COMPASS TRAVERSE :-

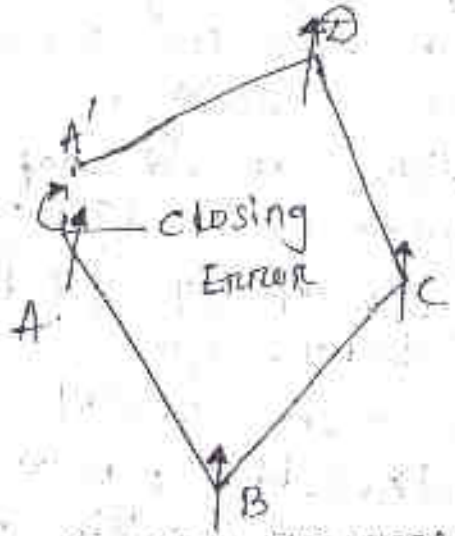
The following are the various methods of plotting compass traverse.

(1) BY parallel Meridian Through Each station

The starting point A is suitably selected on the paper and a line representing the north line. The bearing of the line AB is plotted by protactor and its length is plotted to any suitable scale.

At station B, the north line is drawn parallel to the north line which was drawn at A. Then the bearing of the line BC is plotted and its length marked according to the previous scale.

Similarly, all the traverse legs are plotted. In case of closed traverse, there may be a closing error which should be adjusted graphically.



(Plotting Compass Traverse by parallel Meaddian)

(2) BY considering Included Angles :-

The starting station A is suitably selected on the sheed. A line representing the north line is drawn through the station A. The bearing of the line AB is plotted by a protractor and the distance AB marked to a suitable scale. At the station B, the angle B is plotted and the distance BC marked according to the previous scale. Angle C is plotted at the station C

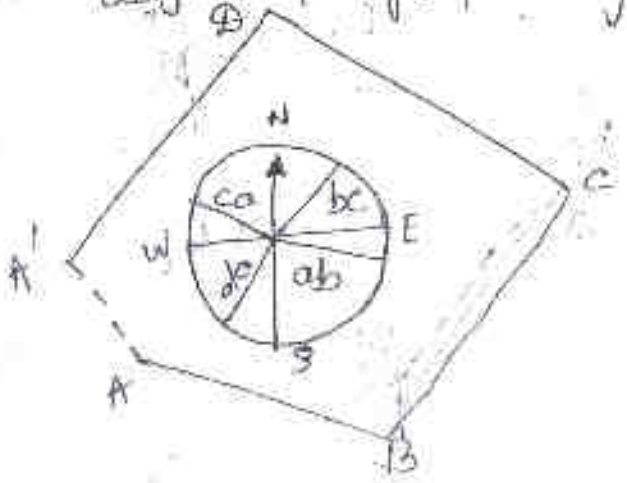
The process is continued until all the lines have been plotted In this case also, there may be a closing error which has to be adjusted graphically

(3) By considering the central Meridian :-

A suitable point O is selected at the centre of the drawing sheet. A line representing the magnetic meridian is drawn through this point. Then a protractor is placed at O and all the lines, namely ab, bc, cd and da, are drawn according to their bearings.

Then a starting point A is suitably selected on the sheet. A line AB is drawn parallel to ab, and the length AB is plotted to a suitable scale. Again from B a line Bc is drawn parallel to the line bc and the distance Bc is plotted to the previous scale.

The process is continued until all the lines have been drawn. In this case also there may be a closing error is adjusted graphically.



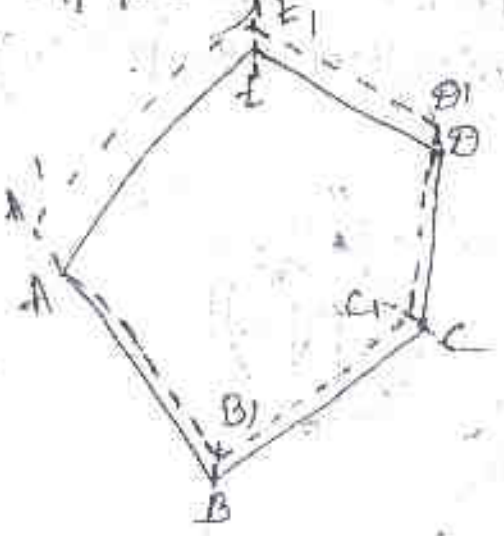
NOTE After adjustment of the closing error, the objects are plotted according to the offsets noted in the field book.

ADJUSTMENT OF CLOSING ERROR

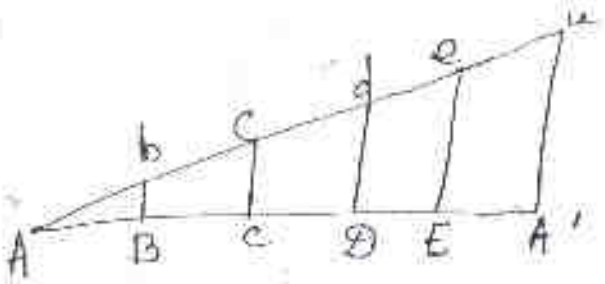
When a closed traverse is plotted the finishing and starting points may not coincide. The distance by which the traverse fails to close is said to be the closing error. Such an error may occure due to mistakes made in the measurement of lengths and bearings of the lines, or because of an error in plotting.

If the closing error exceeds a certain permissible limit, the field work should be repeated. But when the error is within the permissible limit, it is adjusted graphically by Bowditch's rule as explained below.

Suppose a traverse AB, C, D, E, A, is plotted according to any suitable scale (RF = 1/400).



DE - 1 (SAH)



$RF = \frac{1}{2000}$ (say)

Adjusting closing Error

In this case, the traverse fails to close by a distance AA', which is the closing error.

To adjust this error, a horizontal AA'. is drawn to represent the perimeter of the traverse to another scale (RF = 1/2,000) on this line, distance AB', B'C', C'D', D'E', and E'A', are set off according to the corresponding measured lengths of the traverse legs. A perpendicular A'a is drawn equal to the amount of closing error, after which the line Aa is drawn. From the points B', C', D', and E' the lines.

B'b, C'c, D'd and E'e are drawn parallel to A'a. These intercept represent the amount by which the respective stations are to be shifted.

Lines are drawn parallel to the closing error through stations B', C', D', and E'. Then the intercepts B'b, C'c, D'd and E'e are set off along the parallel lines drawn through the respective stations. In this manner, the adjusted traverse ABCDEA is obtained.

OBJECT AND USE OF LEVELLING

object

The aim of levelling is to determine the relative heights of different objects on or below the surface of the earth and to determine the undulation of the ground surface.

USES Levelling is done for the following purposes:

(1) To prepare a contour map for fixing sites for reservoirs, dams, barrages, etc. and to fix the alignment of roads, railway, navigation canals, and so on.

(2) To determine the altitudes of different important points on a hill or to know the reduced levels of different points on or below the surface of the earth.

(3) To prepare a longitudinal section and cross-sections of a project (roads, railways, navigation canals, etc) in order to determine the volume of earth work.

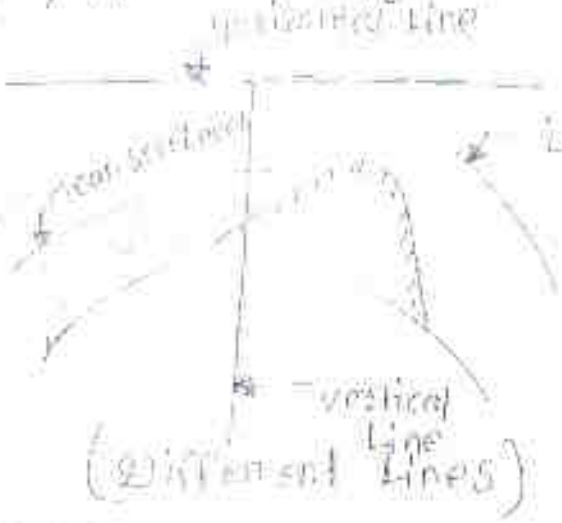
(4) To prepare a layout map for water supply, sanitary or drainage schemes.

DEFINITIONS

(1) Levelling — The art of determining the relative heights of different points on or below the surface of the earth is known as levelling. Thus, levelling deals with measurements in the vertical plane.

(2) Level surface — Any surface parallel to the mean spheroidal surface of the earth is said to be a level surface. Such a surface is obviously curved. The water surface of a still lake is often considered a level surface.

Level line :- Any line lying on a level surface is called a level line. This line is normal to the plumb line (direction of gravity) at all points.



(4) Horizontal plane :- Any plane tangential to the level surface at any point is known as the horizontal plane. It is perpendicular to the plumb line which indicates the direction of gravity.

(5) Horizontal line :- Any line lying on the horizontal plane is said to be a horizontal line. It is a straight line tangential to the level line.

(6) Vertical line :- The direction indicated by a plumb line (the direction of gravity) is known as the vertical line. This line is perpendicular to the horizontal line.

(7) Vertical plane :- Any plane passing through the vertical line is known as the vertical plane. This line is

(8) Datum surface or line :- This is an imaginary level line from which the vertical distance of different points (above or below this line) are measured. In India, the datum

adopted for the Great Trigonometrical Survey (GTS) is the Mean Sea Level (MSL) at Karachi.

(9) Reduced Level (RL) :- The vertical distance of a point above or below the datum line is known as the reduced level (RL) of that point. The RL of a point may be positive or negative according as the point is above or below the datum.

(10) Line of Collimation :- It is an imaginary line passing through the intersection of the cross-hairs of the diaphragm and the optical centre of the object glass and its continuation. It is also known as the line of sight.

(11) Axis of the Telescope :- This axis is an imaginary line passing through the optical centre of the object glass and the optical centre of the eyepiece.

(12) Axis of Bubble Tube :- It is an imaginary line tangential to the longitudinal curve of the bubble tube at its middle point.

(13) Bench - marks (BM) :- These are fixed points or marks of known RL determined with reference to the datum line. These are very important marks. They serve as reference points for finding the RL of new points as for carrying levelling operations in projects involving roads, railways etc.

Bench - marks may be of four types (a) GTS (b) permanent (c) temporary (d) arbitrary.

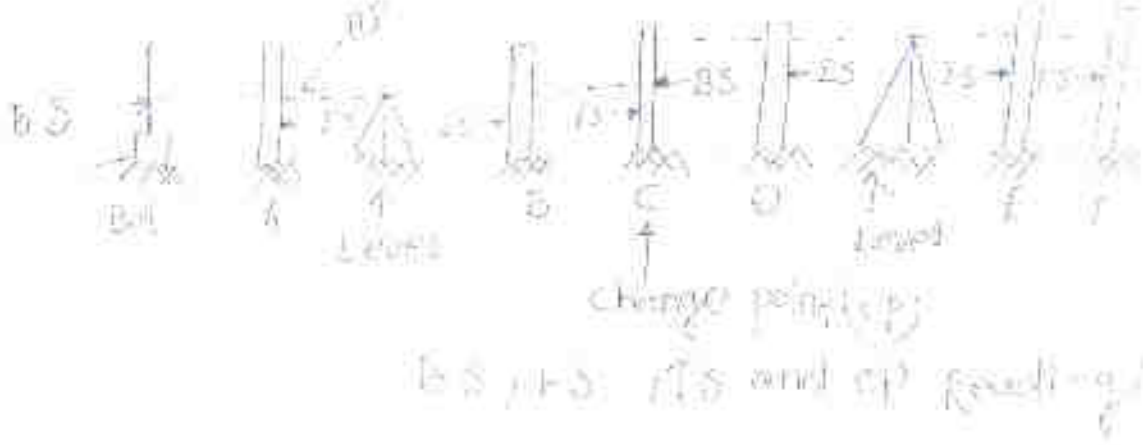
(a) G.T.S. bench-marks :- These bench-marks are established by the survey of India Department at large intervals all over the country. The values of reduced levels & the relevant positions and the number of bench-marks are given in a catalogue published by this department.

(b) Permanent Bench-marks :- These are fixed points or marks established by different government departments like PWD, Railway, Irrigation, etc. The RLs of these points are determined with reference to the G.T.S. bench-mark, and are kept on permanent points like the plinth of a building, parapet of a bridge or culvert, and so on. Sometimes they are kept on underground pillars.

(c) Temporary bench-marks :- when the bench-marks are established temporarily at the end of a day's work, they are said to be temporary bench-marks. They are generally made on the head of a pipe, the parapet of a nearby culvert, a prolong post, iron or a similar place.

(d) Arbitrary bench-marks :- when the RLs of some fixed points are assumed, they are termed arbitrary bench-marks. These are adopted in small survey operations when only the undulation of the ground surface is required to be determined.

Backsight Reading (BS) This is the first staff reading taken in any set-up of the instrument after the levelling has been perfectly done. This reading is always taken on a point of known RL i.e. on a bench-mark or change point.



Intermediate Reading (IS) It is the last staff reading in any set-up of the instrument, and indicates the shifting of the latter.

Intermediate Sight Reading (IS) - It is any other staff between the BS and FS in the same set-up of the instrument.

Change point (CP) - The point indicates the shifting of the instrument. At this point, one BS is taken from one setting and a FS from the next setting.

Height of Instrument :- When the levelling instrument is properly levelled, the RL of the line of collimation is known as height of instrument. This is obtained by adding the BS reading to the RL of the BM or TP on which the staff reading was taken.

Focussing - The operation of setting the eyepiece and the object glass at proper distance apart for clear vision of the object is known as focussing. This is done by turning the focussing screw clockwise or anticlockwise.

The function of the object glass is to bring the object into focus on the diaphragm and that of the eyepiece is to magnify the cross-hairs and objects.

Focussing is done in two steps as follows:

a) Focussing the eyepiece - A sheet of white paper is held in front of the telescope and the eyepiece is turned slowly clockwise or anticlockwise slowly until the cross-hairs appear distinct and clear.

b) Focussing the object glass - The telescope is directed to the object and the focussing screw is turned clockwise or

anticlockwise until the image is clear and sharp.

parallax The apparent of the image relative to the cross-hairs is known as parallax. This occurs due to imperfect focussing when the image does not fall in the plane of the diathragm.

The parallax is tested by moving the eye up and down. If the focussing is perfect the image appearance fixed to the cross-hairs. The parallax may be eliminated by properly focussing the telescope.

DIFFERENT TYPES OF LEVELS

i) Dumpy level :- The telescope of the dumpy level is rigidly fixed to its supports it cannot be removed from its supports nor can it be rotated about its longitudinal axis. The instrument is stable and retains its permanent adjustment for a long time. This instrument is commonly used.

ii) Wye Level (y level) :- The telescope is held in two 'y' supports it can be removed from the supports and reversed from one end to the other end of the telescope. The 'y' supports consist of two curved clips which may be raised. Thus the telescope can be rotated about its longitudinal axis.

iii) Cooke's Reversible Level :- This is a combination of the dumpy level and the y-level. It is supported by two rigid sockets.

The telescope can be rotated about its longitudinal axis without removing from the socket and replaced from one end of the telescope to the other end.

(iii) Cushing's Level :- The telescope cannot be removed from the sockets and rotated about its longitudinal axis. The eye-piece and object glass are removable and can be interchanged from one end of the telescope to the other end.

(iv) Modern Tilting Level :- The telescope can be tilted slightly about its horizontal axis with the help of a tilting screw. In this instrument the line of collimation is made horizontal for each observation by means of the tilting screw.

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

(vi) Description of Dumpy level :-

(a) Tripod stand :- The tripod stand consists of three legs which may be solid or framed. The legs are made of light and hard wood. The lower ends of the legs are fitted with steel shoes.

(b) Levelling head :- The levelling head consists of two parallel triangular plates having three grooves to support the foot screws.

(c) Foot screws :- Three foot screws are providing between the trivet and tribrach. By turning the foot screws the tribrach can be raised or lowered to bring the bubbles to the centre of its run.

(d) Telescope :- The telescope consists of two metal tubes, one moving within the other. It also consists of an objective and an eyepiece at opposite ends. A diaphragm is fixed with the telescope just in front of the eyepiece. The diaphragm consists cross-hairs. The telescope is focussed by means of the focussing screw and may have either external focussing or internal focussing.

In the external focussing telescope, the diaphragm is fixed to the outer tube and the objective to the inner tube. By turning the focussing screw, the distance between the objective and diaphragm is altered to form a real image on the plane of cross-hairs.

In the internal focussing telescope the objective and eyepiece do not move when the focussing screw is turned. Here

Levelling staff :- The levelling staff is a graduated wooden rod used for measuring the vertical distance betn the points on the ground and the line of collimation. Levelling staves are classified into two groups :(i) the target staff and (ii) The self reading staff.

(i) Target staff :- The target staff consists of a movable target. The target is provided with a vernier which is adjusted by the staffman, according to directions from the leveller, so that the target coincides with the collimation hair. After this, the reading is taken by either the staffman or the levelman. This staff is used for long sightings.

(2) Self reading staff

The following are the different types of self-reading staffs:

(i) Sop with telescopic staff :- Such a staff is arranged in three lengths placed one into the other. It can be extended to its full length by pulling. The top portion is solid and of 1.25 m length, the central box portion is hollow and of 1.25 m length, and the bottom box portion is hollow and 1.5 m long. The total length of the staff is 4m. The top portions are held in rest on the left and those m-decimeters are marked in black, vertical position by brass spring catches.

The staff is graduated in such a way that the smallest division is of 5 mm (0.005m). The values in meters are marked in red on the left and those in decimeters are marked in black on the right.

(b) Folding metric staff :- This staff is made of well-seasoned timber and is of 16 mm width, 18 mm thickness, and 4m length. It is divided into two parts of 2m length having a locking arrangement. It can be folded as declared when required. It is graduated like the telescopic staff.

(c) One length staff :- The one-length staff is solid and made of seasoned timber. It is 3m long and graduated in the same way as the telescopic staff.

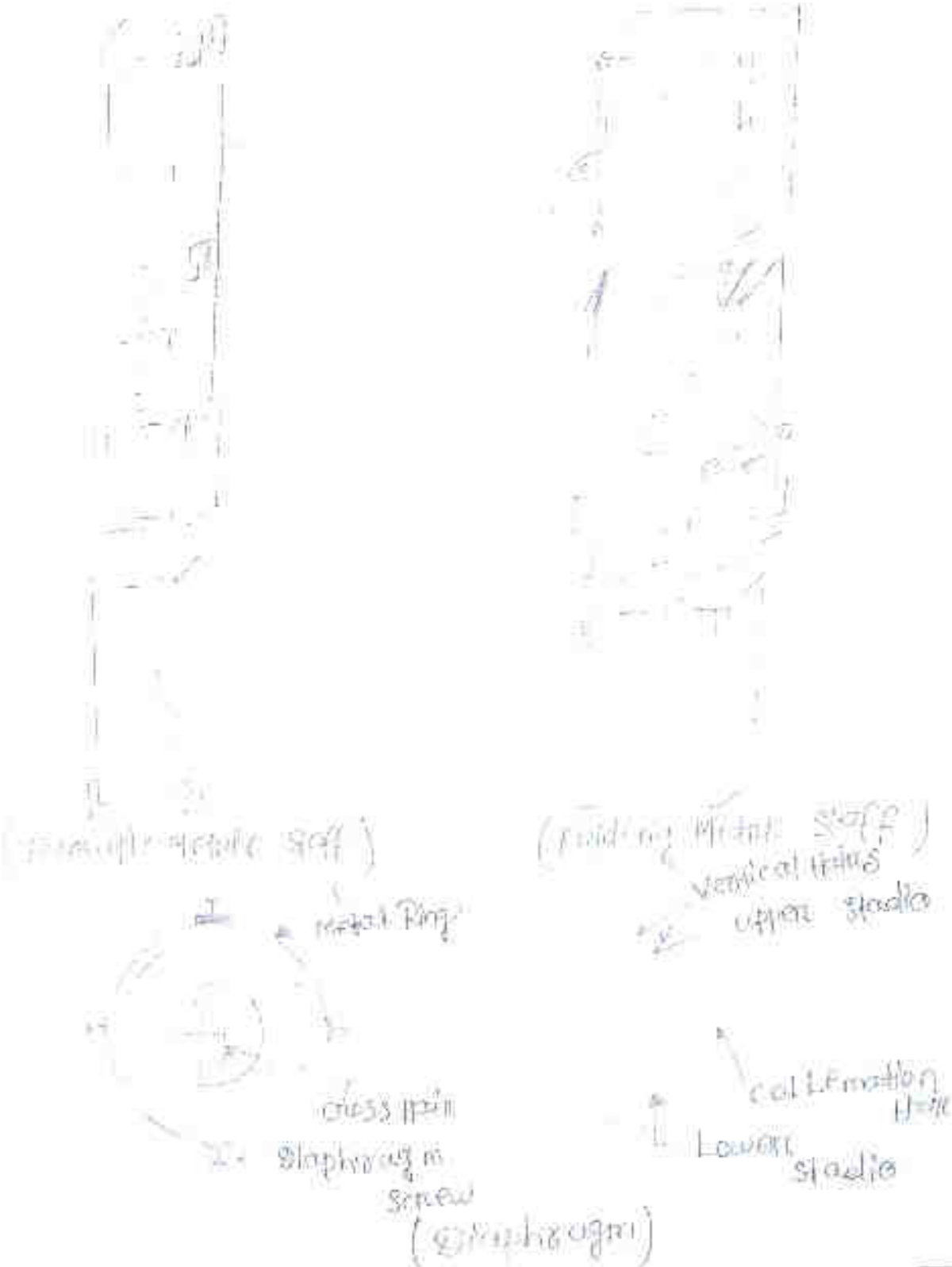
(d) Invar staff :- The invar staff is also 3m long. An invar band is fitted to a wooden staff. The band is graduated in millimetres. It is used for precise levelling work.

(e) Diaphragm :- The diaphragm is a brass ring fitted inside the telescope, just in front of the eyepiece. It can be adjusted by four screws. The ring carries the cross-hairs, which get magnified when viewed through the eyepiece. The cross-hairs may be marked in the following ways :

① With spider webs stretched across the ring.

② By very fine scratch marks in a glass fitted with the ring.

③ By means of platinum wires or silk threads stretched across the ring.



The cross-hair consists of the following lines:-

(1) Two vertical hairs meant for maintaining the verticality of the staff.

(2) Middle horizontal hair representing the line of collimation and

(3) upper stadia hair and lower stadia hair; both horizontal and short in length. The stadia hairs are meant for determining the horizontal distance between the position of level and that of the staff.

TEMPORARY ADJUSTMENT OF LEVEL :-

The adjustments made at every set-up of the level before the staff readings are taken are known as temporary adjustments. The following are the different steps to be followed in temporary adjustment.

(1) selection of suitable position :-

A suitable position is selected for setting the level. From this position, it should be possible to take the greatest number of observations without any difficulty. The ground should be fairly level and firm.

(2) Fixing level with tripod stand :-

The tripod stand is placed at the required position with its legs well apart and pressed firmly into the ground.

The level is fixed on the top of the tripod stand according to the fixing arrangement provided for that particular level. It should be remembered that the level is not to be set up at any station on a point along the alignment.

(a) Approximate Levelling using the tripod stand

The foot screws are brought to the centre of their run. Two legs of the tripod stand are firmly fixed into the ground. Then the third leg is moved to the left or right in or out until the bubble is approximately at the centre of its run.

(b) Perfect Levelling by foot screws:-

As the longitudinal bubble is on the top of the telescope, the latter is placed parallel to any pair of foot screws and the bubble is brought to the centre by turning the foot screws equally either both inwards or both outwards. The telescope is then turned through 90° and brought over the third foot screw. The bubble is brought to the centre by turning this foot screw clockwise or anticlockwise. The telescope is again brought to its original position (the first position) and the bubble is brought to the centre. The process is repeated several times until the bubble remains in the central position in the first as well as the second position when the telescope is turned through 180°.

third foot screw

first position of telescope

second foot screw

Second position of telescope

(Levelling of foot screws)

180°. If the bubble still remains in the central position, the temporary adjustment is perfect and so is the permanent adjustment. But if the bubble is deflected from its central position, the permanent adjustment is not perfect and needs to be modified.

(5) Focussing the Eyepiece :- A piece of white paper is held in front of the object glass and the eyepiece is moved in or out by turning it clockwise or anticlockwise until the cross-hairs can be seen clearly.

(6) Focussing the object glass :-

The telescope is directed towards the leveling staff. Looking through the eyepiece, the focussing screw is turned clockwise or anticlockwise until the graduation of the staff is distinctly visible and the parallax is eliminated. To eliminate the parallax, the eye is moved up and down to verify whether the graduation of the staff remains fixed relative to the cross-hairs.

taking the staff Reading. Finally, the levelling of the instrument is verified by turning the telescope in any direction when the bubbles (the longitudinal bubble and cross-bubble) remain in the central position for any direction of the telescope the staff readings are taken.

16. The following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15m. The first reading was taken at a chainage of 165 m when the RL is 93.085. The instrument was shifted after the fourth & ninth readings 3.150, 2.245, 1.125, 0.860, 3.125, 2.160, 1.835, 1.470, 0.955, 1.225, 0.390, 3.035 m. Find the RL of different points?

Shell Wall Rod

Station A :-

HI = RL of BM + B.S = 98.085 + 3.150 = 101.235

station point	chainage (m)	BS	IS	FS	HI	RL	Remarks
1	165	3.150			101.235	98.085	B.M
2	180		2.845			98.390	
3	195		1.185			100.050	C.P
4	210	3.185		0.860	103.560	100.375	C.P
5	225		2.760			100.800	
6	240		1.855			101.705	
7	255		1.470			102.090	
8	270	1.229		1.760	103.029	101.800	C.P
9	285		2.395			100.634	
10	300			3.055		99.974	
		7.564		5.675			

Station B :- RL of C.P at BS = 100.375 + 3.185

= 103.560

Station C :- RL = 101.800 + 1.760

= 103.560

Arithmetical check

$\Sigma BS - \Sigma FS$ = Last RL - 1st RL

$7.564 - 5.675$ = $99.974 - 98.085$

1.889 = 1.889

∴ Hence it is ok.

(8) The following consecutive readings were taken with a leveling instrument at interval of 20 m:
0.325, 1.730, 2.615, 3.150, 3.935, 2.670, 1.835, 0.790, 0.135, 1.630, 2.295 & 3 (some) The instrument was shifted after the fourth & eight reading. The last reading was taken on a staff held on a point. Find the RL of all the points.

Station point	Chainage	BS	IS	FS	HI	RL	Remark
					112.575	110.200	B.M
1	0	2.375					
						110.825	
2	20		1.750				
						111.960	
3	40		0.615				
				3.450	111.960	109.125	C.P
4	60	2.835					
						109.890	
5	80		2.070				
						110.125	
6	100		1.835				
				0.985	111.410	110.975	
7	120	0.435					
						109.780	
8	140		1.630				
						109.155	
9	160		2.255				
				3.630		107.780	
10	180						
				8.065			

Total : 5.645

Arithmetical check :-

$\Sigma BS - \Sigma FS$ = Last RL - 1st RL

$\Rightarrow 5.645 - 8.065$ = 107.780 - 110.200

$\Rightarrow -2.420$ = -2.420

∴ Hence it is ok.

RL of 2nd IS = HI - FS of 2nd IS = RL of different [illegible]

TYPES OF LEVELLING OPERATIONS

(1) Simple levelling :-

When the difference of level between two points is determined by setting the levelling instrument midway between the points the process is called simple levelling.

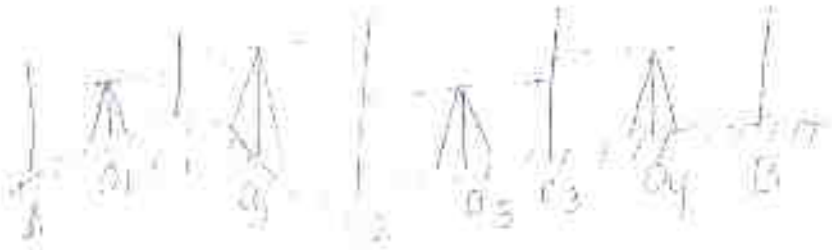
Suppose A and B are two points whose difference of level is to be determined. The level is set up at O, exactly midway between A and B. After proper temporary adjustment, the staff readings on A and B are taken. The difference of these readings gives the difference of level between A and B.

(Simple levelling)

(2) Differential Levelling :- Differential levelling is adopted when (i) the points are a great distance apart, (ii) the difference of elevation between the points is large, & (iii) there are obstacles between the points. This method is also known as compound levelling or continuous levelling. In this method, the levels is

set up at several suitable positions and staff readings are taken at all of these.

Suppose it is required to know the difference of level between A and B. The level is set up at points O_1, O_2, O_3 etc. After necessary adjustments, staff readings are taken at every set-up. The points C_1, C_2 and C_3 are known as change points. Then the difference of level between A and B is found out. If the difference is positive, A is lower than B. If it is negative, A is higher than B. Knowing the RL of A, that of B can be calculated.



(Differential Leveling)

(2) Fly leveling :- When differential leveling is done in order to connect a bench mark to the starting point of the alignment of any project, it is called fly leveling. Fly leveling is also done to connect the BM for any intermediate point of the alignment for checking the accuracy of the work. In such leveling only the backsight and foresight readings are taken at every set up of the level and no

distance are measured along the direction of levelling. The level should be set up just midway between the BS and the FS.

(Fly Levelling)

(3) Longitudinal or profile Levelling:-

The operation of taking levels along the centre line of any alignment (road, railway, etc.) at regular intervals is known as longitudinal operation, the backsight, intermediate sight and foresight readings are taken at regular intervals, at every set up of the instrument. The changes of the points are noted in the level book. This operation is undertaken in order to determine the undulations of the ground surface along the profile line.

(Profile Levelling)

5) Cross - sectional Levelling :-

The operation of taking levels transverse to the direction of longitudinal levelling, is known as cross-sectional levelling. The cross-sections are taken at regular intervals (such as 20m, 40m, 50m etc) along the alignment. Cross-sectional levelling is done in order to known the nature of the ground across the centre line of any alignment.

6) Check Levelling :- The fly levelling done at the end of a day's work to connect the finishing point with the starting point on that particular day is known as check levelling. It is undertaken in order to check the accuracy of the day's work.

Q the following consecutive readings were taken with a levelling instrument at intervals of 20m:

2.375, 1.735, 0.615, 3.450, 2.835, 0.970, 1.835, 0.985, 0.135, 1.630, 2.255 and 3.630m.

The instrument was shifted after the fourth and eighth readings. The last reading was taken on a peg of RL 112.620m. Find the RLs of all the points.

Station point	Chainage	BS	IS	FS	HI	RL	Remark
1	0	2.375			114.995	112.620	
2	20		1.735			113.260	
3	40		0.615			114.380	
4	60	2.835		3.450	114.380	111.545	C.P
5	80		0.970			113.410	
6	100		1.835			112.545	
7	120	0.435		0.985	113.830	113.395	C.P
8	140		1.630			112.200	
9	160		2.255			111.575	
10	180			3.630		110.200	B.M
		5.645		8.065			

Page =

(ii) HI = RL of BS − 110.300 + 3.630 = 113.930

RL of change point = HI − FS = 113.930 − 0.435

= 113.495

(iii) HI = RL of CP + FS = 113.495 + 0.950 = 114.380

RL of CP = HI − FS = 114.380 − 2.835

= 111.545

(iv) HI = RL of P + FS = 111.545 + 3.450 = 114.995

RL of P = HI − FS = 114.995 − 2.375

= 112.620

Arithmetical check

ΣBC − ΣFS = last RL − 1st RL

⇒ 5.645 − 8.065 = 110.300 − 112.620

⇒ −2.420 = −2.420

∴ Hence it is ok.

23 July 2021 0.5.M

Q2. The following consecutive readings were taken with a dumpy level along a chainline at common interval of 20m the first reading was taken on a chainage of 140m. The RL of the second change point was 107.915 m. The instrument was shifted after third & seventh readings calculate the RL of all the points.

3.150, 2.945, 1.125, 3.860, 2.125, 0.760, 2.935, 0.410, 1.935, 3.925, 3.890 m.

Sol

station point	chainage	BS	IS	FS	HI	RL	Remark
					106.115	103.565	
1	140	3.150				104.470	
2	160			2.945		105.590	C.P
3	180	3.860		1.125	109.450	107.325	
4	200			2.125		108.690	
5	220			0.760		107.215	C.P
6	240	0.410			107.625	105.750	
7	260			1.935		104.460	
8	280			3.925		103.175	
9	300						
		7.48		7.25			

Arithmatic check

ΣBS - ΣFS = Last RL - 1st RL

⇒ 9.48 - 7.95 = 103.795 - 103.563

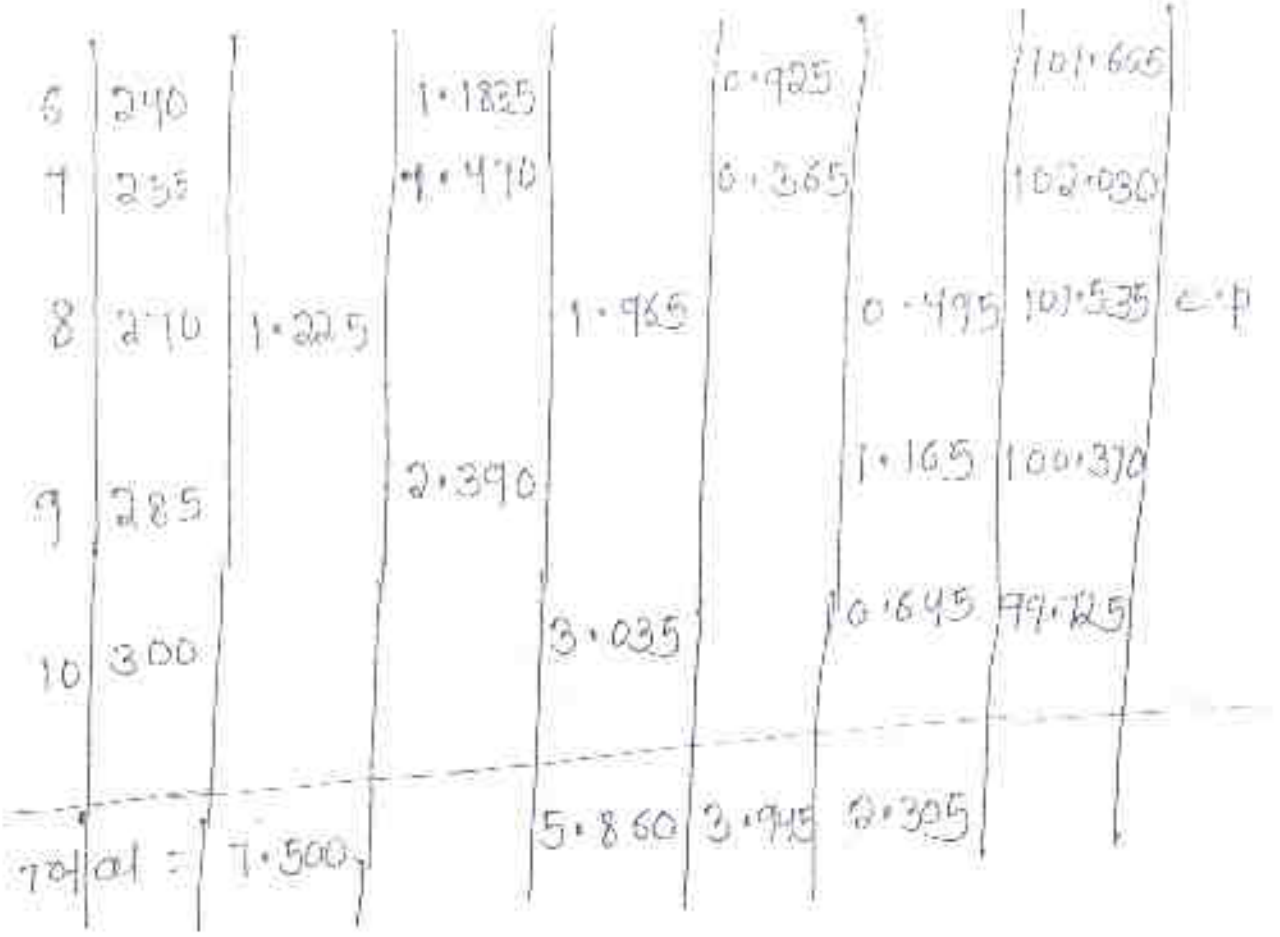
⇒ 0.23 = 0.23

∴ Hence it is ok.

6 July 2021 RISE AND FALL METHOD

16. The following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15 mt. The first reading was 1.69 mt. when the instrument was shifted after the fourth & ninth readings. RL is 98.089. The readings are 3.150, 2.245, 1.125, 0.860, 3.125, 0.760, 1.835, 1.470, 1.985, 1.225, 2.390, 3.025 m. Find the RL of different points?

Station point	Chainage	BS	IFS	FS	Rise (+ve)	Fall (-ve)	RL	Remark
							98.085	B.M
1	165	3.150			0.805		98.890	
2	180		2.245		0.905		100.110	
3	195		1.125		1.120		100.375	
4	210	3.125		0.860	0.265		100.375	C.P
5	225		0.760		0.365		100.740	



Arithmetic check :-

$\Sigma BS - \Sigma FS$ = Last RL − 1st RL : $\Sigma Rise - \Sigma Fall$

$\Rightarrow 7.500 - 5.860 = 99.725 - 98.085 = 3.945 - 2.305$

$\Rightarrow 1.640 = 1.640 = 1.640$

∴ Hence it is OK.

25 The following consecutive Readings were taken with a levelling instrument at interval of 20m :

2·955, 1·130, 0·615, 3·150, 2·835, 2·520, 1·855, 0·989, 2·435, 1·630, 2·455 & 3·630 m. The instrument was shifted after the fourth & eighth readings. The 1st reading was taken

on a S.M of R.L 110.200 and find the R.L of all the points.

Station points	Chainage	B.S	I.S	F.S	Rise (+ve)	Fall (-ve)	R.L	Remark
							110.200	B.M
1	0	2.375						
2	20		1.730		0.645		110.845	
3	40		0.615		1.115		111.960	
4	60	2.835		3.450		2.835	109.125	C.P
5	80		2.070		0.765		109.890	
6	100		1.835		0.235		110.125	
7	120	0.435		0.985	0.850		110.975	C.P
8	140		1.630			1.195	109.780	
9	160		2.283			0.625	109.155	
10	180			3.630		1.375	107.780	
		5.645		8.065	3.610	6.030		

Arithmatic check :-

$\Sigma BS - \Sigma FS$ = last RL − 1st RL = $\Sigma Rise - \Sigma Fall$

$\Rightarrow 5.645 - 8.065 = 107.780 - 110.200 = 3.610 - 6.030$

$\Rightarrow -2.420 = -2.420$

$\Rightarrow$

$\therefore$ Hence it is OK.

5. The following consecutive readings were taken with a levelling instrument at intervals of 20 m:
2.375, 1.730, 0.615, 3.450, 0.835, 2.070, 1.835, 0.985, 1.630, 2.255 and 0.630 m.
The instrument was shifted after the fourth and eighth readings. The last reading was taken on the staff held on a BM of RL 110.200 m. Find the RL of all the points.

Station point	Chainage	BS	IS	FS	Rise (+)	Fall (−)	RL	Remarks
1	0	2.375					112.620	
					0.645			
2	20	1.730	1.730				113.265	
					1.115			
3	40	0.615	0.615			2.835	111.380	
								C.P
4	60	2.835		3.450	0.765		111.515	
5	80		2.070		0.235		112.310	
6	100		1.835		0.850		112.545	
7	120	0.985		0.985		1.195	113.395	C.P
8	140		1.630			0.625	113.020	
9	160		2.255			1.315	115.515	
10	180			0.630			114.200	B.M
					3.610	6.030		

8.065

Check: 5.615

Arithmetic check

$\Sigma BS - \Sigma FS$ = Last RL - 1st RL = $\Sigma Rise - \Sigma fall$

$\Rightarrow 5.645 - 8.065 = 110.200 - 112.620 = 3.610 - 6.030$

$\therefore -2.420 = -2.420 = -2.420$

$\therefore$ Hence it is ok.

46 The following consecutive readings were taken with a dumpy level along a chainage at common interval of 20m the reading was taken on a chainage of 140m. The RL of the second change point was 107.215m. The instrument was shifted after third & seventh readings calculate the R.L of all points.

3.150, 0.245, 1.125, 3.865, 2.125, 0.760, 2.235, 0.470, 1.935, 0.225, 3.890mt.

Station point	Chainage	BS	IS	FS	Rise	Fall	RI	Remark
							103.555	
1	140	3.150			0.905		104.460	
2	160		2.245		1.100		105.560	C.P
3	180	3.865		1.125	1.735		107.295	
4	200		2.125		1.365		108.660	
5	220		0.760			1.375	107.285	C.P
6	240	0.470		2.235				
						1.965	105.320	
7	260		1.935					

R	2.80		3.235		1.290	104.460
P	3.00			3.890	0.665	103.795
Total		7.480		7.250	5.125	4.895	104.460

Arithmetic check

$\Sigma BS - \Sigma FS = \Sigma R - \Sigma F$ = Last RL - 1st RL

$7.480 - 7.250 = 5.125 - 4.895 = 104.460 - 104.230$

$= 0.23 = 0.23 = 0.23$

∴ Hence it is ok.

27 July 2021

Q The following are the consecutive readings were taken with a level & a staff on a continuously sloping ground at a common interval of 30m.
0.835 (on A), 1.545, 2.335, 3.115, 3.525, 0.455, 1.380, 2.055, 2.855, 2.455 (on B). The RL of A was 350.500. Find the RL of different point & apply usual check & determine the gradient of AB.

Station point	Chainage	BS	IS	FS	Rise (m)	Fall (m)	RL	Remark
							380.500	BM
1	0	0.1855				0.690	379.810	
2	30		1.545			0.190	379.620	
3	60		2.335			0.780	378.840	
4	90		3.115			0.110	378.730	C.P
5	120	0.455		3.825		0.905	377.825	
6	150		1.380			0.615	377.210	
7	180		2.053			0.800	376.410	
8	210		2.855			0.600	375.810	C.P
9	240	0.585		3.459		0.430	375.380	
10	270		1.015			0.835	374.545	
11	300		1.850			0.905	373.640	
12	330		2.755			0.096	373.544	
13	360			3.845			371.370	
	Total	1.895		11.125		9.23		

Arithmetic check:-

$\Sigma BS - \Sigma FS$ = Last RL - First RL = Σ Rise - Σ Fall

$\Rightarrow 1.895 - 11.125 = 371.370 - 380.500 = -9.230$

$\Rightarrow -9.230 = -9.230$

Gradient :

$$\text{Gradient} = \frac{\text{difference of level}}{\text{Horizontal distance}}$$

$$= \frac{380.500 - 371.270}{360}$$

$$= \frac{9.23}{360} = \frac{1}{\frac{360}{9.23}} = \frac{1}{39} \; (1 \text{ in } 39)$$

Methods of calculation of Reduced level

The following are the two systems of calculating reduced level :

(i) The collimation system or height of instrument system (HI)

(ii) The rise-and-fall system

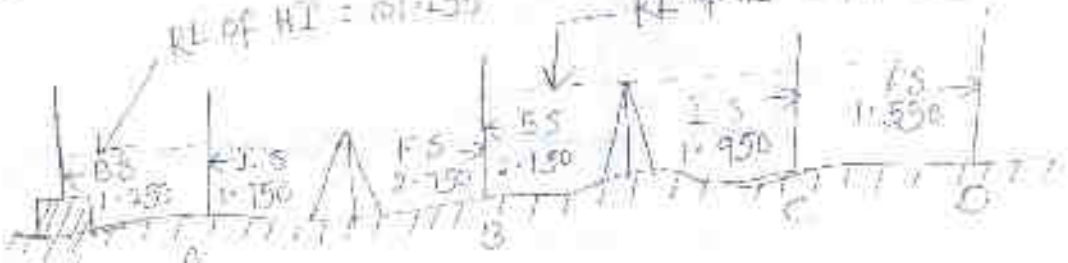
(i) The collimation system

The reduced level of the line of collimation is said to be the height of the instrument. In this system, the height of the line of collimation is found out by adding the backsight reading to the RL of the BM on which the BS is taken. Then the RL of the intermediate points and the change point are obtained by subtracting the respective staff readings from the height of the instrument (HI).

The level is then shifted for the next setup and again the height of the line of collimation is obtained by adding the backsight reading to the RL of the change point (which was calculated in the first set-up)

So, the height of the instrument is different in different setup of the level. Two adjacent planes of collimation are correlated at the change point by an FS reading from one setting and a BS reading from the next setting.

It should be remembered that in this system the RLs of unknown points are to be found out by deducting the staff readings from the RL of the height of the instrument.



(Collimation system)

(a) RL of HI in first setting = 100·000 + 1·255 = 101·255

RL of A = 101·255 − 1·750 = 99·505

RL of B = 101·255 − 2·150 = 99·105

(b) RL of HI in second setting = 99·105 + 2·750
= 101·855

RL of C = 101·855 − 1·950 = 99·905

RL of D = 101·855 − 1·550 = 100·305 and so on.

Arithmetical check ΣBS − ΣFS = last RL − 1st RL

The difference between the sum of backsights and that of foresights must be the difference between the last RL

and the first RL. This check verifies the calculation of the RL of the HI and that of the change point. There is no check on the RLs of the intermediate points.

(2) Rise-and-fall system

In this system, the difference of level between two consecutive points is determined by comparing each forward staff reading with the staff reading of the immediately preceding points.

If the forward staff reading is smaller than the immediately preceding staff reading, a rise is said to have occurred. The rise is added to the RL of the preceding point to get the RL of the forward points.

If the forward staff reading is greater than the immediately preceding staff reading. It means there has been a fall. The fall is subtracted from the RL of preceding point to get the RL of the forward points

(Rise and fall system)

point A (with respect to BM) = 0.75 - 1.25 = -0.50 fall

point B (with respect to A) = 1.25 - 2.75 = -1.50 fall

point C (with respect to B) = 2.75 - 1.50 = +1.25 rise

point Q (with respect to C) = 1.50 - 1.75 = -0.25 (fall)

RL of BM = 100.00

RL of A = 100.00 - 0.50 = 99.50

RL of B = 99.50 - 1.50 = 98.00

RL of C = 98.00 + 1.25 = 99.25

RL of Q = 99.25 - 0.25 = 99.00

Arithmetical check : ΣBS - ΣIS = Σrise - Σfall = last RL - 1st RL

In this method, the difference between the sum of BSs and that of FSs) the difference between the sum of rises and that of falls and the difference between the last RL and the first RL must be equal.

Comparison of the two systems :-

Collimation system

(i) It is rapid as it involves few calculations.

(ii) There is no check on the RL of intermediate points.

(iii) Errors in intermediate RLs cannot be detected

(iv) There are two checks on the accuracy of RL calculation.

(v) This system is suitable for longitudinal levelling where there are a number of intermediate sights.

Rise and fall system

(i) It is laborious, involving several calculations.

(ii) There is a check on the RL of intermediate points.

(iii) Errors in intermediate RLs can be detected as all the points are connected.

(iv) There are three checks on the accuracy of RL calculation.

(v) This system is suitable for fly levelling where one no intermediate these sights.

PRINCIPLE OF EQUALISING BACKSIGHT AND FORESIGHT DISTANCES.

In levelling the line of collimation should be horizontal when the staff readings are taken. Again the fundamental rectation is that the line of collimation should be exactly parallel to the axis of the bubble, so when the bubble is at the centre of its run, the line of collimation is just horizontal. But sometimes the permanent adjustment of level may be disturbed and the line of collimation may not be parallel to the axis of the bubble. In such a case, due to the inclination of the line of collimation, error in levelling are likely to occur. But it is found that if the back sight and foresight distances are kept equal then the error due to the inclination of the collimation line is automatically eliminated, as illustrated below.

Case I — When the line of collimation is inclined upwards

Let A and B be to points whose true difference of level is required. The level is set up at a exactly midway between A and B.

Case II — When the line of collimation is inclined downwards.

Let α - angle of inclination of collimation line.

Aa : true @ reading

Aa_1 : observed staff reading on A

$\therefore$ Error $= Aa_1 - Aa = aa_1 = D\tan\alpha$

So true reading $Aa = Aa_1 - aa_1 = Aa_1 - D\tan\alpha$

Similarly Bb : true reading

Bb_1 : observed staff reading on B

$\therefore$ Error : $Bb_1 - Bb = bb_1 = D\tan\alpha$

So true reading $Bb = Bb_1 - bb_1 = Bb_1 - D\tan\alpha$

true difference of level between A and B

$= Aa - Bb$ (fall from B to A)

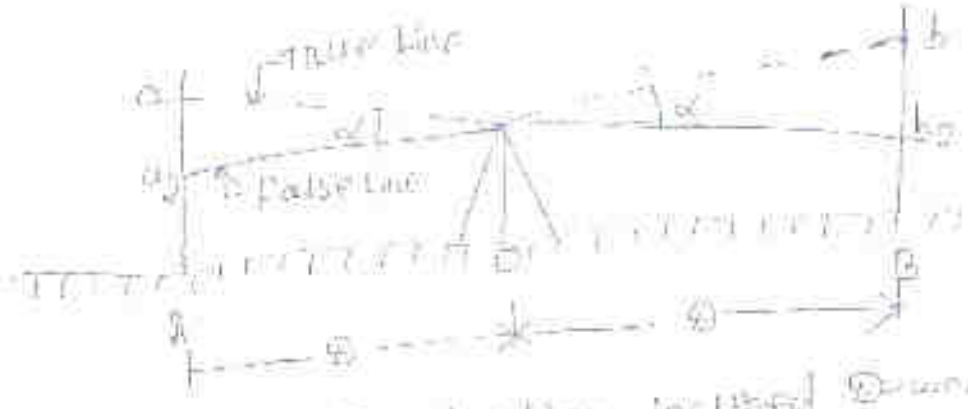
$= Aa_1 - D\tan\alpha - Bb_1 + D\tan\alpha$

$= Aa_1 - Bb_1$

Thus : It is seen that the error due to inclination of the collimation line is completely eliminated and the apparent difference is equal to the true difference.

Case - II -- when the line of collimation is inclined downwards

The staff readings on A and B are taken after setting the level at a. Suppose the readings are a_2 and b_2.



Line of collimation inclined downwards

Here, Aa = true staff reading

Aa_2 = observed staff reading on A

$\therefore$ Error = $Aa - Aa_2 = aa_2$ = α tan α

So true reading $Aa = Aa_2 + aa_2 = Aa_2 + \alpha$ tan α

Similarly, Bb = true reading

Bb_2 = observed staff reading on B

$\therefore$ Error = $Bb - Bb_2 = bb_2$ = α tan α

So true reading $Bb = Bb_2 + bb_2 = Bb_2 + \alpha$ tan α

true difference of level between A and B

= $Aa - bb$ (last from B to A)

= $Aa_2 + \alpha$ tan $\alpha - Bb_2 - \alpha$ tan α

= $Aa_2 - Bb_2$

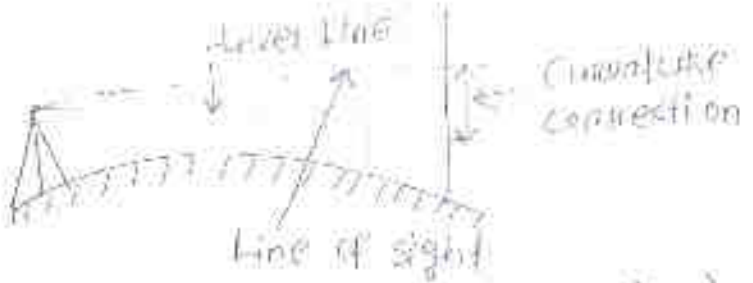
Thus, it is seen that the error due to inclination of the collimation line is completely eliminated.

NOTE :-

So, always remember that the level should be placed exactly midway between back sight and foresight in order to eliminate only error

CORRECTIONS TO BE APPLIED :-

1) Curvature correction :- For long sights the curvature of the earth affects staff readings. The line of sight is horizontal but the level line is curved and parallel to the mean of the earth.



(curvature correction)

The vertical distance between the line of sight and the level line at a particular place is called the correction. Due to curvature, objects appear lower than they really are. Curvature correction is always subtractive (i.e negative)

The formula for curvature correction is derived as follows:

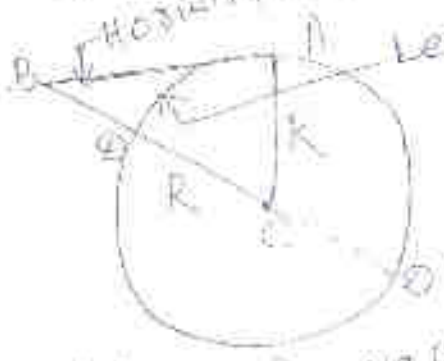
Let AB = D : horizontal distance in kilometres

BD = Cc = curvature correction

DC = AC = R = radius of earth

DD' = diameter, considered 12,742 km.

From right-angled triangle ABC



Deriving formula for curvature correction

$$BC^2 = AC^2 + AB^2$$

$$(R + Cc)^2 = R^2 + D^2$$

$$R^2 + 2RCc + Cc^2 = R^2 + D^2$$

$$\text{or} \quad Cc \times 2R = D^2$$

Curvature correction

(Ci^2 is neglected as it is very small in comparison to the diameter of the earth).

$$C_c = \frac{S^2 \times 1,000}{12,740} = 0.0785 S^2 \text{ m (negative)}$$

Hence, true staff reading = observed staff reading − curvature correction.

(ii) Refraction correction

Rays of light are refracted when they pass through layers of air varying density. So, when long sights are taken, the line of sight is refracted towards the surface of the earth in a curved path. The radius of this curve is seven times that of the earth under normal atmospheric conditions. Due to the effect of refraction, objects appear higher than they really are. But the effect of curvature varies with atmospheric conditions.

However, on an average, the refraction correction is taken as one-seventh of the curvature correction.

$$C_r = \frac{1}{7} \times \frac{S^2}{2k}$$

Refraction correction $= \frac{1}{7} \times 0.0785 S^2 = 0.0112 S^2$ m (positive)

Refraction correction is always additive

true staff reading = observed staff reading + refraction correction

3) Combined correction :-

The combined effect of curvature and refraction is as follows :

Combined correction = curvature correction + Refraction correction

$= -0.0785 D^2 + 0.0112 D^2$

$= -0.0673\ D^2$ m

So, combined correction is always subtractive (i.e negative) -ve

True staff reading = observed staff reading - combined correction

NOTE combined correction may also be expressed as

$\frac{D^2}{2R} - \frac{1}{7} \times \frac{D^2}{2R} = \frac{6}{7} \cdot \frac{D^2}{2R}$ (negative)

4) visible horizon distance :-

Let AB = D = visible horizon distance in kilometres.

h = height of the point above mean sea level in metres.

visible horizon distance

Considering curvature and refraction correction.

$h = 0.0673\ D^2$

$D = \sqrt{\frac{h}{0.0673}}$

(5) Dip of Horizon :-

AB = E = tangent to the earth at A

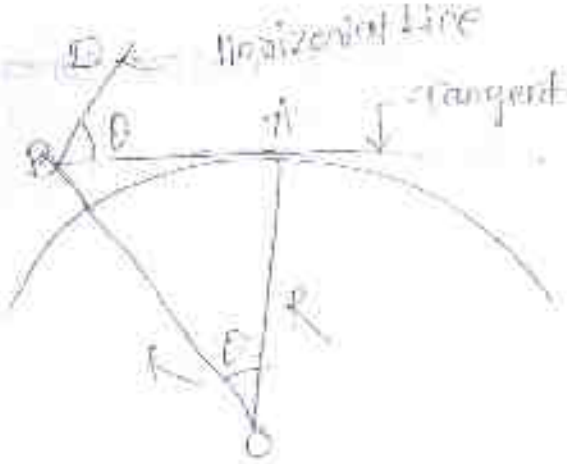
BD = horizontal line perpendicular to OB

θ = dip of horizon

The angle both the horizontal line and the tangent line is known as the dip of the horizon. It is equal to the angle subtended by the arc CA at the centre of the earth.

$$\text{Dip } \theta = \frac{\text{arc CA}}{\text{radius of the earth}} \text{ in radians}$$

$$\theta = \frac{D}{R} \text{ in radians (taking CA approx. equal to AB)}$$



(Dip of Horizon)

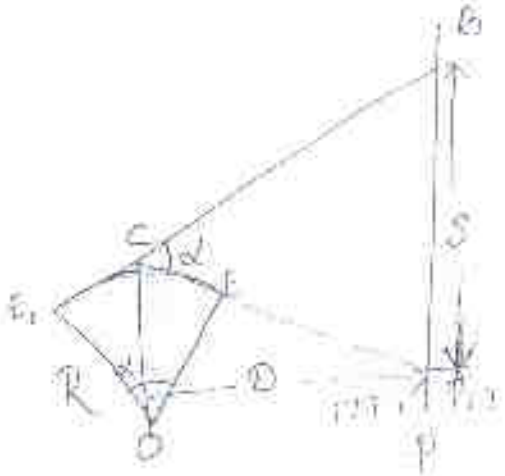
Here D and R must be expressed in the same units.

(6) Sensitiveness of the Bubble :-

This term sensitiveness in the context of a bubble means the effect caused by the deviation of the bubble per division of the graduation of the bubble tube.

Sensitiveness is expressed in terms of the radius of curvature of the upper surface of the tube or by an angle through which the axis is tilted for the deflection of one division of the graduation.

Determining sensitiveness:- Suppose the level was set up at O at a distance D from the staff at P. The staff reading is taken with the bubble at the extreme left end. Let it be PB.



Determining sensitiveness of the bubble

Let D = distance between the level and staff.

S = intercept between the upper and lower sights.

n = number of division through which the bubble is deflected.

R = radius of curvature of the tube.

α = angle subtended by arc EE1 and

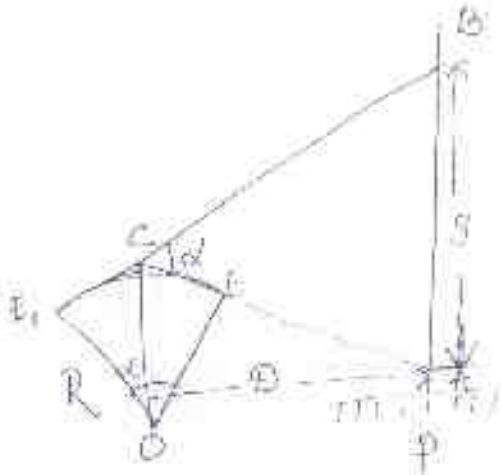
d = length of one division of the graduation.

expressed in the same units as D and S

Movement of centre of bubble = EE1 = nd.

Sensitiveness is expressed in terms of the radius of curvature of the upper surface of the bubble or by an angle through which the axis is tilted for the deflection of one division of the graduation.

Determining sensitiveness :- Suppose the level was set up at O at a distance D from the staff of P. The staff reading is taken with the bubble at the extreme left end. Let it be PE.



Determining sensitiveness of the Bubble

Let D = distance between the level and staff.

s = intercept between the upper and lower sights.

n = number of division through which the bubble is deflected.

R = radius of curvature of the tube.

α = angle subtended by arc E_1E, and

d = length of arc division of the graduation, expressed in the same units as D and S

Movement of centre of bubble = $EE_1 = nd$.

Triangles OLE, and ACB are similar

Hence $\angle\alpha$ = angle EE_1

$$\alpha = \frac{EE_1}{R} = \frac{nd}{R}$$

Again $\frac{EE_1}{R} = \frac{S}{\theta}$ (height of $\triangle OLE_1$ may be considered as R)

(ii) $$\frac{nd}{R} = \frac{S}{\theta}$$

$$\alpha = \frac{nd}{R} = \frac{S}{\theta}$$

$$\therefore R = \frac{nd \times \theta}{S}$$

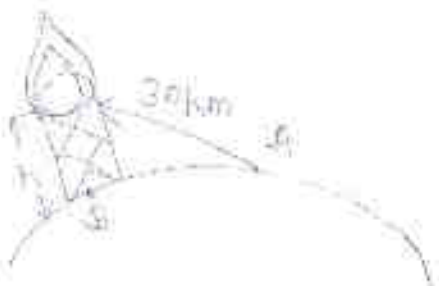
Let α' = angular value for one division in radians

$$\alpha' = \frac{\alpha}{n} = \frac{S}{\theta} \times \frac{1}{n} \text{ radians}$$

(OR)

$$\alpha = \frac{S}{\theta n} \times 206,265 \text{ seconds} \quad (1 \text{ radian} = 206,265 \text{ seconds})$$

4.Q A lamp at the top of a lighthouse is visible just above the horizon from a station at sea level the distance of the lamp from the station is 30km find the height of the light house?



D = 30 km

height = combined correction

h = c = 0.0673 D

= 0.0673 × 30 = 60.57 m.

∴ Height of the light house is 60.57 m.

2) what is the visible horizon distance from a 50m high tower ?

Sol^n

Height = 50m (h)

$D = \sqrt{\frac{h}{0.0673}} = \sqrt{\frac{50}{0.0673}} = 27.25 km$

3) what is the dip of horizon assuming the radius of the earth 6,370 km take D from previous problem ?

Sol^n

Radius (R) = 6,370 km

distance (D) = 27.25 km

Dip of horizon = $\frac{D}{R}$ radian

[1 radian = $\frac{180}{\pi}$

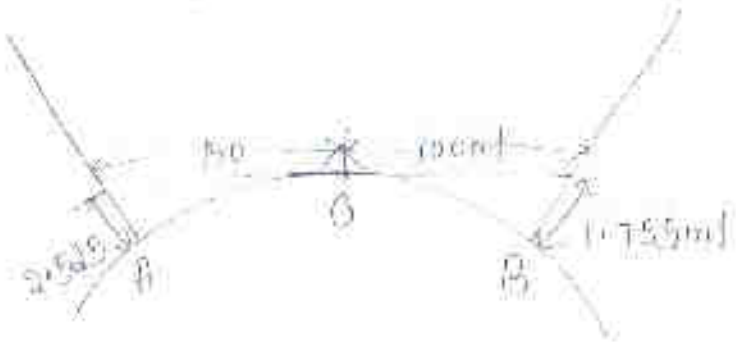
1 degree = $\frac{\pi}{180}$ radian]

$\frac{27.25}{6370} \times \frac{180}{\pi} = 0.24^0$

$= 0.24^0 \times 60 min = 14.4 minute$

30 July 2021

16 A level is set up at a point 150 m from A & 100 m from B. The observed staff readings of A & B are 2.525 & 1.755 find the true difference of level between A & B ?



Soln $D_1 = 150 m$

$= \frac{150}{1000}$ Km

combined correction = Cct + Cc = 0.0673

station A C.C = $0.0673 \times \left(\frac{150}{1000}\right)^2$

$= 1.51 \times 10^{-4}$

$= 0.00151$ m (-ve)

true reading = 2.525 - 0.00151

= 2.5235 m

station B C.C = $0.0673 \times \left(\frac{100}{1000}\right)^2$

$= 6.73 \times 10^{-4} = 0.000673$ m

true reading = observed reading - C.C

= 1.755 - 0.000673

= 1.7543 m

Difference of level bet^n A & B

2.5235 − 1.7543

= 0.7692 mt

[Rise from A to B
Fall from B to A]

50 A man on the deck of a ship observes a luminous object which is 50 mt above sea level. If the man's eye level is 10 mt above sea level. Find the distance bet^n him & the object ?

Sol^n height (h_1) = 50 mt

height (h_2) = 10 mt

$D_1 = \sqrt{\frac{h_1}{0.0673}} = \sqrt{\frac{50}{0.0673}} = 27.25$ Km

$D_2 = \sqrt{\frac{h_2}{0.0673}} = \sqrt{\frac{10}{0.0673}} = 12.18$ Km

distance bet^n him $D_1 + D_2$ = 27.25 + 12.18

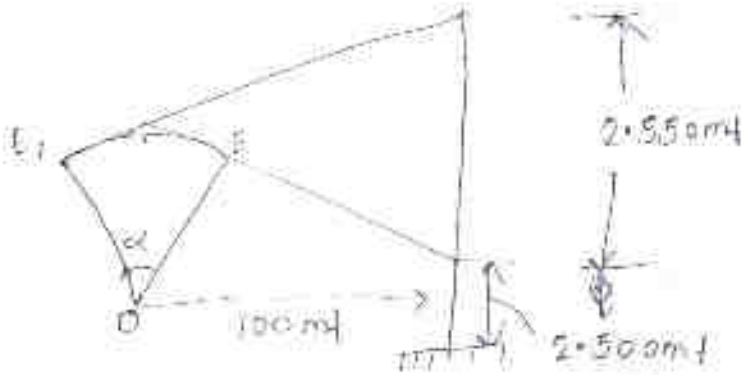
= 39.43 Km .

66 A man at a position 10 mt above sea observes the angle of depth of a hill. The angle of the hill is 60°. Find the height of the hill ?

Sol^n d = 10 mt

h = 0.0673d^2 = 0.0673×10 = 0.673 mt

10. When the bubble is at the centre the reading on the staff is 2.500m. The bubble is then moved by 5 height division & the staff reading is 2.550m. If the length of one division of the bubble is 2mm, determine the radius of curvature of the bubble & the angular value of one division of the bubble?



Soln

$D = 100\ m$

$d = 2mm = 0.002\ m$

$n = 5$

$R = ?$

$\alpha = ?$

$s = 2.550 - 2.500 = 0.050\ m$

$$R = \frac{nd \times D}{s} = \frac{5 \times 0.002 \times 100}{0.050} = 20\ m$$

$$\alpha'' = \frac{s}{Rn} \times 206,265''$$

$$= \frac{0.050}{100 \times 5} \times 206,265 = 20.63\ \text{second}$$

∴ Hence the radius of curvature is 20 m & the angle of 1 division is 20.63 second.

Reciprocal Levelling

We have already found by the principle of equalising backsight and foresight distances that if the level is placed exactly midway between two points and staff readings are taken to determine the difference of level then the errors (due to inclined collimation line, curvature and refraction) are automatically eliminated But in the case of a river or valley, it is not possible to set up the level midway between two points on opposite banks. In such cases, the method of reciprocal Levelling is adopted which involves reciprocal observations from both banks of the river or valley.

In reciprocal levelling, the level is set up on both banks of the river or valley and two sets of staff readings are taken by holding the staff on both banks. In this case, it is found that the errors are completely eliminated and the true difference of level is equal to the mean of the two apparent differences of level.

The principle is explained as follows:

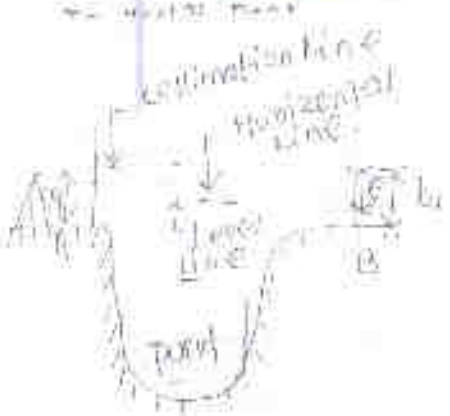
Procedure

A. Suppose A and B are two points on the opposite banks of a river. The level is set up near A and after proper temporary adjustment, staff readings are taken on A and B.

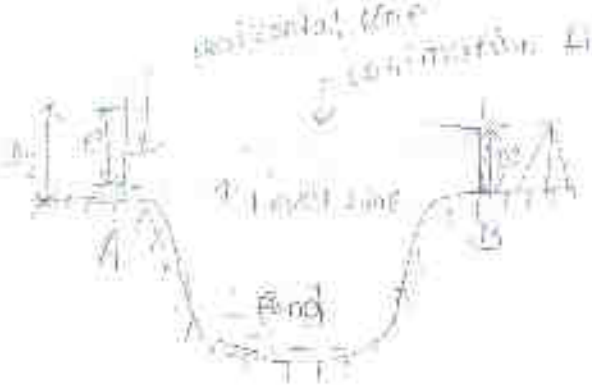
B. Suppose the readings are a1 and b1.

Let, h = true difference of level between A & B

e = combined error due to curvature, refraction and collimation (the error may be positive or negative but the error is assumed positive)



(a)



(b)

e = e_c

Case-1

true reading at A = a_1

B = $b_1 - e_1$

true diff(h) = $a_1 - (b_1 - e)$ ---- (1)

Case-2. true reading at B = b_2.

B = $a_2 - e$

true diff(h) = $(a_2 - e) - b_2$ ---- (2)

Solving eqn (1) & (2)

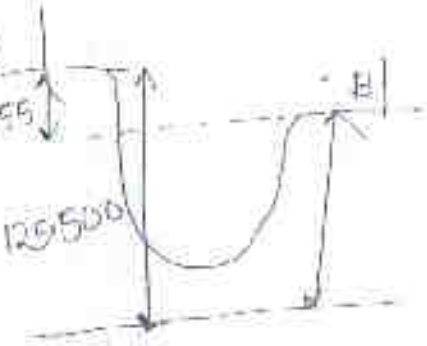
$h = a_1 - (b_1 - e)$

$h = (a_2 - e) - b_2$

$\Rightarrow 2h = a_1 - b_1 + e + a_2 - e - b_2$

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

5) In an operation including involving reciprocal leveling two points A & B are taken on opposite bank of a river, when the level was the level was set up near A, the staff reading on A & B were 2.245 & 3.375 respectively. when the levels were set up near B, the staff readings were 1.955 & 3.055. find the true difference of level of A & B & also the RL of the B if that of A is 125.500 m.



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

$a_1 = 2.245$ $b_1 = 3.375$

$a_2 = 1.955$ $b_2 = 3.055$

$$h = \frac{(2.245 - 3.375) + (1.955 - 3.055)}{2}$$

$= 1.115$ m (-ve) (fall from A to B)

RL of B = RL of A - h

$= 125.500 - 1.115 = 124.385$

Instrument at	reading on		Remark
	A	B	
A	1.955 a_1	2.545 b_1	distance AB = 500 m
B	2.425 (a_2)	3.015 (b_2)	RL of A = 125.500

Find ① true RL of B

② combined correction ?

sp

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

$$= \frac{(1.155 - 2.595) + (0.935 - 2.415)}{2}$$

$$= 1.435 \text{ m}$$

RL of B = RL of A − h

= 525.500 − 1.435

= 524.065 m

(ii) $C_c = 0.0673^2$

$= 0.0673 \left(\frac{500}{100}\right)^2$

$= 0.016 (-ve)$

SOURCES OF ERROR IN LEVELLING

The following are the different sources of error in levelling :-

(i) Instrumental errors :-

(a) The permanent adjustment of the instrument may not be perfect that is the line of collimation may not be parallel to the axis of the bubble tube.

(b) The internal arrangement of the focusing tube is not perfect.

(c) The graduation of the levelling staff may not be perfect.

i) personal errors :-

(i) The instrument may not be levelled perfectly.
(ii) The focussing of the eyepiece and object glass may not be perfect and the parallax may not be eliminated entirely.
(iii) The position of the staff may be displaced at the change point at the time of taking FS and BS readings.
(iv) The staff may appear inverted when viewed through the telescope. By mistake the staff readings may be taken upwards instead of downwards.
(v) The reading of the stadia hair rather than the central collimation hair may be taken by mistake.
(vi) A wrong entry may be made in the level book.
(vii) The staff may not be properly and fully extended

ii) Errors due to natural causes :-

(i) When the distance of sight is long, the curvature of the earth may affect the staff reading.
(ii) The effect of refraction may cause a wrong staff reading to be taken.
(iii) The effect of high winds and a shining sun may result in a wrong staff reading.

CONTOURING

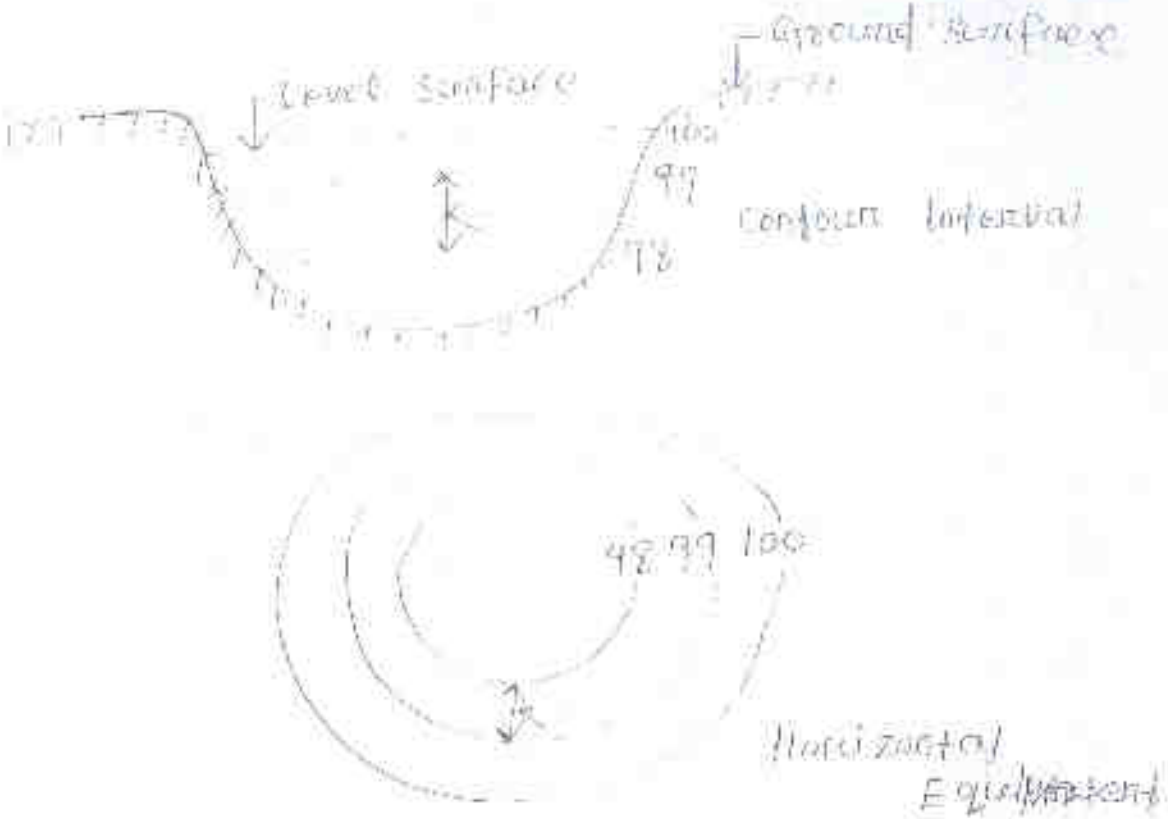
Contouring is, basically, a levelling operation. The equipment are the same for levelling and contouring. The main objective of contouring is to determine the points on the ground having the same reduced level (RL). The contour lines joint the point of same elevation directly or by interpolation technique. It gives the topographical features of the ground, containing different contour lines of different elevation for a closed area. Based on the topographical features. calculations for engineering projects can be carried out.

DEFINITIONS

(1) Contour Line: The line of intersection of a level surface with the ground surface is known as the contour line or simply the contour. It can also be defined as a line passing through points of equal reduced levels.

For example, a contour of 100m indicates that all the points on this line have an RL of 100m. Similarly in a contour of 99m, all points have an RL of 99m. and so on.

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



(ii) Contour interval :- The vertical distance between any two consecutive contours is known as a contour interval. Suppose a map includes contour lines of 100m, 98m, 96m and so on. The contour-interval here is 2m. This ~~mat~~ interval depends upon the nature of the ground (i.e whether flat or steep) & the scale of the map & also the purpose of the survey. Contour intervals for flat country are generally small. eg. 0.25m, 0.15m, etc. The contour interval for a steep slope in a hilly area is generally greater e.g. 5m, 10m, 15m, etc.

Again for a small-scale map, the interval may be of 1m, 2m, 3m, etc. and for large scale map, it may be 0.25m, 0.50m, 1.25m etc.

It should be remembered that the contour interval for a particular map is constant.

(2) Horizontal Equivalent :-

The horizontal distance between any two consecutive contours is known as horizontal equivalent. It is not constant. It varies according to the steepness of the ground.

For steep slopes, the contours lines run close together, and for flatter slopes they are widely spaced.

Object of preparing contour map

The general map of country includes the locations of roads, railways, rivers, villages, towns, and soon. But the nature of the ground surface cannot be realised from such a map. However for all engineering projects involving roads, railways, and soon a knowledge of the nature of ground surface is required for locating suitable alignments and estimating the volume of earth work. Therefore, the contour map is essential for all engineering projects. This is why contour maps are prepared.

Uses of contour map :-

The following are the specific uses of the contour map.

① The nature of the ground surface of a country can be understood by contour map. Hence, the

possible route of communication between different places can be demarcated.

(iii) A suitable site or an economical alignment can be selected for any engineering project.

(iii) The capacity of a reservoir or the area of a catchment can be approximately computed.

(iv) The intervisibility or otherwise of different points can be established.

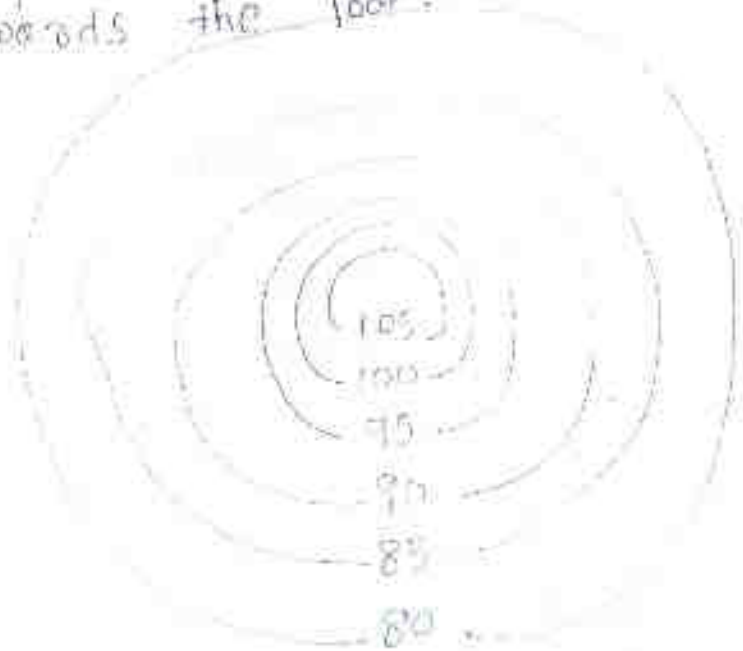
(v) A suitable route for a given gradient can be marked on the map.

(vi) A section of the ground surface can be drawn in any direction from the contour map.

(vii) Quantities of earth work can be approximately computed.

CHARACTERISTICS OF CONTOURS

1. The contour lines are closer near the top of a hill or high ground and wide apart near the foot. This indicates a very steep slope towards the peak and a flatter slope towards the foot.



(HILL)

(5) Contour lines cannot one another; except in the case of an overhanging cliff; but the overlapping portion must be shown by a dotted line.

(overhanging cliff)

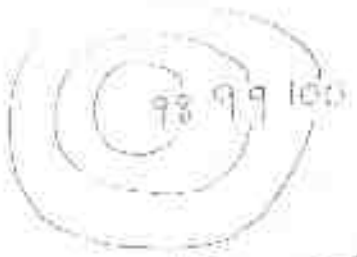
(6) When the higher values are inside the loop, it indicates a ridge line. Contour lines cross ridge lines at right angles.

(Ridge line)

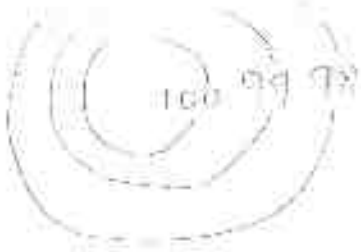
(7) When the lowest values are inside the loop it indicates a valley line. Contour lines cross the valley line at right angles.

(valley line)

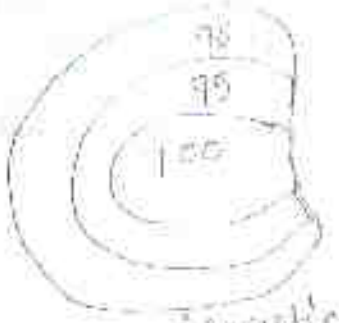
(2) A series of closed contours always indicates a depression or summit. The lower value being inside the loop indicates a depression and the higher values being inside the loop indicates to summit.



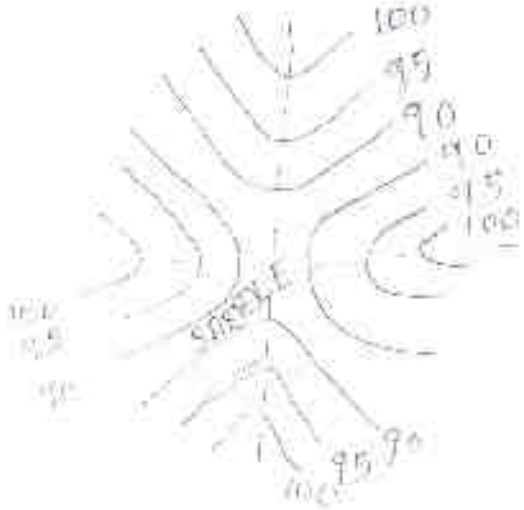
(a) (Depression)



(b) (Summit)



(vertical cliff)



(saddle)