BHADRAK ENGINEERING SCHOOL & TECHNOLOGY
(BEST), ASURALI, BHADRAK

LAND SURVEY-1

(Th-03)

(As per the 2019-20 syllabus of the SCTE&VT, Bhubaneswar, Odisha)

Fourth Semester

Civil Engg.

Prepared By: *Er. A.K.MOHAPATRA*

TOPIC WISE DISTRIBUTION OF PERIODS & MARKS

Sl. No.	Topic	Periods as per syllabus	Expected Marks
01	Introduction To Surveying, Linear Measurements	07	10
02	Chaining and Chain Surveying	07	10
03	Angular Measurement and Compass Surveying	12	15
04	Map Reading Cadastral Maps & Nomenclature	07	05
05	Plane Table Surveying	07	15
06	Theodolite Surveying and Traversing:	15	15
07	Levelling and Contouring	15	20
08	Computation of Area & Volume	05	10
	Total	**75**	**100**

CHAPTER NO-1

INTRODUCTION TO SURVEYING, LINEAR MEASUREMENTS

Learning objectives

1.1 Surveying: Definition, Aims and objectives
1.2 Principles of survey-Plane surveying- Geodetic Surveying- Instrumental surveying.
1.3 Precision and accuracy of measurements, instruments used for measurement of distance, Types of tapes and chains.
1.4 Errors and mistakes in linear measurement – classification, Sources of errors and remedies.
1.5 Corrections to measured lengths due to-incorrect length, temperature variation, pull, sag, numerical problem applying corrections.

1.1. SURVEYING DEFINITION, AIMS AND OBJECTIVES -

Surveying is the art of determining the relative position of different objects on the surface of the earth by means of measurements of distances, directions and elevations and then, preparing a map to any suitable scale.

AIM AND OBJECTIVES OF SURVEYING-

The aim of surveying is to prepare a map to show the relative positions, horizontal distances, and elevation of the objects on the surface of the earth. The map is drawn to some suitable scale. It shows the natural features of a country, such as towns , villages , roads , railways , river etc. The objectives of surveying can be stated as follows.

(i) Collect and record data on the relative positions of points on the surface of the earth.
(ii) Compute areas and volumes using this data,required for various purposes.
(iii) Prepare the plans and maps required for various activities.
(iv) Lay out, using survey data, the various engineering works in correct positions.
(v) Check the accuracy of laid out lines, built of structure.

1.2 PRINCIPLES OF SURVEY-

The two basic principles of surveying need to be followed for accurately locating points on earth.

(i) To work from the whole to part:

The main principle of surveying is to work from whole to part whether it is plane or geodetic surveying. To achieve this in actual practice, a sufficient number of primary control points are established with higher precision in and around the area to be detail surveyed. Minor control points in between the primary control points are then established with less precise method. Further details are surveyed with the help of these minor control points by adopting any of the survey methods. The main idea of working from whole to part is to prevent accumulation of errors and localize minor errors within the frame work of control points. On the other hand if survey is carried out from part to whole, the errors would expand to greater magnitudes and the scale of the survey will be distorted beyond control.

In general practice the area is divided into a number of large triangles and the positions of their vertices are surveyed with greater accuracy, using sophisticated instruments. These triangles are further divided into smaller triangles and their vertices surveyed with less accuracy.

(ii) To locate a new station by at least two measurements from fixed reference points / control points.

The reference points / control points are selected in the area and distance between them, is measured accurately. The line is then plotted to a convenient scale on a drawing sheet. In case, the control points are co-ordinated, their locations may be plotted with the system of coordinates (Cartesian or spherical). The location of the required point may then be plotted by making two measurements from the given control points as explained below.

Let P and Q be two given control points. Any other point R can be located with reference to these points, by any of the following methods

CLASIFICATION OF SURVAYING-

(1) PRIMARY CLASSIFICATION

Surveying is primarily classified as:

(i) Plane surveying

(ii) Geodetic surveying

(i) PLANE SURVEYING:

In plane surveying the curvature of the earth is not taken into consideration. This is because surveying is carried out over a small area so the surface of the earth is consider as plane .Plane surveying is done on an area of less than 250 km^2.

(ii) GEODETIC SURVEYING:

In geodetic surveying the curvature of the earth is taken into consideration. It is extended over a large area. It is carried out over an area exceeding 250 km^2.

(2) INSTRUMENTAL SURVEYING

- Chain surveying
- Compass surveying
- Plane table surveying
- Thedolite surveying
- Tachometric surveying

1.3 PRECISION AND ACCURACY OF MEASUREMENTS

When scoping a project, you want to be as close to the actual workload as possible. Defining the scope means that you and your client are figuring out and documenting a list of specific project goals. That could be features, functionalities, deliverables, deadlines, and ultimately costs of the project. Project scope helps with resource planning and time management of the project. Accuracy and precision are used in the context of measurement, e.g., the size of a project and therefore are both helpful when defining the scoping.

Accuracy and precision are alike only in the fact that they both refer to the quality of measurement, but they are very different indicators of measurement.

Instruments used for measurement of distances

Instruments for measuring distances

(i) Tapes

(ii) Steel Bands

(i) Chains

(ii) Arrows

(iii) Pegs

(iv) Ranging Rods

(v) Ranging Poles

(viii)Offset Rods

(ix)Plumb Bobs

Types of tapes and chains

TAPES: Depending upon the material tapes are classified as

(i) Cloth or linen tape

(ii) Metallic tape

(iii)Steel tape

(iv)Invar tape

(i) **Cloth or linen tape:** Linen tapes are closely woven linen and varnished to resist moisture. They are generally 10 metres to 30 metres in length and 12mm to 15 mm in width. Cloth tapes are generally used for measuring offset measurements only due to following reasons :

(i) It is easily affected by moisture and shrunk.

(ii) Its length gets altered by stretching.

(iii) It is likely to twist and tangle.

(iv) It is not strong as a chain or steel tape.

(v) It is light and flexible and it does not remain straight in strong wind.

(vi) Due to continuous use, its figures get in-distinct.

(ii) **Metallic Tape:** A linen tape reinforced with brass or copper wires to prevent stretching or twisting of fibers is called a metallic tape. As the wires are interwoven and the tape is varnished, these wires are not visible to naked eyes. These tapes are available in different lengths but tapes of 20m and 30m lengths are very common. These are supplied in leather case with winding machine. Each metre is divided into decimeters and each decimeter is sub-divided into centimeters.

(iii) **Steel Tape:** Steel tapes are available with different accuracy of graduation. Steel tapes are available in different lengths but 10m, 20m, 30m and 50m tapes are widely used in survey measurements. At the end of the tape a brass ring is provided. The length of metal ring is included in the length of tape. A steel tape of lowest degree of accuracy is generally superior to a metallic or cloth tape for linear measurements.

(iv) **Invar Tape:** Invar tapes are made of an alloy of nickel (36%) and steel (64%) having very low co-efficient of thermal expansion (0.000000122 per 1ºC). These are 6mm wide and are available in length of 30m, 50m and 100m. These tapes are used for high degree of precision required for base measurements.

CHAINS: The different types of chains are used in surveying and are given below.

(1) **Gunter's chain:** It is 66ft. long and divided into 100 links. Each link measures 0.66 ft.

(2) **Engineer's chain:** It is 100ft. long and divided into 100 links. Each link measures 1 ft.

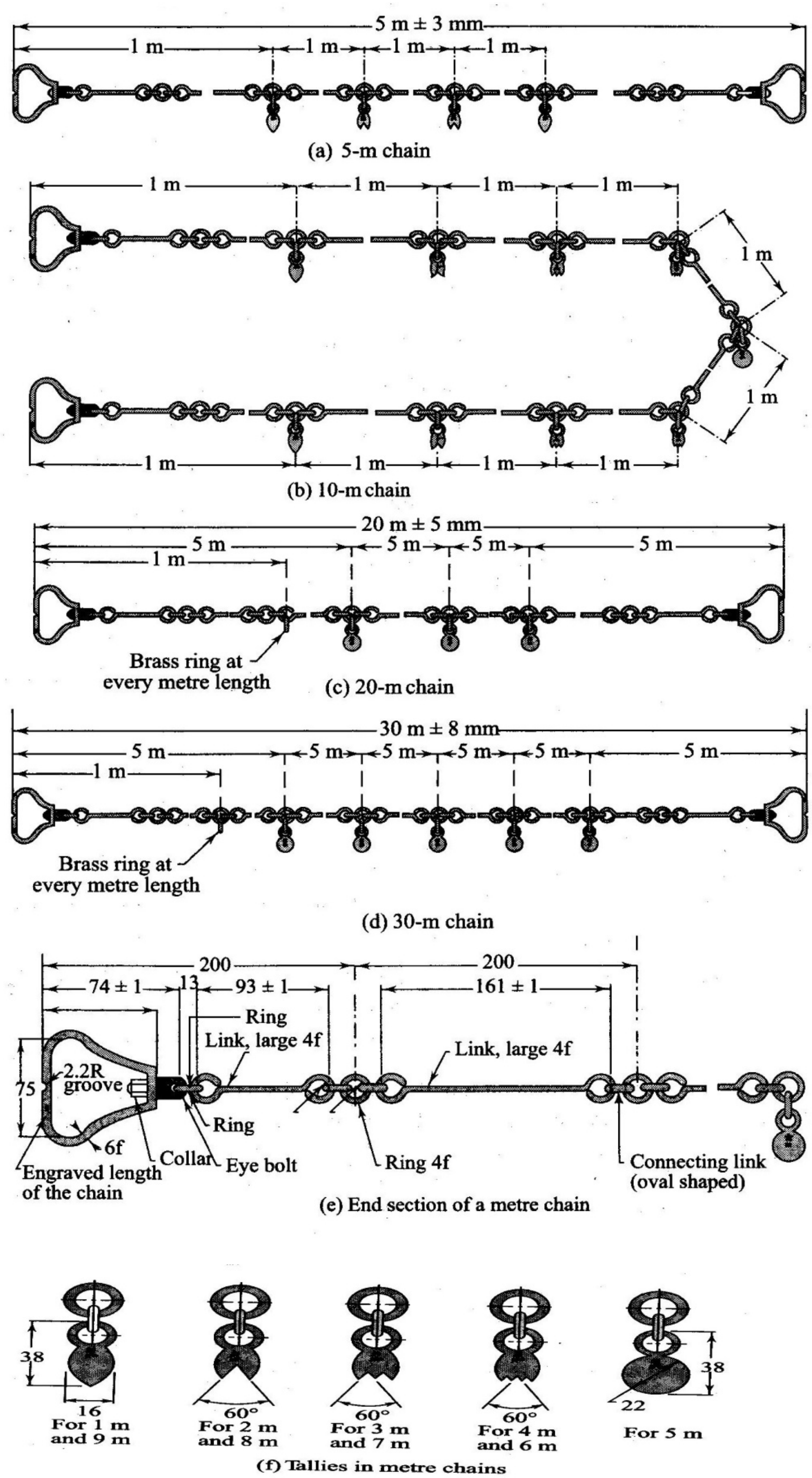


Fig. **Metric chain details**

Fig. 2.1

(3) **Metric Chain:** A metric chain is prepared with 100 or 150 pieces/ links of galvanized mild steel wire of diameter 4mm. The ends of the pieces are bent to form loops and connected together by means of three oval shaped rings which gives flexibility to the chain. Two brass handles are provided at the two ends of the chain with swivel joints so that chain can be turned round without twisting. The outside of the handle is the zero point or the end point of the chain. The length of the chain is measured from the outside of one handle to the outside of the other. The length of a link is the distance between the centres of the two consecutive middle rings as shown in the Fig. 2.1. The end links include the length of handle. Tallies are provided for marking 5m, 10m, etc are marked with letter "m" to distinguish the metric chain from non-metric chain. The length of chain whether 20m 0r 30m is indicated on the handle for easy identification.

Suitability of Chains: The chains are suitable for the following cases.

(i) It is suitable for ordinary or preliminary works as its length alters due to continuous use.

(ii) Its length gets shortened due to bending of links and gets lengthened by flattening of the rings.

(iii) Being heavier, a chain gets sagged considerably when suspended at the ends.

(iv) It can be easily repaired in the field.

(v) Measurement readings can be taken very easily.

(vi) It is only suitable for rough works.

Merits of Chains:

(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.

Demerits of Chains:

(i) They are heavy and take too much time to open or fold.

(ii) They become longer or shorter due to continuous use.

(iii) When the measurement is taken in suspension the chain sags excessively giving incorrect measurements.

ARROWS: Arrow are made of tempered steel wire of diameter 4mm.One end of the arrow is bent into a ring of diameter 50 mm and the other end is pointed. Its overall length is 400mm. Arrows are used for counting the number of chains while measuring a chain line. Generally 10 arrows accompany a chain.

RANGING RODS: Rods, which are used for ranging a line are known as ranging rod. Such rods are made of seasoned timber or seasoned bamboo. Sometimes GI pipes of 25mm/ 30mm diameter are also used as ranging rods. They are generally circular in section of diameter 25mm/30mm and length 2m / 3m.The rod is divided into equal parts of 20cm each and the divisions are painted black and white or red and white alternatively so that the rod is visible from a long distance. The lower end of the rod is pointed or provided with an iron shoe.

RANGING POLES: These are similar to ranging rods except that they are heavier in section of length 4m to 6m. They are used for ranging very long lines in undulating ground.

OFFSET RODS: These are similar to ranging rods and o 3m long. The top is provided with an open hook for pulling or pushing a chain through obstruction like bushes etc.It is used for aligning the offset line and measuring short offsets.

PLUMB BOB: It is used to transfer the end points of the chain onto ground while measuring distances in hilly terrain. It is also used for testing verticality of ranging poles, ranging rods.

PEGS: Wooden pegs usually 2.5cm square and 15cm deep are used to mark the position of survey stations.

1.4 ERRORS AND MISTAKES IN LINEAR MEASUREMENT CLASSIFICATIONS AND SOURCES OF ERRORS AND ITS REMEDIES.

Errors in chaining may be caused due to variation in temperature and pull, defects in instruments etc. They may be classified into two catagories.

(i) Compensating errors

(ii) Cumulative error

(i) **COMPENSATING ERRORS:** Errors, which may occur in both directions (that is both positive and negative) and which finally tend to compensate are known as compensating errors.

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

I. **Positive Cumulative Error:** The error, which make the measured length more than the actual is known as positive cumulative error.

Sources: (a) The length of chain / tape is shorter than its standard length due to

- Bending of links
- Removal of too many rings due to adjustment of its length.
- Knots in connecting links.
- The field temperature is lower than that at which the tape was calibrated.
- Shrinkage of tape when moist
- Clogging of rings with mud.

(a) The slope correction is ignored while measuring along slopping ground.

(b) The sag correction, if not applied when chain / tape is suspended at its ends.

(c) Incorrect alignment.

II. **Negative Cumulative Error:** The error, which make the measured length less than the actual is known as negative cumulative error.

Sources: (a) The length of chain / tape is longer than its standard length due to

- Flattening of connecting rings.
- Opening of the ring joints.
- The field temperature is higher than that at which the tape was calibrated.

MISTAKES: Errors occurring due to the carelessness of the chainman are called mistakes. Following are a few common mistakes:

(1) Once an arrow is withdrawn from the ground during chaining it may not be replaced in proper position, if required due to some reason.

(2) A full chain length may be omitted or added. This happen when arrows are lost or wrongly counted.

(3)The number may be read from the wrong direction; for instance a 6 may be read as a 9.

(4) Some number may be called wrongly. For example 50.2 may be called as fifty two without the decimal point being mentioned

REMEDIES OF ERRORS AND MISTAKES:

(1) The point where the arrow is fixed on the ground should be marked with a cross(×).
(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 number of arrows taken.
(4) The chainman should call the measurement loudly and distinctly and the surveyor should repeat them while booking.
(5) Ranging should be done accurately.
(6) No measurement should be taken with the chain in suspension.

1.5 CORRECTIONS TO MEASURE LENGTH DUE TO INCORRECT LENGTH

(i) Correction for standard length
(ii) Correction for alignment
(iii) Correction for slope
(iv) Correction for tension
(v) Correction for temperature
(vi) Correction for sag

(i) **Correction for standard length:** Before using a tape, its actual length is ascertained by comparing it with a standard tape of known length. The designated nominal length of a tape is its designated length e.g. 30m or 100m. The absolute length of a tape is its actual length under specified conditions.

Let L= measured length of a line

C_a = correction for absolute length

l = nominal designated length of tape

C = correction be applied the tape

Then, $C_a = \dfrac{L.C}{l}$

The sign of the correction C_a will be the same as that of C.

(ii) **Correction for alignment:** Generally a survey line is set out in a continuous straight line. Sometimes, it becomes necessary, due to obstruction to follow a bent line which may be composed of two or more straight portions subtending an angle other than 180º as shown in Fig.2.2.

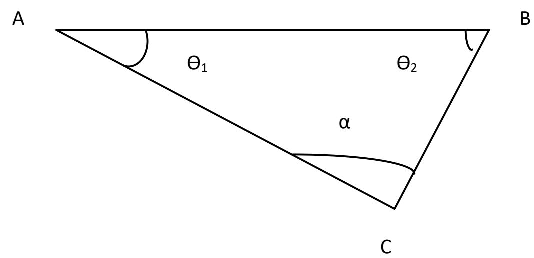


Fig.2.3. Correction for alignment

Let AC=l_1; CB= l_2

Angle $BAC = \Theta_1$; Angle $BAC = \Theta_2$

Length AB= $l_1 \cos \Theta_1 + l_2 \cos \Theta_2$

The required correction $= (l_1 + l_2) - (l_1 \cos \Theta_1 + l_2 \cos \Theta_2)$

(iii) **Correction for slope:** The distance measured along the slope between two stations is always greater than the horizontal distance between them. The difference in slope distance and horizontal distance is known as slope correction which is always substractive.

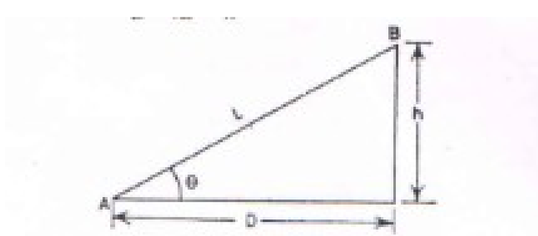


Fig. 2.4 Slope Correction

Let L = slope distance AB

D = horizontal distance AC

h=difference in reduced levels of A and B

$$D = \sqrt{\left(L^2 - h^2\right)}$$

Slope Correction = $L - D = \dfrac{h^2}{2L}$

(iv) <u>Correction for pull/ tension (C_P):</u>

During measurement the applied pull may be either more or less than the pull at which the chain or tape was standardized. Due to the elastic property of materials the strain will vary according to the variation of applied pull and hence necessary correction should be applied. This correction is given by the expression

$$C_P = ((P-P_0)xL)/(AxE)$$

where, P= Pull or tension applied during measurement in Newtons

A= Cross-sectional area of the tape in square cm.

L= Length of the measured line

P_0 = Standard pull

E = Modulus of Elasticity of the tape

If the applied pull is more, tension correction is positive, and if it is less, the correction is negative.

(v) Temperature correction (C_t):

This correction is necessary because the length of the tape or chain may be increased or decreased due to rise or fall of temperature during measurement. The correction is given by the expression as mentioned below.

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

where C_t = correction for temperature

α=coefficient of thermal expansion

T_m=temperature during measurement in degrees centigrade

T_0=temperature at which the tape was standardized in degrees centigrade

L=length of tape

(vi) Correction for sag (C_s)

This correction is necessary when the measurement is taken with the tape in suspension. It is given by the expression as mentioned below.

$$C_s = \frac{L}{24}\left(\frac{W}{P}\right)^2$$

where W= total wt of the tape; L= horizontal distance between the supports

P = pull applied during measurement

Problem 1. The length of a survey line measured with a 30m chain was found to be 631.5m. When the chain was compared with a standard chain, it was found to be 0.1m too long. Find the true length of the survey line.

Solution

The true length of a line = $\frac{L'}{L} \times measured\ length$

L' = 30.1m. L = 30m

and measured length of the survey line = 631.5m

Thus, true length of the survey line = $\frac{30.1}{30} \times 631.5$ = 633.603 m. Ans.

Problem 2. A 20m chain was found to be 4 cm too long after chaining 1400m. It was 8 cm too long at the end of day's work after chaining a total distance of 2420m. If the chain was correct before commencement of the work, find the true distance.

Solution

$$\text{The correct length of the at commencement} = 20\text{m}$$

$$\text{The length of the chain after chaining 1400m} = 20.04\text{ m.}$$

$$\text{The mean length of the chain while measuring} = (20 + 20.04)/2 = 20.02\text{m}$$

$$\text{The true distance for the wrong chainage of 1400m} = (20.02/20)\text{x}1400 = 1401.4\text{ m}$$

$$\text{The remaining distance} = 2420 - 1400 = 1020\text{m}$$

$$\text{The mean length of chain while measuring the remaining distance} = (20.08 + 20.04)/2$$

$$= 20.06\text{m}$$

$$The\ \text{true length of remaining 1020m} = (20.06/20)\text{x } 1020 = 1023.06\text{m}$$

$\text{Hence, the total true distance} = 1401.4 + 1023.06 = 2424.46 \text{ m} \quad \text{Ans.}$

Problem No.3. A line was measured with a steel tape which was exactly 30 meters at 20℃ at a pull of 100N (or 10kgf), the measured length being 1650.00 meters. The temperature during measurement was 30° C and the pull applied was 150N (or 15kgf). Find the length of the line, if the cross-sectional area of the tape was 0.025 sq.cm. The co-efficient of expansion of the material of the tape per 1 ºC =3.5x10^{-6} and the modulus of elasticity of the material of the tape=2.1x10^{5} N/mm^{2} (2.1x10^{6} kg/cm^{2}).

Solution:

(i) Correction of temperature per tape length

$$=\alpha(Tm - To)L$$

$$= 0.0000035(30 - 20)X\ 30$$

= 0.00105m (+ve)

(ii) Correction for pull per tape length

= C_P =((P-P_0)xL)/(AxE)=((150-100)x30)/(2.5x2.1x10^5)

=0.00286m (+ve)

Combined correction = 0.00105+0.00286=0.00391m

True length of the tape = 30+0.0039=30.0039m

True length of the line = (30.0039x1650.00)/30

=1650.21m. Ans.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 What is surveying ? [2012-W]

a – it is the art of determining the relative position of points on above or beneath the surface of the earth by means of direct or indirect measurement of distance direction and elevation and by preparing a map of any suitable scale.

Q-2 What are the uses of surveying? [2015-W]

a- To prepare a topographical map which show a hills , villages ,river, forest and town etc
b- To prepare a cadastral map showing the boundries of houses and other properties

c- To prepare a engg. Map which shows the details of engg. Such as roads railways irrigation canal etc.

Q-3 What is the principle of surveying? [2018-W]

Ans:

a) To work from whole to the part.
b) To locate a new station by at least two measurement from fixed reference point.

Q-4 What is cadastral surveying? [2015-W]

It is the surveying which is conducted in order to determine the boundries of fields houses etc.

POSSIBLE LONG TYPE QUESTIONS

Q-1 Describe what are the stages of survey operation . [2015-W]

Q-2 Describe the principle of surveying.[2018-W]

CHAPTER NO-2

CHAINING AND CHAIN SURVEYING

Learning objectives

2.1 Equipment and accessories for chaining
2.2 Ranging – Purpose, signaling, direct and indirect ranging, Line ranger – features and use, error due to incorrect ranging.
2.3 Methods of chaining –Chaining on flat ground, Chaining on sloping ground – stepping method, Clinometer- features and use, slope correction.
2.4 Setting perpendicular with chain & tape, Chaining across different types of obstacles –Numerical problems on chaining across obstacles.
2.5 Purpose of chain surveying, Its Principles, concept of field book.
Selection of survey stations, base line, tie lines, Check lines.
2.6 Offsets – Necessity, Perpendicular and Oblique offsets, Instruments for setting offset – Cross Staff, Optical Square.
2.7 Errors in chain surveying – compensating and accumulative errors causes & remedies, Precautions to be taken during chain surveying.

2.1 EQUIPMENT AND ACCESSORIES FOR CHAINING

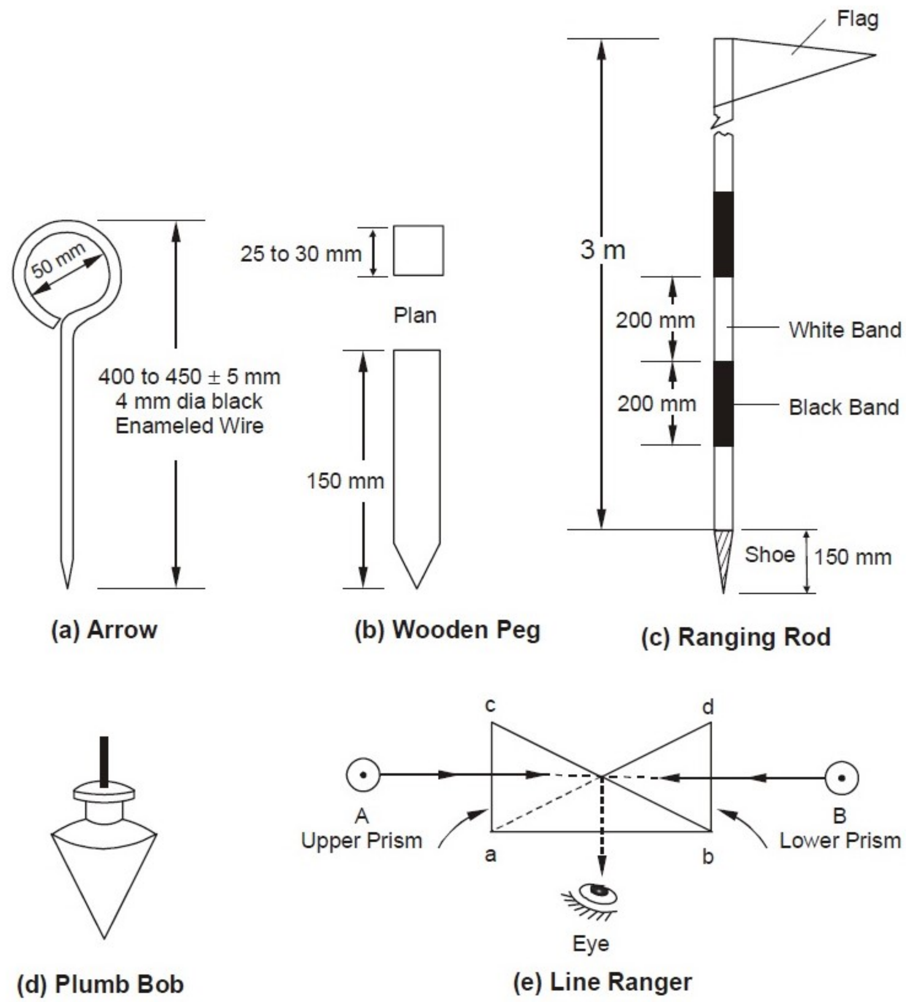
In addition to chain or tape, several other auxiliary equipment are required in a chain surveying These are listed in subsequent paragraphs.

Arrows

Arrows or chain pins, as these are called sometime, are made of stout steel wire 4 mm in diameter, 400 to 450 mm long and black enameled. These are used to mark the end of each chain length as shown in Figure (a).

Wooden Pegs

These are made of stout timber generally 25 to 30 mm square or circular size and 150 mm long as shown in Figure (b). Wooden pegs are normally used to mark station position on ground on a quasi-permanent state. These are tapered at one end so that they can be driven in the ground with a hammer. These are kept at about 40 mm (minimum) projecting above the ground.



Ranging Rods

These are octagonal or circular in plan normally 25 to 30 mm diameter straight timber or tubular steel rods, 3 m in length and provided with an iron shoe at lower end as shown in Figure (c). These are painted in black and white alternate bands and normally have a flag at the top for easy recognition and identification from a distance. If the ranging roads are graduated in meters and one tenth of a meter, they are called offset rods and are used for measurement of short offsets.

Plumb Bob

It is usually heavy spherical or conical ball, as shown in Figure (d), of metal and is used to transfer points on ground by suspending it with the help of a strong thread. It is used in measuring distances on sloping ground by stepping. Compass, Dumpy levels and. Theodolites are also positioned over the station point accurately with the help of plumb bobs.

Line Ranger

A line ranger consists of either two plane mirrors or two right angled isosceles prisms placed one above the other as depicted in Figure (e). The diagonals of both the prisms are silvered so as to reflect the incident rays. Line rangers are provided with a handle to hold the instrument. A line ranger can also be used to draw offset on a chain line.

Use of chain

Unfolding Of Chain: 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 on 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.

Folding of Chain :

After the completion of the work the chain should be folded in to a bundle and fastened with a leather strap. To do this the handles of the chain should be brought together by pulling the chain at the middle. Commencing from the middle, take two pairs of link at a time with the right hand and place them obliquely across the other in the left hand. When the chain is collected in a bundle, it is tied with a leather strap. This process is called the folding of chain.

Reading a chain :

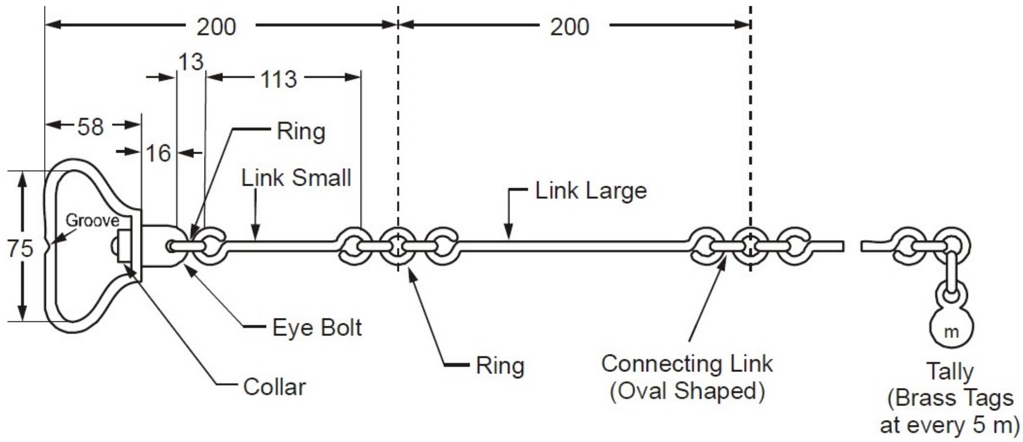
A survey chain is generally composed of 100 or 150 links formed by pieces of galvanised mild steel wire of 4 mm diameter. The ends of each link are looped and connected together by means of three circular or oval shaped wire rings to provide flexibility to chain. The length of each link is measured as the distance between the centres of two consecutive middle rings.

The ends of chain are provided with brass handles with swivel joints. The end link length includes the length of handle and is measured from the outside of the handle, which is considered as zero point or the chain end. Tallies, which are metallic tags of different patterns, are provided at suitably specified points in the chain to facilitate quick and easy reading. A semi-circular grove is provided in the centre on the outer periphery of handle of chain for fixing the mild steel arrow at the end of one chain length. The number of links in a chain could be 100 in a 20 m chain and 150 in a 30 m chain. The details of a metric chain are as shown in Figure

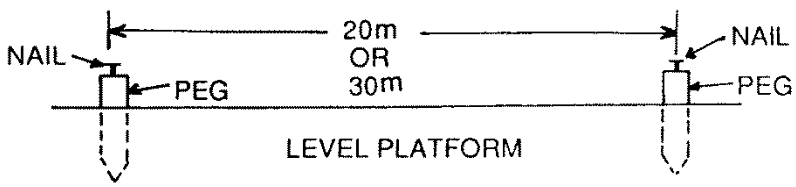
Testing of a chain :

Due to continuous use, a chain may be elongated or shortened. So, the chain should be tested and adjusted accordingly. If full adjustment is not possible, then the amount of shortening (known as ‘too short’) and elongation (known as ‘too long’) should be noted clearly for necessary correction applicable to the chain.

For testing the chain, a test gauge is established on a level platform with the help of standard steel tape. The steel tape is standardised at 20^0C and under a tension of 8 kg. The test gauge consist of two pegs having nails at the top and fixed on a level platform a required distance apart (say 20 or 30m). The incorrect chain is fully stretched by pulling it under normal tension along the test gauge. If the length of the chain does not tally with standard length, then the attempt should be made to rectify the error. Finally the amount of elongation or shortening should be noted.



Details of Metric Chain



The allowable error is about 2mm per 1m length of the chain. The overall length of the chain should be within the following permissible limit :

20 m chain : ± 5mm

30m chain : ± 8mm

2.2 RANGING RANGING – PURPOSE, SIGNALING, DIRECT AND INDIRECT RANGING, LINE RANGER – FEATURES AND USE, ERROR DUE TO INCORRECT RANGING.

Purpose

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

Purpose of ranging :

The purpose of ranging is to mark a number of intermediate points on a survey line joining two stations in the field so that the length between them may be measured correctly.

If the line is short or its end station is clearly visible, the chain may be laid in true alignment. But if the line is long or its end station is not visible due to undulation ground, it is required to mark a number of points with ranging rods.

Code of Signals for Ranging

Sl.No.	Signal by the Surveyor	Action by the Assistant
1	Rapid sweep with right hand	Move considerably to the right
2	Slow sweep with right hand	Move slowly to the right
3	Right arm extended	Continue to move to the right
4	Right arm up and moved to the right	Plumb the rod to the right
5	Rapid sweep with left hand	Move considerably to the left
6	Slow sweep with left hand	Move slowly to the left
7	Left arm extended	Continue to move to the left
8	Left arm up and moved to the left	Plumb the rod to the left
9	Both hands above head and then brought down	Correct
10	Both arms extended forward horizontally and the hands depressed briskly	Fix the rod

Direct ranging :

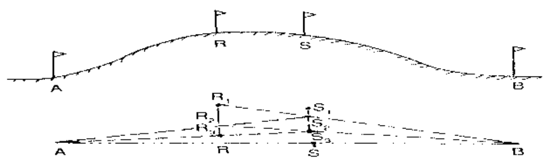
When intermediate ranging rods are fixed along the chain line, by direct observation from either end station, the process is known as "Direct Ranging". Direct ranging is possible when the end stations are inter visible. The following procedure is adopted for direct ranging :

- Erect ranging rods or poles vertically behind each end of the line.
- Stand about 2m behind the ranging rod at the beginning of the line.
- Direct the assistant to hold a ranging rod vertically at arm's length at the point where the intermediate station is to be established.
- Direct the assistant to move the rod to the right or left , until the ranging rods appear to be exactly in a straight line.
- Stoop down and check the position of the rod by sighting over their lower ends in order to avoid error to non-vertically of the ranging rods.
- After ascertaining that the ranging rods are in a straight line, signal the assistant to fix the ranging rod.

Indirect ranging :

When the end stations are not inter visible 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 chain men 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 direct the chainman at S_1 to come to position S_2 so that R_1 , S_2 and B are in the same straight line. Again the chainman at S_2 directs the chainman at R_1 to move the position at 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.



Line ranger features and use

It is an optical instrument used for locating a point on a line and hence useful for ranging. It consists of two isosceless prisms placed one over the other and fixed in an instrument with handle. The diagonals of the prisms are silvered so as to reflect the rays.
To locate point C on line AB (ref. Fig. 12.10) the surveyor holds the instrument in hand and stands near the approximate position of C. If he is not exactly on line AB, the ranging rods at A and B appear separated as shown in Fig. 12.10 (b). The surveyor moves to and fro at right angles to the line AB till the images of ranging rods at A and B appear in a single line as shown in Fig. 12.10 (c). It happens only when the optical square is exactly on line AB. Thus the desired point

C is located on the line AB.
Its advantage is it needs only one person to range. The instrument should be occasionally tested by marking three points in a line and standing on middle point observing the coincidence of the ranging rods. If the images of the two ranging rods do not appear in the same line, one of the prism is adjusted by operating the screw provided for it.

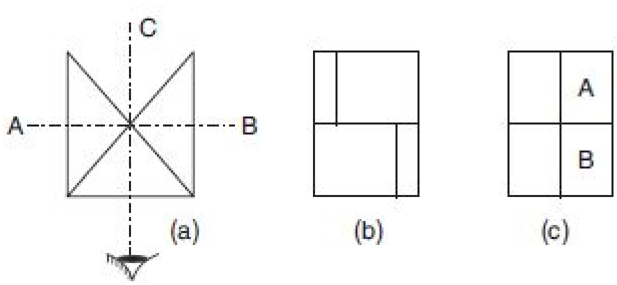


Fig. 12.10. Line ranger

ERRORS DUE TO INCORRECT RANGING

The error that occurs during the chaining process in the same direction is called as cumulative error. This type of error accumulates with the process of chaining.

An error that occurs in either directions during the chaining process is called as compensating error. As these errors takes place in either directions, the values compensate.

Causes of Errors

The basic reasons for errors caused in the chaining process in surveying are due to:

1. The Chain or tape with erroneous length
2. Inefficient Ranging
3. Inefficient Straightening
4. Careless holding and marking
5. Sag in Chain
6. Personal Mistakes
7. Variations in Pull
8. Variations in Temperature
9. Non-Horizontality

2.3 METHODS OF CHAINING -

CHAINING ON FLAT GROUND, CHAINING ON SLOPING GROUND – STEPPING METHOD, CLINOMETER-FEATURES AND USE, SLOPE CORRECTION

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 rods, should be fixed near the end of each chain length, during the ranging operation.

To chain the line, the leader moves forward by dragging the chain and by taking with him a ranging rod and 10 arrows . The follower stands at the starting station by holding the other end of 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 were 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 drag the chain beyond the station and the follower should hold the zero end of the chain at the last arrow. Then the odd links should be counted.

Chaining on Sloping Ground:

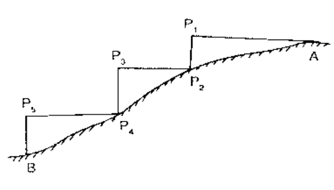
Chaining on the surface of a sloping ground gives the sloping distance. For plotting the surveys, horizontal distances are required. It is therefore, necessary either to reduce the sloping distance to horizontal equivalent or to measure the horizontal distances between the stations directly. The following are the different methods that are generally employed.

a) Direct Method or Stepping Method

b) Indirect Method

Direct Method:

This method is applied when slope of the ground is very steep. In this method, the sloping ground is divided in to a number of horizontal and vertical strips, like steps. So, this method is also known as stepping method. The length of the horizontal portions are measured and added to get the total horizontal distance between the points. The steps may not be uniform, and would depend on the nature of the ground.



Procedure:

Suppose the horizontal distance between points A and B is to be measured.

The line AB is first ranged properly.

Then, the follower holds the zero end of the tape at A.

The leader selects a suitable length AP1 so that P_1 is at chest height and AP_1 is just horizontal.

The horizontal is maintained by eye estimation, by tri-square or by wooden set-square.

The point P_2 is marked on the ground by plumb-bob so that P_1 is just over P_2.

The horizontal length AP_1 is noted then the follower moves to the position P_2 and holds the zero end of the tape at that point.

Again the leader selects a suitable length P_2P_3 in such a way that P_2P_3 is horizontal and P_3P_4 vertical.

Then the horizontal lengths P_2P_3 and P_4P_5 are measured.

So the total horizontal length, $AB = AP_1 + P_2P_3 + P_4P_5$

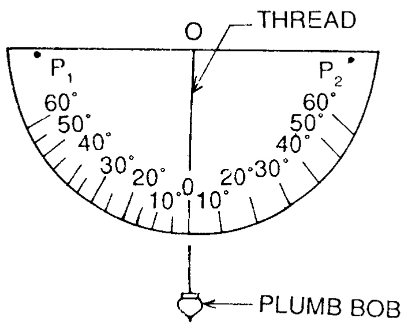
Indirect Method :

When the slope of the ground surface is long and gentle, the stepping method is not suitable. In such a case, the horizontal distance may be obtained by the indirect methods. Those are of following types.

a. By measuring the slope with clinometers.
b. By applying hypotenusal allowance
c. By knowing the difference of level between the points.

a. Measuring slope with a clinometer :

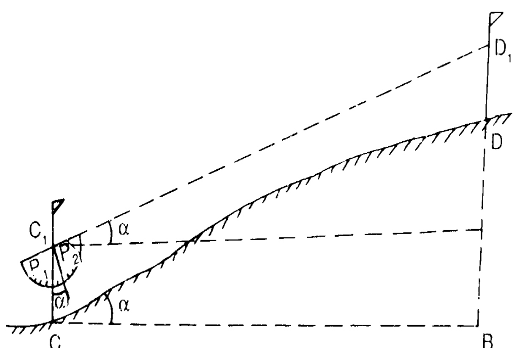
A clinometers is a graduated semicircular protractor. It consists of two pins P_1 and P_2 for sighting the object. A plum bob is suspended from point O with a thread. When the straight edge is just horizontal, the thread passes through 0^0. When the straight edge is tilted, the thread remains vertical, but passes through a graduation on the arc which shows the angle of slope.

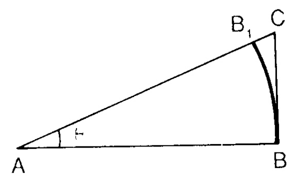


Suppose C and D are two points on sloping ground. Two ranging rods are fixed at these points. Then two other points C1 and D1 are marked on the ranging rods so that CC1 = DD1

The clinometers is placed in such a way that its centre just touches the mark C1. The clinometers is then inclined gradually until the points P1, P2, and D1 are in the same straight line. At this position the thread of the clinometers will show an angle which is the angle of slope of the ground. Suppose this angle is α . The sloping distance CD is also measured.

The required horizontal distance = CB = lcosα





b. Applying hypotenusal allowance

In this method , the slope of the ground is first out by using the clinometers. Hypotenusal allowance is then made for each tape length.

Let θ= angle of slope measured by clinometers

AB = AB1 = 20m = 100 links

AC = AB secθ = 100 sec θ

B1C = AC – AB1

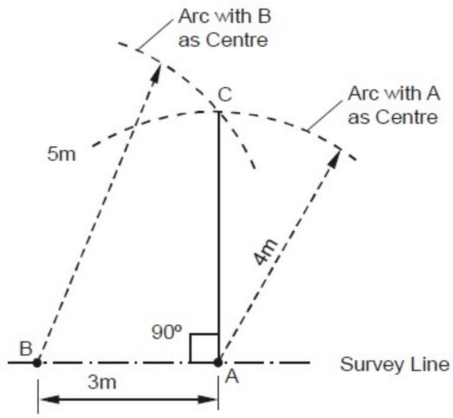
= 100 sec θ - 100

= 100 (sec$\theta - 1$)

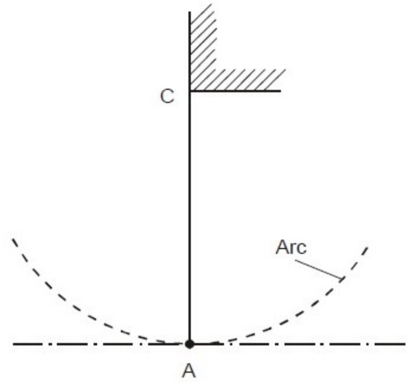
2.4 SETTING PERPENDICULAR WITH CHAIN & TAPE, CHAINING ACROSS DIFFERENT TYPES OF OBSTACLES –NUMERICAL PROBLEMS ON CHAINING ACROSS OBSTACLES.

3-4-5 Offset

Perpendicular offset of chain line at any point A is obtained using the following mathematical expression ($3^2 + 4^2 = 5^2$). A point B is located on chain line at a distance of



(a) 3-4-5 Offset



(b) Swing Offset

3 m from A such that AB = 3 m. Next, an arc is set on ground with centre at A and radius equal to 4 m. Another arc is laid with center at B and radius equal to 5 m intersecting the previous arc at C as shown in Figure (a). Line AC will then be perpendicular to line AB.

Obstacles:

A chain line may be interrupted the following situations:

1. When chaining is free, but vision is obstructed.
2. When chaining is obstructed, but vision is free, and
3. When chaining and vision are both obstructed

1. 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 inter-visible.

Case – I

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

Case – II

The end stations are not visible from intermediate points when jungle are comes across the chain line.

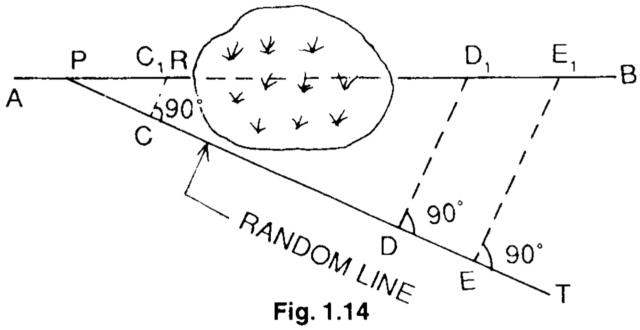


Fig. 1.14

Let **AB** line be the actual chain line which can not be ranged and extended because of interruption by a jungle. Let line extended up to **R**. A point **P** is selected on the chain line and a random line **PT** is taken in a suitable direction. Points **C, D** and **E** are selected on the random line and perpendiculars are projected from them. The perpendicular at **C** meets the line at $\boldsymbol{c_1}$

Theoretically,

$$\frac{DD_1}{PD} = \frac{CC_1}{PC}$$

$$DD_1 = \frac{CC_1}{PC} \text{ x PD} \quad \ldots\ldots\ldots\ldots\ldots \ldots\ldots\ldots \ldots\ldots\ldots\ldots\ldots (1)$$

Again from triangle $PEE_1$and PCC_1

$$\frac{EE_1}{PE} = \frac{CC_1}{PC}$$

$$EE_1 = \frac{CC_1}{PC} \text{ x PE} \quad \ldots\ldots\ldots\ldots\ldots \ldots\ldots\ldots \ldots\ldots\ldots\ldots\ldots (2)$$

From eq 1 and 2, the lengths $\boldsymbol{DD_1}$ and $\boldsymbol{EE_1}$ are calculated. The distance is measured along the perpendiculars at **D** and **E**. Points $\boldsymbol{D_1}$ and $\boldsymbol{E_1}$ should lie in the chain line **AB**

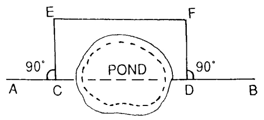
Distance $\boldsymbol{PE_1 = \sqrt{PE^2 + EE_1^2}}$

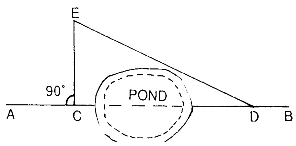
2. Chaining obstructed but vision free:

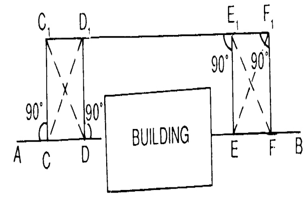
Such a problem arises when a pond or river comes across the chain line. The stations may be tackled in the following ways.

Case – I

When a pond interrupts the chain line, it is possible to go around the obstruction.







CD = EF

CD = $\sqrt{ED^2 + CE^2}$

<u>3. Chaining and vision both obstructed</u> :

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 at one side of the building. Equal perpendiculars CC_1 and DD_1 are erected. The line C_1D_1 is extended until the building is crossed. On the extended line, two points E_1 and F_1 are selected. Then perpendiculars E_1E and F_1F are so erected that

$E_1E = F_1F = D_1D = C_1C$

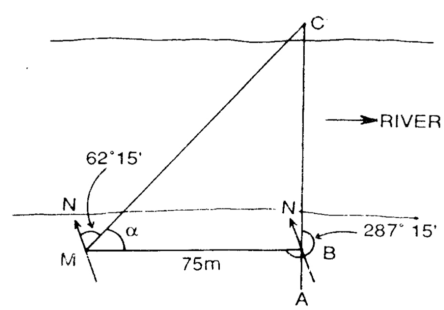
Thus, the points C, D, E and F will lie on the same straight line AB

Here, $DE = D_1E_1$

The distance D_1E_1 is measured , and is equal to the required distance DE.

Problem :

A chain line ABC crosses a river, B and C being on the near and distant banks respectively. The line BM of length 75 m is set out at right angles to the chain line at B. If the bearings of BM and MC are 287^0 15' and 62^0 15' respectively, find the width of the river.



Solution :

<BMC =BB of BM – FB of MC

i.e. $\alpha = (287^0 15' - 180^0 0') - 62^0 15' = 45^0 0'$
From triangle MBC, $\frac{BC}{BM} = tan 45^0 0'$
$BC = BM\ tan 45^0 0' = 75\ m$
So the width of river is 75 m

2.5 PURPOSE OF CHAIN SURVEYING, ITS PRINCIPLES, CONCEPT OF FIELD BOOK. SELECTION OF SURVEY STATIONS, BASE LINE, TIE LINES, CHECK LINES.

The chain surveying is one of the method of land surveying. It is the system of surface in which sides of different triangular are measured directly in the field and no angular measurement are taken

Principle of Chain Surveying:

The principle of chain surveying is triangulation. This means that the area to be surveyed is divided in to a number of small triangles which should be well conditioned. In chain surveying the sides are directly measured by chain or tape.

Chain surveying is recommended when:

1. The ground surface is more or less leveled.
2. A small area is to be surveyed.
3. A small – scale map is to be prepared and
4. The formation of well conditioned triangles is easy

Concept of field book**:**

1. The book in which the chain or tape measurements are entered or sketched of detail points are recorded is called field book.
2. Its size is 20c.m.X 12c.m.
3. The chain line may be represented about 1.5c.m. to 2.0c.m. a part rolled down the middle of each page.
4. The chain line is started from the bottom of page and work up words.
5. It should be well bounded and a size of convenient for the pocket.
6. All distance along the chain line are entered either to the left or to the right of the chain line.
7. The new line should be started from a new fresh page and name of line should be noted at the foot and booking proceeds from the bottom of the page to up wards.
8. At the different feature within the offset are reached, surveyor draw them and enters the chain and length of each offset.
9. Field books may be two types
    I. Single Line
    II. Double Line

Selection of Surveying Stations:

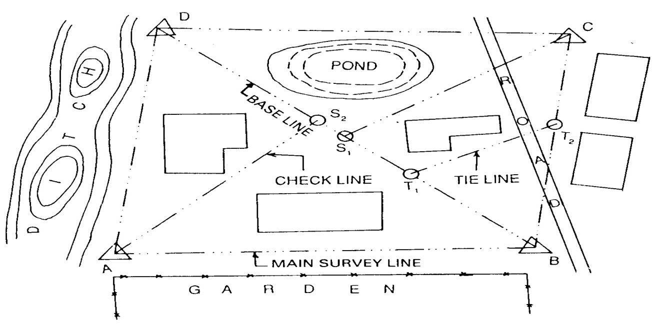
Survey stations are the points at the beginning and the ending of a chain line. The stations are classified under 3 categories

i.e - (a) Main Station
(b) Subsidiary Station
(c) Tie Station

1. Main survey station at the end of chain line should be inter-visible.
2. Survey line should be minimum as possible.
3. The main principle of surveying such as working from whole to part and from part to whole.
4. The stations should be well conditioned triangle.
5. Every triangle should be provided with a check line.
6. Tie line should be provided to avoid too long offsets.
7. Obstacles to ranging and changing if any should be avoided.

The larger side of the triangle should be placed parallel to the boundaries, roads, buildings, etc. to have short offsets.

1. Chain line should be lie over leveled ground.
2. Line should be laid on one side of the road to avoid disturbance of chaining by passing of traffic.



INDEX SKETCH

Base line:

The line on which the frame work of the survey is built is known as the "Base line". It is the most important line of the survey work. Generally, the longest of the main survey line is considered as base line.

Tie line:

The tie line is a line which joins subsidiary stations on the main line.

Check line:

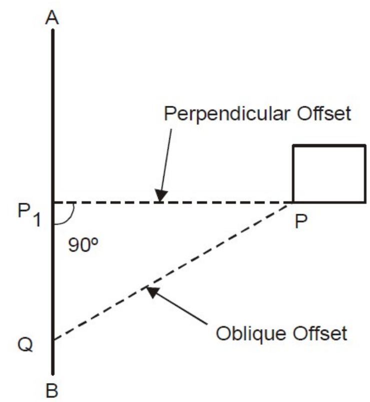
The line joining the apex point of triangle to some fixed point on its base is known as check line. It is taken to check the accuracy of the triangle.

2.6 OFFSETS – NECESSITY, PERPENDICULAR AND OBLIQUE OFFSETS, INSTRUMENTS FOR SETTING OFFSET – CROSS STAFF, OPTICAL SQUARE.

Necessity

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 are two types:

I. Perpendicular Offset
II. Oblique Offset



Perpendicular Offset:

When the lateral measurement for fixing the detail points are maid perpendicular to the chain line. The offsets are known as perpendicular offset.

Oblique Offset:

When the lateral measurement for fixing the detail points are maid at any angle to the chain line. The offsets are known as oblique offset. It can be done by following two(2) process

i.e -

a. Long offset
b. Short offset

Instruments for setting offset

Optical Square:

1. It is a most suitable instrument for setting out a line at a right angle to another line.
2. It consists of a circular metal box about 5c.m. in diameter and 1.25c.m. in deep. It consists of two inclined mirror at an angle of 45^0.
3. The upper glass is known as ***horizontal*** glass and the lower end glass is known as ***index*** glass.

Principle:

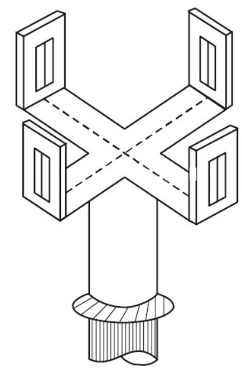
If the two mirror's are inclined with the surface at an angle of 45^0. The plane is successfully reflected under deviation of twice the angle.

Uses:

1. It is used to find out foot of perpendicular to the chain line.
2. To set out a perpendicular to a chain line.

Cross staff:

The cross-staff consists of four metal arms with vertical slits. The two pairs of arms are at right angles to each other. The vertical slits are meant for sighting the ranging rods. The cross-staff is mounted on a wooden pole of length 1.5m. and diameter 2.5c.m. The pole is fitted with an iron shoe.



Cross Staff

2.7 ERRORS IN CHAIN SURVEYING – COMPENSATING AND ACCUMULATIVE ERRORS CAUSES & REMEDIES, PRECAUTIONS TO BE TAKEN DURING CHAIN SURVEYING.

ERRORS IN CHAIN SURVEYING:

Errors in chaining may be caused due to variation in temperature and pull, defects in instruments, etc. They may be either;

1. Compensating Error
2. Cumulative Error

Compensating Error:

Errors which may occur in the both directions (i.e. both positive and negative) and which finally tend to compensate are known as compensating errors. They are proportional to $\sqrt{L}$, where $\sqrt{L}$ - is the length of the line. Such error may be caused by

1. Incorrect holding f the chain.
2. Inaccurate measurement of right angles with chain, tape.
3. Horizontality and verticality of steps not being properly maintained during the stepping operations.
4. Fractional parts of the chain or tape not being uniform throughout its length

Cumulative Error:

Errors which may occur in the same direction and which finally tend to accumulative. They seriously affect the accuracy of work, the length of the line (L).

Positive Error: when the measured length is greater than the actual length,(the chain length is too short), the error is said to be positive error. Such error 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 Error: When the measured length is less than the actual length,(the chain length is too long), the error is said to be negative . These errors occur when length of chain or tape is greater than the standard length due the following reasons :

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

Precautions against Error:

Following are the precautions should be taken to guard against errors and mistakes.

1. The point where the arrow 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. The chain man should call the measurement loudly and distinctly and the surveyor should repeat them while booking.
4. During chaining , the number of arrows carried by the follower and leader should always tally with the total numbers of arrows taken.
5. Measurements should not be taken with tape in suspension in high wind.
6. In stepping operations, horizontality and verticality should be properly maintained.
7. Ranging should be done accurately.
8. No measurement should be taken with the chain in suspension.
9. Care should be taken so that the chain is properly extended.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 What are the types of chain? [2016-W,2017-W,2018-W]

- Metric chain
- Steelband chain
- Engineers chain
- Gunter's chain
- Revenue chain

Q-2 Where and why brass rings and tallies are provided in a chain? [2011-W,2019-S]

Ans:

- Small brass rings are provided at every mtr length except at 5 m , 10 m,15 m , 20m, 25 m. where tallies are fixed.
- The tallies used for marking 5 m to 10 m etc. are marked with letter M to distinguish matric chain from a non metric chain.

Q-3 What are the types of tape? [2011-W]

Ans:

Cloth tape

Metallic

Steel tape

Invar tape

Q-4 Write down the formula for correction for pull? [2019-S]

Ans-the correction is necessary when the pull used during measurement is different from that at which the tape is standardized.

The correction for pull=

C1=$\frac{(P-Po)xL}{AE}$

POSSIBLE LONG TYPE QUESTIONS

Q-The length of a line measure with a 30 mtr chain was found to be 380 m. the true length of line was known to be 381.5 m find the error in the chain. [2015-W]

Q-2 Explain the sources of error in chaining. [2018-W]

Q-3 Describe briefly ranging across a high ground. [2018-W]

Q-4 Describe with a sketches the construction and use of cross staff? [2019-S]

CHAPTER NO-3

ANGULAR MEASUREMENT AND COMPASS SURVEYING

Learning objectives

3.1 Measurement of angles with chain, tape & compass
3.2 Compass – Types, features, parts, merits & demerits, testing & adjustment of compass
3.3 Designation of angles- concept of meridians – Magnetic, True, arbitrary; Concept of bearings – Whole circle bearing, Quadrantal bearing, Reduced bearing, suitability of application, numerical problems on conversion of bearings
3.4 Use of compasses – setting in field-centering, leveling, taking readings, concepts of Fore bearing, Back Bearing, Numerical problems on computation of interior & exterior angles from bearings.
3.5 Effects of earth's magnetism – dip of needle, magnetic declination, variation in declination, numerical problems on application of correction for declination.
3.6 Errors in angle measurement with compass – sources & remedies.
3.7 Principles of traversing – open & closed traverse, Methods of traversing.
3.8 Local attraction – causes, detection, errors, corrections, Numerical problems of application of correction due to local attraction.
3.9 Errors in compass surveying – sources & remedies. Plotting of traverse – check of closing error in closed & open traverse, Bowditch's correction, Gales table

3.1 MEASUREMENT OF ANGLES WITH CHAIN, TAPE & COMPASS

TRIANGULATION

Because, at one time, it was easier to measure angles than it was distance, triangulation was the preferred method of establishing the position of control points. Many countries used triangulation as the basis of their national mapping system. The procedure was generally to establish primary triangulation networks, with triangles having sides ranging from 30 to 50 km in length. The primary trig points were fixed at the corners of these triangles and the sum of the measured angles was correct to ±3. These points were usually established on the tops of mountains to afford long, uninterrupted sight lines. The primary network was then densified with points at closer intervals connected into the primary triangles. This secondary network had sides of 10– 20 km with a reduction in observational accuracy.

3.2 COMPASS – TYPES, FEATURES, PARTS, MERITS & DEMERITS, TESTING & ADJUSTMENT OF COMPASS

The compass works on the principle that a freely suspended magnetic needle takes the direction of the magnetic lines of force at a place. This provides us a reference direction with respect to which all angles can be measured.

There are two types of compasses

1. The prismatic compass
2. The surveyor's compass.

The surveyor's compass is rarely used in comparison purposes. The principle of the operation of both the compass is the same but they are made differently used in the field

1) The prismatic compass.

It is the most suitable type of surveying compass which consists of a circular box about 100 mm in diameter.

It can be used as a hand instrument or on a tripod.

It can be accurately centered over ground station marks.

The main parts of a prismatic compass is as follows

Magnetic Needle:

The magnetic needle is the most important of the measurement. The needle, generally of the board form, is supported on a hard, steel pivot with an agate tip. When not in use, the needle can be lifted off the pivot, by a lifting needle, actuated by the folding of the objective vane. This is done to on sure that the pivot tip is not subjected to undue wear. The magnetic needle should be perfectly symmetrical and balanced at its midpoint on the hard pointed pivot. It should be weighted with an adjustable weight to compensate for the dip angle. The needle should be sensitive and take up the north-south direction speedily. The needle should lie in the same horizontal plane as the pivot point, and a vertical plane should be made in such a way that the centre of gravity of the needle lies as much below the pivot point as possible.

Graduated ring:

An aluminum graduated ring 85 to 110 mm diameter is attached in the needle on its top a diametrical arm of the ring. Aluminum, being a non- magnetic substance, is used to ensure that the ring does not influence the behavior of the needle. The graduation of the ring is from 0 to 360°. 0°/360° is marked on the south end of the needle and the graduation go in a clockwise direction., with 90° marked on the west, 180 on the north, and 270° on the east directions. The graduations are marked to half degrees, but it is possible to read the angle as per least count.. The graduations on the ring are inverted as they are to be read by a prism.

Eye vane prism:

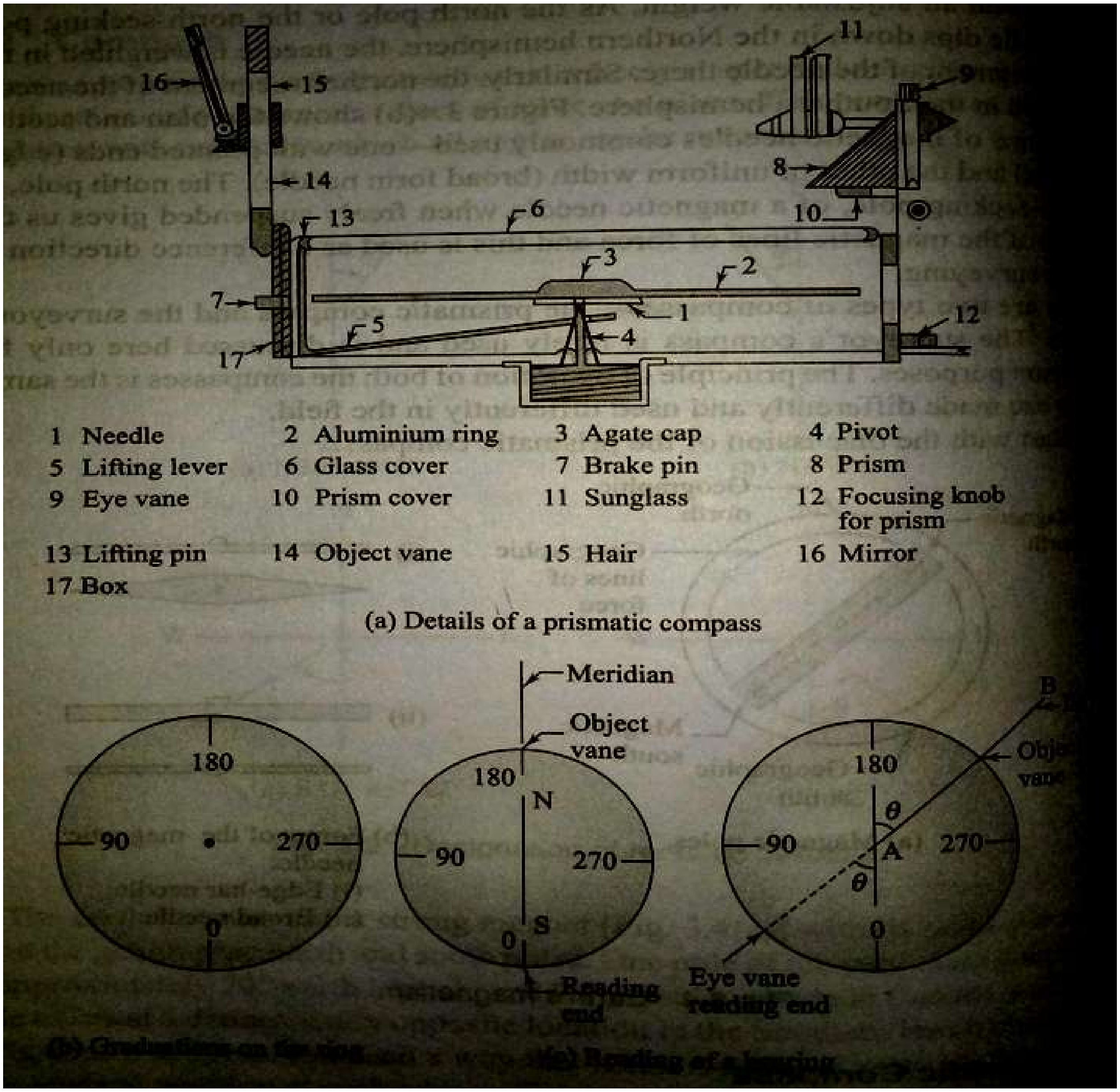
The point on the prismatic compass from where the straighting is done is known as the eye vane, which is made up of a rectangular frame to the graduated ring when it is folded over the glass plate cover of the compass. The prism has convex surfaces, which magnify the graduations on the ring. A metal cover is used to cover the reading face of the prism when it is not in use. The prism can be raised or lowered on the metal frame for adjusting to the eye of the observer. Dark glasses may be provided on the frame, which can be brought in view while shiting bright objects to reduce glare.

Object vane:

Diametrically opposite the eye vane the object vane, which is a metal frame hinged at the bottom for folding over the glass cover when it is not in use. A fine silk thread or hair is shifted on the frame vertically, which can be used to bisect a ranging rod or the hair id fitted on the frame vertically, which can be used to bisect a ranging rod or other objects. When the frame is folded over the glass cover, it pressing against a pin, which actuates the lifting lever of the needle and lifts the needle off the pivot. Also fitted below this frame on the box is a brake pin, which, when, gently passed, stops the oscillation of the needle by pressing agains the graduated aluminium ring. The object vane may be provided with mirrors, which can be moved over the frame for sighting objects at a height or far below.

Compass Box:

The needle and other fittings are enclosed in a metal box with a glass cover to prevent dust. The two vanes are also attached to the box at diametrically opposite ends. The box is attached to a metal plate through a ball and socket arrangement for leaving the compass. While the compass may also be used by holding it in the hand, it is preferable to use it with a tripod, for which the metal plate has a screwed end that can be attached to a tripod. The compass box can be carried in a leather pouch when not in use.



(a) Details of a prismatic compass

Use of Prismatic Compass:

The following steps are required in using prismatic compass.

1. Setting up and centering screw the prismatic compass onto the tripod and place the tripod over the station. it is centered over the tripod. Centering is done by adjusting the tripod legs.
2. Level the compass using the ball and socket arrangement. Levelling is done approximately so that the needle can move freely in a plane, after opening the objective and eye vanes.
3. Open the object vane and eye vane see that needle moves freely. Direct the object vane towards the ranging rod or any other objects at the next station. Sighting is done by bisecting the object with the cross hair on the object vane while looking through the eye vane. The prism

of the eye vane has to be adjusted for a clear view of the graduations by moving it up of down. It is clear that the graduated ring along with the attached needle always points to the north direction while the box is rotated with the vanes. The line of straight between the stations is through the eye vane and the cross hair of the object vane and should pass through the centre of the pivot.

4. Once the object has been clearly sighted, damp the oscillation of the needle with the breaking pin if required. Once the object has been pin if required. Once the needle comes to rest, looking through the prism, record the reading at the point on the ring corresponding to the vertical hair seen directly through the slit in the prism holder.

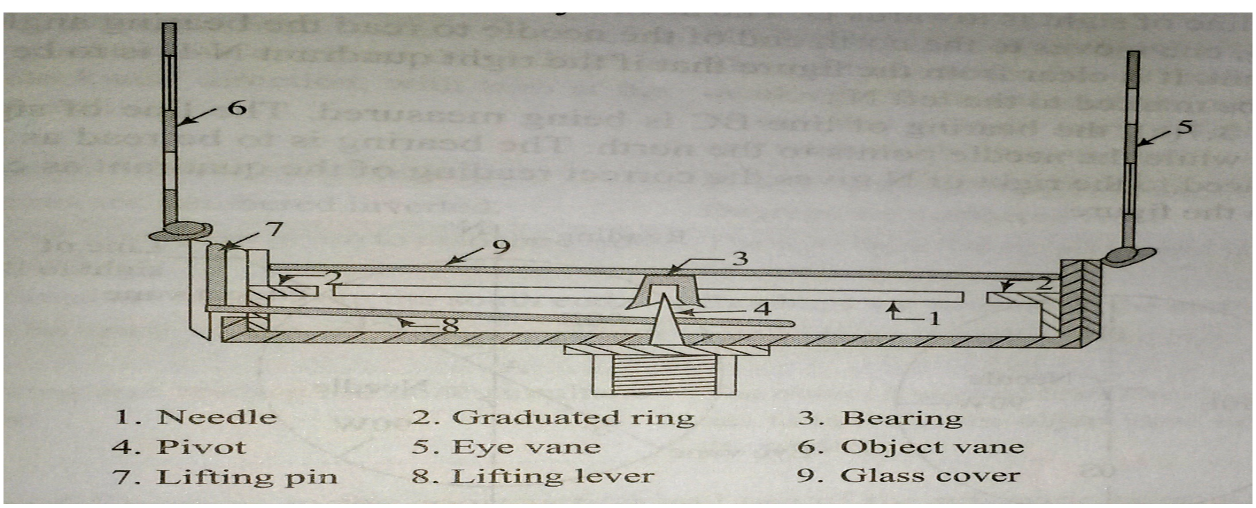
Graduation on ring:

It is clear from the graduations that the prismatic compass gives the WCBs of the lines. The reading taken through the prism has to be zero when the lines. The reading taken through the prism has to be zero when the line of sight is pointing to the north. The reading end is the south end of the needle. Therefore, the zero graduation is marked at the south end .

Temporary adjustments:

At every station where prismatic compass is placed, The following adjustments, as described above, have to be made: centering leveling, and focusing the prism. The prism has to be focused once if the same person has to take the prism. The prism has to be focused only once if the same person has to take the readings. Centering is done by adjusting only the legs to bring the compass exactly over the station. Leveling is done to ensure that as the compass is rotated it moves very nearly in a horizontal plane and the needle moves freely.

Surveyor's Compass



The surveyor's compass is an old type of instrument finding rare use today. A brief description of the instrument is given below. The surveyor's compass has the following components.

Magnetic needle: The edge bar magnetic needle rests on a pivot of hard metal and floats freely.

Graduation ring:

The graduated ring is not attached to the needle but to the cover box of the compass and inside it. The graduations are in the quadrennial system. The letters N, W, S, and E are marked on the ring along with graduations from 0° to 90° in each quadrant. The graduations are marked to Half-degrees but can be read to one-fourth of a degree by judgement.The E and W half-degrees but can be read in the ring. The moves with the compass as the box is roated for sighting, the needle pointing to the north always.

Object vane and eye vane:

The object vane consists of a fine thread or hair fitted onto a metal frame for sighting objects. The eye vane is a similar frame with a fine slit but has no prism to read the graduations.

Base and tripod:

The surveyor's compass cannot be used without a tripod. A base with a ball and socket arrangement and a screwing end for the tripod is used.

An arrangement for lifting the needle off the pivot is provided. This is actuated when the object vane is folded onto the cover glass.

Uses of surveyor's compass:

The following steps are required.

1. Attach the compass box to the tripod. Place the tripod over the station and centre and level the instrument.
2. Rotate the instrument to bring the object vane in line with the ranging rod at the adjacent station. Looking through the eye vane, finely bisect the ranging rod.
3. Note the reading, by going around to the objective vane side, at the north end of the needle by looking through the glass. Take the reading along with the quadrant by nothing down the letters on either side of the reading.

Graduation on ring:

Fig explain the graduations on the ring. N and S are marked along the north-south direction. E and W are marked along the east-west direction but their positions are interchanged, with E marked to the left of N and W to the right of the N. This is done to ensure that the correct quadrant is noted when the reading is taken at the north end of the needle.

Testing and Adjustment of compass

Adjustments of Prismatic Compass

In this article, we will be discussing step by step method for making Temporary adjustments in Prismatic Compass necessary for carrying out Compass Surveys.

1. Fixing the compass to thr tripod
2. Centering the compass
3. Levelling the compass
4. Sighting the object
5. Observation of bearings

Fixing the compass to the tripod

The box of prismatic compass is fixed to a spindle of ball and socket joint. By the ball and socket arrangement, this can be quickly levelled and rotated in any direction.

Centering the compass

The prismatic compass is centered over a survey station correctly by means of a plumb bob or by dropping a pebble from the centre of the instrument.

Levelling the compass

The compass is quickly levelled by ball and socket arrangement by eye judgement.

It should be levelled in such a way that dial moves freely and does not touch the rim of the bob.

Sighting the object

The object is sighted with the help of eye vane and object vane in the compass.

The surveyor views through the eye vane and rotate the box until the ranging rod at a station is bisected.

Observation of Bearing

After citing the object correctly, the bearing of the survey lines are noted through prism at which the line of sight and object cuts the image of the graduation on the dial.

3.3 DESIGNATION OF ANGLES- CONCEPT OF MERIDIANS – MAGNETIC, TRUE, ARBITRARY; CONCEPT OF BEARINGS – WHOLE CIRCLE BEARING, QUADRANTAL BEARING, REDUCED BEARING, SUITABILITY OF APPLICATION, NUMERICAL PROBLEMS ON CONVERSION OF BEARINGS

Meridians:

The fixed direction on the surface of the earth with reference to which bearings of survey lines are expressed is called as Meridians .

Bearing:

The horizontal angle between the reference meridian and the survey line measured in a clockwise direction is call bearing.

There are four different types of meridians which can be used as reference directions.

True meridian:

The true or geographic meridian at a point is the line of intersection of a plane passing through the north and south poles and the point with the surface of the earth. Since the earth is approximately a sphere, it is clear that the meridians through different points meet at the north and south poles. The true meridians through different points are not parallel. The true meridian at a place can be established through astronomical observations. The direction of the true meridian remain constant. If the magnetic bearing of the sun is taken at noon, the location of the true meridian at the point can be found. The sun is taken at noon is on a plane passing through the north and south poles at a place. The true bearing of survey line is the horizontal angle that line makes with the true meridian passing through one of its ends.

Magnetic meridian:

The magnetic meridian through a point on the ground is the direction taken by a freely suspended magnetic needle placed at that point. The magnetic meridian can be affected by any serious magnetic interference. Such as an overhead electric cable or the presence of magnetic substance, such be explained later. The magnetic bearing of a survey line is the horizontal angle compass measures the magnetic bearing of a line.

Grid Meridian :

State survey maps are based on one or more true meridians of places so that they placed centrally.The northsouth lines of the grid are parallel to the line representing the central meridian.The direction of the grid lines along the north-south directin is known as Grid Meridian. The bearing of survey lines referred to and reckoned from grid lines are called Grid Bearing

Arbitrary meridian:

The arbitrary meridian at a point is any well-defined direction between any two points, such as the spire of a church, a well-defined point on the ground, or a tower. Such meridians can be used for local surveys as they will serve the purpose of a reference direction, and the required computations are possible with such data. The arbitrary bearing of a line is the horizontal angle between the line direction of the direction of the arbitrary meridian through one end of the line.

Designation of Bearings:

The bearing of survey lines are designated in the following systems

1. Whole Circle bearing system (W.C.B)
2. Quadrental bearing system (Q.B)

1.Whole circle bearing system (WCB):

In this system of bearing of a line measured from the true north or magnetic north in clockwise direction. The value of bearing may vary from 0 to 360.It is also known as Azimuthal System.

2.Quadrental Bearing system (WCB):

In this system of bearing of a line measured eastward or westward from the north or south which ever is nearer .In this system both North and South direction are used as referencemeridians.The bearings are measured either clockwise or anticlockwise depending upon the position of the survey line .It is also called Reduced Bearing.

Conversion of bearings:

If the WCBs are given, convert them to quadrental or reduces bearings. Similarly, QBs can also be converted to WCBs.

Whole circle bearing to reduced bearing:

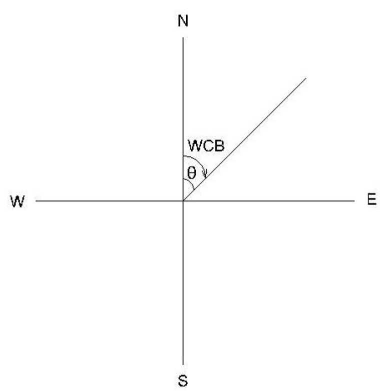
To convert WCB (measured clockwise from the north direction) to RBs, the following simple rules are followed.

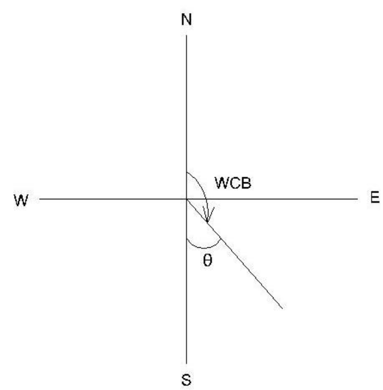
(a) If the WCB is less than 90°, the RB is numerically equal to the WCB. The quadrant designation is N-E.

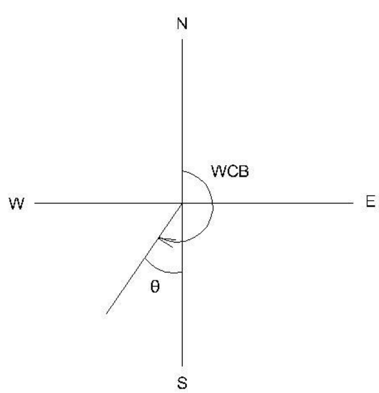
(b) If the WCB is between 90° and 180°, the RB is equal to 180° - WCB. The quadrant designation is S-E.

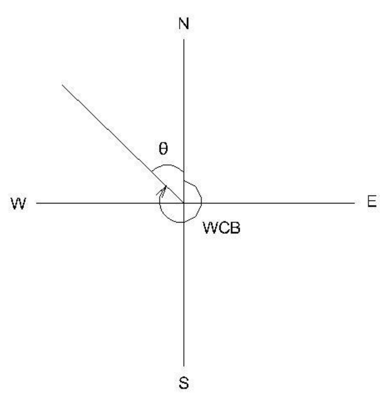
(c) If the WCB is between 180° and 270°, the RB is equal to WCB _ 180°. The quadrant designation is S-W.

(d) If the WCB is between 270° and 360°, the equal to 360° _ WCB. The quadrant designation is N-W.









Quadrantal bearing to whole circle bearing:

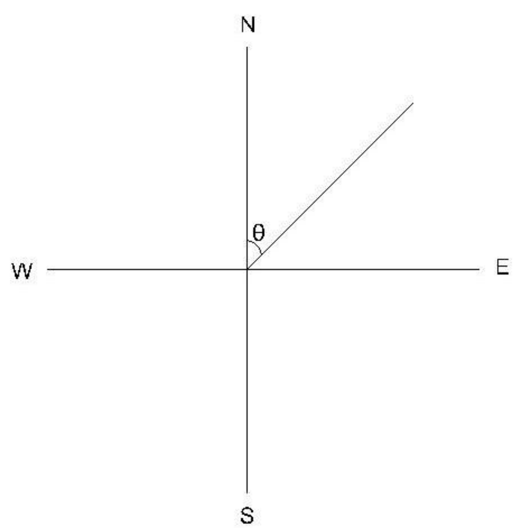
To convert given QBs to WCB, the following simple rules are to be followed.

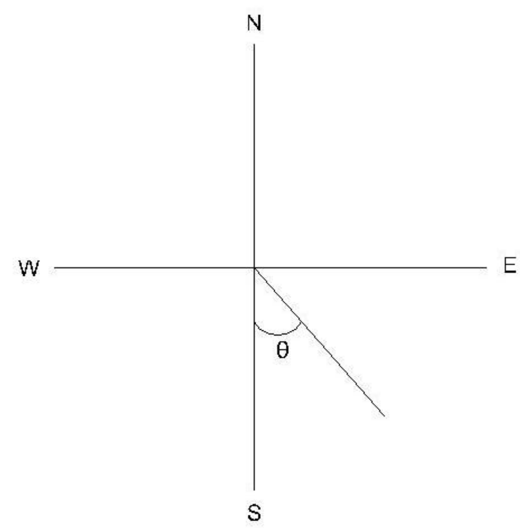
(a) If the quadrant designation is N-E, the WCB is numerically equal to the RB.

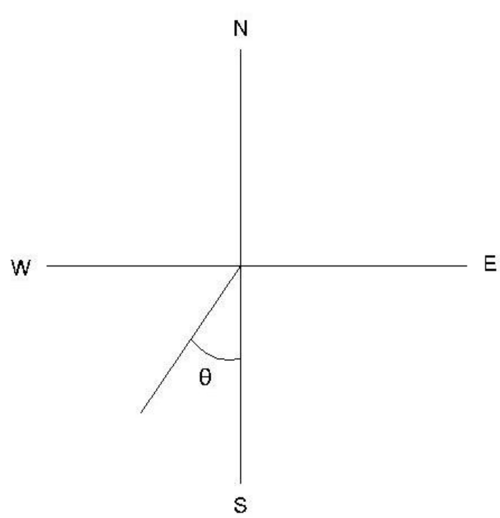
(b) If the quadrant designation is S-E, the WCB is equal to 180°-QB.

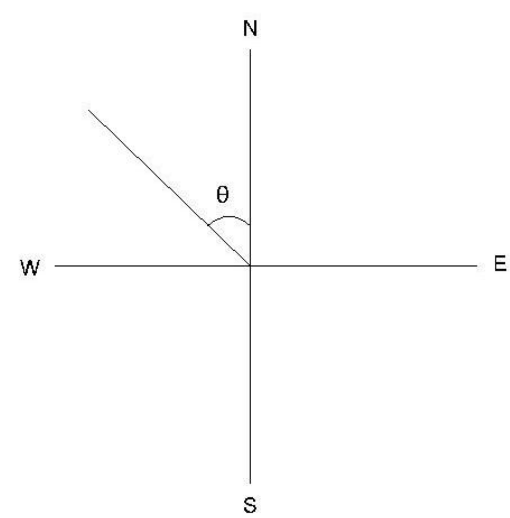
(c) If the quadrant designation is S-W, the WCB is equal to 180+QB.

(d) If the quadrant designation is N-W, the WCB is equal to 360°-QB.









Example : Convert the following WCBs to RBs and RBs to WCBs.

a) 187°30', 48°15' , 295° 0 , 126° 30'
b) N30°30'W, S45°15'E, S38°15'W, N49°30'E.

Sol.:

a)

187°30' This lies in the S-W quadrant. RB== 187°30'-180°= S7°30'W
48°15' lies in the N-E quadrant. RB=N 48°15' E
295° 00' , this lies in the N-W quadrant. RB=360°-29°5= N65°00'W
126° 30' this lies in the S-E quadrant. RB=180°-126° 30' = S 53°30'E

b)

N30°30'W This lies in the N-W quadrant. WCB= 360°00'- 30°30'= 329°30'
S45°15'E, This lies in the S-E quadrant. WCB= 180°00'- 45°15' = 134°45'
S38°15'W, This lies in the S-W quadrant. WCB= 180°00'+S38°15' = 218°15'
N49°30'E. This lies in the N-E quadrant. WCB = 49°30'

Fore and Back Bearings:

Fore Bearing :

The bearing of a line in the direction of progress of the survey is called Fore or forward Bearing(FB).

Back Bearings:

The bearing of a line in the opposite direction of progress of the survey is called Back d Bearing(BB).

The relation between the FB& BB is

Back Bearing= Fore Bearing ± 180°

Use + sign if FB is less than 180° & Use - sign if FB is greater than 180°

If the fore bearing is given, in the Quadrantal System ,the back bearing is equal to the fore bearing but the designating letters will be exactly opposite. N will be changed to S and vice versa and E will be changed to W and vice versa.

3.4 USE OF COMPASSES – SETTING IN FIELD-CENTERING, LEVELING, TAKING READINGS, CONCEPTS OF FORE BEARING, BACK BEARING, NUMERICAL PROBLEMS ON COMPUTATION OF INTERIOR & EXTERIOR ANGLES FROM BEARINGS.

The following steps are required in using compass.

1. Setting up and centering screw the prismatic compass onto the tripod and place the tripod over the station. it is centered over the tripod. Centering is done by adjusting the tripod legs.
2. Level the compass using the ball and socket arrangement. Levelling is done approximately so that the needle can move freely in a plane, after opening the objective and eye vanes.

3. Open the object vane and eye vane see that needle moves freely. Direct the object vane towards the ranging rod or any other objects at the next station. Sighting is done by bisecting the object with the cross hair on the object vane while looking through the eye vane. The prism of the eye vane has to be adjusted for a clear view of the graduations by moving it up of down. It is clear that the graduated ring along with the attached needle always points to the north direction while the box is rotated with the vanes. The line of straight between the stations is through the eye vane and the cross hair of the object vane and should pass through the centre of the pivot.
4. Once the object has been clearly sighted, damp the oscillation of the needle with the breaking pin if required. Once the object has been pin if required. Once the needle comes to rest, looking through the prism, record the reading at the point on the ring corresponding to the vertical hair seen directly through the slit in the prism holder.

Concept of fore bearing and back bearing

Fore and Back Bearings:

Fore Bearing :

The bearing of a line in the direction of progress of the survey is called Fore or forward Bearing(FB).

Back Bearings:

The bearing of a line in the opposite direction of progress of the survey is called Back d Bearing(BB).

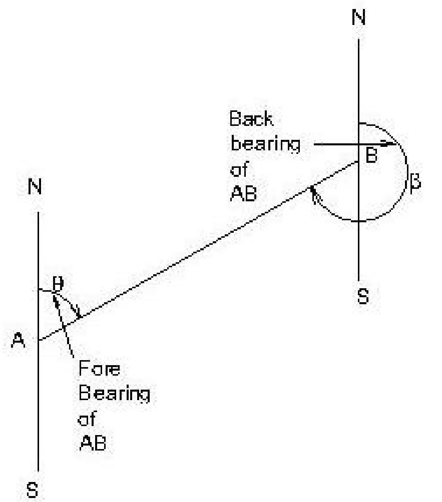
The relation between the FB& BB is

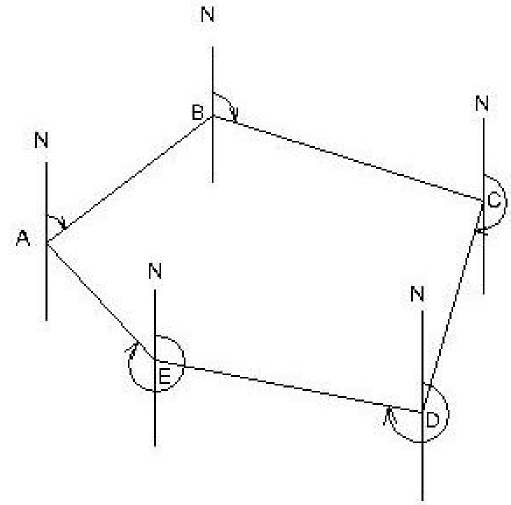
Back Bearing= Fore Bearing ± 180°

Use + sign if FB is less than 180° & Use - sign if FB is greater than 180°

If the fore bearing is given, in the Quadrantal System ,the back bearing is equal to the fore bearing but the designating letters will be exactly opposite. N will be changed to S and vice versa and E will be changed to W and vice versa.

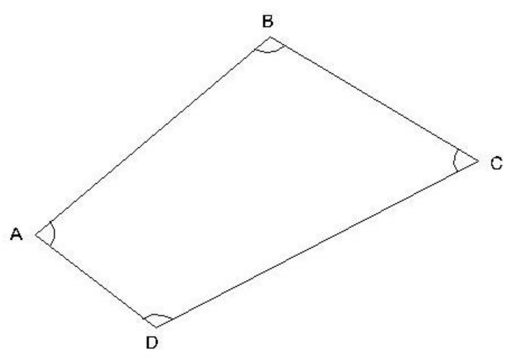
Numerical problems on computation of interior and exterior angle

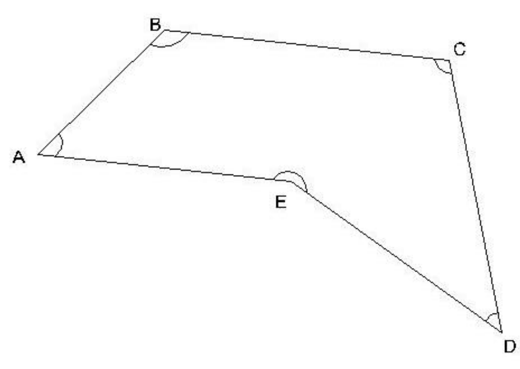




Calculation of Included Angles from Bearings

At the point where two survey lines meet, two angles are formed – an exterior angle and an interior angle. The interior angle or included angle is generally the smaller angle (<180°) . the difference of bearing of two adjacent lines is the included angle measured clockwise from the line whose bearing is less.





Calculation of Bearings from Included Angles.

In order to calculate the bearing of the next line the following statement may be made.

Add the included angle measured clockwise to the bearing of the previous line . If the sum is :

More than 180^o, deduct 180^o

More than 540^o,deduct 540^o

Less than 180^o,add 180^o ,to get the bearing of the next line.

Note :

- In a closed traverse run in anticlockwise direction, the observed included angles are interior angles.
- In a closed traverse run in clockwise direction, the observed included angles are exterior angles.

Example : Find the included angle between lines AB and AC ,if their reduced bearing are

i)	AB N40°10'E	ACN89°45'E
ii)	AB N10°50'E	ACS40°40'E
iii)	AB S35°45'W	ACN45°20'E
iv)	AB N30°25'E	ACN30°25'W

Bearings of AB = N 40^0 $10^/$ E; Bearing of AC = N 89^0 $45^/$ E
both lines lie in NE quadrant.
Included angle BAC = difference in the bearings = $89^0\,45^/ - 40^0\,10^/ = 49^0\,35^/$. **Ans.**

(ii)

Bearing of AB = N 10^0 $50^/$ E; Bearing of AC = S 40^0 $40^/$ E
lines lie in adjacent quadrants.

Included angle BAC = 180^0 –sum of the bearings= $180^0 – (10^0\ 50^/ + 40)^{/\,=}\ 128^0\ 30^/$

(iii)

Bearing of AB = S 35^0 45' W
Bearing of AC = N 45^0 $20^/$ E
The lines lie in opposite quadrants,

Included angle CAB = 180^0 – (difference in bearings)= $180^0 – (45^0\ 20^/ - 35^0\ 45^/)$
$=170^0\ 25^/$.

(iv)

Bearing of AB = N 30^0 $25^/$ E
Bearing of AC = N 30^0 $25^/$ W
The lines lie in adjacent quadrants.
The Included angle CAB = sum of the bearings=$30^0\ 25^/ + 30^0\ 25^/ =60^0\ 50^/$.

Example 2

The bearings of the sides of a closed transverse ABCDEA are as follow :

Side F.B. B.B.

AB	$107^0 15^/$	$287^0 15^/$
BC	$22^0 00^/$	$202^0 00^/$
CD	$281^0 30^/$	$101^0 30^/$
DE	$181^0 15^/$	$1^0 15^/$
EA	$124^0 45^/$	$304^0 45^/$

Compute the interior angles of the traverse and exercise necessary checks.,

Solution:

(i) The included angle A = The difference in bearings of AB and AE.

As the bearing of AB is less than of AB, add 360^0.

Included angle A= $107^0\ 15^/ + 360^0 - 304^0 45^/ = 162^0 30^/$.

The included angle at B= The difference in bearings of BC and BA

$= 22^0\ 00^/ + 360^0 - 287^0 15^/$

Included angle B = $94^0\ 45^/$.

The included angle at C= The difference in bearings of CD and CB

$= 281^0\ 30^/ - 202^0 00^/ = 79^0 30^/$

Included angle C = $794^0\ 45^/$.

The included angle at D= The difference in bearings of DE and DC

$= 181^0\ 15^/ - 101^0 30^/ = 79^0 45^/$

Included angle D = $79^0 45^/$.

The included angle at E= The difference in bearings of EA and ED

$= 124^0\ 45^/ - 1^0 15^/ = 123^0 30^/$

. Included angle E = $123^0\ 30^/$. **Ans.**

Check :

Sum of the included angles of a pentagon

= (2x5-4) = 6 right angles.

And, sum of the included angles A+B+C+D+E

$= 162^0 30^/ + 94^0 45^/ + 79^0 30^/ + 79^0 45^/ + 123^0 30^/$

$= 540^0 00^/$ or 6 right angles Hence ,O.K.

Example

A closed compass traverse ABCD was conducted round a lake and the following bearings were obtained. Determine which of the stations are suffering from local attraction and give the values of the corrected bearings:

AB	$74^0 20^/$	$256^0 0^/$
BC	$107^0 20^/$	$286^0 20^/$
CD	$224^0 50^/$	$44^0 50^/$
DA	$306^0 40^/$	$126^0 00^/$

Solution:

On examination the fore and back bearings of CD differ exactly by 180^0. Hence, stations C and D are free from local attraction. Stations affected by local attraction are A and B.

Calculation of included angles:

Interior angle at A = bearing of AD – bearing of AB

$= 126^0\ 00' - .74^0\ 20' = 51^0\ 40'$

Exterior angle A = $360^0 - 51^0\ 40' = 308'\ 20'$

Interior angle at B = bearing of BA – bearing of BC

$256^0\ 0' - 107^0\ 20' = 148^0\ 40'$

Exterior angle at B = $360^0 - 148^0\ 40' = 211^0\ 20'$

Interior angle at C = bearing of CB – bearing of CD

$= 286^0\ 20' - 224^0\ 50' = 61^0\ 30'$

Exterior angle at C = $360^0\ 00' - 261^0\ 30' = 298^0\ 30'$

Exterior angle D = bearing of DA – bearing of DC

$= 306^0\ 40' - 44^0\ 50' = 261^0\ 50'$

Check : Sum of exterior angles of the quadrilateral ABCD

(2x4+4) = 12 right angles. O.K.

Total sum of exterior angles

$= 308^0\ 20' + 211^0\ 20' + 298^0\ 30' + 261^0\ 50'$

$= 180^0$ = 12 right angles. O.K.

Calculation of bearing :

Bearing of CD	$224^0\ 50'$	(given)
Add angle at D	$= + 261^0\ 50'$	
Sum	$= 486^0\ 40'$	
Sum is more than 180^0, subtract	$=(-)\ 180^0\ 00'$	
. Bearing of DA	$= 306^0\ 40'$	
Add angle at A	$= +308^0\ 20'$	
	$= 615^0\ 00'$	
Sum is more than 540^0, subtract	$= (-)\ 540^0\ 00'$	
. Bearing of AB	$= 75^0\ 00'$	
Add traverse angle at B	$+ 211^0\ 20'$	
Sum	$= 286^0\ 20'$	
Sum is more than 180^0, subtract	$- 180^0\ 00'$	
. Bearing of BC	$= 106^0\ 20'$	
Add traverse angle at C	$+ 298^0\ 30'$	
Sum	$= 404^0\ 50'$	
Sum is more than 180^0, subtract	$- 180^0\ 00'$	

Bearing of CD = $224^0 50^/$ checked

Result: Corrected bearings of the lines are:

Side	FB	BB
AB	$75^0 00^/$	$225^0 0^/$
BC	$106^0 20^/$	$286^0 20^/$
CD	$224^0 50^/$	$44^0 50^/$
DA	$106^0 40^/$	$126^0 40^/$

3.5 EFFECTS OF EARTH'S MAGNETISM – DIP OF NEEDLE, MAGNETIC DECLINATION, VARIATION IN DECLINATION, NUMERICAL PROBLEMS ON APPLICATION OF CORRECTION FOR DECLINATION.

The earth behaves like a strong magnet with its poles placed away from the geographic north and south poles. One pole of the earth's magnet is placed at approximately 70° north latitude and 96° west longitude in Canada and similar pole exists at a diametrically opposite location in the Southern hemisphere. A magnetic needle supported in such a way that it can rotate in a vertical plane will take up a vertical position at such a place. Since one end of a magnetic needle points to the north direction and is designed as the north pole of the needle, it is clear that the imaginary magnet inside the earth has its south pole there. This is because unlike poles attract each other. The north pole of a magnet is strictly the north-seeking pole.

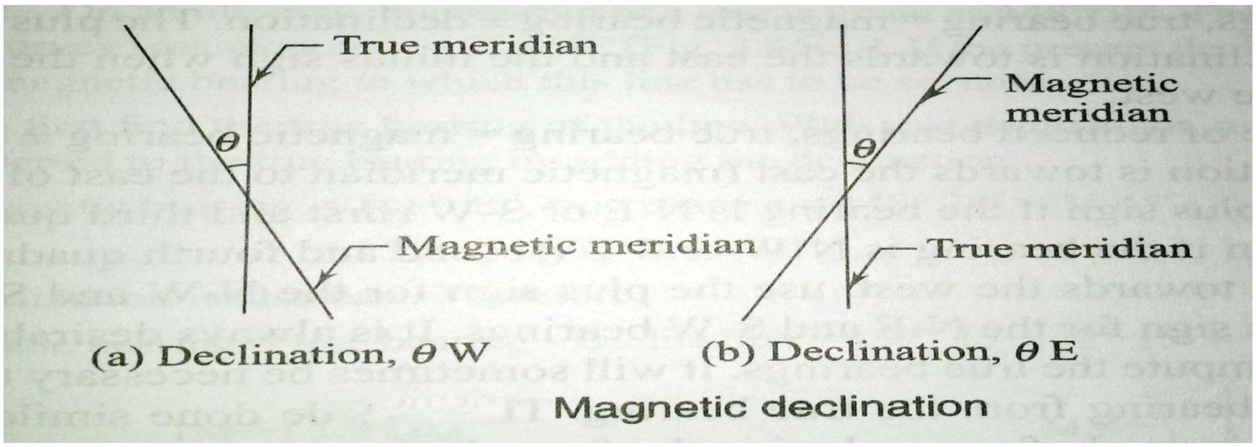
Dip of needle

The magnetic lines or forces due to earth'[s magnetism generally go from near the South pole to North pole. Such lines of force are parallel to the surface (horizontally) only near the equator. At other places, as these lines to the poles, they are direction as the lines of forces; it will dip (from the horizontal) by a small angle. This is known as the dip angle. The dip angle increases as we go from the equator to the poles.

Magnetic Declination:

- The horizontal angle between true north and magnetic north at a place at the time of observation, is called magnetic declination.
- The angle of convergence between the true north and magnetic north at any place does not remain constant.
- It depends upon the direction of the magnetic meridian at the time of observation.
- If the magnetic meridian is on eastern side of true meridian, the angle of declination is said to be eastern declination of positive declination.

- On the other hand if the magnetic meridian is on western side, the declination is said to be western declination or negative declination is zero.
- The imaginary lines joining the places of equal declination either positive or negative, on the surface of the earth, are called "Isogonic lines".
- The isogonics lines having zero declination are known as 'Agonic lines'.



Mariners generally call magnetic declination as '**variation**'.

1. **Determination of Magnetic Declination:**
   - True meridians at a number of places in the area, are determined by making astronomical observations (specially to stars).
   - Compass observations are made by sighting the true meridians at the places
   - The angle of inclination between true meridian and magnetic meridian given by a compass reading, is the desired magnetic declination at the place.
   - Magnetic declination = True bearing –Magnetic bearing".*

2. **Calculation of True Bearing**.

   True Bearing = Magnetic bearing $\pm$ magnetic declination,

   use + ve sign if declination is east

   and –ve sign, if it is west.

3. **Calculation of Magnetic Bearing**.

   Magnetic bearing = True Bearing $\pm$ magnetic declination, use –ve sign for eastern declination and + ve sign for western declination.

Variation of Declination

Declination at my place does not remain constant but keeps on changing from time to time. These variations may be classified under four heads *viz.*

1.Secula variation 2. Annual variation

3. Diurnal variation 4. Irregular variation

1. Secular Variation.: The earth magnetic poles are continually changing their positions relatively to the geographical poles. Earth Magnetic meridian also changes and affects the declination of places. Secular variation is a slow continuous change and declination of places. Alters in a more and less regular manner from year to year. Due to its magnitude, secular variation is the most important for land surveyors. It appears to be of periodic character and follows a sine curve. The swing of declination at a place over a period of centuries, may be compared to a simple harmonic motion. A secular change from year to year is also not uniform for any given place. It is also different for different places. To convert magnetic bearings into true bearings, an accurate amount of declination is essentially required. As such it is very important for a surveyor to know the exact amount of declination. When observations for the declination are made in different years of a century, it is revealed that magnetic meridian moves from one side of true meridian to the other. The change produced annually by secular variation at different places amounts from 0.02 minute to 12 minute. The variation at depends upon the geographical position of different place. The annual secular change is greatest near the middle point of a complete cycle and least at it extreme limits.

2. Annual Variation.: Change in declination at a place over a period of one year, is known as annual variation. From the observations made at different places over a period of 12 months, it is found that annual variation is about 1 minute to 2 minutes, depending upon their geographical positions.

3. Diurnal Variation. The departure of declination from its mean value during a period of 24 hours at any place is called diurnal variation. The diurnal variation depends upon the following factors:

(1) **The geographical position of the place.** It is greatest for the places in higher latitudes and lesser near the equator.
(2) **Season of the year.** It is comparatively more in summer than in winter at the same place.
(3) **The time.** It is more in day and less at night.
(4) **The year of the cycle.** It is different for different years in the complete cycle of secular variation.

(4) Irregular Variation. Abrupt change of declinations at places due to magnetic storms, earthquakes and other solar influences, are called irregular variations. These disturbances may occur at any time at any place and cannot be predicted. The displacement of a needle may vary in extent from 1^0 to 2^0.

Example. The true and magnetic bearings of a line are 7 8^0 45 and7 $5^0 30^/$ respectively. Calculate the magnetic declination at the place.

Solution.

Magnetic declination = True bearing-Magnetic bearing

$= 78^0 45^/ - 75^0 30^/$

$= 3^0 15^/$

As the sign is + ve, declination is east of true meridian.

. Magnetic declination = $3^0 15^/$ East.

3.6 ERRORS IN ANGLE MEASUREMENT WITH COMPASS SOURCES AND REMEDIES

The following errors are common in surveying with compass.

Instrumental errors:

It is caused by the defective parts of the instrument. These are

(a)The needle may not be straight, giving wrong readings.

(b)The pivot point may have become blunt and the needle may not move freely.

(c)The line of sight may not pass through the centre of the graduated ring.

(d)The ring may not move in a horizontal plane due to the dip of the needle as a result of the wrong adjustment of the balancing weight.

(e)The cross hair in the objective vane4 may not be straight or may have become loose.

Personal errors:

(a)Reading the graduations in the wrong direction or reading the quadrants wrongly.

(b)Improper centering of the compass over the station.

(c)Not leveling the compass properly.

(d)Not bisecting the signal at a station properly.

Other errors:

(a) Variation in declination during the day, when the survey is carried out over a long duration during the day
(b) Local attraction due to the proximity of external magnetic influences at one or more stations
(c) Other variations due to magnetic storms, cloud cover, etc, which affect the magnetic needle.

Precautions to be taken in compass survey

The instrumental and observational errors during a compass survey may be minimized by taking the following precautions:

- Set up and level the compass carefully.
- Stop the vibrations of the needle by gently pressing the brake-pin so that it may come to rest soon.
- Always look along the needle and not across it, to avoid parallax.
- When the instrument is not in use, its magnetic needle should be kept off the pivot. If it is not done, the pivot is subjected to unnecessary wear which may cause sluggishness of the magnetic needle.
- Before taking a reading, the compass box should be gently tapped to ensure that the magnetic needle is freely swinging and has not come to rest due to friction of the pivot.
- Stations should be selected such that these are away from the sources of local attraction.
- Surveyor should never carry iron articles, such as a bunch of keys which may cause local attraction.
- Fore and back bearings of each line should be taken to guard against the local attraction. If the compass is not be set at the end of a line, the bearings may be taken from any intermediate point along that line.
- Two sets of readings should be taken at each station for important details by displacing the magnetic needle after taking one reading.
- Avoid taking a reading in wrong direction viz**.** 25° to 20° instead 20° to 25° and so.
- If the glass cover has been dusted with a handkerchief, the glass gets charged with electrostatic current and the needle adheres to the glass cover .This may be obviated by applying a moist finger to the glass.
- Object vane and eye vane must be straightened before making observations.

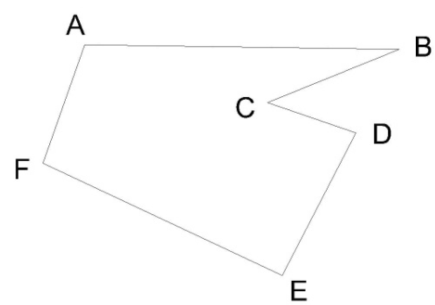
3.7 PRINCIPLES OF TRAVERSING – OPEN & CLOSED TRAVERSE, METHODS OF TRAVERSING.

A series of connected straight line each joining two points on the ground is called a traverse. End points are known as traverse stations and straight lines between two consecutive station s are called traverse legs.

Traverse may be either a closed traverse or an open traverse.

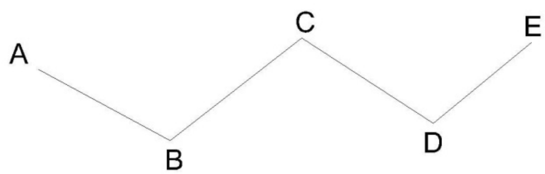
Closed Traverse:

A traverse which either originate from a station and closes on same station or runs between two station whose co ordinates are known in terms of a common system of co ordinates is known as closed traverse. In closed traverse accuracy of linear as well as angular measurements may be known.



Open Traverse:

A traverse which neither returns to its starting station nor ends on another known station is known as open traverse .In open traverse accuracy of linear as well as angular measurement may not be checked.



Difference between Chain survey And Compass Survey

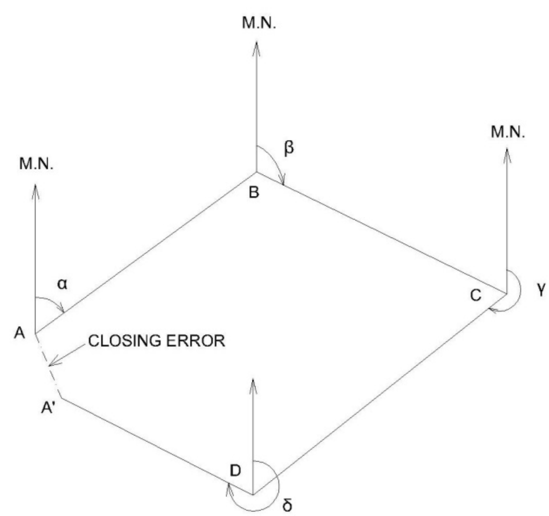
Chain survey is preferred to if the area to be surveyed is small in extent and higher accuracy is aimed at where as if the area is comparatively large with undulation and less accuracy is required, compass survey is adopted.

Methods of Traverse:

Before plotting of traverse survey it should be checked whether the observed bearing are correct. If not the required correction to each bearing may be made so that the traverse will perfect in the geometrical figure based on field data.

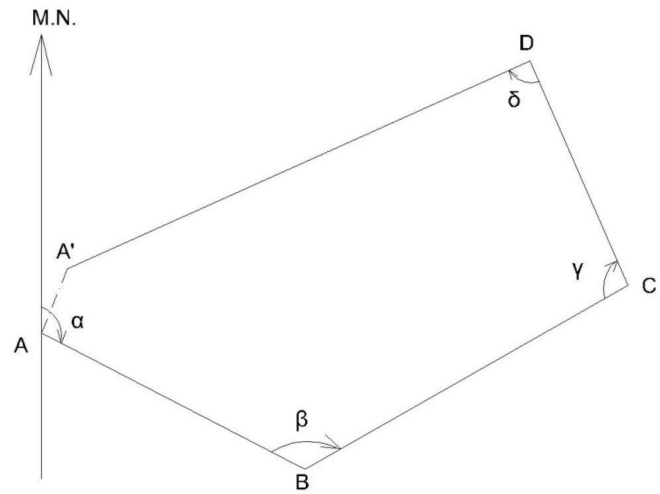
The traverse may be plotted by one of the method

1.By Parallel Meridians; After deciding the layout of the traverse a line representing the magnetic meridian through the location of the station is drawn on the paper .The bearing of the line AB is plotted with the ordinary protractor and its length duly reduced to scale, is marked off to get the location of station A is drawn . The bearing of BC is plotted and length BC is plotted to scale .The process is continued till last station is plotted. In a close traverse last line should be end on the starting station A. In case of a closed circuit or at any other known station in case of linear closed traverse. If dose not the distance between two locations of the same station is termed as closing error.



2.By Included Angles;

After deciding the location of the station A on the paper draw a line to represent the magnetic meridian passing through A . Plot the magnetic bearing of the chain line AB and plot AB duly reduced to scale. Now plot the included angle ABC by a protractor and plot the location of station C .The process is continued till all the station are plotted. It may be noted that for a closed traverse if linear measurement between stations are correct and plotting is error less the closing station will coincide with the station A .If not the distance between two location of the starting station is known as closing error.

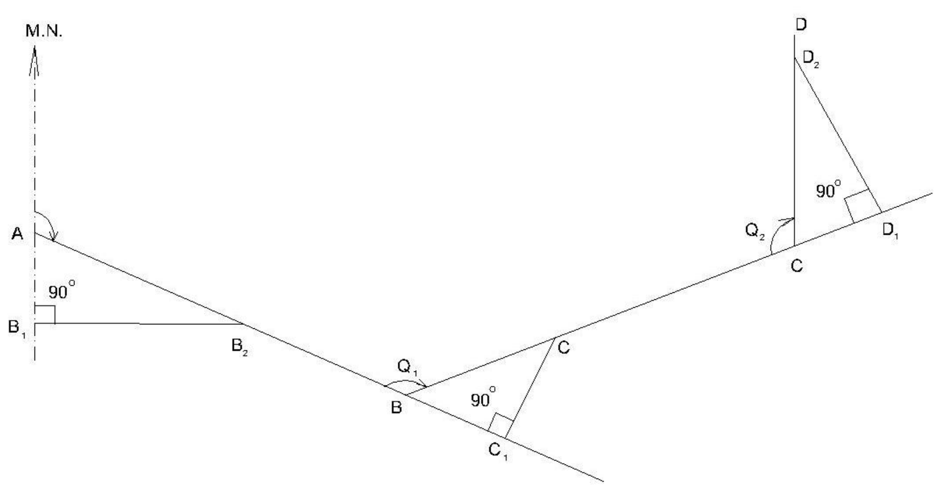


3.Plotting By tangents.

Defection angles of the chain lines are plotted by geaometry construiction with the help of their natural tangents. The traverse may be plotted as followed.

From the location of the starting station A draw a line passing through A to represent its magnetic meridian .To draw the bearing of traverse leg AB cut a length of 10 cm on the magnetic meridians of station A at B_1. At B_1 erect a perpendicular B_1B_2 on the proper side of the meridian .Take B_1B_2 equal to 10 x tangent of the reduced bearing i,e angle of deflection of the line AB in centimeter.

Join AB2 and produce it to get the direction of traverse line AB plot length of AB on the line AB2 to a desired scale.



4.
5.

The Deflection angles of the successive chain lines for the purpose of plotting are obtained by the following formulae.

1.If the included angle between adjacent lines is between 0° and 90°, deflection angle is equal to the included angle.

2. If the included angle is between 90° and 180°, subtract the given included angle from 180° to get the deflection angle.

3. If the included angle is between 180° and 270°, subtract180° from the given included angle.

4. If the included angle is between 270° and 360°, subtract the given included angle from 360° to get the deflection angle.

Continue the process till all the traverse legs are plotted.

3.8 LOCAL ATTRACTION – CAUSES, DETECTION, ERRORS, CORRECTIONS, NUMERICAL PROBLEMS OF APPLICATION OF CORRECTION DUE TO LOCAL ATTRACTION.

Causes

North end of a freely suspended magnetic needle always points to the magnetic north ,if not influenced by any other external forces except the earth's magnetic field.

The magnetic needle gets deflected from its normal position, if placed near magnetic rocks ,iron ores cables etc. such a disturbing force is known as local attraction.

Detection of local attraction:

The presence of local attraction at any station may be detected by observing the fore and back bearings of the line. If the difference between fore and back bearing is 180°,both end stations are free from local attraction. If not, the discrepancy may be due to:

(1)An error in observation of either fore or back bearings or both.

(2) Presence of local attraction at either station.

(3) Presence of local attraction at both the stations.

The correction to other stations may be made according to the following methods.

i) By calculating the included angles at the affected stations

ii) By calculating the local attraction of each station and then applying the required corrections starting from the unaffected bearing.

Method of elimination of local attraction by in closed :

i) Compute the included angles at each station from the observed bearing, in case of a closed traverse.

ii) Starting from the unaffected line run down the correct bearing of the successive sides.

3.9 ERRORS IN COMPASS SURVEYING – SOURCES & REMEDIES.

PLOTTING OF TRAVERSE – CHECK OF CLOSING ERROR IN CLOSED & OPEN TRAVERSE, BOWDITCH'S CORRECTION, GALES TABLE

The following errors are common in surveying with compass.

Instrumental errors:

It is caused by the defective parts of the instrument. These are

(a)The needle may not be straight, giving wrong readings.

(b)The pivot point may have become blunt and the needle may not move freely.

(c)The line of sight may not pass through the centre of the graduated ring.

(d)The ring may not move in a horizontal plane due to the dip of the needle as a result f the wrong adjustment of the balancing weight.

(e)The cross hair in the objective vane4 may not be straight or may have become loose.

Personal errors:

(a)Reading the graduations in the wrong direction or reading the quadrants wrongly.

(b)Improper centering of the compass over the station.

(c)Not leveling the compass properly.

(d)Not bisecting the signal at a station properly.

Other errors:

(d) Variation in declination during the day, when the survey is carried out over a long duration during the day
(e) Local attraction due to the proximity of external magnetic influences at one or more stations
(f) Other variations due to magnetic storms, cloud cover, etc, which affect the magnetic needle.

PRECAUTIONS TO BE TAKEN IN COMPASS SURVEY

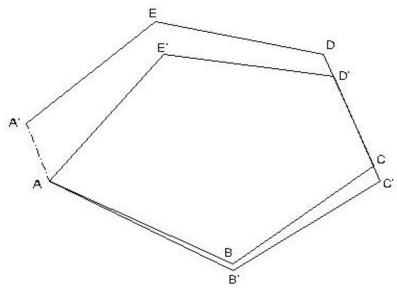
The instrumental and observational errors during a compass survey ,may be minimized by taking the following precautions:

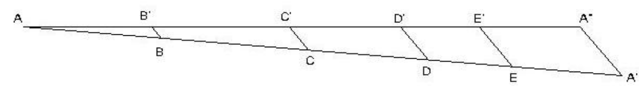
- Set up and level the compass carefully.
- Stop the vibrations of the needle by gently pressing the brake-pin so that it may come to rest soon.
- Always look along the needle and not across it, to avoid parallax.
- When the instrument is not in use ,its magnetic needle should be kept off the pivot. If it is not done, the pivot is subjected to unnecessary wear which may cause sluggishness of the magnetic needle.
- Before taking a reading ,the compass box should be gently tapped to ensure that the magnetic needle is freely swinging and has not come to rest due to friction of the pivot.
- Stations should be selected such that these are away from the sources of local attraction.
- Surveyor should never carry iron articles, such as a bunch of keys which may cause local attraction.

Adjustment of Closing Error:

When a closed traverse is plotted from the field measurements, the end station of a traverse generally does not coincide exactly with its starting station. This discrepancy is due to the errors in the field observations i.e. magnetic bearings and linear distances. Such an error of the traverse is known as **closing error or error of closure.**

When the angular and linear measurements are of equal precision, graphical adjustment of the traverse may be made. This method is based on the Bowditch's rule. Corrections are applied to lengths as well as to bearings of the lines in proportion to their lengths. Graphical method is also sometimes known as proportionate method of adjustment.





Method. The adjustment of a compass traverse graphically, may be made as follow:

Let ABCDEA’ be a closed traverse as plotted from the observed magnetic bearings and linear measurements of the traverse legs. A is thestarting station and A’ is the location of the station A as plotted. Hence, A’A is the closing error.

Adjustment. Following procedure may be adopt.

- Draw a straight line AA’ equal to the perimeter of the traverse to any suitable scale.
- Set off the distances AB,BC,CD,DE, and EA’ equal to the lengths of the sides of the traverse.
- Draw A’A” parallel and equal to the closing error A’A.
- Draw parallel lines through points B,C,D, and E to meet AA” at B’,C’D’ and E’.
- Draw parallel lines through the plotted stations B,C,D,E and plot the errors equal to BB’,CC’,DD’ in the direction of A’A’.
- Join the points AB’C’D’E’ A to get the adjusted traverse.

1) Bowditch’s Rule:

It is also known as the compass rule and is most commonly used in traverse adjustment. It is used when the angular and linear measurements are equally precise. By this rule, the total error in latitude and that in departure is distributed in proportion to the lengths of the sides.

Gales Table

Traverse computations are usually done in a tabular form, a more common form being gales traverse table is shown below figure. For complete traverse computations, the following steps are usually necessary.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 What is a 12 cm compass? [2017-W]

The size of a compass is designated by its diameter therefore 12 cm compass of diameter 12 cm.

Q-02 Fundamental difference between prismatic compass and surveyor compass?[2019-S]
Ans:

- The prismatic compass shows the WCB of line
- Where as the surveyor compass shows in quadrantal bearing of a line

Q-3 Define fore bearing and back bearing?[2014-S,2017-W,2018-W]

Ans- The bearing of a line measure in the direction of progress of surveying is called FB of the line
The bearing of a line is measure in the direction of opposite to the survey .

Q-4 The fore bearing of line is $145^0 30'$ what is its back bearing? [2018-W]
Ans- BB= $145^0 30' + 180^0 = 325^0 30'$

POSSIBLE LONG TYPE QUESTIONS

Q-1 Write down the essential parts of a prismatic compass with neat sketch.[2014-S,2017-W]
Q-2 Write down the sources of error in compass surveying?[2015-W,2018-W]
Q-3 Define wcb and qb in compass survey.[2014-S,2018-W]

CHAPTER NO-04

MAP READING CADASTRAL MAPS & NOMENCLATURE:

Learning objectives

4.1 Study of direction, Scale, Grid Reference and Grid Square
Study of Signs and Symbols
4.2 Cadastral Map Preparation Methodology
4.3 Unique identification number of parcel
4.4 Positions of existing Control Points and its types
4.5 Adjacent Boundaries and Features, Topology Creation and verification.

4.1 STUDY OF DIRECTION, SCALE, GRID REFERENCE AND GRID SQUARE
STUDY OF SIGNS AND SYMBOLS

The Elements of a Map

1. Title
2. Legend
3. Scale
4. Directional Indicator
5. Inset Maps

Study of direction

- Directions are indicated on the maps by north south lines with arrow head pointing towards north. If the north south line with arrow is not drawn on the map is taken as north and buttom of the map is south.

Scale of map

- The scale of a map is the ratio of a distance on the map to the corresponding distance on the ground.
- The simple concept is completed by the curvature of earth surfaces which forces scale to vary across a map

Grid reference

- Grid references defined as location on maps using cartasian co-operation systems. Grid lines on maps defines the coordinate system and are numbered so provide a unique reference to each location on the map.
- This reference is normally based on projected eastings and northings.
- Grid square
- Grid square are the squares of a map denoted by a grid formed by a series of numbers horizontal *(northings) and vertical (southings) lines.*

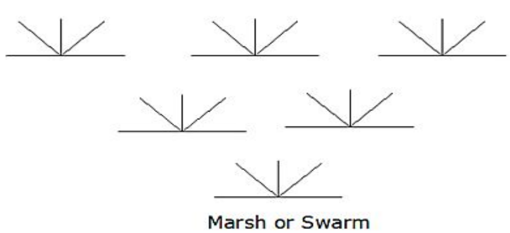
Signs and symbols

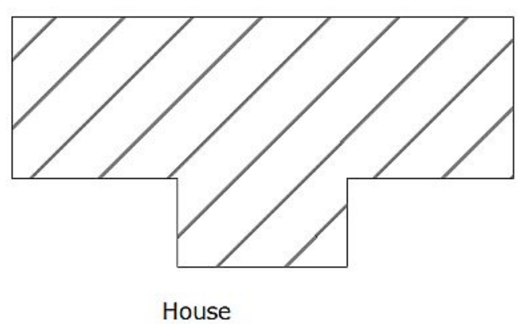
- Most map symbols are conventional signs as them understood by everyone around world.
- Example

A light house, a church etc.

Here are a few important Conventional Signs and symbols:

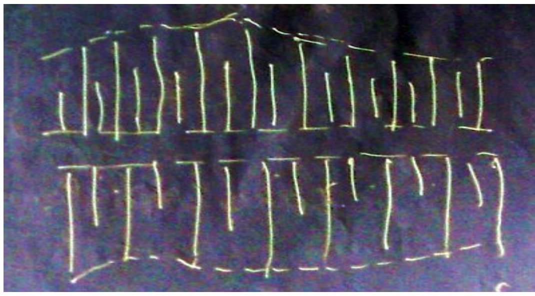
1. Marsh or Swarm
2. House
3. Embankment
4. Cutting
5. Single Line Railway
6. Double Line Railway
7. Lake or Pond
8. Road
9. Railway Bridge
10. Road Bridge
11. River
12. Fence





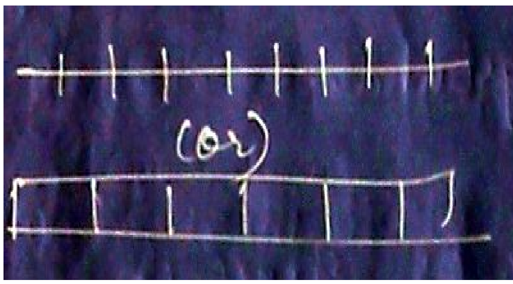
Marsh or Swarm

House

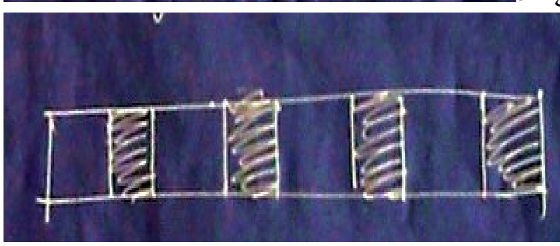


Embankment

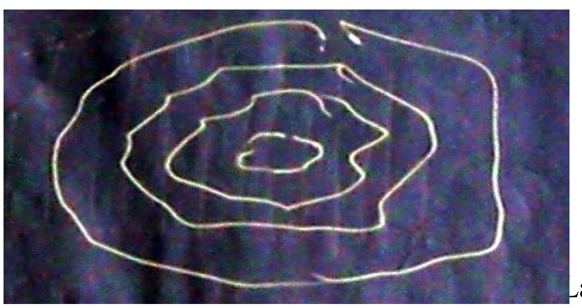
Cutting



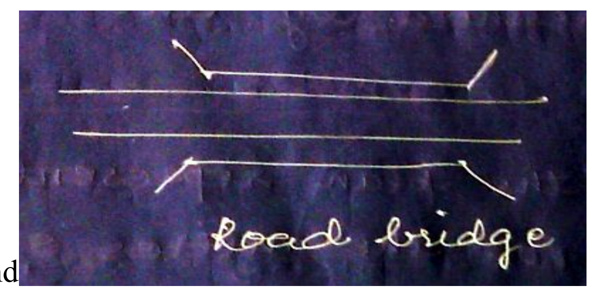
Single Line Railway



Double Line Railway

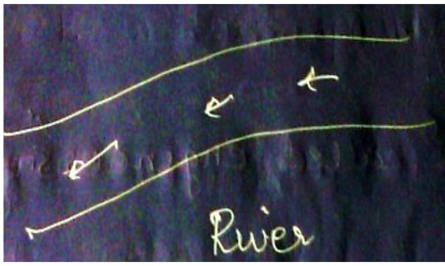


Lake or Pond

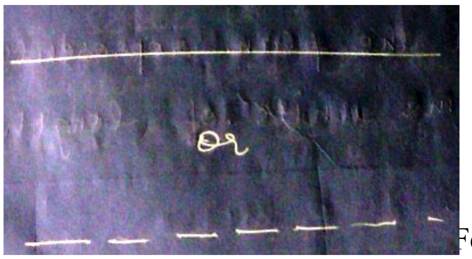


Road Bridge

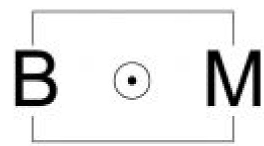
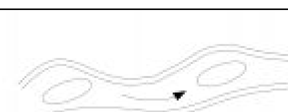
Road

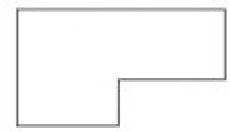
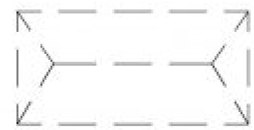
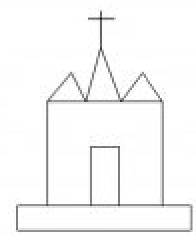
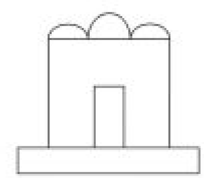
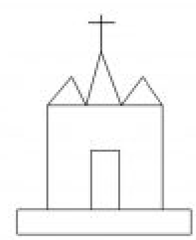
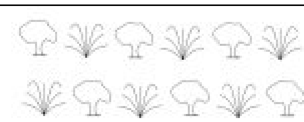
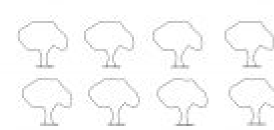


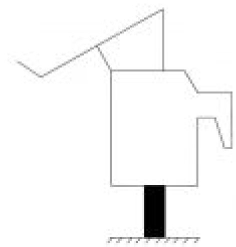
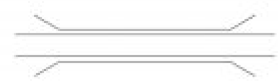
River

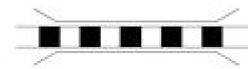
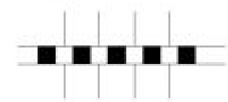
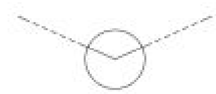
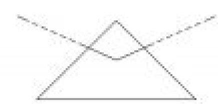
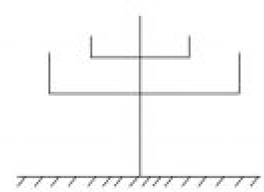
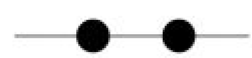


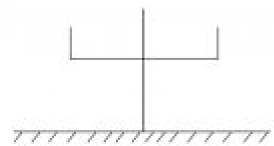
Fence

Objects	**Symbols**
Bench Mark which is used as reference point or called as permanent point this is usually marked with respect to known elevation (Mean Sea Level)	
Lake or Pond	
River	
Canal	

Building (Pukka)	
Huts	
Temple	
Masjid	
Church	
Tree	
Orchard	

Tube well	
Objects	Symbols
Metalled Road Roads which are made of bitumen of coal or cement concrete	
Un-Metalled Road Roads which are laid using sand	
Road Bridge	
Footpath	
Railway line (Single)	

Railway line (double)	
Railway bridge	
Level Crossing	
Main stations or triangulation stations in chain survey	
Traverse stations or sub stations in chain surveying	
Electric post	
Electric line	

Telegraphic Line	
Telegraphic post	

4.2 CADASTRAL MAP PREPARATION METHODOLOGY

Cadastral map

- Cadastral mapping is a compressive resistance of the details related to the property of an area. This cadastral mapping service provided all the includes solution of the land information system problems by forcing varies forms land records.
- This is a map that shows the boundary and ownership and land reprisal.
- Some map shows additional details such as survey district name , unique identification number parcel for certificate of the title numbers or plot numbers , strict name .selected boundary nos and reference to prior map.
- Most countries, legal systems have developed around the original administrative systems and use the cadastre to define the dimensions and location of land parcels described in legal documentation. A **land parcel** or **cadastral parcel** is defined as "a continuous area, or more appropriately volume, that is identified by a unique set of homogeneous property rights".[3]
- Cadastral surveys document the boundaries of land ownership, by the production of documents, diagrams, sketches, plans (*plats* in the us), charts, and maps. They were originally used to ensure reliable facts for land valuation and taxation. An example from early england is the domesday book in 1086. Napoleon established a comprehensive cadastral system for france that is regarded as the forerunner of most modern versions.
- Cadastral survey information is often a base element in geographic information systems (gis) or land information systems (lis) used to assess and manage land and built infrastructure. Such systems are also employed on a variety of other tasks, for example, to track long-term changes over time for geological or ecological studies, where land tenure is a significant part of the scenario.

4.3 UNIQUE IDENTIFICATION NUMBER OF PARCEL

Parcel identifiers

- A unique identification numbers should be assigned parcel or code that links the parcel with files containing data such as ownership value use and zoning.
- Parcel identifiers provide index for all property for all property records.

The Unique Land Parcel Identification Number (ULPIN) is a 14-digit identification number accorded to a plot of land.

- It is an alpha-numeric unique ID for each land parcel that contains ownership details of the plot besides its size and longitudinal & latitudinal details.
- It is part of the Digital India Land Records Modernisation Programme (DILRMP), a programme that had been initiated in 2008.
- The identification will be based on the longitude and latitude coordinates of the land parcel, and depends on detailed surveys and geo-referenced cadastral maps.
- The number is developed by the National Informatics Centre (NIC).
- The ULPIN scheme was rolled out in 2021 in ten Indian states. The government plans to launch it in all states and UTs by March 2022.
- The idea behind the program is to check land fraud, especially in the rural hinterlands of India, where there are no clear land records and often, land records are ambiguous and land ownership disputed.
- It will eventually integrate its land records database with revenue court records and bank records, as well as Aadhaar numbers on a voluntary basis.
- It is being touted as ‘Aadhar for Land’.
- Proper land statistics and land accounting through the ULPIN scheme will aid in developing land banks and usher in the Integrated Land Information Management System (ILIMS).

4.4 POSITIONS OF EXISTING CONTROL POINTS AND ITS TYPES

Control point

- A control point is a point on the ground or any permanate structure whose horizontal and vertical on or position is known.
- Control point are use as a starting point of all type of survey a control point is marked way point used in orienting at related sports such as adventure receiving and regaining.
- It is located compitation area marked both on an orienting map and in the terrain and described on a control description.

Types of control point

1. Ground control points
2. Check points

4.5 ADJACENT BOUNDARIES AND FEATURES, TOPOLOGY CREATION AND VERIFICATION.

In 1736, the mathematician Leonhard Euler published a paper that arguably started the branch of mathematics known as topology. The problem that led to Euler's work in this area, known as "The Seven Bridges of Königsberg," is described in the accompanying article "Conundrum Inspires Topology." More recently, the United States Census Bureau, while preparing for the 1970 census, pioneered the application of mathematical topology to maps to reduce the errors in tabulating massive amounts of census data. Today, topology in GIS is generally defined as the spatial relationships between adjacent or neighboring features.

Enter Shape files

Shapefiles were introduced with the release of ArcView 2 in the early 1990s. A shape file is a non topological data structure that does not explicitly store topological relationships. However, unlike other simple graphic data structures, shapefile polygons are represented by one or more rings. A ring is a closed, non-self-intersecting loop. This structure can represent complex structures, such as polygons, that contain "islands." The vertices of a ring maintain a consistent, clockwise order so that the area to the right, as one "walks" along the ring boundary, is inside the polygon, and the area to the left is outside the polygon.

Moreover, polygon features in shape file format can contain one or more parts, so that dis junct and overlapping features can be represented. For example, an individual parcel that is split by a road can be represented alternatively as two separate polygons with two rings and two records in the attribute table or as one polygon with two parts and one record in the attribute table. A source of confusion for some users is that some Arc View GIS commands can result in spatially disjunct, multipart features.

Topographical features

- The edges of earths lithosphere atmosphere plate boundary which are both well defined and poorly defined.
- Well defined boundary include mid-ocean ridge and ocean trenches.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 What are the elements of a map?

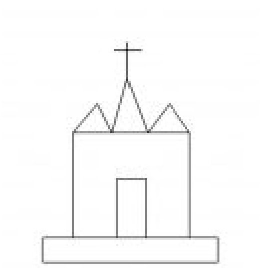
Ans: The Elements of a Map

1. Title
2. Legend
3. Scale
4. Directional Indicator
5. Inset map

Q-2 Define scale of map.

- The scale of a map is the ratio of a distance on the map to the corresponding distance on the ground.
- The simple concept is completed by the curvature of earth surfaces which forces scale to vary across a map

Q-3 Draw the symbol for a temple



Ans:

Q-4 Define control points in a map?

- Ans: A control point is a point on the ground or any permanate structure whose horizontal and vertical on or position is known.
- Control point are use as a starting point of all type of survey a control point is marked way point used in orienting at related sports such as adventure receiving and regaining.

POSSIBLE LONG TYPE QUESTIONS

Q-1 What is scale of a map and Study of direction in a map?

Q-2 What are the elements of a map. Briefly describe about it?

CHAPTER NO-5

PLANE TABLE SURVEYING

Learning objectives

5.1 Objectives, principles and use of plane table surveying.
5.2 Instruments & accessories used in plane table surveying.
5.3 Methods of plane table surveying –
(1) Radiation, (2) Intersection, (3) Traversing, (4) Resection.
5.4 Statements of TWO POINT and THREE POINT PROBLEM.
Errors in plane table surveying and their corrections, precautions in plane table surveying.

5.1 OBJECTIVES, PRINCIPLES AND USE OF PLANE TABLE SURVEYING

A **plane table** is a device used in surveying and related disciplines to provide a solid and level surface on which to make field drawings, charts and maps. The early use of the name *plain table* reflected its simplicity and plainness rather than its flatness.

Objectives:-

- It is suitable for location of details as well as contouring for large scale maps directly in the field.
- As surveying and plotting are done simultaneously in the field, chances of getting omission of any detail get less.
- The plotting details can immediately get compared with the actual objects present in the field. Thus errors as well as accuracy of the plot can be ascertained as the work progresses in the field.
- Contours and specific features can be represented and checked conveniently as the whole area is in view at the time of plotting.
- Only relevant details are located because the map is drawn as the survey progresses. Irrelevant details get omitted in the field itself.
- The plane table survey is generally more rapid and less costly than most other types of survey.
- As the instruments used are simple, not much skill for operation of instruments is required. This method of survey requires no field book.

Disadvantage:-

- The plane table survey is not possible in unfavorable climates such as rain, fog etc.
- This method of survey is not very accurate and thus unsuitable for large scale or precise work.

- As no field book is maintained, plotting at different scale require full exercise.
- The method requires large amount of time to be spent in the field.
- Quality of the final map depends largely on the drafting capability of the surveyor.
- This method is effective in relatively open country where stations can be sighted easily

Principle :-

The principle of plane table survey is ***Parallelism*** ,It means that the ray drawn from station to objects on the paper are parallel to the lines from the station to the objects on the ground.

5.2 INSTRUMENT AND ACCESSORIES USED IN PLANE TABLE SURVEYING

Accessories of plane table:-

a. Plane table

b. Alidade

c. The Spirit level

d. The compass

e. The U – Fork or plumbing Fork with plum bob

a. The Plane Table:-

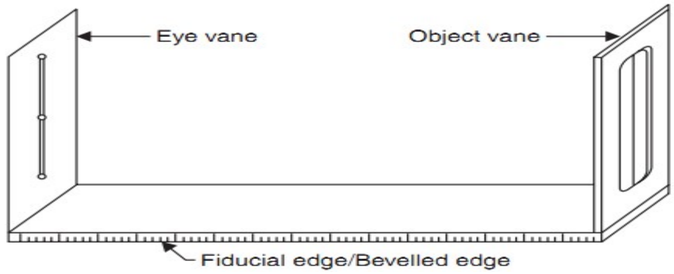
i. The plane table is a drwing board of size 750mm X 600mm made of well seasoned wood like Teak, pine,etc.

ii. The top surfaceof the table is well levelled .

iii. The bottom surface consists of a threaded circular plate for fixing the table with the tripod stand by a wing nut.

iv. The plane table is meant for fixing the a drawing sheet over it.

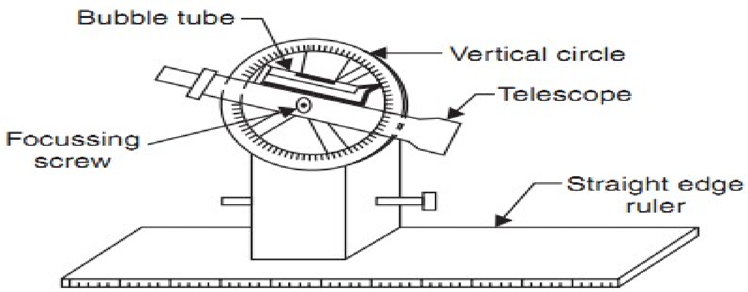
v. The position of the objects are located on this sheet by drawing rays and plotting to any suitable scale.



b. **<u>Alidade:-</u>**

There are two types of alidade –Plain and telescopic aldade.

1.Plain alidade:-the plain alidade consists of a metal or wooden ruler of length about 50cm. one of its edge is beveled, and is known as fiducial edge. It consists of two vanes at both ends which are hinged with the ruler. One is known as object vane and the other is known as sight vane.



2.Telescopic alidade:-The telescopic alidade consists of a telescope meant for inclined sight or sighting distant objects clearly. The alidade has no vanes at the ends, but is provided with the fiducial edge

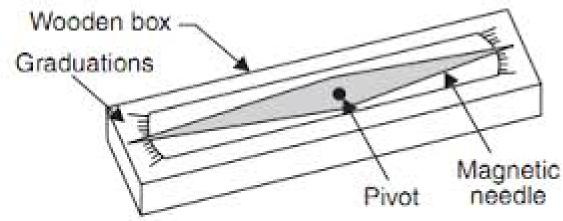
The function of the alidade is to sight objects. The rays should be drawn along the fiducial edge.

c.**<u>The Spirit level:-</u>** It is a smaller metal tube containing a small bubble of spirit . The bubble is visible on the top along a graduated glass tube. The spirit level is meant for leveling the plane table.

d. **The compass:-** There are two kinds of compass

i.The trough compass

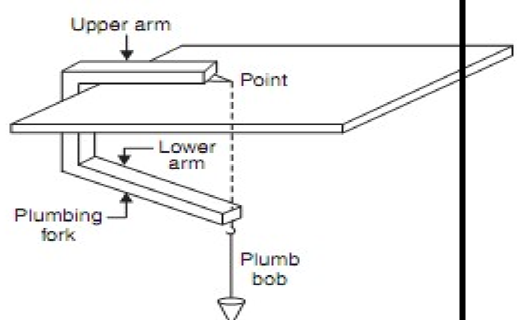
ii. The circular box compass.



i. **The trough compass**:- It is rectangular box made of non magnetic metal containing a magnetic needle pivoted at the centre. This compass consists of '0' mark at both the ends to locate N-S direction.

ii.**The Circular box compass**:- It carries a pivoted magnetic needle at the centre. The circular box is fitted on square base plate . Sometimes two bubble tubes are fixed at the right angles to each other on the base plate. The compass is meant for making the north direction of the map.

e. **The U – Fork or plumbing Fork with plum bob:-** The U- fork is a metal strip bent in the shape of a 'U' (Hair pin) having equal arm lengths. The top arm is pointed and the bottom arm carries a hook for suspending a plumb bob .



This is meant for centering the table over a station.

5.3 METHODS OF PLANE TABLE SURVEYING

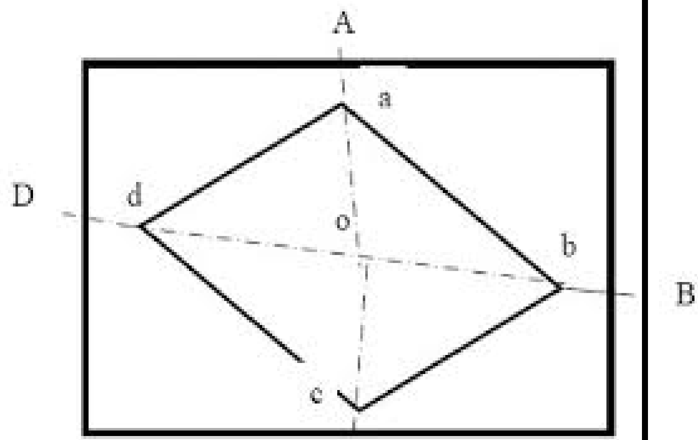
Methods Of Plane Table.:-

There are four methods of plane table. They are

Radiation
Intersection
Traversing
Resection

Radiation:-

This method is suitable for locating the objects from a single station.



In this method rays are drawn from the station to the objects and the distances from the station to the object are measured and plotted to any suitable scale along the respective rays.

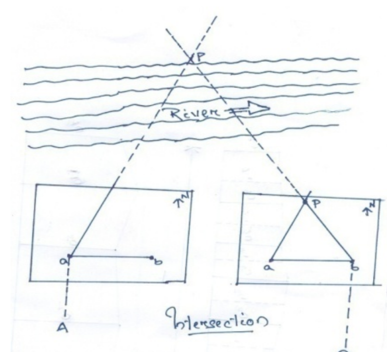
Procedure:-

i. Suppose O is a station on the ground from where the objects A, B, C, & D are visible.
ii. The plane table set up over at P. A drawing sheet is fixed on the table ,which is then leveled and centered . A point *o* is selected on the sheet to represent the point *o*.
iii. The North line is marked on the right hand top corner of the drawing sheet with the trough compass .
iv. With the alidade touching the point *o*, Ranging rod at A, B, C, & Dare bisected and the rays are drawn.
v. The distances OA,OB,PC, & OD are measured and plotted to any suitable scale to obtain the points a,b,c, & d representing A, B, C, & D on the paper.

Intersection:-

This method is suitable for locating inaccessible points by the intersection of the ray drawn from two station instrument station.

Procedure:-



i. Suppose A & Bare two station and P is an object on the far bank of the river . It is required to fix the position of P on the sheet by the intersection of the rays drawn From A and B.
ii. The table is set up at A. it is leveled and centered so that a point a on the sheet is just over the station A. The North line is marked on the right hand top corner of the drawing sheet with the trough compass.
iii. With the alidade touching the point *a* the object P and the ranging rod at B are bisected and rays are drawn trough the fiducial edge of the alidade.

iv. The distance AB is measured and plotted to any suitable scale to obtain the point *b*.

v. The table is shifted and centered over B and leveled properly. Now the alidade is placed along the line ba and orientation is done by back sighting.. While backsighting it should be kept in mind that the centering and leveling is not disturbed. In case it is disturbed it should be adjusted immediately.

vi. With the alidade touching b, the object P is bisected and ray is drawn . Suppose this ray intersects the previous ray at a point *p*. This point p is the plotted position of P.

Traversing:-

This method is suitable for connecting the traverse station

Procedure:-

i. Suppose the P,Q,R,& S are the traverse stations.

ii. The table is set up at the station P. A suitable point is selected on the drawing sheet let it be *p.* such that the whole area may plotted on the drawing sheet..the table well leveled, centered and the north line is marked on right hand top corner of the sheet.

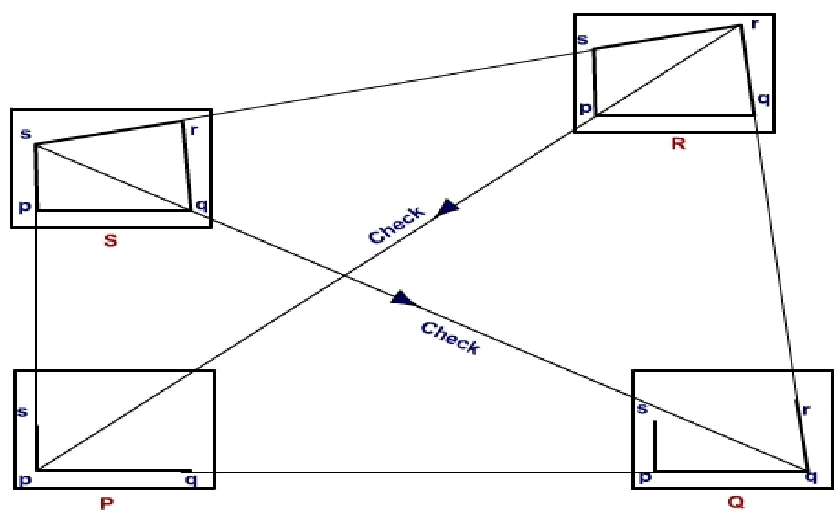
iii. With the alidade touching the point *p* the ranging rod at Q is bisected and the ray is drawn . The distance PQ is measured and plotted to any suitable scale to obtain the point *q*

iv. The table is shifted and set up over the station Q. It is then well leveled, centered , and oriented by back sighting and clamped.

v. With the alidade touching the point *q* the ranging rod at R is bisected and the ray is drawn . The distance QR is measured and plotted to any suitable scale to obtain the point *r*

vi. The table is shifted and set up over the station R. It is then well leveled, centered , and oriented by back sighting and clamped .

vii. With the alidade touching the point *r* the ranging rod at S is bisected and the ray is drawn . The distance RS is measured and plotted to any suitable scale to obtain the point *s*



viii. The table is shifted and set up over the station S. It is then well leveled, centered , and oriented by back sighting and clamped.

ix. With the alidade touching the point *s* the ranging rod at P is bisected and the ray is drawn.

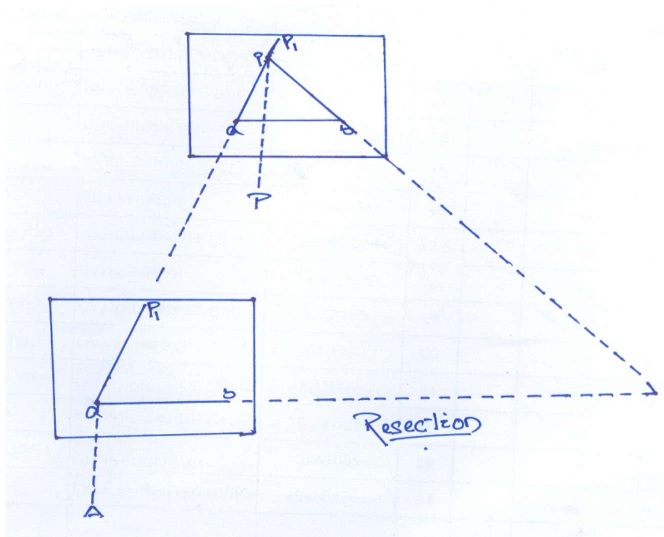
x. At the end the finishing point may not coincide with the starting point and there may be closing error. This error is adjusted graphically by Bowditch's rule.

xi. After making the correction for closing error the table is again setup over at A. After (well leveled, centered , and oriented by back sighting the surrounding are located by radiation).

xii. The table is then shifted and set up at all station of the traverse and proper adjustments the details are located by the radiation and intersection methods.

Resection method:-

This method is suitable for establishing new stations at a place in order to locate missing details.



Procedure

(a) Suppose it is required to establish a station at position on P. Let us select two points A and B on the ground .The distance AB is measured and plotted to any suitable scale. This line AB is known as the "base line".

(b) The table is set up at A. It is leveled; centered and oriented by bisecting the ranging rod at B. the table is then clamped.

(c) With the alidade touching point a, the ranging rod at P is bisected and a ray is drawn .Then a point P_1 is marked on this ray by estimating with the eye.

(d) The table is shifted and centered in such a way that P_1 is just over P. It is then oriented by back sighting the ranging rod at a.

(e) With the alidade touching point b, the ranging rod at B is bisected and a ray is drawn .Suppose this ray intersects the previous ray at a point P. This point represents the position of the station P on the sheet. Then the actual position of the station P is marked on the ground by U-fork and plumb bob.

Resection method based on (1)the two-point problem, and (2) the three-point problem.

5.4 TWO POINT PROBLEM AND THREE POINT PROBLEM

Two point problem:-

In problem ,two well defined points whose position have already been plotted on the plan and selected . then by perfectly bisecting these points a new station is established at the required position.

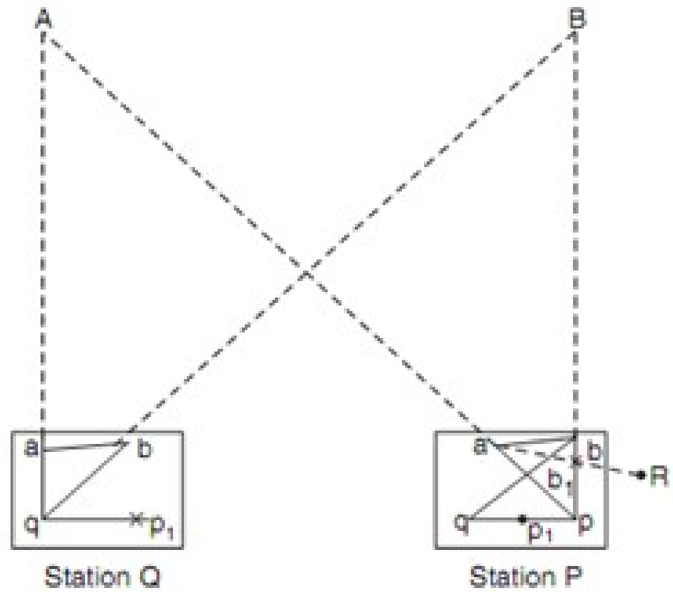
Procedure:-

a.Suppose A and B are two well defined points whose position are plotted on map as *a* and *b* . it is required to locate a new station at P by perfectly bisecting A and B

b. An auxiliary station Q is selected at a suitable position on the ground.The table is set up at Q and it is leveled; centered and oriented by an eye estimate. It is then clamped.

c. With the alidade touching *a* and *b* the points A and B are bisected and a ray is drawn suppose these ray meet at *q*

d. with the alidade centered on *q* the ranging rod at A is bisected and a ray is drawn . Then by eye estimation a point p_1 is marked on this ray.



e. The table is then shifted and centered on P with p_1 just over P. It is then leveled and oriented by the backlighting. With the alidade touching the point *a* the point A is bisected and the ray is drawn . Suppose this ray intersects at pq_1 at the point q_1 as assumed previously.

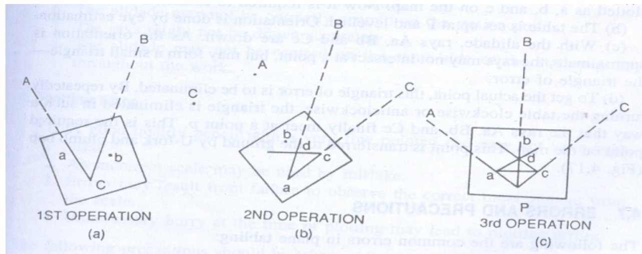
f. With the alidade centered on p_1 the point B is bisected and a ray is drawn .Suppose this ray intersect the ray qb at a point b_1. The triangle abb_1 is known as triangle of error and is to be eliminated.

g. The alidade placed along the line ab_1 and a ranging rod R is fixed at some distance from the table. Then the alidade placed along the line ab and the table is turned to bisect R. at this position the table is said to be perfectly oriented.

(h) Finally, with the alidade centered on p and q , the points p and Q are bisected and rays are drawn. Suppose these rays intersect at a point a. This would represent the exact position of the required station A. Then the station A is marked on the ground

The Three-point problem

in this problem, three well defined points are selected whose positions have already been plotted on the map. Then, by perfectly bisecting these three well-defined points, a new station is established at the required position.



No auxiliary station is required in order to solve this problem. The table is directly placed at the required position. The problem may be solved by three methods (a) the graphical or Bessel's method, (b) the mechanical method, and (c) the trial and error method.

(a) The Graphical method

(1) Suppose A, B and C are three well-defined points which have been plotted as a, b and c. Now it is required to locate a station at P.

(2) The table is placed at the required station P and leveled. The alidade is placed along the line ca and the point A is bisected and ray drawn.

(3) Again the alidade is placed along the line ac and the point c is bisected and the table is clamped. With the alidade touching a, the point b is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point d .

The alidade is placed along db and the point B is bisected. At this position the table is said to ba perfectly oriented. Now the rays Aa, Bb and Cc are drawn. These three rays must meet at a point p which is the required point on the map. This point is transferred to the ground by U-fork and plumb bob.

<u>Errors and Precautions:-</u>

A. <u>Instrumental Errors</u>

1. The surface of table may not be perfectly level.
2. The fiducial edge the alidade might not be straight.
3. The vanes may not be vertical.
4. The horsehair may be loose and inclined.
5. The table may be loosely joined with the tripod stand.
6. The needle of the through compass may not be perfectly balanced. Also it may not be able to move freely due to sluggishness of the pivot point.

B. <u>Personal Errors</u>

1. The leveling of the table may not be perfectly.
2. The table may not be centred properly.
3. The orientation of the table may not be proper.

4. The table might not be perfectly clamped.
5. The objects may not be bisected perfectly.
6. The alidade may not be correctly centred on the station point.
7. The rays might not be drawn accurately.
8. The alidade may not be centred on the same side of the station point throughout the work.

C. Plotting Error

1. A good quality pencil with a very fine pointed end may not have been used.
2. An incorrect scale may be used by mistake.
3. Errors may result from failure to observe the correct measurement from the scale.
4. Unnecessary hurry at the time of plotting may lead to plotting errors.

The following precautions should be taken while using the plane table;

1. Before starting the work the equipments for survey work should be verified. Defective accessories should be replaced by perfect equipment.
2. The centering should be perfect.
3. The leveling should be proper.
4. The orientation should be accurate.
5. The alidade should be centred on the same side of the station-pin until the work is completed.
6. While shifting the plane table from one station to another, the tripod stand should be kept vertical to avoid damage to the fixing arrangement.
7. Only the selected scale should be on the table.
8. Measurements should be taken carefully from the scale while plotting.
9. The stations on the ground are marked A, B, C, D etc. while the station points on the map are marked a, b, c, d etc.

Procedure of Field work

1. Reconnaissance

The area to be surveyed is thoroughly examined to find the best possible way for traversing. The traverse stations should cover the whole area and should indivisible. The provisions for check lines should be kept in mind.

2. Marking the stations

The selected stations are marked on the ground by wooden pegs. Reference sketches should be prepared for the stations so that they can be readily located in case the station pegs are removed.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 What is the principle of plane table surveying? [2014-S,2017-W,2018-W,2019-S,2020-W]

The principle of plane table surveying is parallelism this means that the rays drawn from the station to object on the paper are parallel to the line from the station to the object on the ground.

Q-2 What are the accessories is used in plane table surveying ? [2018-W]

Ans:

- Plane table
- Alidade
- Spirit level
- Compass
- U-fork

Q-3 What is alidade and its type? [2010-w]

Ans:

It is an instrument for sighting the object.

It may be of two types

- Plane alidade
- Telescopic alidade

Q -4 What are the working operation of plane table surveying? [2015-W]

Ans-

- Levelling
- Centering
- Orientation

POSSIBLE LONG TYPE QUESTIONS

Q-1 Describe the working operation of plane table surveying. [2015-W]

Q-2 Write down the procedure of radiation method.[2015-W,2018-W,2018-W-N]

Q-3 Write down the procedure of traversing method? [2014-W]

Q-4 Write down the components of a plane table?[2016-W,2018-WN]

CHAPTER N0-06

THEODOLITE SURVEYING AND TRAVERSING:

Learning objectives

6.1 Purpose and definition of theodolite surveying
6.2 Transit theodolite- Description of features, component parts, Fundamental axes of a theodolite, concept of vernier, reading a vernier, Temporary adjustment of theodolite
6.3 Concept of transiting –Measurement of horizontal and vertical angles.
6.4 Measurement of magnetic bearings, deflection angle, direct angle, setting out angles, prolonging a straight line with theodolite, Errors in Theodolite observations.
6.5 Methods of theodolite traversing with – inclined angle method, deflection angle method, bearing method, Plotting the traverse by coordinate method, Checks for open and closed traverse.
6.6 Traverse computation – consecutive coordinates, latitude and departure, Gale's traverse table, Numerical problems on omitted measurement of lengths & bearings
6.7 Closing error – adjustment of angular errors, adjustment of bearings, numerical problems
6.8 Balancing of traverse – Bowditch's method, transit method, graphical method, axis method, calculation of area of closed traverse.

6.1PURPOSE AND DEFINITION OF THEODOLITE SURVEYING

An instrument used for measuring horizontal and vertical angles accurately, is known as a theodolite. Theodolite is also use for prolongation or survey line, finding difference in elevation and setting out engineering work requiring higher precision I.e. ranging the highway and railway curves ,aligning tunnels ,etc.

Purposes of theodolite

Theodolites are commonly used for the following operations.

i. Measurements of horizontal angles.
ii. Measurements of vertical angles.
iii. Measurements of magnetic bearing of lines.
iv. Measurements of direct angles.
v. Measurements of deflection angels.
vi. Prolongation of straight lines.
vii. Running a straight line between two points.

viii. Laying off an angle by repetition method.

6.2 TRANSIT THEODOLITE- DESCRIPTION OF FEATURES, COMPONENT PARTS, FUNDAMENTAL AXES OF A THEODOLITE, CONCEPT OF VERNIER, READING A VERNIER, TEMPORARY ADJUSTMENT OF THEODOLITE

Features of theodolite

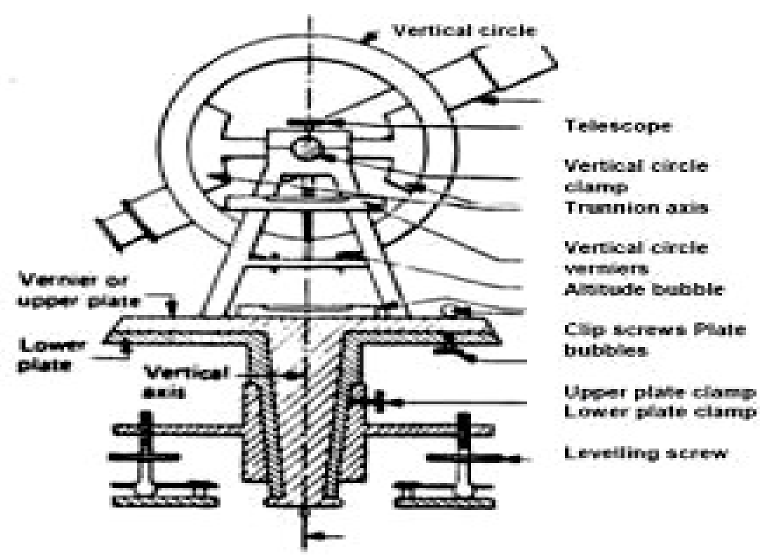
it consists of a moveable telescope mounted so it can rotate around horizontal and vertical axes and provide angular readouts. These indicate the orientation of the telescope, and are used to relate the first point sighted through the telescope to subsequent sightings of other points from the same theodolite position.

These angles can be measured with accuracies down to microradians or seconds of arc. From these readings a plan can be drawn, or objects can be positioned in accordance with an existing plan. The modern theodolite has evolved into what is known as a total station where angles and distances are measured electronically, and are read directly to computer memory.

In a transit theodolite, the telescope is short enough to rotate about the trunnion axis, turning the telescope through the vertical plane through the zenith; for non-transit instruments vertical rotation is restricted to a limited arc.

The optical level is sometimes mistaken for a theodolite, but it does not measure vertical angles, and is used only for levelling on a horizontal plane (though often combined with medium accuracy horizontal range and direction measurements).

Main Components



- **Upper Plate:** It is the base on which the standards and vertical circle are placed. For the instrument to be in corrent adjustment it is necessary that the upper plate must be perpendicular to the alidade axis and parallel to the trunnion axis.
- **Telescope:**It has the same features as in a level graticule with eyepiece and internal focussing for the telescope itself.
- **Vertical Scale (Circle)**: It is a full 400g scale. It is used to measure the angle between the line of sight (collimation axis) of the telescope and the vertical axis.
- **Vertical Clamp and Tangent Screw** : This allow free transiting of the telescope. When clamped, the telescope can be slowly transited using vertical tangent screw.
- **The Lower Plate:** It is the base of the whole instrument. It houses the foot screws and the bearing for the vertical axis. It is rigidly attached to the tripod mounting assembly and does not move.
- **Horizontal Scale (Circle)**: It is a full 400g scale. It is often placed between the upper and lower platesItis capable of full independent rotation about the trunnionaxis.
- **The Upper Horizontal Clamp and Tangent Screw:** used during a sequence or "round" of horizontal angle measurements.
- **The Lower Horizontal Clamp and Tangent Screw:** Thesemustonlybe usedat thestart of horizontal angle measurements to set the first reading to zero
- **Circle Reading and Optical Micrometer:** The vertical and horizontal circles require illumination in order to read them. This is usually provided by small circular mirrors

Definations and other technical terms

Following terms are used while making observations with a theodolite.

1. **Vertical axis:-** The axis about which the theodolite, may be rotated in a horizontal plane, is called vertical axis. Both upper and lower plates may be rotated about vertical axis.

2. **Horizontal axis:**- The axis about which the telescope along with the vertical circle of a theodolite, may be rotated in vertical plane, is called horizontal axis. It is also sometimes called trunnion axis or traverse axis.

3. **Line of collimation:** - The line which passes through the intersection of the cross hair of the eye piece and optical center of the objective and its continuation is called line ofcollimation. The angle between the line of collimation and the line perpendicular to the horizontal axis is called error of collimation.
The line passing through the eye piece and any point on the objective is called
line of sight.

4. **Axis of telescope: -** The axis about which the telescope may be rotated is called axis of
telescope.

5. **Axis of the level tube:** - The straight line which the tangential to longitudinal curve of the level tube at its center is called the axis of the level tube. When the bubble 0f the level tube is central, the axis of the level tube becomes horizontal.
Fig.3.2 Cross section of level tube

6. **Centering: -** The process of setting up a theodolite exactly over the ground station mark, is known as centering. It is achieved when the vertical axis of the theodolite is made to pass through the ground station mark.

7. **Transiting:** - The process of turning the telescope in vertical plane through 180° about its horizontal axis is known as transiting. The process is also sometimes known as reversing or plunging.

8. **Swing:** - A continuous motion of the telescope about the vertical axis in horizontal plane is called swing. The swing may be in either direction i.e. left or right. When the telescope is rotate in the clockwise right direction, it is known as right swing. If it is rotated in the anticlockwise left direction it is known as left swing.

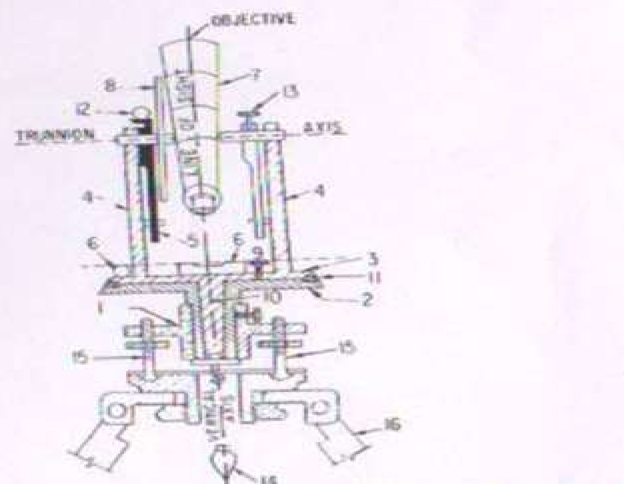
9. **Face left observations:** - When the vertical circle is on the left. of the telescope at the time of observations, the observations of the angles are known as face left observations.

10. **Face right observations:** - When the vertical circle is on the right of the telescope at the time of observations, the observations of the angles are known as face right observations.

11. **Changing face:-** It is the operation of changing the face of the telescope from left tom right and vice-versa.

12. **Telescope normal:** - Telescope is said to be normal when its vertical circle is to its left and the bubble of the telescope is up.

13. **Telescope inverted:** - A telescope is said to be inverted or reversed when its vertical circle is to its right and the bubble of the telescope is down.



1. Levelling head, 2. Lower plate or vernier plate, 3. Upper plate or vernier plate, 4. The standards (or A frames), 5. T-frame (or index bar), 6. Plate levels, 7. Telescope, 8. Vertical circle, 9. Upper clamp, 10. Inner axis, 11. Plate vernier, 12. Altitude bubble, 13. Vertical clamping screw, 14. Plumb bob, 15. Levelling screws, 16. Tripod

Fundamental lines of a transit theodolite

The fundamental lines of a transit theodolite are:
1) The vertical axis
2) The axis of plate bubble
3) The line of collimation which is also sometimes called line of sight.
4) The horizontal axis, transverse axis or trunnion axis.
5) The bubble line of telescope bubble or altitude bubble.

Concept of vernier

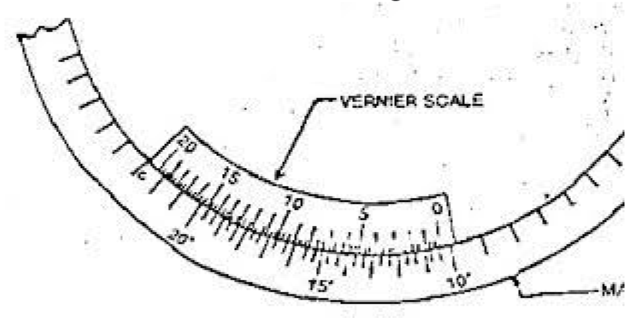
- Since the 17th century, theodolites have been the primary tool used in geodetic surveys. The theodolite employs a Vernier scale, which, with a little practice, can accurately measure angles to the minute.
- Learn angle basics. Before you can understand the Vernier scale on a theodolite, you must know how angles are broken up. A full circle consists of 360 degrees. Each degree (or 1/360 of a circle) can be divided into 60 minutes.
- Look at the Vernier scale on your theodolite or refer to the Vernier model in the References. The outer circle on the scale is fixed, while the inner scale is rotated as the scope of the theodolite is rotated. Each interval marking signifies 30 minutes, or 1/2 of a degree.
- Study the outer scale. The top number tells you what the angle is to the right. If you were to draw an arc to the right from the theodolite to the object being surveyed the top number would

give you the angle measure of that arc. The complementary arc to the left is read using the bottom number. Adding the angle to the right and the angle to the left will always equal 360 degrees. Note that the number on top decreases from left to right while the bottom number decreases.

- Find the "0" mark on the inner (movable) scale. This is your baseline. Wherever the 0 falls in relation to the outer scale is your degree measure. If the 0 mark falls right on the 130 (top) 230 (bottom) line, then your angle right is 130 degrees and your angle left is 230 degrees.
- Count the minutes. If the 0 mark of the inner scale falls directly on a mark on the outer scale, your degree measure exact. If, however, the 0 mark falls in between two interval markings, then your degree measure is not exact. Moving toward the right of the 0 mark, count the interval markings until one lines up directly with the interval mark beneath it. This is the minutes of your angle.

Reading a vernier

The value of 1big division on the vernier plate is 1'. The big division is divided into three small division then the value of 1 small division will be 1'÷3 or 60" ÷3=20". Reading the above picture ----> First saw on main scale reading, The vernier indicator 0 is crossed between 9° 40' and 10 °.



Adjustments of theodolite

The adjustments of theodolites are of two kinds.
1. Temporary adjustment
2. Permanent adjustment

Temporary adjustments: The adjustments which are required to be made at every instrument station before making observations are known as temporary adjustments.
The temporary adjustments of a theodolite include the following:

i. Setting up the theodolite over the station.
ii. Leveling of the theodolite
iii. Elimination of the parallax.

1) **Setting up: -** The operation of setting up a theodolite includes the centering of the theodolite over the ground mark and also approximate leveling with the help of tripod legs.

2) Centering: -

The operation with which vertical axis of the theodolite represented by a plumb line, is made to pass through the ground station mark is called centering.

The operation of centering is carried out in following steps:

i. Suspend the plumb bob with a string attached to the hook fitted to the bottom of the instrument to define the vertical axis.
ii. Place the theodolite over the station mark by spreading the legs well apart so that telescope is at a convenient height.
iii. The centering may be done by moving the legs radially and circumferentially till the plumb bob hangs within 1cm horizontally of the station mark.
iv. By unclamping the center shifting arrangement, the finer centering may now be made.

3. Approximate leveling with the help of the tripod:

It is very necessary to ensure that the level of the tripod head is approximately level before centering is done. In case there is a considerable dislevelment, the centering will be disturbed when leveling is done. The approximate levelling may be done either with reference to a small circular bubble provided on the tribarch or by eye judgment.

4. **Levelling of a theodolite:** The operation of the making the vertical axis of a theodolite truly vertical is known as leveling of the theodolite.

After having leveled approximately and centered accurately, accurate leveling is done with the help of plate levels. Two methods of leveling are adopted to the theodolites, depending upon the number of leveling screws.

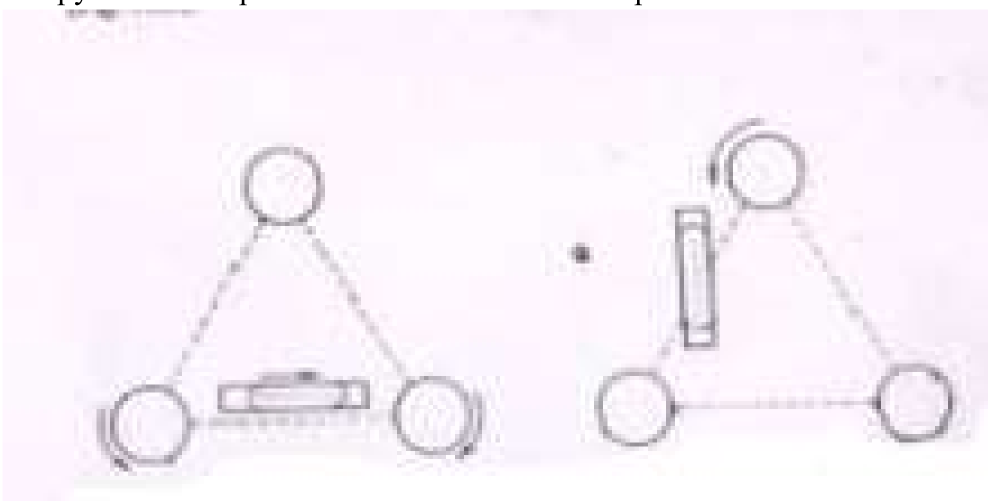
5. **Levelling with three screw head: -** The following steps are involved

Leveling of a theodolite with a three screw head

1) Turn the horizontal plate until the longitudinal axis of the plate level is approximately parallel to line joining any two leveling screws.
2) Bring the bubble to the center of its run by turning both foot screws simultaneously in opposite directions either inwards or outwards. The movement of the left thumb indicates the direction of movement of the bubble.
3) Turn the instrument through 180° in azimuth.
4) Note the position of the bubble. If it occupies a different position, move it by means of the same foot screws to the approximate mean of the two positions.

5) Turn the theodolite through 90° in a azimuth so that the plate level becomes perpendicular to the previous position.
6) With the help of the third foot screw move the bubble to the approximate mean position already indicated.
7) Repeat the process until the bubble retains the same position for every setting of the instrument in azimuth.

The mean position of the bubble is called the zero of the level tube. If the theodolite is provided with two plate levels placed perpendicular to each other, the instrument is not required to be turned through 90°.in this case, the longer plate level is kept parallel to any two foot screws and the bubble is brought to central position by turning both the foot screws simultaneously. Now with the help of the third foot screw, bring the bubble of second plate level central. Repeat the process till both the plate bubbles occupy the central position of their run for all the positions of the instrument.



6.3 CONCEPT OF TRANSITING –MEASUREMENT OF HORIZONTAL AND VERTICAL ANGLES.

Transiting: -

The process of turning the telescope in vertical plane through 180° about its horizontal axis is known as transiting. The process is also sometimes known as reversing or plunging.

Swing: -

A continuous motion of the telescope about the vertical axis in horizontal plane is called swing. The swing may be in either direction i.e. left or right. When the telescope is rotate in the clockwise right direction, it is known as right swing. If it is rotated in the anticlockwise left direction it is known as left swing.

Measurement of horizontal angles

1. To measure the angle by method of repetition:

Let ABC be the required angle between sides BA and BC to be measured by repetitio0n method as shown in Fig 3.5. When the measure of an angle is small, slight error in its sine value introduce a considerable error in the computed sides as the sine value of the angle changes rapidly. Therefore, for accurate and precise work, the method of repetition is generally used. In this method, The value of the angle is added several times mechanically and the accurate value of the angular measure is determined by dividing the accumulated reading by the number of repetition.

2) To measure the angle by reiteration method:

When several angles having a common vertex, are to be measured the reiteration method is generally adopted. In this method angles are measured successively starting from a reference station and finally closing on the same station. The operation of making last observation on the starting station is known as closing horizon. Making observations on the starting station twice provides a check on the sum of all angles around a station.

The sum should invariably be equal to 360°, provided the instrument is not disturbed during observations. As the angles are measured by sighting the stations in turn, this method is sometimes known as direction method of observation of the horizontal angles.

3. Measurement of vertical angles:

A vertical angle may be defined as the angle subtended by the inclined line of sight and the horizontal line of sight at the station in vertical plane. If the point sighted is above the horizontal axis of the theodolite, the vertical angle is known as angle of elevation and if it is below, it is known as angle of depression.

Procedure:

To measure a vertical angle subtended by the station B at the instrument station A, The following steps are involved:

i. Set up the theodolite over the ground station mark A. Level it accurately by using the altitude bubble.

ii. Set the zero of the vertical vernier exactly in coincidence with zero of the vertical scale using vertical clamp and vertical tangent screw. Check up whether the bubble of the altitude level is central of its run. If not, bring it to the centre of its run by means of the clip screw. In this position, the line of collimation of the telescope is horizontal and the verniers read to zero.

iii. Loosen the vertical circle clamp and move the telescope in vertical plane until the station B is brought in field of view. Use vertical circle tangent screw for accurate bisection.

iv. Read both the verniers of the vertical circle. The mean of two vernier readings gives the value of the vertical angle.

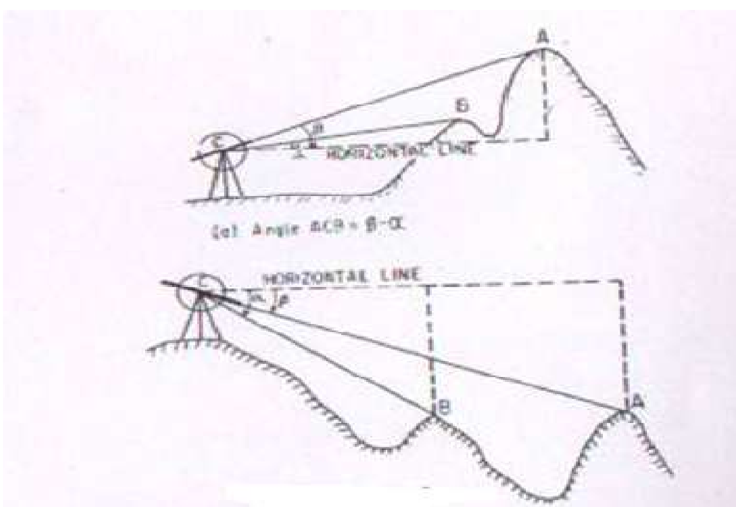
v. Change the face of the instrument and make the observations exactly in similar way as on the face left.

vi. The average of two values of the vertical angle is the required value of the vertical angle.

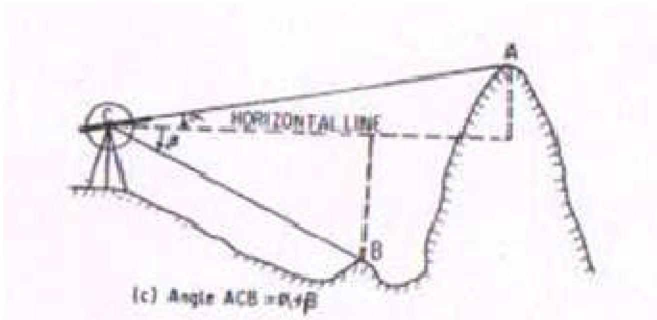
6.4 MEASUREMENT OF MAGNETIC BEARINGS, DEFLECTION ANGLE, DIRECT ANGLE, SETTING OUT ANGLES, PROLONGING A STRAIGHT LINE WITH THEODOLITE, ERRORS IN THEODOLITE OBSERVATIONS

Measurement of magnetic bearing of a line:

To measure the magnetic bearing of a line AB, the theodolite should be provided with either a circular or a trough compass.
The following steps are involved:
(i) Centre and level the instrument accurately on station A.
(ii) Set the vernier to read zero.
(iii) Loosen the lower plate and also release the magnetic needle.
(iv) Swing the telescope about its vertical axis until the magnetic needle points SN graduations of the compass box scale.
(v) Clamp the lower plate. Using the lower tangent screw bring the needle exactly against the zero graduation is exact coincidence with the north end of the needle.
(vi) In this position, the line of collimation of the telescope lies in the magnetic meridian at the place while verniers still reads to zero. The setting of the instrument is now said to be oriented on the magnetic meridian.



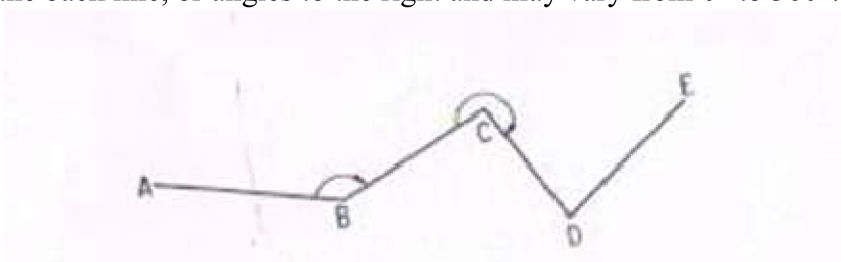
(vii) Loosen the upper plate, swing the instrument and bisect B accurately, using the upper tangent screw.



Measurement of vertical angle

(viii) Read both the vernier. The means of the two readings is the required magnetic bearing of the line AB.
(ix) Change the face of the instrument and observe the magnetic bearing exactly in a similar way as on the left face.
(x) The mean of magnetic bearings observed on both faces is the accurate value of the magnetic bearing of line AB.

Measurement of direct angles: The angle measured clockwise from the preceding line to the following line is called a direct angle. These angles are also sometimes known as azimuths from the back line, or angles to the right and may vary from 0° to 360°.



Setting out angles

To set out right angles in the field, a measuring tape, two ranging poles, pegs and three persons are required.

The first person holds together, between thumb and finger, the zero mark and the 12 metre mark of the tape. The second person holds between thumb and finger the 3 metre mark of the tape and the third person holds the 8 metre mark.

When all sides of the tape are stretched, a triangle with lengths of 3 m, 4 m and 5 m is formed (see Fig. 20), and the angle near person 1 is a right angle.

NOTE: Instead of 3 m, 4 m and 5 m a multiple can be chosen: e.g. 6 m, 8 m and 10 m or e.g. 9 m, 12 m and 15 m.

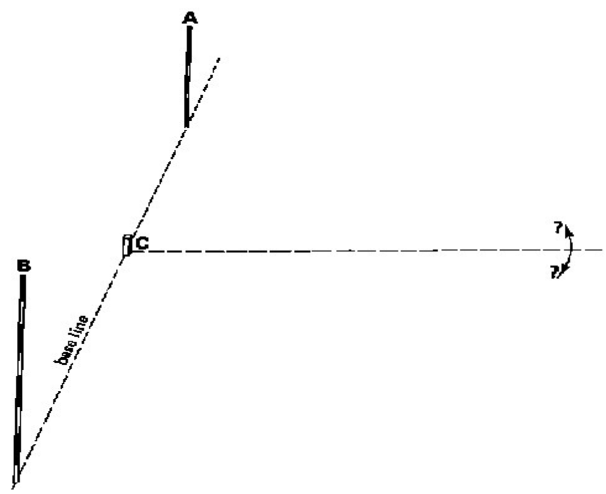
The 3-4-5 method

EXAMPLE: Setting out a right angle

Step 1

In Fig. 2 la, the base line is defined by the poles (A) and (B) and a right angle has to be set out from peg (C). Peg (C) is on the base line.

Fig. a Setting out a right angle, Step 1

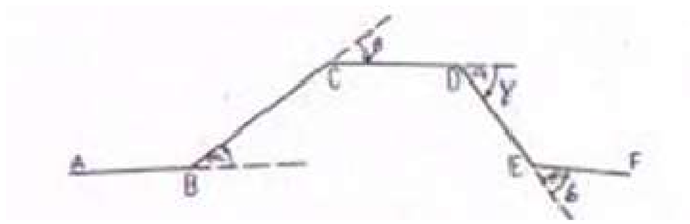


Step 2

Three persons hold the tape the way it has been explained above. The first person holds the zero mark of the tape together with the 12 m mark on top of peg (C). The second person holds the 3 m mark in line with pole (A) and peg (C), on the base line. The third person holds the 8 m mark and, after stretching the tape, he places a peg at point (D). The angle between the line connecting peg (C) and peg (D) and the base line is a right angle (see Fig.). Line CD can be extended by sighting ranging poles.

Measurement of deflection angles:

The angle which any survey line makes with the prolongation of the preceding line is called deflection angle. Its value may vary from 0° to 180° and is designated as right deflection angle if it is measured in clockwise direction and as left deflection angle if it is measured in an anticlockwise direction. In fig. the deflection angles stations B and E respectively are left deflection angles whereas angles stations C and D are right deflection angles.



Prolongation of a straight line: Prolongation of any straight line AB to a point F may be done by any one of the following methods:

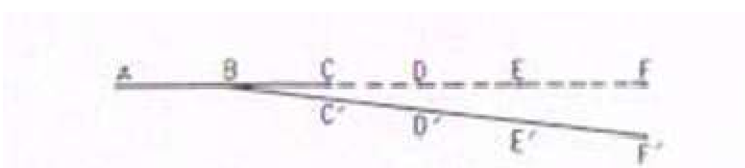
First method: -

The following steps are involved:

ii. Set up the theodolite at A, center and level it accurately.

ii. Bisect an arrow centered over the mark at B.
iii. Establish a point C in the line of sight at a convenient distance.
iv. Shift the instrument to B.
v. Centered t5he theodolite over B, level it and sight C. Establish another point
vi. Proceed in a similar manner until the desire point F is established.

Second method:-



The following steps are involved:
I. Set up the theodolite at B and centered it carefully.
II. Bisect A accurately and clamp both the plates.
III. Plunge the telescope and establish a point C in the line of sight.
IV. Shift the instrument to C and center it carefully.
V. Bisect B and clamp both the plates.
VI. Plunge the telescope and establish the point D in the line of sight.
VII. Continue the process till the last point F is established.

Sources of error in theodolite work:

The sources of error in theodolite work may be broadly divided into three categories, i.e.

1. Instrument error.
2. Personal error
3. Natural errors

Instrumental errors: - The theodolites are very delicate and sophisticated surveying instrument. In spite of best efforts during manufacturing perfect adjustment of fundamental axes of the theodolite, is not possible. The unadjusted errors of the instrument are called residual errors. We shall now discuss how best to avoid the effect of these residual error while making field observations.Instrumental errors may also be divided into different types as discussed below:

1. **Error due to imperfect adjustment of plate level: -** If the plate bubbles are not adjusted properly, the vertical axis of the instrument does not remain vertical evenif plate bubbles remain at the center of their run. Non verticality of the vertical axis introduced error in the measurements of both the horizontal and vertical angles. Due to non verticality of vertical axis the horizontal plate gets inclined and it does not remaining in horizontal plane. The error is especially important while measuring the horizontal angles between stations at considerable different elevations.

Elimination of the error: - this error can be eliminated only by leve3lling the instrument carefully, with the help of the altitude or telescope bubble, before starting the observations.

2. **Error due to line of collimation not being perpendicular to the trunnion axis: -** If the line of collimation of the telescope is not truly perpendicular to the trunnion axis, it generates a cone when it is rotated about the horizontal axis. The trace of the intersection of the conical surface with the vertical plane containing the station sighted the hyperbolic. This imperfect adjustment introduces errors in horizontal angels measured between stations at different elevations.

Personal errors:

Personal errors are due to mainly following causes.

(i) inaccurate centring over a station
(ii) slip of instrument when not put firmly on the tripod
(iii) faulty manipulation of instrument controls like clamping the instrument and operating wrong tangent screw
(iv) inaccurate leveling, inaccurate bisection of target
(v) non-verticality of ranging rod
(vi) displacement of target stations, parallax
(vii) errors in sighting, reading and recording

Natural errors

Errors due to natural causes include the followings.

(i) settlement of tripod due to soft soil
(ii) wind causing vibrations and turning
(iii) high temperature causing faults in reading due to refraction, differential expansion of different parts
(iv) direct sunlight on the instrument making sighting and reading difficult.

6.5 METHODS OF THEODOLITE TRAVERSING WITH – INCLUDED ANGLE METHOD, DEFLECTION ANGLE METHOD, BEARING METHOD, PLOTTING THE TRAVERSE BY COORDINATE METHOD, CHECKS FOR OPEN AND CLOSED TRAVERSE.

Traversing

The method in which the magnetic bearings of traverse lines are measured by a theodolite fitted with s compass is called traversing by fast needle method. The direction of the magnetic meridian is not established at each station but instead, the magnetic bearings of the lines are measured with a reference so that direction of the magnetic meridian established at the first station. There are three methods of observing the bearings of lines by fast needle method.

1. Direct method with transiting,
2. Direct method without transiting,
3. Back bearing method.

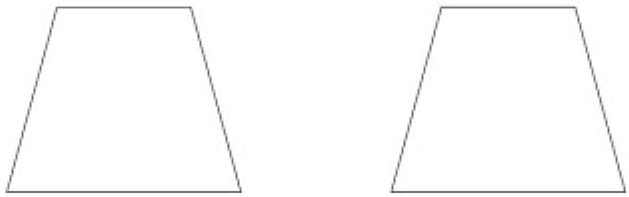
Traversing By Direct Observation Of Angles

In this method, the angles between the lines are directly measured by a theodolite and the magnetic bearing of other lines can be calculated in this method. The angles measured at different stations may be either

1. Included Angles and
2. Deflection Angles
3. Bearing method

Traversing by Included Angle

An included angle at a station is either of the two angles formed n\by two survey lines meeting there and these angles should be measured clockwise. The method consists simply in measuring each angle directly from a back sight on the preceding station. The angled may also be measured by repetition. The angles measured from the back station may be interior or exterior depending on the direction of progress.

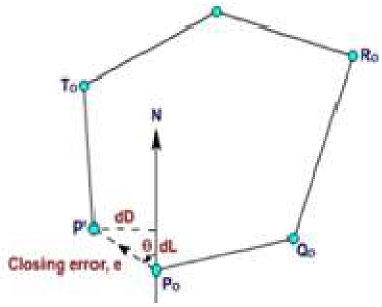


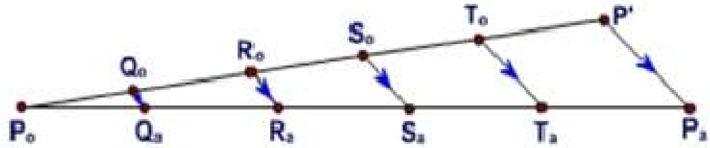
Traverse by Deflection Angles

A deflection angle is an angle in which a survey line makes with the prolongation of the preceding line. It is designated as right (R) or left (L) as it is measured clockwise or anti-clockwise from the prolongation of the previous line. This type of traversing is more suitable for the survey of roads, railways, pipe-lines, etc where the survey lines make small deflection angles.

Bearing Method:

- Set up and level the theodolite at station P of the traverse PQRSTP, a closed traverse.
- Using the upper clamp and upper tangent screw, set vernier A to read zero.
- Loosen the magnetic needle. Release the lower clamp and point the telescope in the direction of the magnetic meridian till the magnetic needle comes to rest at the zero position using the lower tangent screw the north end of the magnetic needle to read exactly zero.
- Release the upper plate and swing the instrument to bisect the signal at Q. With theupper tangent screw, bisect the station mark exactly. Read Vernier A, this gives the bearing of the line PQ.
- Keeping both the clamps tight, shift the instrument to Q. Set up and level the instrument.
- Check the reading on Vernier A. It should be the same as the magnetic bearing of the line PQ (if not, this can be corrected and the bearing value noted earlier be set on Vernier A).
- Release the upper clamp. Swings the instrument clockwise to bisect the station mark at R. Using upper tangent screw bisect mark R exactly. Read the Vernier at A and note down the reading.
- With both clamps tight, shift the instrument to R and repeat the procedure. The work is continued at all stations in a similar manner.





Plotting the traverse by coordinate method

This method is generally used for plotting precise work, mainly a theodolite traverse, both closed and open.

In this method, the position of different points are plotted on a plan with reference to two lines yy_1 (y-axis) and xx_1 (x-axis) which are respectively parallel and perpendicular to the meridian (Fig. 5.25). These reference lines are called the axes of the co-ordinates, and the point of their intersection O, called the origin.

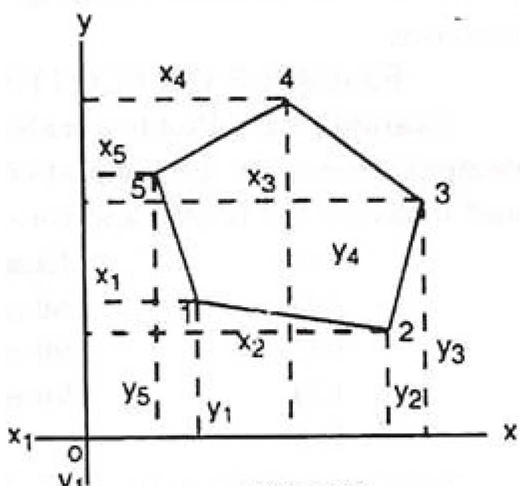


FIG.5.25

The origin may either be any traverse station or entirely outside the traverse. The distance of a point from each of the axes are called its co-ordinates.

Checks for open and closed traverse

Checks in Close traverse

- Sum of All Latitude is always Zero in close Traverse.
- Sum of all Departures is always zero in close a traverse.
- Sum of the internal angle is (2n-4)90°
- Sum of the external angle is (2n+4)90°

If the Sum doesn't come zero then there is some error.

CHECK ON OPEN TRAVERS

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

a). *Taking cut-off lines.* Cut-off lines are taken between some intermediate stations of the open traverse. Suppose ABCDEFG represents an open traverse. Let AD and DG be the cut-off lines. The length and the magnetic of the cut-off lines are measured accurately. After plotting the traverse, the distances and bearings are noted from the map. These distances and bearings should tally with the actual records obtained from the field.

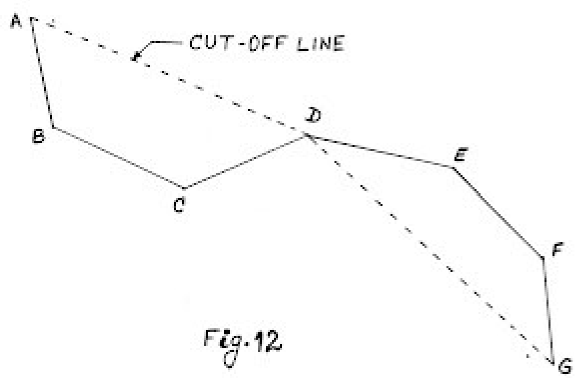


Fig.12

b). *Taking an auxiliary point.* Suppose ABCDEF an open traverse. A permanent point P is selected on the side of it. The magnetic bearings of this point are taken from traverse stations A, B, C, D, etc. If the survey carried out accurately and so is the 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

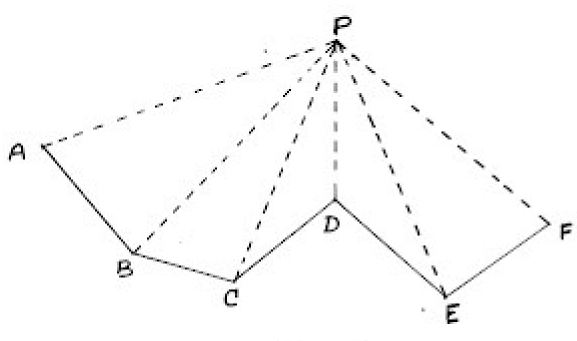


Fig. 13

6.6 TRAVERSE COMPUTATION – CONSECUTIVE COORDINATES, LATITUDE AND DEPARTURE, GALE'S TRAVERSE TABLE, NUMERICAL PROBLEMS ON OMITTED MEASUREMENT OF LENGTHS & BEARINGS

CONSECUTIVE COORDINATES,

Consecutive Coordinates : The latitude & departure of a point calculated with reference to the preceding point for what are called consecutive coordinates. Consecutive coordinates may be positive or negative, depending upon the quadrant in which they lie.

Latitude:

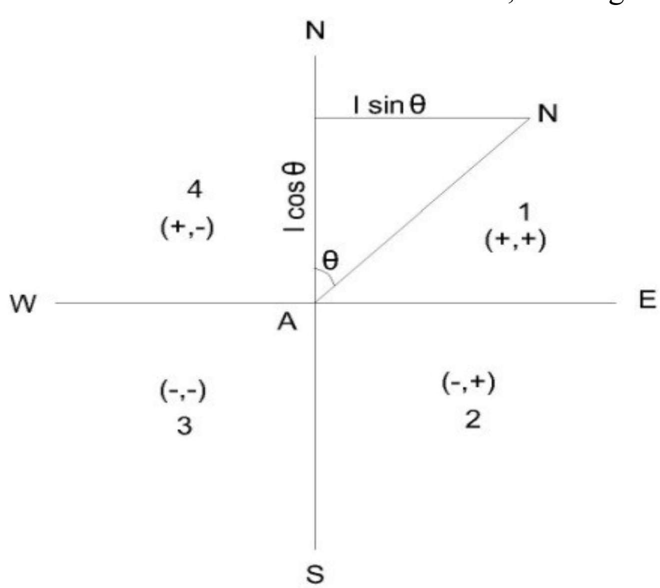
The projection of the line parallel to the meridian (N-S line) is called the "latitude" of the line.

Departure:

The projection of the line perpendicular to meridian (N-S line) is called the "departure" of the line.

The latitude when measured upward or northward along the meridian, is positive and termed as "northing" and when it is measured downward or southward along the meridian it is negative and is called "southing".

The departure when measured eastward or towards right, is positive and is known as "easting" and when it is measured westward or to the left, it is negative and is known as "westing".



if the length of line AB is known and its reduced bearing from meridian (i,e,_) is known, the latitude and the departure may be determined.

Latitude of a line = Length of line x The cosine of reduced bearing of line
= Length x cos Θ

Departure of a line = Length of line x The sine of reduced bearing of line
= Length x sin Θ

Whole circle bearing Quadrant (WOB) Latitude Departure
0° to 90 ° (1) or NE + +
90° to 180° (2) or SE - +
180° to 270° (3) or SW - -
270° to 360° (4) + -

Consecutive and independent coordinates
The latitude and departure of any point with reference to the preceding point are known as “consecutive coordinates”. And the coordinates of any point with reference to a common origin are called the “independent coordinates” of the point. The independent coordinates are also known as “total latitude” and “total departure” of the points.

The independent coordinates of any point may be determined by adding (algebraically) the latitudes and departures of the lines between that point and the origin. Thus, x-coordinate (or total = x-coordinate of the first point of the departure) of any point traverse+ Algebraic sum of the departures of the lines between the first point and that point y-coordinate (or total = y-coordinate of the first point of the latitude) of any point traverse + Algebraic sum of latitudes of the lines between the first point and that point

The above rule follows that x-or y-coordinate of last = x. or y-coordinate of the first point point of the traverse + Algebraic sum of all departures or Latitudes

Note
The theodolite traverse should always be plotted with the help of rectangular coordinates.

Adjustment of closing errors in a closed traverse:

If the survey work is correct, in a closed traverse, the algebraic sum of latitudes (I e, ∑L) should be equal to zero and also the algebraic sum of latitudes (I e, ∑D) should be equal to zero.

It follows that the sum of northings should be equal to sum of southings and the sum of eastings should be equal to sum of westings. closing errors in a closed traverse
There are always errors in a theodolite traverse mainly due to two sources, viz :
(1) the angles between the sides, and
(2) the lengths of the sides.
Out these, the first is usually less important than the second. The traverse is plotted according to the field measurements, the same will not close on paper and the end point of the traverse will not coincide exactly with the starting point. The distance by which the last point of the traverse falls short to coincide with the starting point is called the “error of closure” or “closing error”. As per the Fig shows plotting of traverse ABCD. The traverse is not closing at

Gales Traverse Table:

The following steps may observed.

(1) Find out the sum of the observed included angles which should be equal to (2n-4) right angles according as the interior or exterior angles are measured. If they are not equal, apply the necessary corrections to the angles so that the sum of the corrected angles is exactly equal to (2^{nd} right angles.

(2) From the observed bearing of the first line AB and the corrected included angles, calculate the WCB of all the other lines BC, CD, etc. As a check, find out the bearing of the first line which should be equal to its observed bearing.

(3) Calculate the reduced bearings of lines from the whole circle bearing and find out the respective quadrants.

(4) Compute the latitudes and departures of lines, I e, the consecutive coordinates (coordinates with respect to preceding point, e g, in a closed traverse ABCD, the coordinates of A, with respect to D and coordinates of B with respect to station A, and so on) from their observed lengths and corrected reduced bearings. For example in the above traverse, to compute the coordinates of A, the length and reduced bearing of line DA should be taken, and similarly for point B, the length and reduced bearing of line AB should be considered.

(5) Find out the algebraic sum of latitudes (∑L) and that of departures (∑D). Apply necessary corrections to the latitudes and departures so that their sum is equal to zero for closed traverse.

(6) From the corrected consecutive coordinates, obtain the independent coordinates of the lines such that they are all positive and the whole traverse lies in the first quadrant (NE).

Q-1 The following records are obtained in a traverse survey, where the length and bearings of the last line were not recorded :

Line	Length (m)	Bearing
AB	75.50	$30^0 24^/$
BC	180.50	$110^0 36^/$
CD	60.25	$210^0 30^/$
DA	?	?

Compute the length and bearing of line DA

Solution

Let L=Length of DA

ϴ=bearing of DA

The calculation of latitudes and departures of the traverse are arranged in tubular form as follows:

Line	Length in m (L)	WCB	RB (Θ)	Latitude (L cosΘ)	Departure (L sin θ
AB	75.50	$30^0 24^/$	$30^0 24^/$NE	75.50 cos $30^0 24^/$=+65.12	75.50 sin $30^0 24^/$=+38.21
BC	180.50	$110^0 36^/$	$69^0 24^/$SE	180.50 cos$69^0 24^/$ =-63.51	180.50 sin $69^0 24^/$=+168.95
CD	60.25	$210^0 30^/$	$30^0 30^/$SW	60.25 cos $30^0 30^/$=-59.91	60.25 sin $30^0 30^/$=-30.60
DA	L	-------	θ	L cos θ	L sin θ

In a closed traverse, the algebraic sum of latitudes as also that of departures must be equal to zero.

So +65.12-63.51-51.91+L cos θ =0

L cos θ =50.3

Again +38.21+168.95-30.60+ L sin θ=0

L sin θ=-176.56

Since the latitude of line DA is positive and the departure is negative , the line DA will be in the NW quadrant.

$$\frac{L\ cos\theta}{L\ sin\theta} = \frac{176.56}{50.30}$$

Tan θ= 3.5101

θ =$74^0 5^/$

Therefore , Bearing of DA = N $74^0 5^/$W

Distance DA=$\sqrt{(50.3)^2 + (176.56)^2}$

=183.58 m ANS

6.7 CLOSING ERROR – ADJUSTMENT OF ANGULAR ERRORS, ADJUSTMENT OF BEARINGS, NUMERICAL PROBLEMS

CLOSING ERROR

Closing Error: The closing error is the actual distance by which the traverse fails to close. Error in distance occurs when the end point does not coincides with the starting point.

While plotting a closed traverse, the end point coincides exactly with the starting point provided that work is correct. Also in a closed traverse, the algebraic sum of the latitudes- (the sum of northings-the sum of southings) should equal zero, and the algebraic sum of the departures (the sum of eastings – the sum of westings) should equal zero.

But due to errors in the field measurements of angles and distances, the traverse if plotted according to the field measurements will not close on the starting point. The distance by which the end point of a survey fails to meet with the starting one is called the closing error or error of closure. In fig. 9.23, A and A_1 are the starting and end points respectively, and AA_1 represents the closing error.

The two components of this error (A_1A_2 and AA_2) parallel and perpendicular to the meridian may be determined by finding the algebraic sum of the altitudes (ΣL), and that of the departures (ΣD). Since the triangle A_1A_2A is right angled at A_2, the linear closing error (AA_1) is equal to the square root of the sum of the squares of the algebraic sum of the latitudes and that of the departures.

$= AA_1$

closing error $=\sqrt{\sum L^2 + \sum D^2}$

The direction of the closing error is given by the relation.,

$\tan\theta = \frac{\Sigma D}{\Sigma l}$ where θ is the reducing bearing. The signs of ΣL and ΣD will define the quadrant of the closing error.

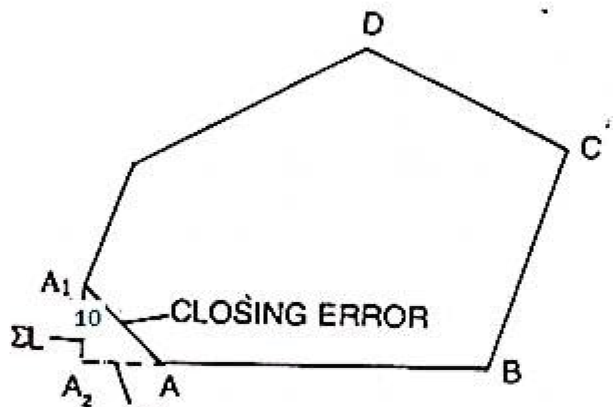


Fig. 9 23

Adjustment of Angular Error:

(a) Angular Error:

The theoretic sum of the interior angles of a traverse should equal (2N-4) right angles, and that of the exterior angles should equal (2N+4) right angles, where N is the number of sides of a closed traverse. The difference between the theoretic sum and the sum of the measured angles in a closed traverse is called the angular error of closure. It should not exceed the least count x$\sqrt{N}$.

When all angles are measured with equal care and under similar conditions, this error is distributed equally among all the angles. However, if the accuracy of some angle or angles is suspected due to peculiar field conditions, the whole or the most of the angular error may be assigned to that angle or angles.

Adjustment of Bearings:

(a) Closing Error in Bearings:

If traversing is done by taking bearings of the lines, the closing error in bearing may be determined by comparing the back and fore bearings of the last line of the closed traverse as observed at the first and last stations of the traverse respectively. When the traverse ends on a line of known bearing, the error in bearing may be determined by finding the difference between its observed bearing and known bearing.

b) Adjustment:

If e is the closing error in bearing, and N is the number of the sides of the traverse, then the correction applied to the bearings of the sides are:

Correction to the Bering =e/N

‘‘ ‘‘ second ‘‘ = 2e/N

‘‘ ‘‘ third ‘‘ = 3e/N

and so on to the last bearings Ne/N=e

The same results will be obtained if a correction of e/N is applied to each of the observed angles. Then the first bearing will be changed by e/N the second by 2e/N, the third by 3e/N, and so on.

Q-3

The following are the details of closed theodolite traverse? Find consecutive coordinates of points A,B,C,D also find closing error, relative error of closure and direction of closing error.

Line	Length	Point	Included angle	Remark
AB	250	A	$95^0 19^/$	Observed bearing of line AB= $86^0 40^/$
BC	126	B	$88^0 31^/$	
CD	257	C	$90^0 13^/$	
DA	120	D	$86^0 01^/$	

Solution

Coordinates of points

Line	Length	Point	Included angle	WCB	RB	Latitude	Departure
AB	250	A	$95^0 19^/$	$86^0 40^/$	$N86^0 40^/E$	14.53	249.57
BC	126	B	$88^0 31^/$	$355^0 11^/$	$N4^0 49^/W$	125.55	-10.57
CD	257	C	$90^0 13^/$	$265^0 24^/$	$S85^0 24^/W$	-20.61	-256.17
DA	120	D	$86^0 01^/$	$171^0 25^/$	$S8^0 35^/E$	-118.65	17.90

By rule

$\sum$Northing=140.08

$\sum$southing =139.26

$\sum$Latitude=0.82

$\sum$Easting=267.47

$\sum$Westing=266.74

$\sum$Departure=0.73

ii) closing error =$\sqrt{\sum L^2 + \sum D^2}$

=$\sqrt{0.82^2 + 0.73^2}$= 1.0978 m

iii) Relative error =$\frac{closing\ error}{perimeter\ of\ traverse}$=$\frac{1.0978}{753}$=1.457 X 10^{-3}

iv) Direction of closing error =tanθ=$\frac{\sum D}{\sum L}$=$\frac{0.73}{0.82}$=$41^0 40^{/} 36.77^{//}$.

6.8 BALANCING OF TRAVERSE – BOWDITCH'S METHOD, TRANSIT METHOD, GRAPHICAL METHOD, AXIS METHOD, CALCULATION OF AREA OF CLOSED TRAVERSE.

Balancing the traverse

When the closing error in latitude and in departure is determined, the latitudes, and departures should be adjusted such that the algebraic sum of the latitudes should be adjusted such that the algebraic sum of the latitudes and departures should each be equal to zero. This operation of applying correction to the latitudes and the departuresis called the "balancing of the traverse". If one or more sides of a traverse have not been measured with equal care due to some typical field conditions, the whole or the largest part of the error may be adjusted to the same side or sides. But, if all the sides have been measured with equal precision and care, the following rules may be applied to determine the corrections foe balancing the traverse.

Bowditch's rule

The Bowditch's Rule or the "compass rule" is generally used to balance the traverse when the angular as well as linear measurements are taken with equal precision. By this rule, the total error in latitude and in departure is distributed in proportion to the length of the sides.

Correction to the latitude or to the departure of any line

=Total error in latitude or departure X $\frac{Length\ of\ the\ line}{Perimeter\ of\ the\ travers}$

The traverse can also be adjusted as it is explained in "compass traverse adjustment".

Transit rule

The transit rule may be applied to balance the traverse when the angular measurementsare taken with greater care and precision than the linear measurements as in the case of a theodolite and stadia traverse. According to this rule,

Correction to latitude of any line

$= Total\ error\ in\ latitude\ X\ \ \frac{Latitude\ of\ that\ line}{Arithmetica\ \ sum\ of\ all\ the\ latitudes}$

Similarly,

Correction to departure of any line

= Total error in departure X $\frac{Departure\ of\ that\ line}{Arithmetical\ sum\ of\ all\ the\ departure}$

Graphical method

Graphical method uses Bowditch's formula for solving graphically whereas Axis method is used in case of length corrections which are having accurate angles.

Axis method:

(i) This method is used to balance a traverse where angles are measured more precisely than the lengths and thus this axis method is used for correction of lengths only.

(ii) Since the angles are measured more precisely and therefore the direction of lines do not change much while applying the correction and thus the general shape of the traverse do not change much.

(iii) In this method, the correction applied to any traverse line is expressed as

(one half of closing error × Length of traverse line)/Axis length

Area of closed traverses:

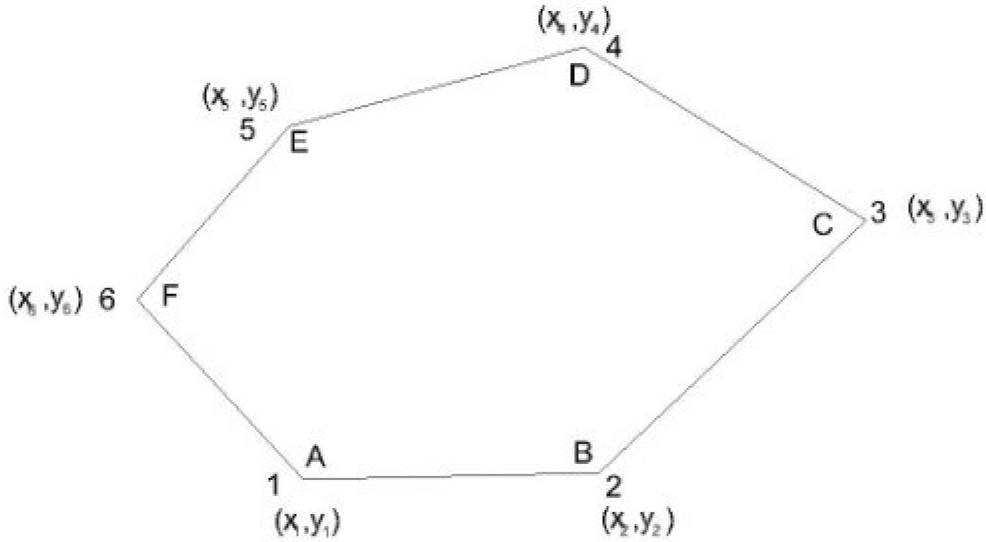
The following methods are generally used for calculating the area of closed traverses :

a)Area from coordinates (y and x).

b)Area from latitudes and double meridian.

c)Area from departures and total latitudes.

a) Area of closed traverses from coordinates:



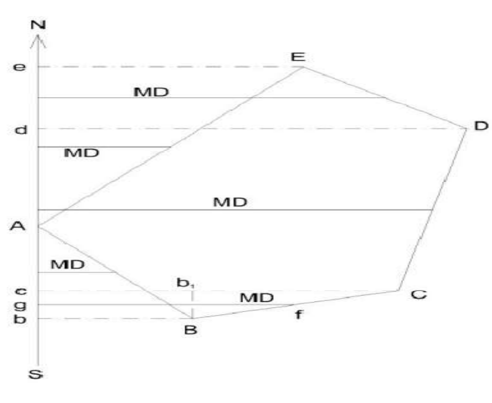
Thus, the area of closed traverse is given below:

Area = ½[y1(x2, x6) +y2(x3, x1)+ y3(x4– x2) + y4(x5, x3) + y5(x6, x4) + y6(x1, x5)]
Area = ½[y1(x2 – x6) + y2(x3– x1) + y3(x4 – x2) + … +yn(xn+1 –xn-1)]
Where x1, x2, x3, etc. are the abscissae and y1, y2, y3 etc. are the ordinates.
The abscissae or x-coordinates are along X-axis or east-west line.

The ordinates or y-coordinates are along Y-axis or north-south line.

Area of closed traverse from latitudes and double meridian distances (DMD):

The meridian distance (MD) of a line or "longitude" is the perpendicular distance of the middle point of the line from the reference meridian.
The double meridian distance (DMD) or the double longitude of a line is equal to the sum of the meridian distance of the two ends of the lines. The MD of various lines can be calculated by the following principles:



(1) The DMD of first line is equal to the departure of that line.

(2) The DMD of each succeeding line is equal to DMD of the preceding line plus the departure of the line itself.
(3) The DMD of the last line is numerically equal to the departure of the last line but with opposite sign.

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 Define vertical and horizontal axis of a theodolite? [2014-S]

Ans : . Vertical axis:- The axis about which the theodolite, may be rotated in a horizontal plane, is called vertical axis. Both upper and lower plates may be rotated about vertical axis. Horizontal axis:- The axis about which the telescope along with the vertical circle of a theodolite, may be rotated in vertical plane, is called horizontal axis. It is also sometimes called trunnion axis or traverse axis.

Q-2 Define transiting? [2010,2014-S]

Ans : The process of turning the telescope in vertical plane through 180° about its horizontal axis is known as transiting. The process is also sometimes known as reversing or plunging.

Q-3 what do you mean by telescopic inverted and telescopic normal? [2014-S]

Ans : Telescope normal: - Telescope is said to be normal when its vertical circle is to its left and the bubble of the telescope is up.
Telescope inverted: - A telescope is said to be inverted or reversed when its vertical circle is to its right and the bubble of the telescope is down.

Q-4 Define latitude and departure of a line? [2011-S,2012-S,2015-S,2016-S,2019-S,2019-W]

Ans: Latitude:-The projection of the line parallel to the meridian (N-S line) is called the "latitude" of the line.
Departure:-The projection of the line perpendicular to meridian (N-S line) is called the "departure" of the line

Q-5 Define closing error of a traverse? [209-W]

Ans: The distance by which the last point of the traverse falls short to coincide with the starting point is called the "error of closure" or "closing error".

POSSIBLE LONG TYPE QUESTIONS

Q-1 write the procedure for measuring the vertical distance by the theodolite? [2018-W]

Q-2 Describe about the temporary adjustment of a theodolite?[2011-S,2012-S,2017-W,2019-S]

Q-3 Write the procedure for prolongation of a line? [2017-W]

Q -5 Describe the process of measuring the magnetic bearing of a line by theodolite. [2018-W]

CHAPTER NO-07

LEVELLING AND CONTOURING :

Learning objectives

7.1 Definition and Purpose and types of leveling– concepts of level surface, Horizontal surface, vertical surface, datum, R. L., B.M.
7.2 Instruments used for leveling, concepts of line of collimation, axis of bubble tube, axis of telescope, Vertical axis.
7.3 Levelling staff – Temporary adjustments of level, taking reading with level, concept of bench mark, BS, IS, FS, CP, HI.
7.4 Field data entry – level Book – height of collimation method and Rise & Fall method, comparison, Numerical problems on reduction of levels applying both methods, Arithmetic checks.
7.5 Effects of curvature and refraction, numerical problems on application of correction.
7.6 Reciprocal leveling – principles, methods, numerical problems, precise leveling.
7.7 Errors in leveling and precautions, Permanent and temporary adjustments of different types of levels.
7.8 Definitions, concepts and characteristics of contours.
7.9 Methods of contouring, plotting contour maps, Interpretation of contour maps, toposheets.
7.10 Use of contour maps on civil engineering projects – drawing cross-sections from contour maps, locating proposal routes of roads / railway / canal on a contour map, computation of volume of earthwork from contour map for simple structure.
7.11 Map Interpretation: Interpret Human and Economic Activities (i.e.: Settlement, Communication, Land use etc.), Interpret Physical landform (i.e.: Relief, Drainage Pattern etc.), Problem Solving and Decision Making

7.1 DEFINITION AND PURPOSE AND TYPES OF LEVELING– CONCEPTS OF LEVEL SURFACE, HORIZONTAL SURFACE, VERTICAL SURFACE, DATUM, R. L., B.M.

Levelling is the art of finding the relative heights and depths of the objects on the surface of the earth. It is that part of surveying which deals with the measurements in vertical plane.

Purpose of levelling:

- Levelling is of prime importance to an engineer for the purpose of planning, designing and executing various engineering projects such as roads, Railways, canals, dams, water supply and sanitary schemes etc. The Principle of leveling lies in furnishing a horizontal sight and finding the vertical distances of the points above this line. This is done with the help of a level and a levelling staff respectively.

Types of leveling

1. Direct leveling
2. Trigonometric leveling
3. Barometric leveling
4. Stadia leveling

Direct Leveling

It is the most commonly used method of leveling. In this method, measurements are observed directly from leveling instrument. Based on the observation points and instrument positions direct leveling is divided into different types as follows:

- Simple leveling
- Differential leveling
- Fly leveling
- Profile leveling
- Precise leveling
- Reciprocal leveling

Simple Leveling

It is a simple and basic form of leveling in which the leveling instrument is placed between the points which elevation is to be find. Leveling rods are placed at that points and sighted them through leveling instrument. It is performed only when the points are nearer to each other without any obstacles.

Differential Leveling

Differential leveling is performed when the distance between two points is more. In this process, number of inter stations are located and instrument is shifted to each station and observed the elevation of inter station points. Finally difference between original two points is determined.

Fly Leveling
Fly leveling is conducted when the benchmark is very far from the work station. In such case, a temporary bench mark is located at the work station which is located based on the original benchmark. Even it is not highly precise it is used for determining approximate level.

Profile Leveling
Profile leveling is generally adopted to find elevation of points along a line such as for road, rails or rivers etc. In this case, readings of intermediate stations are taken and reduced level of each station is found. From this cross section of the alignment is drawn.

Precise Leveling
Precise leveling is similar to differential leveling but in this case higher precise is wanted. To achieve high precise, serious observation procedure is performed. The accuracy of 1 mm per 1 km is achieved.

Reciprocal Leveling
When it is not possible to locate the leveling instrument in between the inter visible points, reciprocal leveling is performed. This case appears in case of ponds or rivers etc. in case of reciprocal leveling, instrument is set nearer to 1st station and sighted towards 2nd station.

Trigonometric Leveling
The process of leveling in which the elevation of point or the difference between points is measured from the observed horizontal distances and vertical angles in the field is called trigonometric leveling. In this method, trigonometric relations are used to find the elevation of a point from angle and horizontal distance so, it is called as trigonometric leveling. It is also called as indirect leveling.

Barometric Leveling
Barometer is an instrument used to measure atmosphere at any altitude. So, in this method of leveling, atmospheric pressure at two different points is observed, based on which the vertical difference between two points is determined. It is a rough estimation and used rarely.

Stadia Leveling
It is a modified form of trigonometric leveling in which Tacheometer principle is used to determine the elevation of point. In this case the line of sight is inclined from the horizontal. It is more accurate and suitable for surveying in hilly terrains.

Defination of terms used in levelling-concepts of level surface, Horizontal surface, Vertical surface, Datum, R.L, B.M.

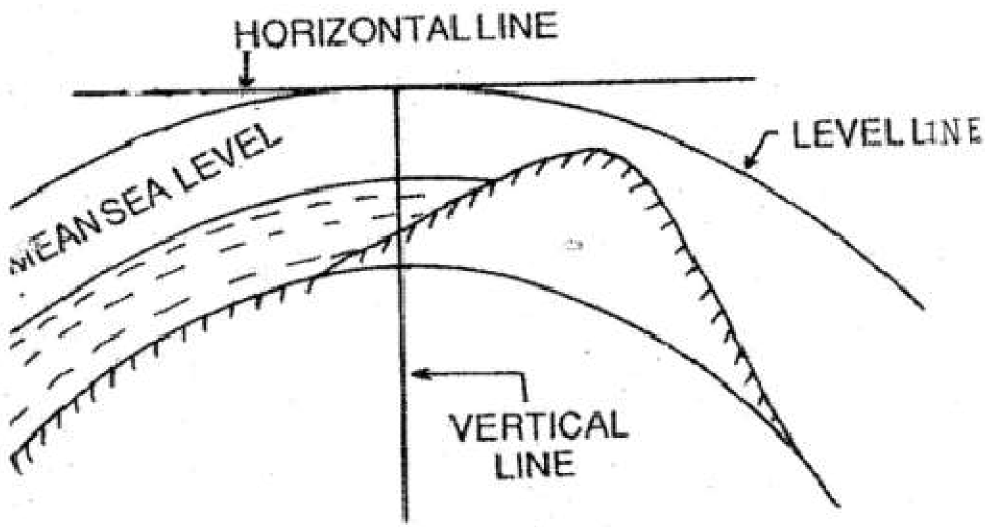
Level Surface: This is a surface parallel to the mean spheroidal surface of the earth is said to bea level surface.The water surface of a still lake is also considered to be a level surface.

Horizontal Plane/surface: Any plane tangential to the level surface at any point is known as the horizontal plane.It is Perpendicular to the plumb line.

VerticalPlane/surface: Any plane passing through the vertical line is known as the vertical Plane.

Datum Surface or Line: This is an imaginary level surface or level line from which the vertical distances of different points(above or below this line) are measured. In India the datum adopted for the Great Trigonometrically Survey(GTS) is the mean sea level(MSL) at Karachi.

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



Bench Mark: 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 or for conducting leveling operations in projects involving roads, Railways.

Bench mark are of four types.(a)**GTS (Great Trigonometric Survey)Bench mark**: This Bench mark s are established by Survey of India at large intervals all over the country(Mumbai). The values of Reduced levels, the relevant positions and the number of benchmarks are given in a catalogue published by this department

Permanent Bench marks: These are fixed points or marks established by different Government Departments like PWD, Railway, Irrigation, etc..The R.L's of these points are determined with reference to the GTS bench mark., and kept on permanent points like the plinth of building, parapet of a bridge or culvert, and so on.

(c)Arbitrary Bench marks: When the RL's of some fixed points are assumed, they are termed arbitrary bench-mars. These are adopted in small survey operations, when only undulation of the ground surface is required to be determined.

(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 root of a tree, the parapet of a nearby culvert, a furlong post, or on a similar place.

7.2 INSTRUMENTS USED FOR LEVELING, CONCEPTS OF LINE OF COLLIMATION, AXIS OF BUBBLE TUBE, AXIS OF TELESCOPE, VERTICAL AXIS.

INSTRUMENTS USED FOR LEVELING

Auto Level:

The Auto level is a naturally leveling elite optical instrument valuable amid site overviews and building development to assemble, exchange or set horizontal levels and grade applications.

An optical level consists of a precision telescope with **crosshairs** and **Stadia marks.** The cross hairs are used to establish the level point on the target, and the stadia allow range-finding; stadia are usually at ratios of 100:1, in which case one metre between the stadia marks on the levelling staff repesents 100 metres from the target.

Stadia marks on a crosshair while viewing a metric levelling rod. The top mark is at 1.5 m and the lower is at 1.345 m. The distance between the marks is 0.155 m, yielding a distance to the rod of 15.5 m.

Auto Level Labelled Diagram:

2. Digital Level:

Digital automatic **levels** are a precise instruments used for precise **leveling**. Operation of **digital levels** is based on the **digital** processing of video indications of a coded staff. At the beginning of measurement a visual pointing of the instrument into the surface of **leveling** meter is performed.

With all the advances in electronics in distance measurement, angle measurement, and positioning, it is not too surprising that dramatic advances also have been made in leveling.

Above picture is of **NA 2000** by **Wild Heerbrug.** The NA 2000 features digital, electronic image-processing for determining heights and distances with automatic recording of data for future transfer to computer.

3. Tilting Level:

It consists of a telescope attached with a level tube which can be tilted within few degrees in vertical plane by a tilting screw.

The main peculiarity of this level is that the vertical axis need not to be truly vertical, since the line of collimation is not perpendicular to it. The line of collimation, is, however, made horizontal for each pointing of telescope by means of tilting screw. It is mainly designed for precise leveling work.

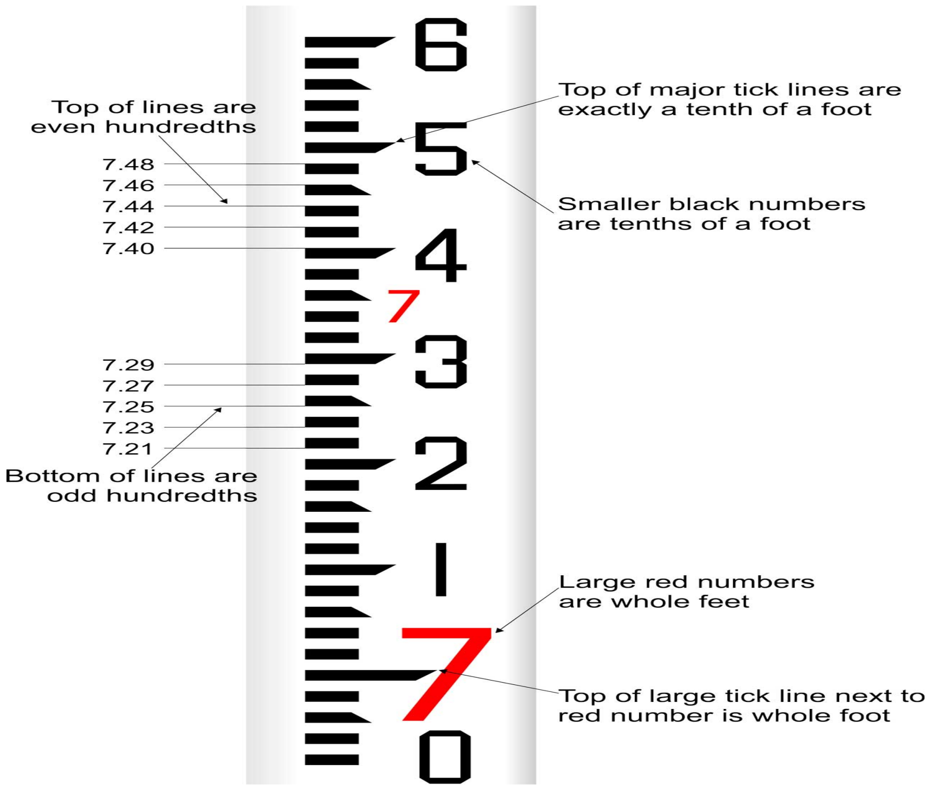
Labelled Diagram:

Picture:

4. Level Rods:

A **level staff**, also called **leveling rod**, is a graduated wooden or aluminium rod, used with a leveling instrument to determine the difference in height between points or heights of points above a vertical datum. It cannot be used without a leveling instrument.

How to Read Staff:



5. Level Vials:

Level vails, also known as **Spirit Level** is the heart of any level. It is also known as **Bubble Level**. The name "spirit level" comes from the liquid inside the level vial. This liquid is a mineral spirit solution with additives which give the vial its color, protection from ultraviolet (UV) fading in sunlight and freezing in the cold. These additives are carefully researched to mix together and never settle out. Level manufacturers also want to make sure their vials maintain their color and other features for a long time.

6. Telescopes:

The telescopes of leveling instruments define the line of sight and magnify the view of a graduated rod. The components of a telescope are mounted in a cylindrical tube. Its four main components are objective lens, negative lens, reticle and eyepiece. Two of these parts the objective lens and eyepiece are external to tge instrument, and are shown in the auto level below.

Parts of automatic level

Objective Lens: Its main function is to gather incoming light rays and direct them toward the negative focusing lens.

Negative Lens: It is located between objective lens and reticle. Its function is to focus rays of light that pass through the objective lens on to the reticle plane.

Reticle: The reticle consists in a pair of perpendicular reference lines(usually called crosshairs) mounted at the principal focus of the objective optical system. The point of intersection of the crosshairs together with the optical centre of the objective system, forms the so called *line of sight* sometimes called *line of collimation.*

Eyepiece: The eyepiece is a microscope (usually with the magnification from about 25 to 45 power) for viewing the image.

7. Hand Level:

The hand level is a handheld instrument used on low precision work, or to obtain quick checks on more precise work. It consists of a brass tube approximately 6 inches long, having a plain glass objective and peep-sight eyepiece.

The instrument is held in one hand and leveled by raising or lowering the objective end until the cross line bisects the bubble. Resting the level against a rod or staff provides stability and increases accuracy. This instrument is especially valuable in quickly checking proposed locations for instrument setups in differential leveling.

Concept of line of collimation:

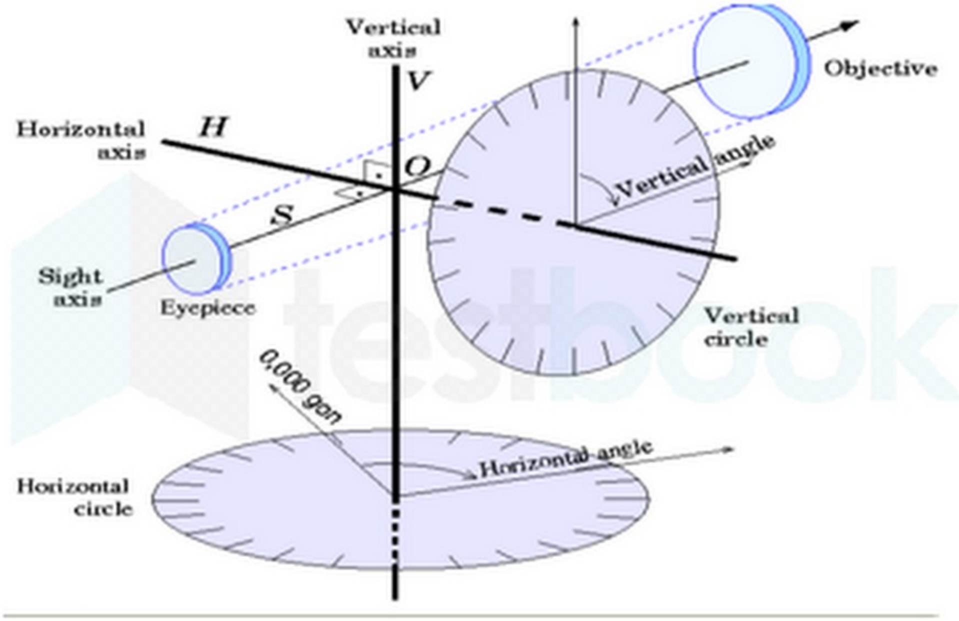
It is an imaginary line passing through the intersection of the cross hairs at the diaphragm and the optical centre of the object glass and its continuation. It is also known as line of sight.

Axis of the telescope: This is an imaginary line passing through the optical centre of the object glass and the optical centre of the eye piece.

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

Horizontal Axis: The horizontal axis is the axis about which the telescope can be rotated in a vertical plane. It is also called the trunnion axis or transverse axis.

Vertical Axis: The vertical axis is the axis about which the telescope can be rotated in a horizontal plane.



7.3 LEVELLING STAFF – TEMPORARY ADJUSTMENTS OF LEVEL, TAKING READING WITH LEVEL, CONCEPT OF BENCH MARK, BS, IS, FS, CP, HI.

Temporary Adjustment of a Level

At each set up of a level instrument, temporary adjustment is required to be carried out prior to any staff observation. It involves some well defined operations which are required to be carried out in proper sequence.

The temporary adjustment of a dumpy level consists of
(1)Setting ,
(2)Leveling and
(3) Focusing .

During **Setting**, the tripod stand is set up at a convenient height having its head horizontal (through eye estimation). The instrument is then fixed on the head by rotating the lower part of the instrument with right hand and holding firmly the upper part with left hand. Before fixing, the leveling screws are required to be brought in between the tribrach and trivet. The bull's eye bubble (circular bubble), if present, is then brought to the centre by adjusting the tripod legs.

Next, **Leveling** of the instrument is done to make the vertical axis of the instrument truly vertical. It is achieved by carrying out the following steps:

Step 1: The level tube is brought parallel to any two of the foot screws, by rotating the upper part of the instrument.

Step 2: The bubble is brought to the centre of the level tube by rotating both the foot screws either inward or outward. (The bubble moves in the same direction as the left thumb.)

Step 3: The level tube is then brought over the third foot screw again by rotating the upper part of the instrument.

Step 4: The bubble is then again brought to the centre of the level tube by rotating the third foot screw either inward or outward.

Step 5: Repeat Step 1 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 2.

Step 6: Repeat Step 3 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 4.

Step 7: Repeat Steps 5 and 6, till the bubble remains central in both the positions. **Step 8:** By rotating the upper part of the instrument through 180 o , the level tube is brought parallel to first two foot screws in reverse order. The bubble will remain in the centre if the instrument is in permanent adjustment.
In the case of four foot screws the levelling is to be carried out as follows

Focusing is required to be done in order to form image through objective lens at the plane of the diaphragm and to view the clear image of the object through eye-piece. This is being carried out by removing parallax by proper focusing of objective and eye-piece. **For focusing the eye-piece**, the telescope is first pointed towards the sky. Then the ring of eye-piece is turned either in or out until the cross-hairs are seen sharp and distinct. Focusing of eye-piece depends on the vision of observer and thus required whenever there is a change in observer.

For focusing the objective, the telescope is first pointed towards the object. Then, the focusing screw is turned until the image of the object appears clear and sharp and there is no relative movement between the image and the cross-hairs. This is required to be done before taking any observation.

TAKING READING WITH LEVEL

The staff readings are taken as follows: (1) The instrument is set up and carefully leveled. (2) The staff is held vertically over the staff station. To hold the staff vertically, the staff man stands behind the levelling staff with his heels together and the bottom of the staff in between his toes.

Concept of Bench Mark, BS, IS, FS,CP, HI:

1.Station: This is appoint where a leveling staff is held for taking observations with a level.

2.Height of the Instrument(HI): It means elevation of the line of sight or line of collimation with respect to the datum.

3.Back Sight(BS): It is the first reading taken at a station of known elevation after setting up of the instrument.This reading gives the height of Instrument(elevation of line of collimation), elevation of line of collimation=Known elevation+backsight

4.Intermediate Sight(IS): These are readings taken between the 1st and last reading before shifting the instrument to a new station.

5.Fore Sight(FS): This is the last reading taken before shifting an instrument to a new station.

6.Turnig Point or Change Point:For leveling over a long distance,the instrument has to be shifted a number of times.Turning point or change point connects one set of instrument readings with the next set of readings with the changed position of the Instrument.A staff is held on the turning point and a foresight is taken before shifting the instrument.From the next position of the instrument another reading is taken at the turning point keeping the staff undisturbed,which is known as back sight.

7.Reduced Level(RL): Reduced level of a point is its height relative to the datum. The Level is calculated or reduced with respect to datum.

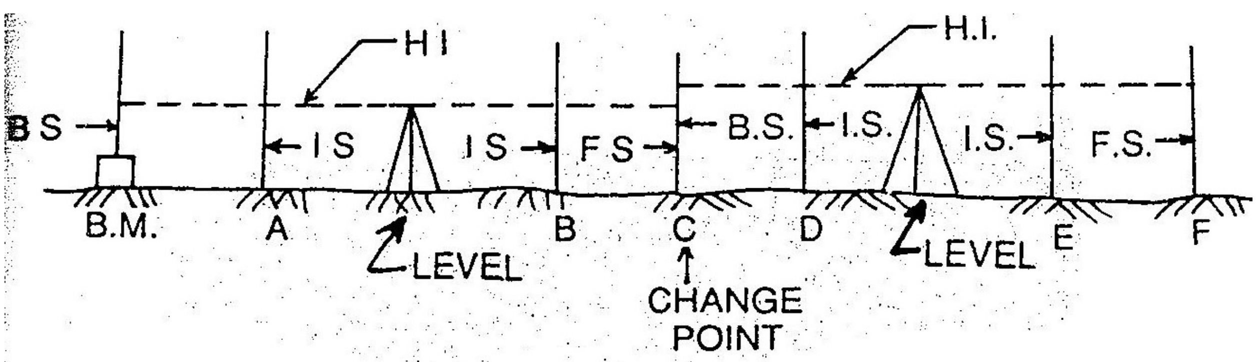
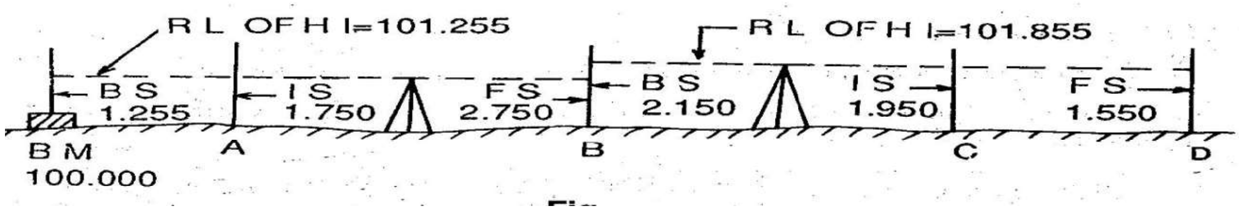


Fig.

7.4 FIELD DATA ENTRY – LEVEL BOOK – HEIGHT OF COLLIMATION METHOD AND RISE & FALL METHOD, COMPARISON, NUMERICAL PROBLEMS ON REDUCTION OF LEVELS APPLYING BOTH METHODS, ARITHMETIC CHECKS.

Field Data entry:Level book
Height of collimation or Height of Instrument method:



The reduced level of the line of collimation is said to be the height of instrument.In this system,the height of the line of collimation is found by adding the backsight reading to 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 Instrument(HI).The level is then shifted for the next set up and again the height of the line of collimation is obtained by adding the backsight reading to the RL of the change point(which is calculated in the first setup).So the ht.of instrument is different in different set ups of the level.Two adjacent places of collimation.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.The RLs of of unknown points are to be found out by deducting the staff readings from the RL of the height of instrument. Referring to Fig.

a)RL of HI in 1st setting=100.00+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 2nd 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.,

Arithmatic check:$\sum BS - \sum FS$ =Last RL -1st RL.The difference between the sum of backsights and that of foresights must be equal to 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.

The Rise and Fall method:

In this method, the difference in level between two consecutive points is determined by comparing each forward staff reading with the staff reading at the immediately preceeding point.If the forward staff reading is smaller than the immediately preceeding staff reading,a rise is said to have occurred. The rise is added to the RL of the preceeding point to get the RL of the forward point.If the forward staff reading is greater than the immediately preceeding staff reading, it means there has been a fall.The fall is subtracted from the RL of the preceeding point to get the RL of the forward point. Refer. to Fig.

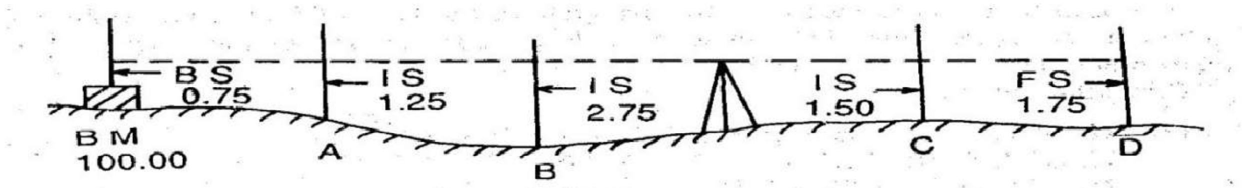


Fig.

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 D(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=98.50, RL of C=98.00+1.25=99.25, RL of D=99.25-0.25=99.00

Arithmatic check:
:$\sum BS - \sum FS$ =Last RL -1st RL=$\sum R-\sum F$

In this method,the difference between the sum of BS's and that ofFS's,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.

Note: The arithmetical check is meant only for the accuracy of calculation to be verified. It does not verify the accuracy of field work. There is a complete check on the RLs of the intermediate points in the rise and fall system.

Points to be remembered while entering the level Book:

1.The first reading of any support is entered in the BS column,the last reading in the FS column and the other readings in the IS column.

2.A page always starts with BS and finishes with an FS reading.
3.If a page finishes with an IS reading, the reading is entered in the IS and FS columns on that page and brought forward to the next page by entering it in the BS and IS columns.

4.The FS and BS of any change point are entered in the same horizontal line.

5.The RL of the line of collimation is entered in the same horizontal line in which the corresponding are entered.

6.Bench Mark(BM) and change point(CP) should be clearly described in the remark column.

Q -1 the following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15 m. the first reading was taken at a chainage of 165 m where the RL is 98.085. the instrument was shifted after the fourth and ninth readings.

3.150, 2.245, 1.125, 0.860, 3.125, 2.760, 1.835, 1.470, 1.965, 1.225, 2.390, and 3.035 m

Mark rules on a page of your notebook in the form of a level book page and enter on it the above readings and find the RL of all the points by:

The collimation method:

Station point	Chainage	BS	IS	FS	RL of the collimation line(HI)	RL	Remark
1	165	3.150			101.235	98.085	
2	180		2.245			98.990	
3	195		1.125			100.110	
4	210	3.125		0.860	103.500	100.375	Change point
5	225		2.760			100.740	
6	240		1.835			101.665	
7	255		1.470			102.030	
8	270	1.125		1.965	102.760	101.535	Change point
9	285		2.390			100.370	
10	300			3.035		99.725	
Total =		7.500		5.860			

Arithmetical check:

∑BS-∑FS=Last RL –First RL

=7.500-5.860=99.725-99.085

=+1.640=+1.640

By Rise and Fall method

Q -2 the following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15 m. the first reading was taken at a chainage of 165 m where the RL is 98.085. the instrument was shifted after the fourth and ninth readings.

3.150, 2.245, 1.125, 0.860, 3.125, 2.760, 1.835, 1.470, 1.965, 1.225, 2.390, and 3.035 m

Mark rules on a page of your notebook in the form of a level book page and enter on it the above readings and find the RL of all the points by:

The rise and fall method:

Station point	Chainage	BS	IS	FS	Rise(+)	Fall (-)	RL	Remark
1	165	3.150					98.085	
2	180		2.245		0.905		98.990	
3	195		1.125		1.120		100.110	
4	210	3.125		0.860	0265		100.375	Change point
5	225		2.760		0.365		100.740	
6	240		1.835		0.925		101.665	
7	255		1.470		0.365		102.030	
8	270	1.125		1.965		0.495	101.535	Change point
9	285		2.390			1.165	100.370	
10	300			3.035		0.645	99.725	
Total =		7.500		5.860	3.945	2.305		

Arithmetical check:

$\sum$BS-$\sum$FS=Last RL –First RL=$\sum$R-$\sum$F

=7.500-5.860=99.725-99.085=3.945-2.305

=+1.640=+1.640=+1.640

7.5 EFFECTS OF CURVATURE AND REFRACTION, NUMERICAL PROBLEMS ON APPLICATION OF CORRECTION.

Effects of Curvature and refraction:

Leveling instruments provide horizontal line of sight and as a result curvature error occurs. In addition due to refraction in earth's atmosphere the ray gets bent towards the earth introducing refraction errors. Fig. illustrates these errors.

Neglecting small instrument Height SA,OA can be taken as the radius of earth ,From Geometry of a circle, AB(2R+AB)=$d2$,as AB is very small compared to diameter of the earthAB.2R= $d2$,

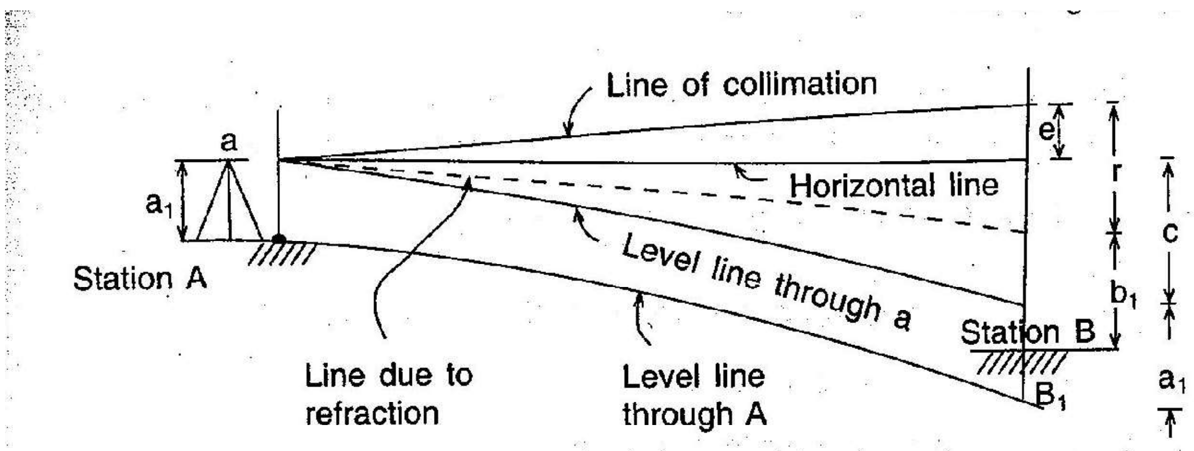
AB= $\frac{d2}{2R}$,

The dia. Of earth is taken as12734KM ,

Hence curvature correction: AB=*d2*/12734Km=0.078 *d2* m, *d* is expressed in Km. The radius of ray IC isbent due to refraction is taken as seven times the radius of earth.

The refraction correction is taken as1/7th of the curvature correction. Refraction correction reduces the curvature

correction and hence combined correction is 6/7th of0.078*d2m,*i,e0.067*d2* and *d* is expressed inKm. The correction is subtractive from staff reading.



Q-2 A level is set up at a point 150 m from A and 100 m from B thee observed staff readings at A and B are 2.525 and 1.755 respectively. Find the true difference of level between A and B.

Solution

Combined correction for curvature and refraction to staff reading at

A=0.0673 X D^2 =0.0673 X $(\frac{150}{1000})^2 = 0.0015\ m$

Correct reading on A = 2.5250-0.0015

=2.5235 m

Combined correction for curvature and refraction to staff reading at

B=0.0673 X $(\frac{100}{1000})^2$

=0.000673 m =0.007 m (say)

Correct reading at B =1.7550-0.0007=1.7543 M

True difference of level between A and B =2.5235-1.7543

= 0.7692 M (fall from B to A)

7.6 RECIPROCAL LEVELING – PRINCIPLES, METHODS, NUMERICAL PROBLEMS, PRECISE LEVELING.

Reciprocal levelling:

While crossing a river or ravine it is not possible to put the level midway so that the back sight and foresight are equal. Sight distance ,however, is long and errors due to

(i) Collimation,

(ii) Curvature and refraction are likely to occur.

Reciprocal levelling helps in compensating for the error due to curvature and refraction and also the line of collimation errors in surveying. It is one of the best methods to eliminate curvature and refraction errors.

In reciprocal levelling, the level is set up on both sides of the levels. Two sets of staff reading are taken. This helps in compensating for the error due to curvature and refraction & also the line of collimation errors in surveying.

Finding Difference in Levels

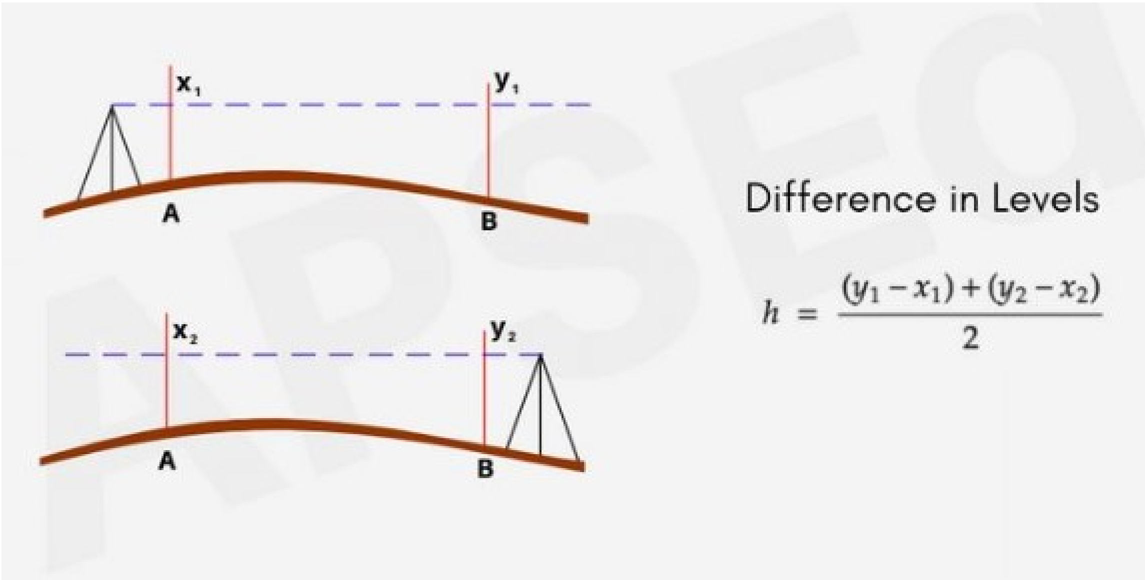
Let us say A and B are two points.

- The instrument is set up near A and readings on the staff are noted (x1 and y1)
- The instrument is shifted near B and readings on staff are again noted (x2 and y2)

Assuming the curvature and refraction error combinedly be e.

The error occurred when the instrument is close to 'A' is considered equal to the error occurred when the instrument is close to 'B' because the instrument is shifted quickly and there is not much change in refraction error.

The difference in levels, h = (y1 - x1) = (y2 - x2)
Considering a combined errors e,
The difference in level h = ((y1 - e) - x1)
It can also be written as, h = (y2 - (x2 - e))
On adding these two equations, 2h = (y1 - x1) + (y2 - x2)
So, h = 0.5((y1 - x1) + (y2 - x2))



Q -1

The following records refer to an operation involving reciprocal levelling.

Instrument at	Staff reading on		Remarks
	A	B	
A	1.155	2.595	Distance AB = 500 m
B	0.985	2.415	RL of A = 525.500

Find :

a) The true RL of B
b) The combined correction for curvature and refraction
c) The collimation error and
d) Whether the line of collimation is inclined upwards or downwards

Solution

a) True difference of level between A and B

$= \frac{(2.595-1.155)+(2.415-0.985)}{2}$

=1.435 m (fall from A to B) ---------(1)

R L of B = 525.500-1.435 =524.065 m

b) Combined correction for 500 m = 0.0673 x $(0.5)^2$ = 0.0168 m (negative)
c) Let us assume that the line of collimation is inclined upwards.

Let , collision error in 500 m = e (positive , as it is inclined upwards)
When the instrument is at A ,
Correct staff reading at A = 1.155 m (as level is near A)
Correct staff reading at B = (2.595-0.0168-e)
True difference of level between A and B
=(2.595-0.0168-e)
=1.4232-e ----------------(2)
From (1) and (2)
1.4232-e =1.4350e =-0.0118
Collimation error per 100 m =$\frac{-0.0118\ X\ 100}{500} = -0.0023\ m$

Precise leveling

Precise leveling aims at establishing the RL of a point with highest precision. The objective of providing such high order of precision in the measurement is to establish network of Bench marks. Precise leveling employs precise level, precise leveling staff and all possible corrections in order to attain accuracy of highest order in the Instrument.

7.7 ERRORS IN LEVELING AND PRECAUTIONS, PERMANENT AND TEMPORARY ADJUSTMENTS OF DIFFERENT TYPES OF LEVELS.

Errors in levelling and precautions:

1.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 leveling staff may not be perfect.

2.Personal errors: (a)The instrument may not be leveled perfectly.(b)The focusing of eye piece and object glass may not be perfect and the parallax may not be eliminated entirely.(c)The position of the staff may be displaced at the change point at the time of taking FS and BS readings.(d)The staff may appear inverted when viewed through the telescope.By mistake,the staff readings may be taken upwards instead of downwards..(e)The reading of the stadia hair rather than the central collimation hair may be taken by mistake.(f)A wrong entry may be made in the level book.(g)The staff may not be properly and fully extended.

3.Errors due to natural Causes:(a)When the distance of sight is too long, the curvature of the earth may effect the staff reading.(b)The effect of refraction may cause a wrong staff reading to be taken.(c)The effect of high winds and a shining Sun may result in a wrong staff reading.

Permissible Errors in Levelling:

The precision of levelling is ascertained according to the error of closure.The permissible limit of closing error depends upon the nature of work for which the leveling is to be made.It is expressed as:E=C__,Where,E= closing error in meter,C=the constant,D=distance in Km.The following are the permissible errors for different types of leveling

PRECAUTIONS,

- The staff should be kept vertically for accurate reading. There is also an alternative method, i.e., to wave the staff and take the lowest reading. Both methods are equally effective.
- The bubble in the dumpy level should be central to obtain an accurate line of sight.
- The foot screws in the dumpy level should be used carefully
- To avoid errors, read the staff in the increasing direction of readings
- Take the reading on the portion of the staff between two vertical crosshairs.
- Equalize foresight distance and backsight distance
- Tripod should be carefully placed in the required point and should not be disturbed at any cost.
- Use a firm and fixed point for turning point/changing point
- Eliminate all the minor and major errors especially parallax before readings are taken
- Avoid mistakes in reading (make sure that the metre and decimeter are correctly noted).
- Avoid work in very hot climate because it may harm the instrument. So the instrument should be protected from heat.

The permanent and temporary adjustments of different level

The permanent and temporary adjustments of different level are made to establish the fixed relationships between its fundamental lines:

1. Permanent Adjustments of a Dumpy Level
2. Permanent Adjustments of a Cooke's Reversible Level
3. Permanent Adjustments of a Cushing's Level
4. Permanent Adjustments of the Y-Level
5. Permanent Adjustments of a Tilting Level.

1. Permanent Adjustments of a Dumpy Level:

In a dumess level, there are only two adjustments as he telescope is rigidly fixed to the spindle.

1. The axis of the bubble tube should be perpendicular to the vertical axis.

2. The line of collimation should be parallel to the axis of the bubble tube.

First Adjustment:
To make the axis of the bubble tube perpendicular to the vertical axis.

Object:
The object of this adjustment is to ensure that if the instrument is once levelled up, the bubble remains in the centre of its run for all positions of the telescope.

(i) Set-up the level on firm ground and level it carefully by tripod-legs and foot-screws. The bubble will now be central in two positions at right angles to each other, one being parallel to a pair of foot-screw and the other over the third foot-screw.

(ii) Bring the telescope over a pair of foot-screws or over the third foot-screw and turn it through 180 in the horizontal plane. If the bubble still remains central, the adjustment is correct.

Adjustment:
(i) If the bubble does not remain in the centre, note down the deviations of the bubble from the centre, say it is '2n' division over the bubble half way back i.e., 'n' divisions by raising or lowering end of the bubble tube by means of capstan headed must and the remaining half with the pair of foot-screws beneath the telescope at its present position.

(ii) Turn the telescope through 90° so that it lies over the single foot- screw below the telescope or parallel to a pair of this screw or pair of foot -screws and bring the bubble in the centre of its run by means of this screw pair of foot-screws.

(iii) Rotate the telescope and see if the bubble remains central for all positions of the telescope. If not repeat the whole process until the adjustment is correct.

Second Adjustment:
To make the Line of collimation parallel to the axis of the bubble tube.

Test and Adjustments:
The collimation error may be tested by any of the following three methods and then the necessary adjustments are made:
First Method (Two-Peg Method.):

Test:

(i) Drive two pegs A and B at a distance of (D) metres say 60 to 100 metres on a fairly level ground. Drive another peg at O exactly midway between A and B

(ii) Set up and level the instrument at O and take the staff readings on A and B. The bubble must be in the centre while the readings are being taken. Let the staff readings on A and B, be a and b respectively.

(iii) Shift the level and set it up a point O_1, d metres away from A (or B) and along the same line BA (Fig. 7.37). levels the instrument accurately and take staff readings on A and B with the bubble central. Let the readings be a_1 and b_1 respectively. (The level may also be set up at a point between A and B, d metres away from A or B

(iv) Find the difference between the staff readings a and b, and that between the staff readings a_1 and b_1. The difference of staff readings a and b gives the true difference in elevation between A and B as the instrument was set up exactly midway between A and B and that the back and for sight distances were exactly difference, whereas the difference between a_1 and b_1 gives the apparent difference. If the two differences are equal, the line of collimation is in adjustment, otherwise it is inclined and needs adjustment.

Adjustment:

(i) Find out whether the difference is a rise or a fall from the peg A to B. If a is greater than b, the peg A is lower than peg B and the ground is rising from A to B. If b is greater than a, the ground is falling from A to B.

(ii) Find out the reading on the far peg B at the same level are of a_1 by adding the true difference to a_1 if it is a fall, or by subtracting the true difference from a_1 if it is a rise. Let the reading be b_2.

(iii) If b_1 is greater than b_2, the line of collimation is Inclined upwards and if b_1 is smaller than b_2, the line of collimation is inclined downwards. $b_1 – b_2$ (difference of b_1 and b_2) is the collimation error in the distance "D". ∴ the collimation error for unit distance:

(iv) The corrections to be applied for readings on the pegs A and B may be found out as under:

These corrections are additive if the of collimation is inclined downward and subtractive if the same is inclined upwards.

(vi) Now place the staff on the near peg (A) and take the staff reading. If the adjustment is correct, this reading should agree with the calculated correct reading on the peg A, otherwise repeat the adjustment until perfect.

2. Permanent Adjustments of a Cooke's Reversible Level:

By slackening the stop-screw, the telescope of a Cooke's reversible level can be rotated about its longitudinal axis in the sockets and can also be withdrawn from the socket s and replaced end for end.

There are three adjustments for this level viz:

1. The line of collimation should coincide with the axis of the telescope.

2. The line of collimation should be at right angles to the vertical axis.

3. The axis of the bubble tube should be perpendicular to the vertical axis.

First Adjustment:

To make the line of collimation coincident with the axis of the telescope.

Test:

(i) Set up the level at a distance of 30 to 60 m from a wall and level it. The instrument is so set up that the telescope lies over one foot- screw when directed towards the wall.

(ii) Direct the telescope towards the wall and focus it. Mark a point A on the wall coinciding with the cross-hairs

(iii) Slacken the stop-screw and withdraw it and rotate the telescope about its longitudinal axis through 180° so that the stop-screw-hole is brought to the top and the cross-hair is horizontal. See whether the point A is bisected again or not. If A is bisected, then the instrument is in adjustment.

(iv) Otherwise, mark another point B coinciding with the cross-hair, above or below the point A.

(v) Measure the distance AB accurately. The actual error is half the vertical distance AB. Mark a point C exactly midway between A and B.

Adjustment:

(i) Move the diagram up or down by means of the diagram screws until the mark C is bisected by the cross-hair. If the diagram is to be raised, loosen the lower screw and tighten the upper one and vice versa.

(ii) Repeat the test and adjustment until the adjustment is correct.

Alternatively, instead of making mark A, B and C on the wall, a staff may be held at a distance of 30 to 60 m from the instrument, a reading coinciding with the cross-hair (say a_1) is first taken. The telescope is then rotated through 180° and the second reading on the staff (say b_1) is observed.

Then adjust the diagram to bisect this reading on the staff.

Second Adjustment:

To place the Line of Collimation a right angles to the vertical axis.

Test:

(i) The first two steps are the same as in the first adjustment of this level.

(ii) Remove the stop'-screw and carefully withdraw the telescope from its socket. Turn the socket end for end and gently replace the telescope. See that the cross-hair is exactly horizontal.

(iii) Again sight the test mark A and see if it is bisected again not. If A is bisected, the adjustment is correct.

(iv) If not, mark another point B coinciding with the cross-hair above or below the point A.

(v) Measure AB and mark a point C exactly midway between the two.

Adjustment:

The adjustment is made by means of the base plate or limb-nuts:

(i) Raise or lower the socket by means of the base-plate limb-nuts until the point C is bisected by the cross-hair.

(ii) Turn the foot-screw beneath the telescope until the cross-hair again bisects the mark A. Now reverse the telescope end for end and see whether the mark A is still bisected by the cross-hair.

(iii) If not, repeat the test and adjustment till correct.

Alternatively, instead of marking points A, B and C on the way the staff readings a_1 and b_1 may be observed and the socket is adjusted such that the cross-hair bisects the reading

Third Adjustment:

To make the axis of the bubble tube perpendicular to the vertical axis.

Test and Adjustment:

The test and adjustment are the same as in the first adjustment of the dumpy level. This adjustment was made only for convenience of taking quick readings in the dumpy level but in the case of this level, it is a necessity because the parallelism of the line of collimation and the bubble axis also depends on this adjustment.

3. Permanent Adjustments of a Cushing’s Level:

In Cushing’s level, the telescope can neither be rotated about its longitudinal axis nor it can be withdrawn from its socket and changed end for end directly, but both the above properties of the telescope of this level are attained indirectly by making the end collars exactly similar which can be interchanged to reverse the telescope end for end and also fixed after the desired rotation in its fittings.

The fundamental lines of this level and the relations between them are the same as those for Cooke’s reversible level. The methods of testing and adjusting are also exactly similar to those employed in the case of Cookes’ in the level.

First Adjustment:

To make the line of collimation coincident with the axis of the telescope.

Test and Adjustment:

This is made in the same way as the first adjustment of Cooke’s level except that in step (iii), rotate the eye-piece end (i.e. eye-piece and diagram) through 180° in its fittings instead of rotating the telescope about its axis through 180°.

Second Adjustment:

To place the Line of collimation at right angles to the vertical axis.

Test and Adjustment:

Similar to the second Adjustment of Cooke’s level except that interchange the eye-piece and objective ends instead of reversing the telescope end for end.

Third Adjustment:

To make the axis of the bubble tube perpendicular to the vertical axis.

Test and Adjustment:

Performed in the same manner as the first adjustment of dumpy level.

4. Permanent Adjustments of the Y-Level:

In the Y-level, the telescope can be revolved about its longitudinal axis. It can also be lifted from the Y-supports and changed end for end.

There are three permanent adjustments viz.:

1. The line of collimation should coincide with the axis of the telescope.

2. The line of collimation should be parallel to the axis of the bubble tube.

3. The axis of the bubble tube should be perpendicular to the vertical axis.

First Adjustment:
To make the line of collimation coincident with the axis the telescope. The instrument is manufactured such that the axis of the telescope collars coincide with the optical axis of the telescope and also it is parallel to the bearing surface of the collars.

Therefore the line of collimation should coincide with the axis of the telescope collars or parallel to bearing surface of the collars. This adjustment is essential before performing the second adjustment in which the bubble axis is made parallel to the bearing surface, but with this adjustment it becomes parallel to the line of collimation.

Test and Adjustment:
Similar to the first adjustment of the Cooke's level except that for rotating the telescope through 180-, loosen and raise wye-clips and turn the telescope about the longitudinal axis.

Second Adjustment:
To make the Line of collimation parallel to the axis of the bubble tube. With this adjustment, the line of collimation remains horizontal when the bubble lies in the centre of its run. This is very important relationship as it constitutes the fundamental principle (to furnish horizontal line of sight) of spirit levelling.

This adjustment is made in two stages:
(1) The axis of the bubble tube is placed in the plane of the line of collimation,

(2) The axis of the bubble tube is made parallel to the line of collimation (or the bearing surface of the collars).

First Stage:
To place the axis of the bubble tube in the plane of the line of collimation.

Test:
(i) Set up the instrument and level it accurately.

(ii) Loosen and raise the wye-clips. Revolve the telescope about its longitudinal axis through a small angle, (say 10°). If the bubble still remains central, the adjustment is correct.

Adjustment:

If the bubble does not remain central, bring it entirely back to the central position by means of the azimuth screw. The bubble tube is thus adjusted laterally. It may be noted here that in this case the whole deviation of the bubble is the actual error since no reversal is made in the test. Repeat the test and adjustment till it is correct.

This adjustment is necessary only for those Y-levels in which the level tube is fitted with the azimuth screw.

Second Stage:

To make the axis of the bubble tube, parallel to the line of collimation (or the bearing surface of the collars.).

Test:

(i) Set up and level the instrument and bring the telescope over one foot-screw and clamp it. Bring the bubble to the centre of its run by means of this screw.

(ii) Loosen and raise the Y-clips. Lift the telescope gently, turn it end for end and replace it in wyes. If the bubble remains central, the adjustment is correct.

Adjustment:

(i) If the bubble is deviated, note its deviation from the centre. Let it be 2n divisions. Then the actual errors is n divisions.

(ii) Bring the bubble half way back towards centre i.e. n division by means of the level-tube nuts and the remaining half by the foot-screw beneath the telescope.

(iii) Repeat the test and adjustment till correct.

In such a case, the two-peg method as already explained in the adjustment of dumpy level may be employed. The method may be used in the same way except that the intersection of the crosshairs is brought to the calculated correct staff reading on the far peg by means of levelling screw beneath the telescope. The line of collimation is now horizontal but the bubble is displaced from its central position. Then bring the bubble to the centre by means of level tube nuts.

Third Adjustment:

To make the axis of the bubble tube perpendicular to the vertical Axis. The method is similar to that of the first adjustment of dumpy level except that the bubble is brought half way back by Y-nuts and the remaining half by foot-screw under the telescope.

5. Permanent Adjustments of a Tilting Level:

In the tilting level, the telescope along with the main bubble tube can be tilted by means of the tilting screw independent of the vertical axis. Therefore, there is only one adjustment viz, the axis of the bubble tube should be parallel to the line of collimation so that the line of collimation is horizontal when the bubble lies in the centre of its run.

There are two types of the tilting level:

(1) Reversible, and

(2) Non-Reversible.

In the reversible type, the telescope along with the main bubble tube can be rotated about its longitudinal axis through 180 whereas in non-reversible type, the telescope cannot be rotated as above.

Adjustment:

To make the axis of the bubble tube parallel to the line of collimation:

(1) Reversible Type:

Necessity:

Same as for second adjustment of the dumpy level.

Test:

(i) Set up the level such that the main level tube is on the left face of the instrument. Level it approximately by foot-screw. Hold the staff at a distance of about 100 m.

(ii) Bring the main bubble exactly to the centre of its run by using the tilting-screw and take the staff reading. Let the reading be a_1.

(iii) Rotate the telescope about its longitudinal axis through 180° so that the main bubble tube is now on the right face of the instrument.

(iv) After making the bubble exactly central, again read the staff.

Let the reading be a_2. If the two readings agree, the adjustment is correct.

Method of Adjustment:

(i) If the two readings disagree rotate the telescope to the original position so that the bubble tube is again on the left face of the instrument and then set the telescope to the mean of the above readings by turning the tilting screw. The line of collimation is now horizontal but the bubble becomes out of centre.

(ii) Bring the bubble exactly to the centre of its run by means of the screws attached with the bubble tube.

(iii) Repeat the procedure until correct.

(2) Non-Reversible Type:

Necessity:

Same as for the second adjustment of the dumpy level.

Test:

Same as the two-peg test for the second adjustment of the dumpy level.

Method of Adjustment:

(i) If the apparent and true differences of level are not equal, find the correct staff readings on the two pegs.

(ii) Hold the staff at the far peg B and set the telescope to the corrected staff reading by means of the tilting-screw. The bubble is divided from its central position.

(iii) Bring the bubble to the centre of its run by means of bubble adjusting screws.

(iv) Repeat the operation until correct.

Temporary Adjustment of a Level

At each set up of a level instrument, temporary adjustment is required to be carried out prior to any staff observation. It involves some well defined operations which are required to be carried out in proper sequence.

The temporary adjustment of a dumpy level consists of
(1)Setting ,
(2)Leveling and
(3) Focusing .

During **Setting**, the tripod stand is set up at a convenient height having its head horizontal (through eye estimation). The instrument is then fixed on the head by rotating the lower part of the instrument with right hand and holding firmly the upper part with left hand. Before fixing, the leveling screws are required to be brought in between the tribrach and trivet. The bull's eye bubble (circular bubble), if present, is then brought to the centre by adjusting the tripod legs.

Next, **Leveling** of the instrument is done to make the vertical axis of the instrument truly vertical. It is achieved by carrying out the following steps:

Step 1: The level tube is brought parallel to any two of the foot screws, by rotating the upper part of the instrument.

Step 2: The bubble is brought to the centre of the level tube by rotating both the foot screws either inward or outward. (The bubble moves in the same direction as the left thumb.)

Step 3: The level tube is then brought over the third foot screw again by rotating the upper part of the instrument.

Step 4: The bubble is then again brought to the centre of the level tube by rotating the third foot screw either inward or outward.

Step 5: Repeat Step 1 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 2.

Step 6: Repeat Step 3 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 4.

Step 7: Repeat Steps 5 and 6, till the bubble remains central in both the positions. **Step 8:** By rotating the upper part of the instrument through 180 o , the level tube is brought parallel to first two foot screws in reverse order. The bubble will remain in the centre if the instrument is in permanent adjustment.
In the case of four foot screws the levelling is to be carried out as follows

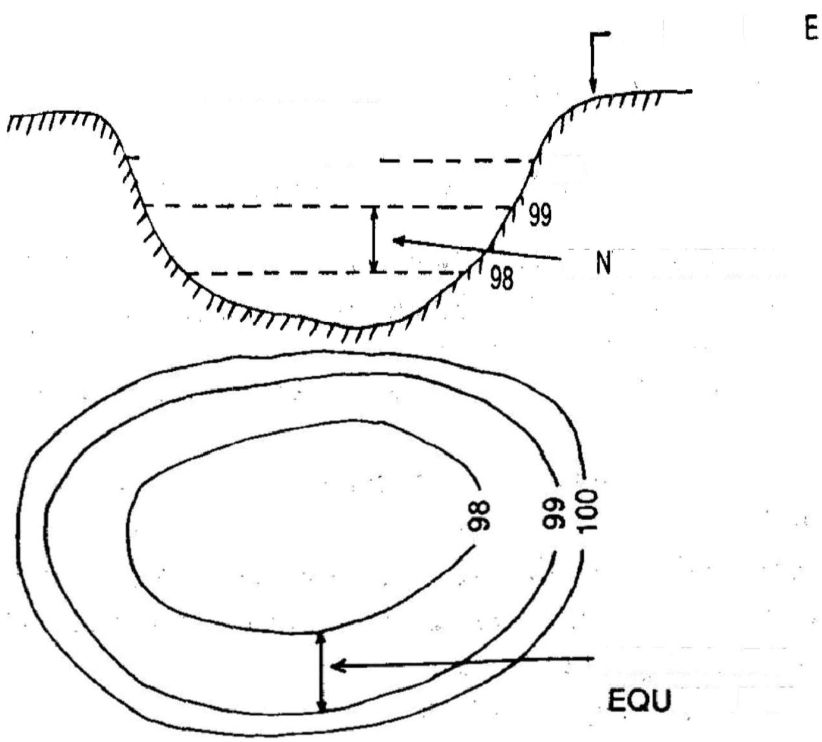
Focusing is required to be done in order to form image through objective lens at the plane of the diaphragm and to view the clear image of the object through eye-piece. This is being carried out by removing parallax by proper focusing of objective and eye-piece. **For focusing the eye-piece**, the telescope is first pointed towards the sky. Then the ring of eye-piece is turned either in or out until the cross-hairs are seen sharp and distinct. Focusing of eye-piece depends on the vision of observer and thus required whenever there is a change in observer.

For focusing the objective, the telescope is first pointed towards the object. Then, the focusing screw is turned until the image of the object appears clear and sharp and there is no relative movement between the image and the cross-hairs. This is required to be done before taking any observation.

7.8 DEFINITIONS, CONCEPTS AND CHARACTERISTICS OF CONTOURS.

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.

Contour line:
The line of intersection of a level surface with the ground surface is known asthe contour line or simply the contour. It can also be defined as a line passing through points of equal reduced levels



Contour Interval: The vertical distance between any two consecutive contours is known as a contour interval. Suppose a map includes contour lines of 100 in, 98 in, 96 in, and so on. The contour interval here is 2 m. This interval depends upon: (i) the nature of the ground (i.e. whether flat or steep), (ii) the scale of the map, and (iii) the purpose of the survey.

Characteristics of Contours:
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.

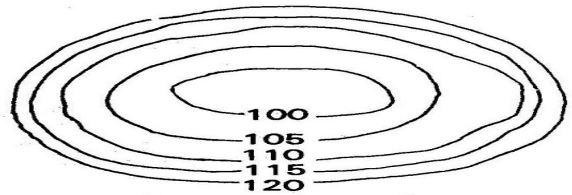


Fig. Depression

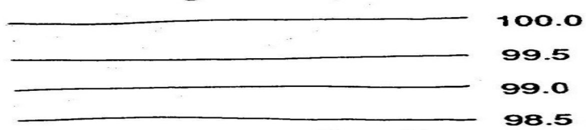


Fig. Uniform Slope

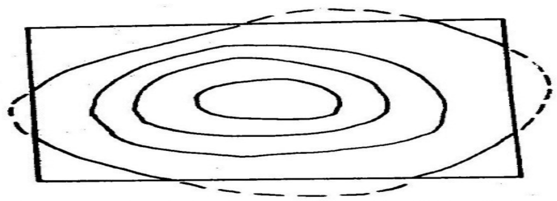


Fig. Contour Closed within Map

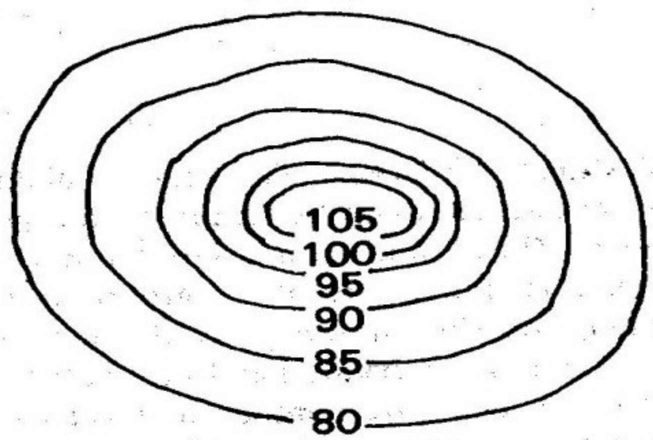


Fig.

2. In Fig. the contour lines are closer near the bank of a pond or depression and wide apart towards the centre. This indicates a steep slope near the bank and a flatter slope at the centre.
3.Uniformly spaced, contour lines indicate a uniform slope (Fig. . '-).
4.Contour lines always form a closed circuit. But these inesmay be within or outside
the limits of the map (Fig**.)**

5.Contour lines cannot cross one another, except in the case of an overhanging cliff. But the over-lapping portion must be shown by a dotted line (Fig.)
.
6.When the higher value are inside the loop, it indicates a ridge line. Contour lines cross ridge linesat right angles (Fig. ').

7.When the lower values are insidie the loop, it indicates a valley line. Contour lines cross the valley line at right angles (Fig.).

8.Aseries of closed contours always indicates a depression or summit. The lower values being inside the loop indicates a depre- ssion and the higher values being Inside the loop indicates a summit. (Fig.).

9.Depressions b e t w e e n summit are called saddles (Fig.)

10.Contour lines meeting at a point indicate a vertical cliff.

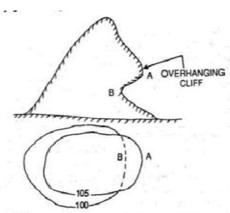


Fig. Overhanging Cliff

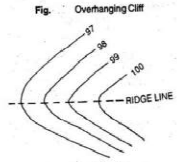


Fig. Ridge Line

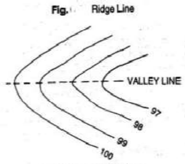
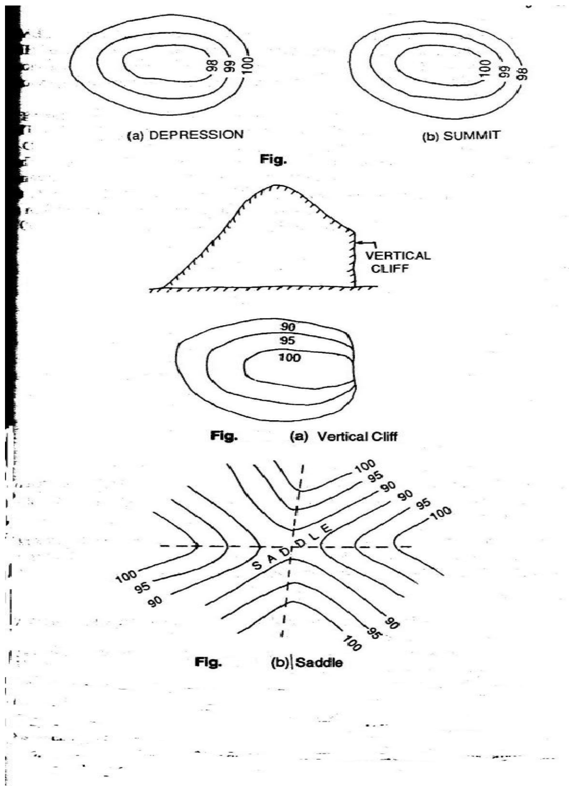


Fig. Valley Line



7.9 METHODS OF CONTOURING, PLOTTING CONTOUR MAPS, INTERPRETATION OF CONTOUR MAPS, TOPOSHEETS.

There are two methods of contouring-
direct and indirect.

A)Direct Method:

There may be two cases, as outlined below.
Case-I: When the area is oblong and cannot be controlled from a single station: In this method, the various points on any contour are located on the ground by taking levels. Then these points are marked by pegs. After this,the points are plotted on the map to any suitable scale ,by plane table. This method is very slow. And tedious.But it gives accurate contour lines.

Procedure:1)Suppose a contour map is to be prepared for an oblong area.A temporary Bench mark is set up near the site by taking fly level readings from a permanent bench mark.(2)The level is then set up at a suitable position L from where maximum area can be covered.(3)The Plane table is set up at a suitable station P from where the above area can be plotted.(4)A back sight reading is taken on the TBM. Suppose the RL of the TBM is249.500 m and that the BS reading is 2.250 m. then the RL of HI251.750 m. If s contour of 250.000 is required, the staff reading should be 1.750 m. If a contour of 249.000 m is required, the staff reading should be 2.750 m, and so on.(5)The staff man holds the staff at different point of the area by moving Up and down, or left and right, until the staff reading is exactly 1.750. Then, he points are marked by pegs. Suppose, these points are A, B, C, D

Case-II: When the area is small and can be controlled from a single station: In this case, the method of radial lines is adopted to obtain contour map. This is alsovery slow and tedious, but gives the actual contour lines.

Direct Method:

1)The plane table is set up at a suitable station P from where the whole area can be commanded.
2)A point p is suitably selected on the sheet to represent the station P. Radial lines are then drawn in different directions.

3) A temporary bench-mark is established near the site. The level is set upat a suitable position L and a BS reading is taken on the TBM. Let the HI in this setting be 153.250 m. So, to Find the contour of RL, 152.000 in a staff reading of 1.250 in is required at a particular point, so that the

RL of contour of that point comes to 152.000 m.

RL = HI — Staff reading
= 153.250 — 1.250 = 152.000 in
4.The staff man holds the staff along the rays drawn from the plane table station in such a way that the staff reading on that point is exactly 1.25In this manner, points A, B, C, D and E are located on the ground, where the staff readings are exactly 1.250.
(5)The distances PA, PB, PC, PD and PE are measured and plotted to anysuitable scale. Thus the points a, b, c, d and e are obtained which are joined in order to obtain a contour of 152.000.(6)The other contours may be located in similar fashion (Fig.).

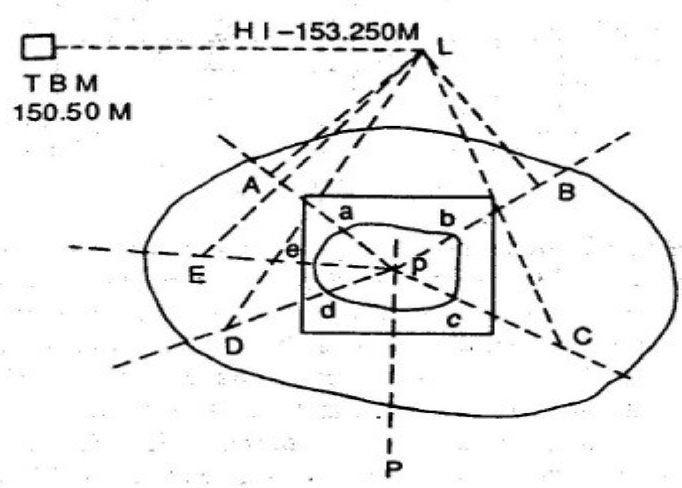


Fig.

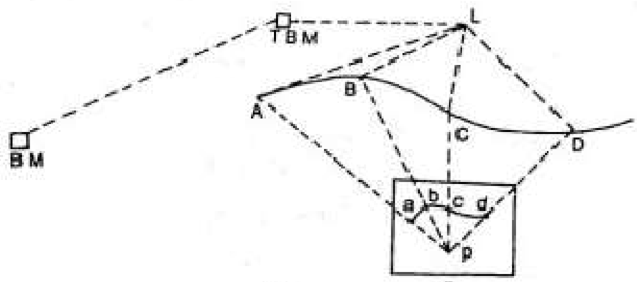


Fig.

PLOTTING CONTOUR MAPS

- A contour map is a map illustrated with contour lines, for example a topographic map, which thus shows valleys and hills, and the steepness or gentleness of slopes. The contour interval of a contour map is the difference in elevation between successive contour lines.
- The gradient of the function is always perpendicular to the contour lines. When the lines are close together the magnitude of the gradient is large: the variation is steep. A level set is a generalization of a contour line for functions of any number of variables.
- Contour lines are curved, straight or a mixture of both lines on a map describing the intersection of a real or hypothetical surface with one or more horizontal planes. The configuration of these contours allows map readers to infer the relative gradient of a parameter and estimate that parameter at specific places.
- Contour lines may be either traced on a visible three-dimensional model of the surface, as when a photogrammetrist viewing a stereo-model plots elevation contours, or interpolated from the estimated surface elevations, as when a computer program threads contours through a network of observation points of area centroids. In the latter case, the method of interpolation affects the reliability of individual isolines and their portrayal of slope, pits and peaks.

Interpretetion of contour maps, toposheets:

The process of locating the contours proportionately between the plotted points is termed interpolation. This can be done by

(1)Arithmetical Calculation:Let A and B be two corners of the squares

.The RL of A is 98.75m,and that of B 100.75m.The horizontal distance between A and B is 10m,

Horizontal distance between A and B=10m,Vertical distance between A and B=100.75-98.75=2m,Let a contour of 99.00m be required.

Then, difference of level between A and 99.00m contour=99.00-98.75=0.25m

,∴ Distance of 99.00m contour line from A= 1.01 x0.25=1.25m.

This calculated distance is plotted to the same scale in which the skeleton was plotted, to obtain a point of RL of 99.00m,Similarly the other point is located.

Toposheets

Toposheets is a topographic map which is a two dimensional representation of a three dimensional land surface.

Topographic maps are differentiated from the other maps in that they show both the horizontal and vertical position of the terrain.

Through a combination of contour lines, colours, symbols, labels and other graphical representation.

Topographic maps portray the shapes, location of mountains, and many other natural and manmade features.

To identify a map of a particular area, a map numbering system has been adopted by survey of India.

Uses:

1. Firstly toposheet contains valuable reference information for surveyors.
2. It is also map makers including bench mark, baseline and meridian and are used in civil engineers .
3. Therefore it is also used environmental managers and urban planners as well as by emergency service agencies and historians.
4. Toposheet are extremely useful for planning various projects.
5. As they provide the required data in most convenient form so that the construction can be planned.
6. One can use toposheets for planning of a building complex, an industrial plant, a railway and an irrigation project.
7. One can plan Bridges, tunnels and dams from the toposheets.
8. These are also helpful for directing military operations at the time of war.
9. Therefore one can use them for the development of hydroelectric schemes, landscape, architecture, environmental protection and agriculture.
10. Lastly one uses it in earth science and many other graphic disciplines mining and other earth based endeavors.

7.10 USE OF CONTOUR MAPS ON CIVIL ENGINEERING PROJECTS – DRAWING CROSS-SECTIONS FROM CONTOUR MAPS, LOCATING PROPOSAL ROUTES OF ROADS / RAILWAY / CANAL ON A CONTOUR MAP, COMPUTATION OF VOLUME OF EARTHWORK FROM CONTOUR MAP FOR SIMPLE STRUCTURE.

USE OF CONTOUR MAPS ON CIVIL ENGINEERING PROJECTS –

Uses of Contour Maps

Contour maps are extremely useful for various engineering works:

1. A civil engineer studies the contours and finds out the nature of the ground to identify. Suitable site for the project works to be taken up.
2. By drawing the section in the plan, it is possible to find out profile of the ground along that line. It helps in finding out depth of cutting and filling, if formation level of road/railway is decided.
3. Intervisibility of any two points can be found by drawing profile of the ground along that line.
4. The routes of the railway, road, canal or sewer lines can be decided so as to minimize and balance earthworks.
5. Catchment area and hence quantity of water flow at any point of nalla or river can be found. This study is very important in locating bunds, dams and also to find out flood levels.
6. From the contours, it is possible to determine the capacity of a reservoir.

Determination of intervisibility:

Let it be required to ascertain the intervisibility of two stations A and B having elevations 62m and 90m,respectively,as shown in the contour map(Ref:fig.).Join A and B is 28m(90-62=28).The line of sight will have an inclination of 28m in the distance ab. draw projections to mark points of elevation of90,85,…,62 on the line ab. Compare these points with the corresponding points in which the contours cut the line ab.At the point e,the ground has an elevation of 75 and 70m),whereas line of sight will have an elevation less than 75m(between 75 and 70m).It can be seen that there will be obstruction in the range CD.Similarly,checks can be made for the other points.

Drainage area: The extent of drainage area may be estimated on a contour map by locating the ridge line around the watershed. The ridge line should be located in such apposition that the ground slopes are down on either side of it. The area is found out by Planimetric measurements.

Capacity of reservoirs: Reservoirs are made for water supply and for power or irrigation m projects. A contour map is very useful to study the possible location of a dam and the volume of water to be confined.All the contours are closed lines within the reservoir area

Site of structures: The most economical and suitable site for structures such as buildings,bridges,dams etc. can be found from large –scale contour maps**.**

Earthquake estimates: On the contour line of the original surface,the contours of the desired altered surface are drawn .By joining the intersections of the original contours and new ones of equal value ,the line in which the new surface cuts the original is obtained . Exacavation is required within this line ,whereas the surrounding parts will be in the embankment. The volume of cut or a fill is found by multiplying the average by the contour interval.

Route Location: By inspecting a contour map the most sitable site for a road, railway ,canal etc. can be selected.By following the contour lines, steep gradients ,cutting and filling, etc.may be avoided.

7.11 MAP INTERPRETATION: INTERPRET HUMAN AND ECONOMIC ACTIVITIES (I.E.: SETTLEMENT, COMMUNICATION, LAND USE ETC.), INTERPRET PHYSICAL LANDFORM (I.E.: RELIEF, DRAINAGE PATTERN ETC.), PROBLEM SOLVING AND DECISION MAKING

MAP INTERPRETATION

Interpreting Contour Lines Contour lines on a map show topography or changes in elevation. They reveal the location of slopes,depressions, ridges, cliffs, height of mountains and hills, and other topographical features. A contour lineis a brown line on a map that connects all points of the same elevation. They tend to parallel each other,each approximately the shape of the one above it and the one below it.

INTERPRET HUMAN AND ECONOMIC ACTIVITIES (I.E.: SETTLEMENT, COMMUNICATION, LAND USE ETC.)

- Land is not regarded simply in terms of soils and surface topography, but encompasses such features as underlying superficial deposits, climate and water resources, and also the plant and animal communities which have developed as a result of the interaction of these physical conditions.

- The results of human activities, reflected by changes in vegetative cover or by structures, are also regarded as features of the land. Changing one of the factors, such as land use, has potential impacts on other factors, such as flora and fauna, soils, surface water distribution and climate.

- Changes in these factors can be readily explained by ecosystem dynamics and the importance of their relationships in planning and management of land resources has become increasingly evident

INTERPRET PHYSICAL LANDFORM (I.E.: RELIEF, DRAINAGE PATTERN ETC.), PROBLEM SOLVING AND DECISION MAKING

- A contour map and what it looks like from a landscape perspective.

- Note that contour lines are far apart for level land and almost touch for cliffs Evenly and widely spaced contoursindicate type of slope and shape of hilltop.

• Jagged, rough contours indicate large outcrops of rocks, cliffs, and fractured areas.

• "V" shape contours indicate stream beds and narrow valleys with the point of the "V" pointing uphillor upstream.

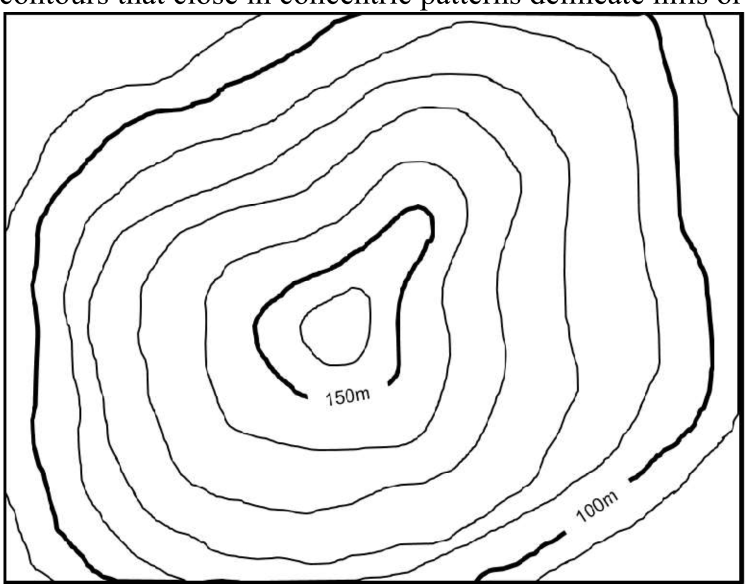
• "U" shape contours indicate ridges with the bottom of the "U" pointing down the ridge. A saddle isa ridge between two hills or summits.

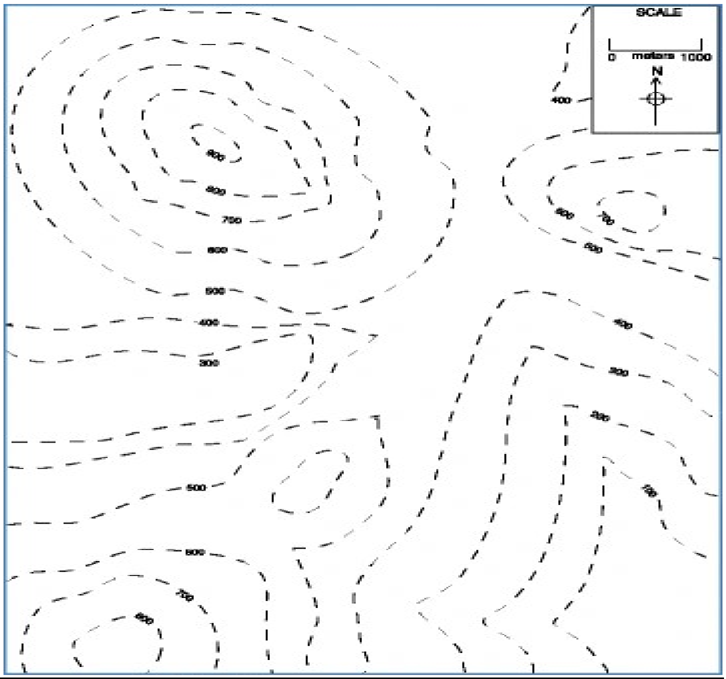
• "M" or "W" shape contours indicate upstream from stream junctions.

• Circles with hachures or hatch lines (short lines extending from the contour line at right angles)indicate a depression, pit, or sinkhole.

• Spot elevations (height of identifiable features) such as mountain summits, road intersections, andsurfaces of lakes may also be shown on the map.

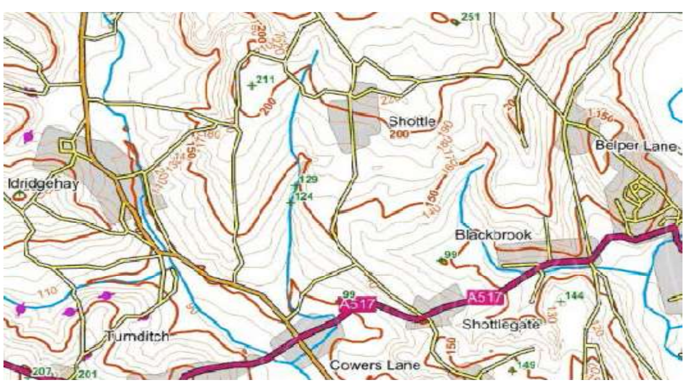
Contours never cross and will at some point close, although this may be off the map. Topographic contours that close in concentric patterns delineate hills or depressions





Contour patterns can be used to recognize distinctive landforms such as ridges, valleys and hills Contours may appear as black or colored lines on maps, and are often supported by color shading to give an impression of relief

•Cross-sections provide a useful way of visualizing the shape of the land surface, but care needs to be taken in their construction, particularly in terms of vertical exaggeration



POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 Define leveling. [2009-S]
Ans : Levelling is the art of finding the relative heights and depths of the objects on the surface of the earth. It is that part of surveying which deals with the measurements in vertical plane.

Q-2 Define level surface and horizontal plane . [2015-W,2018-W]
Ans: Level Surface: This is a surface parallel to the mean spheroidal surface of the earth is said to be a level surface.The water surface of a still lake is also considered to be a level surface.

Horizontal Plane/surface: Any plane tangential to the level surface at any point is known as the horizontal plane.It is Perpendicular to the plumb line.

Q-3 Define RL ans BM . [2011-S,2016-S,2019-W]
Ans: Reduced Level(R.L):The vertical distance of appoint above or below the datum line is known as the reduced level of that point.The Rl of a point may be positive or negative according as the point is above or below the datum.

Bench Mark: 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 or for conducting leveling operations in projects involving roads, Railways.

Q-4 What is a change point in leveling ? [2011-S]
Ans: **Turnig Point or Change Point**: For leveling over a long distance, the instrument has to be shifted a number of times. Turning point or change point connects one set of instrument readings with the next set of readings with the changed position of the Instrument. A staff is held on the turning point and a foresight is taken before shifting the instrument. From the next position of the instrument another reading is taken at the turning point keeping the staff undisturbed, which is known as back sight.

Q-5 Define contour line . [2019-W]
Ans : 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.

POSSIBLE LONG TYPE QUESTIONS

Q-1 Describe about the temporary adjustment of level. [2019-W]
Q-2 Describe about the characteristics of contours. [2014-S]
Q-3 what are the error of leveling? [2015-W]
Q-4 The following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15m.The first reading was at a chainage of165m,where RL is 98.085.The instrument was shifted after the fourth and ninth readings:

3.50,2.245,1.125,0.860,3.125,2.760,1.835,1.470,1.965,1.225,2.390 and 3.035

Mark rules on a page of your note book in the form of a level book page and enter on it the above readings and find the RL of all the points by:(1)The line of Collimation method,(2)The Rise and fall method&apply the usual checks. [2012-S,2016-S,2018-W]

Q -5 Write the uses of a contour map ? [2019-W]

CHAPTER NO-08

COMPUTATION OF AREA & VOLUME:

Learning objectives

8.1 Determination of areas, computation of areas from plans.
8.2 Calculation of area by using ordinate rule, trapezoidal rule, Simpson's rule.

8.3 Calculation of volumes by prismoidal formula and trapezoidal formula, Prismoidal corrections, curvature correction for volumes.

8.1 DETERMINATION OF AREAS, COMPUTATION OF AREAS FROM PLANS.

- The main objective of the surveying is to compute the areas and volumes.
- Generally, the lands will be of irregular shaped polygons.
- There are formulae readily available for regular polygons like, triangle, rectangle, square and other polygons.
- The main objective of the surveying is to compute the areas and volumes.
- Generally, the lands will be of irregular shaped polygons.
- There are formulae readily available for regular polygons like, triangle, rectangle, square and other polygons.
- But for determining the areas of irregular polygons, different methods are used.

Earthwork computation is involved in the excavation of channels, digging of trenches for laying underground pipelines, formation of bunds, earthen embankments, digging farm ponds, land levelling and smoothening. In most of the computation the cross sectional areas at different interval along the length of the channels and embankments are first calculated and the volume of the prismoids are obtained between successive cross section either by trapezoidal or prismoidal formula.

Calculation of area is carried out by any one of the following methods:

a) Mid-ordinate method

b) Average ordinate method

c) Trapezoidal rule

d) Simpson's rule

8.2 CALCULATION OF AREA BY USING ORDINATE RULE, TRAPEZOIDAL RULE, SIMPSON'S RULE.

Calculation of area is carried out by any one of the following methods:

a) Mid-ordinate method

b) Average ordinate method

c) Trapezoidal rule

d) Simpson's rule

The mid-ordinate rule

Consider figure.

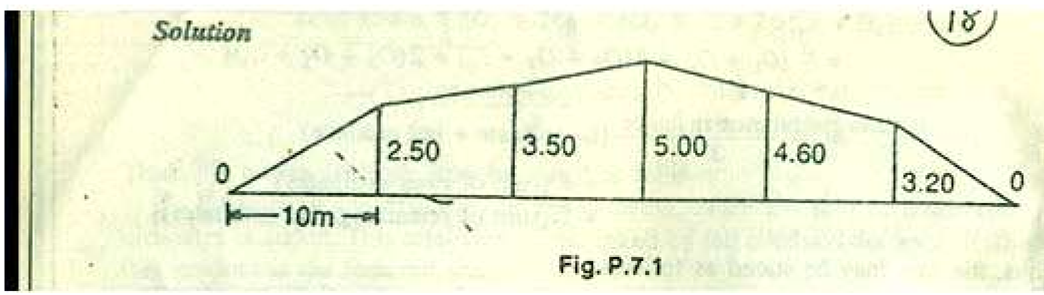


Fig. P.7.1

Let O_1, O_2, O_3, O_4.........On= ordinates at equal intervals

l=length of base line

d= common distance between ordinates

$h_1, h_2, \ldots h_n$=mid-ordinates

$$\text{Area of plot} = h_1 * d + h_2 * d + \ldots + h_n * d$$
$$= d\,(h_1 + h_2 + \ldots + h_n)$$

Area = common distance* sum of mid-ordinates

Average ordinate method

Let $O_1, O_2, \ldots O_n$=ordinates or offsets at regular intervals l= length of base line

n= number of divisions

n+1= number of ordinates

$$\text{Area}= \frac{O_1+O_2+\ldots..+O_n}{n+1} *l$$

$$\text{Area}= \frac{\text{sum of the ordinates * length of base line}}{\text{no of ordinates}}$$

The trapezoidal rule

While applying the trapezoidal rule, boundaries between the ends of ordinates are assumed to be straight. Thus the areas enclosed between the base line and the irregular boundary line are considered as trapezoids.

Let O1, O2, …..On=ordinate at equal intervals, and d= common distance between two ordinates

$$1^{st}\text{ area}= \frac{O_1+O_2}{2} *d$$

$$2^{nd}\text{ area}= \frac{O_2+O_3}{2} *d$$

$$3^{rd}\text{ area}= = \frac{O_2+O_3}{2} *d$$

$$\text{Last area}= \frac{O_{n-1}+O_n}{2} *d$$

Total area=d/2{ $O_1+2O_2+2O_3+\ldots\ldots+2O_n-1+O_n$}

$$\text{AREA} = \frac{\text{common distance ((1}^{st}\text{ ordinate +last ordinate) +2(sum of other ordinates)}}{2}$$

Thus the trapezoidal rule may be stated as follows:

To the sum of the first and last ordinate, twice the sum of intermediate ordinates is added. This total sum is multiplied by the common distance. Half of this product is the required area.

Limitation: There is no limitation for this rule. This rule can be applied for any number of ordinates

Simpson's rule

In this rule, the boundaries between the ends of ordinates are assumed to form an arc of parabola. Hence simpson's rule is some times called as parabolic rule. Refer to figure:

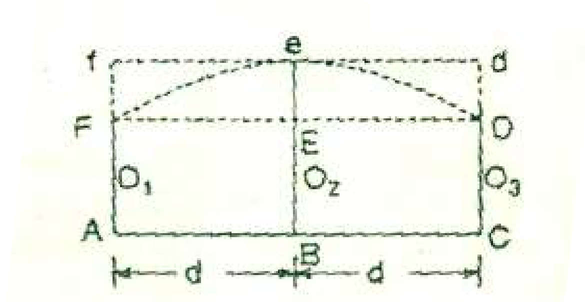


Fig.

Let

O_1, O_2, O_3= three consecutive ordinates

d= common distance between the ordinates

area AFeDC= area of trapezium AFDC+ area of segment FeDEF

Here,

$$\text{Area of trapezium} = \frac{O_1+O_3}{2} * 2d$$

Area of segment= 2/3* area of parallelogram FfdD

$= 2/3 * eE * 2d$

$= 2/3 * \{ O_2 - O_1+O_3 /2 \} * 2d$

So, the area between the first two divisions,

$$\Delta_1 = \frac{O_1+O_3}{2} * 2d + 2/3 * \{ O_2 - O_1+O_3 /2 \} * 2d$$

$= d/3(O_1+4O_2+O_3)$

Similarly, the area of next two divisions

$\Delta_2 = d/3(O_1+4O_2+O_3)$ and so on

Total area = $d/3[O_1+O_n+4(O_2+O_{4+\ldots\ldots}) + 2(O_3+O_5)]$

$$= \frac{\text{Common distance}}{3} \{\text{1st ordinate} + \text{last ordinate}) + 4(\text{sum of even ordinates}) + 2(\text{sum of remaining odd ordinate})\}$$

Thus the rule may be stated as the follows

To the sum of the first and the last ordinate, four times the sum of even ordinates and twice the sum of the remaining odd ordinates are added. This total sum is multiplied by the common distance. One third of this product is the required area.

Limitation: This rule is applicable only when the number divisions is even i.e. the number of ordinates is odd.

The trapezoidal rule may be compared in the following manner:

Trapezoidal rule	**Simpson's rule**
The boundary between the ordinates is considered to be straight	The boundary between the ordinates is considered to be an arc of a parabola
There is no limitation. It can be applied for any number of ordinates	To apply this rule, the number of ordinates must be odd
It gives an approximate result	It gives a more accurate result.

Note: sometimes one or both the end of the ordinates may be zero. However they must be taken into account while applying these rules.

Worked- out problems

Problem 1: The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m:

0, 2.50, 3.50, 5.00, 4.60, 3.20, 0 m

Compute the area between the chain line, the irregular boundary line and the end of offsets by:

a) mid ordinate rule

b) the average –ordinate rule

c) the trapezoidal rule

d) Simpson's rule

Solution: (Refer fig)

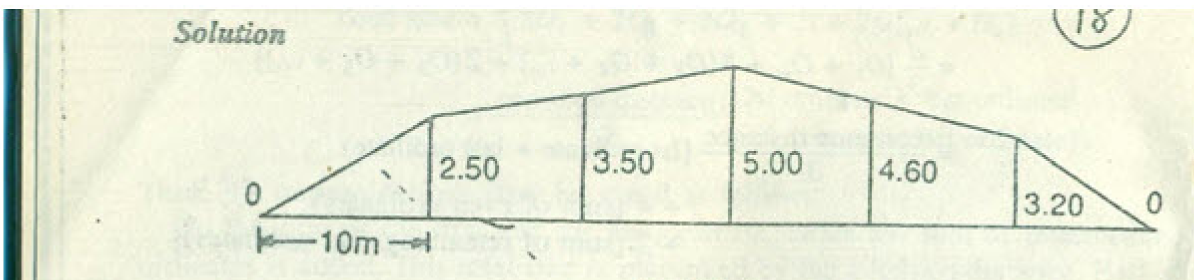


Fig. P.7.1

Mid-ordinate rule:

$$h_1 = \frac{0+2.50}{2} = 1.25 \text{ m}$$

$$h_2 = \frac{2.50+3.50}{2} = 3.00 \text{ m}$$

$$h_3 = \frac{3.50+5.00}{2} = 4.25 \text{ m}$$

$$h_4 = \frac{5.00+4.60}{2} = 4.80 \text{ m}$$

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

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

Required area= 10(1.25+3.00+4.25+3.90+1.60)

= 10*18.80=188 m^2

By average-ordinate rule:

Here d=10 m and n=6(no of devices)

Base length= 10*6=60 m

Number of ordinates= 7

Required area=10((1.25+3.00+5.00+4.60+3.20+0)/7)

$$= \frac{16*18.80}{7} = 161.14 m^2$$

By trapezoidal rule:

Here d=10m

Required area=10/2{0+0+2(2.50+3.50+5.00+4.60+3.20+)}

= 5*37.60=188 m^2

By Simpson's rule:

d=10m

required area=10/3{0+0+4(2.50+5.00+3.20)+2(3.50+4.60)}

= 10/3{ 42.80+16.20}=10/3*59.00

10/3*59= 196.66m^2

Problem 2: The following offsets were taken at 15 m intervals from a survey line to an irregular boundary line

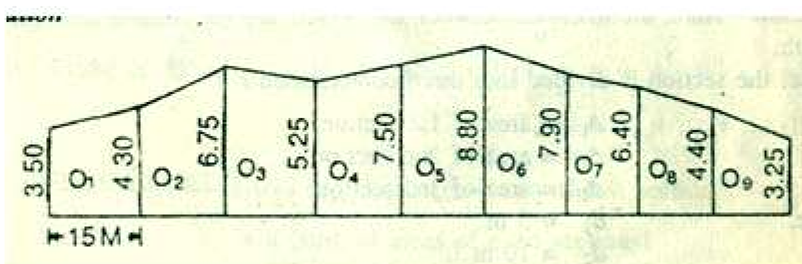
3.50,4.30, 6.75, 5.25, 7.50, 8.80, 7.90, 6.40, 4.40, 3.25 m

Calculate the area enclosed between the survey line, the irregular boundary line, and the offsets, by:

a) the trapezoidal rule

b) simpson's rule

solution:



a) The trapezoidal rule

required area=15/2{3.50+3.25+2(4.30+6.75+5.25+7.50+8.80+7.90+6.40+4.40)}

= 15/2{6.75+102.60} = 820.125 m^2

c) simpson's rule

if this rule is to be applied, the number of ordinates must be odd. But here the number of ordinates must be odd. But here the number of ordinate is even(ten).

So, simpson's rule is applied from O_1 to O_9 and the area between O_9 and O_{10} is found out by the trapezoidal rule.

A_1= 15/3{ 3.50+4.40+4(4.30+5.25+8.80+6.40)}+2(6.75+7.50+7.90)

= 15/3(7.90+99.00+44.30)= 756.00 m^2

A_2= 15/2(4.40+3.25)= 57.38 m^2

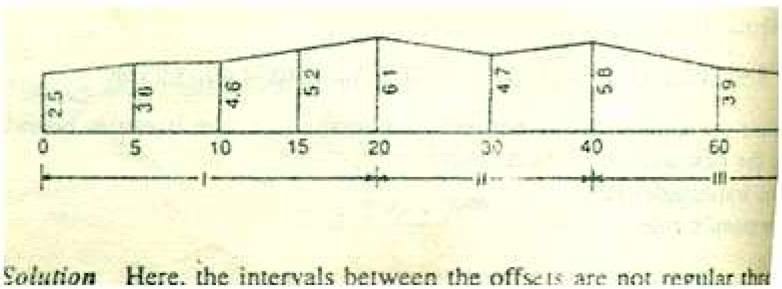
Total area= A_1+ A_2 =756.00+57.38 = 813.38 m^2

Problem 3: the following offsets are taken from a survey line to a curves boundary line, and the first and the last offsets by:

a) The trapezoidal rule

b) Simpson's rule

solution:



here the intervals between the offsets are not reglar through out the length.

So, the section is divided into three compartments

Let

Δ_I= area of the first section

Δ_{II}= area of 2^{nd} section

Δ_{III}= area of 3^{rd} section

Here

d1= 5 m

d2=10 m

d3=20 m

a) by trapezoidal rule

Δ_I= 5/2{2.50+6.10+2(3.80+4.60+5.20)} = 89.50 m^2

Δ_{II}= 10/2{6.10+5.80+2(4.70)} =106.50 m^2

Δ_{III}= 20/2{5.80+2.20+2(3.90)} = 158.00 m^2

Total area = 89.50+106.50+158.00 = 354.00 m^2

b) by simpson's rule

Δ_I= 5/3{2.50+6.10+4(3.8+5.20) + 2(4.60)} = 89.66 m^2

Δ_{II}= 10/3{6.10+5.80+4(4.70)} =102.33 m^2

Δ_{III}= 20/3{5.80+2.20+4(3.90)} = 157.33 m^2

Total area= 89.66+102.33+157.33 = 349.32 m^2

8.3 CALCULATION OF VOLUMES BY PRISMOIDAL FORMULA AND TRAPEZOIDAL FORMULA, PRISMOIDAL CORRECTIONS, CURVATURE CORRECTION FOR VOLUMES.

FORMULA FOR CALCULATION OF VOLUME:

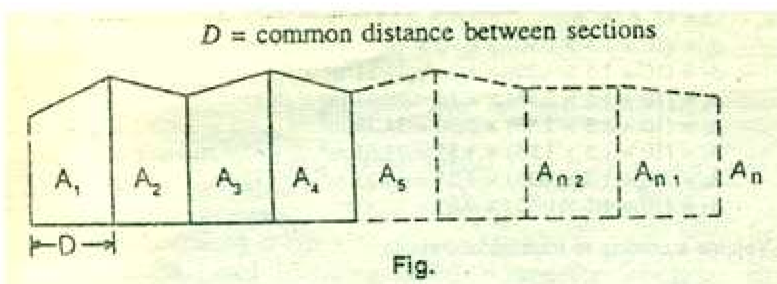


Fig.

D= common distance between the sections

Trapezoidal formula

volume (cutting or filling), V=D/2(A1+An+2(A2+A3+….+An-1))

i.e. volume = $\frac{\text{common distance}}{2}$ {area of first section+ area of last section +2(sum of area of other sections)}

Prismoidal formula

Volume(cutting or filling), V= D/3{A_1+ A_n +4(A_2+ A_4+ A_{n-1})+ 2(A_3+ A_5+….+ An_{n-1})}

i.e. $$V=\frac{\text{common distance}}{3}\{\text{area of }1^{st}\text{ section}+\text{ area of last section}+4(\text{sum of areas of even sections})+2(\text{sum of areas of odd sections})$$

Note: the prismoidal formula is applicable whrn there is an odd number of sections. If the number of sections is even, the end strip is treated separately and the area is calculated according to the trapezoidal rule. The volume of the remaining strips is calculated in the usual manner by the prismoidal formula. Then both the results are added to obtain the total volume.

Prismoidal Correction:

The difference between the volumes computed by the trapezoidal formula and the prismoidal formula is known as prismoidal correction. The volume by prismoidal formula is more nearly correct. Since the volume calculated by trapezoidal formula is usually more than that calculated by prismoidal formula, therefore the prismoidal correction is generally subtractive.

Thus volume by prismoidal formula=volume by trapezoidal formula -prismoidal correction.

In the formulae of prismoidal correction given below, the small and capital letters refer to the notations of the adjacent sections. The prismoidal correction is denoted by C_P.

Curvature Correction for Volumes:

The trapezoidal and prismoidal formulae are derived on the assumption that the sections are parallel to each other and normal to the centre line. But when the centre line is on a curve, the-sections do not remain parallel to each other and a correction for curvature has to be applied.
This effect is not much pronounced and does not involve large quantities of earth work in ordinary cases, therefore it is neglected. But it has to be considered in final estimates and precise results.

This is quite appreciable in the case of road widening and hill side sections which are partly in cutting and partly in filling. Curved volumes are calculated from Pappu's theorem. It states that the volume swept by a constant area rotating about a fixed axis is equal to the product of that area and the length of the path traced by the centroid of the area. When the areas are not uniform, mean distance of the centroid from the centre line is taken as

Problem 1: an embankment of width 10 m and side slopes 1 ½:1 is required to be made on a ground which is level in a direction transverse to the centre line. The central heights at 40 m intervals are as follows:

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

Calculate the volume of earth work according to

i) Trapezoidal formula

ii) Prismoidal formula

Solution: the c/s areas are calculated by

$\Delta = (b+sh)*h$

$\Delta_1 = (10+1.5*0.90)*0.90 = 10.22\ m^2$

$\Delta_2 = (10+1.5*1.25)*0.90 = 14.84\ m^2$

$\Delta_3 = (10+1.5*1.25)*2.15 = 28.43\ m^2$

$\Delta_4 = (10+1.5*2.50)*2.50 = 34.38\ m^2$

$\Delta_5 = (10+1.5*1.85)*1.85 = 23.63\ m^2$

$\Delta_6 = (10+1.5*1.35)*1.35 = 16.23\ m^2$

$\Delta_7 = (10+1.5*0.85)*0.85 = 9.58\ m^2$

(a) Volume according to trapezoidal formula

$V = 40/2\{10.22+ 9.58+2(14.84+28.43+34.38+23.63+16.23)\}$

$= 20\{19.80+235.02\} = 5096.4\ m^2$

(b) Volume calculated in prismoidal formula:

$V = 40/3\ \{10.22+9.58+4(14.84+34.38+16.23)+2(28.43+23.63)\}$

$= 40/3\ (19.80+ 261.80+104.12) = 5142.9\ m^2$

Problem the areas enclosed by the contours in the lake are as follows:

Contour (m)	270	275	280	285	290
Area (m^2)	2050	8400	16300	24600	31500

Calculate the volume of water between the contours 270 m and 290 m by:

i) Trapezoidal formula

ii) Prismoidal formula

Volume according to trapezoidal formula:

=5/2{2050+31500+2(8400+16300+24600)}

=330,250 m^3

POSSIBLE SHORT TYPE QUESTIONS WITH ANSWER

Q-1 Define earthwork computation. [2019-W]

Ans: Earthwork computation is involved in the excavation of channels, digging of trenches for laying underground pipelines, formation of bunds, earthen embankments, digging farm ponds, land leveling

Q-2 What are the methods for calculation area of a plane? [2015-S]

Ans:

a) Mid-ordinate method

b) Average ordinate method

c) Trapezoidal rule

d) Simpson's rule

Q-3 Write the formula for trapezoidal rule. [2018-W]

$$\text{AREA} = \frac{\text{common distance }((1^{st}\text{ ordinate} + \text{last ordinate}) + 2(\text{sum of other ordinates})}{2}$$

Ans:

Q-4 Write the trapezoidal formula for calculation of volume of earthwork. [2019-W]

Ans:

$$\text{i.e. volume} = \frac{\text{common distance}}{2}\ \{\text{area of first section} + \text{area of last section} + 2(\text{sum of area of other sections})\}$$

POSSIBLE LONG TYPE QUESTIONS

Q-1 An embankment of width 10 m and side slopes 1 ½:1 is required to be made on a ground which is level in a direction transverseto the centre line. The central heights at 40 m intervals are as follows: [2014-W,2017-W,2018-W]

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

Calculate the volume of earth work according to

i) Trapezoidal formula

ii) Prismoidal formula

Q-2 The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m: [2017-W]

0, 2.50, 3.50, 5.00, 4.60, 3.20, 0 m

Compute the area between the chain line, the irregular boundary line and the end of offsets by:

a) Mid ordinate rule

b) The average –ordinate rule

c) The trapezoidal rule

d) Simpson’s rule