PART–I

SURVEYING

CHAPTER 1

Introduction

1.1 DEFINITION

Surveying is the art of determining the relative positions of distinctive features on the surface of the earth or beneath the surface of the earth, by means of measurements of distances, directions and elevations.

The branch of surveying which deals with the measurements of relative heights of different points on the surface of the earth, is known as levelling.

1.2 OBJECT OF SURVEYING

The object of surveying is preparation of plans and maps of the ground surface areas. The practical importance of surveying cannot be over-estimated. In the absence of accurate maps, it is impossible to lay out the alignments of roads, railways, canals, tunnels, transmission power lines, and microwave or television relaying towers. Detailed maps of the sites of engineering projects, are necessary for precision establishment of sophisticated instruments. Surveying is the first step for the execution of any project. As the success of any engineering project is based upon accurate and complete survey work, an engineer must, therefore, be thoroughly familiar with the principles and different methods of surveying and mapping. It is for this reason, the subject of surveying has been made compulsory to all the disciplines of engineering at diploma and degree courses.

1.3 PLANE SURVEYING

The surveys in which earth surface is assumed as a plane and the curvature of the earth is ignored, are known as *Plane surveys*. As the plane surveys extend only over small areas, the lines connecting any two points on the surface of the earth, are treated as straight lines and the angles between such lines are taken as plane angles.

Plane surveys are used for the lay-out of highways, railways, canals, fixing boundary pillars, construction of bridges, factories, etc. The scope and use of plane surveys are very wide. For majority of engineering projects, plane surveying is the first step to execute them. For proper, economical and accurate planning of projects, plane surveys are basically needed and their practical significance cannot be over-estimated.

1.4 CLASSIFICATION OF SURVEYING BASED ON INSTRUMENTS USED

According to the instruments or methods employed, surveys are classified as under :

(1) Chain surveying.
(2) Compass surveying.
(3) Plane table surveying.
(4) Theodolite surveying.
(5) Tacheometric surveying.
(6) Traingulation surveying.
(7) Aerial surveying.
(8) Photogrammetric surveying.

1.5 PRINCIPLE OF SURVEYING

The fundamental principles upon which different methods of surveying are based, are very simple. These are stated as under:

(1) Working from the whole to the part. The main principle of surveying is to work from the whole to the part. 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 methods. Further details are surveyed with the help of these minor control points by adopting any one of the survey methods. The main idea of working from the whole to the part is to prevent accumulation of errors and to localise minor errors within the frame work of the control points. On the other hand, if survey is carried out from the part to the 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 are surveyed with lesser accuracy.

(2) **Location of a point by measurement from two control points.** Two control points are selected in the area and the 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 Cartesian co-ordinates. The location of the required point may then be plotted by making two measurements from the given control points.

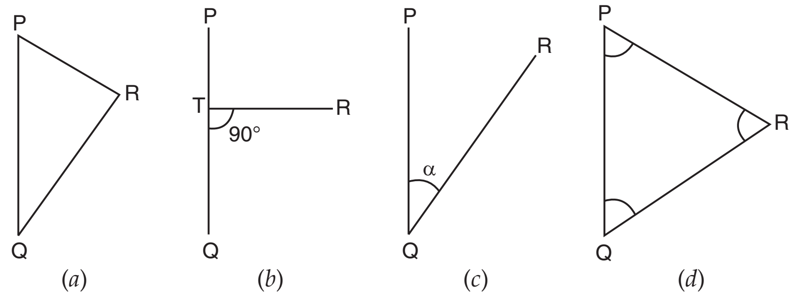


Fig. 1.1. Principle of surveying.

Let *P* and *Q* be two given control points. Any other point, say, *R* may be located with reference to these points, by any one of the following methods (Fig. 1.1).

(*a*) **By measuring the distances PR and QR.** The distances *PR* and *QR* may be measured and the location of *R* may be plotted by drawing the arcs to the same scale to which line *PQ* has been drawn [Fig. 1.1 (*a*)].

(*b*) **By dropping a perpendicular from R on PQ.** A perpendicular *RT* may be dropped on the line *PQ*. Distances *PT*, *TQ* and *RT* are measured and the location of *R* may be plotted by drawing the perpendicular *RT* to the same scale to which line *PQ* has been drawn [Fig. 1.1 (*b*)].

Principles (*a*) and (*b*) are generally used in the method of '**Chain surveying**'.

(*c*) **By measuring the distance QR and the angle PQR.** The distance *QR* and the angle *PQR* say equal to α, are measured and the location of *R* may be plotted either by means of a protractor or trigonometrically [Fig. 1.1 (*c*)]. This principle is used in the method of **Theodolite Traversing.**

(*d*) **By measuring the interior angles of the triangle PQR.** The interior angles *P*, *Q* and *R* of the triangle *PQR* are measured with an angle-measuring instrument such as theodolite. The lengths of the sides *PR* and *QR* are calculated by solving the triangle *PQR* by applying the sine rule. Even without calculating the co-ordinates, or sides, the location of *R* can be obtained by plotting the angles *PQR* and *QPR* [Fig. 1.1 (*d*)]. This principle is used in the method of '**Triangulation**'.

1.6 LINEAR MEASURES

According to the standards of Weight and Measure Act (India) 1956, the metric system has been introduced in India. The units of measurement of distances, have been recommended as metre and centimetre for the execution of land surveys.

(*a*) *Basic units of length in metric system:*

10 milimetres = 1 centimetre
10 centimetres = 1 decimetre
10 decimetres = 1 metre
10 metres = 1 dekametre
10 dekametres = 1 hectametre
10 hectametres = 1 kilometre
1.852 kilometres = 1 nautical mile.

(*b*) *Basic units of area in metric system:*

100 sq. metres = 1 acre
10 acres = 1 deka acre
10 deka acres = 1 hecta acre

(*c*) *Basic units of volume in metric system:*

1000 cub. millimetres = 1 cub. centimetre
1000 cub. centimetres = 1 cub. decimetre
1000 cub. decimetres = 1 cub. metre.

1.7 STAGES OF SURVEY OPERATIONS

The entire work of a survey operation may be divided into three distinct stages:

(*i*) Field work

(*ii*) Office work

(*iii*) Care and adjustment of the instruments

1. Field work. The field work consists of the measurement of distances and angles required for plotting to scale and also keeping a systematic record of what has been done in the form of a field book or measurement book. Field work is further divided into three stages:

(*a*) Reconnaissance

(*b*) Observations

(*c*) Field Records

(*a*) **Reconnaissance.** During reconnaissance the surveyor goes over the area to fix a number of stations, ensuring necessary inter-visibility to establish a system of horizontal control. A few permanent stations are also selected for an extension of the survey in future.

(*b*) **Observations.** The surveyor makes necessary observations with survey instruments for linear and angular measurements. The observations also include determination of differences in elevations between the stations, establishment of points at given elevations and surveying contours of land areas and bathymetric contours of water bodies. Method of observation depends upon the nature of the terrain type of the instruments and the method of surveying.

(*c*) **Field records.** All the measurements are recorded in a field book. Every care is taken to ensure correct entries of all the observations otherwise the survey may be useless. The competency of a surveyor is judged by his field records.

EXERCISE 1

1. Define surveying. Explain its importance for Civil Engineers.

2. Explain the fundamental principle on which the art of surveying is based.

3. (*i*) What are the objects of plane surveying?

(*ii*) Give a classification of surveys based on the instruments used.

CHAPTER 2

Linear Measurements

2.1 INTRODUCTION

For surveying the locations of various objects on the earth surface, the horizontal distances between them are required to be measured. The method of determining the distances between various points on the surface of the earth, is called the method of *linear measurement*.

2.2 INSTRUMENTS FOR MEASURING DISTANCES

For the measurement of distances, the following instruments are required:

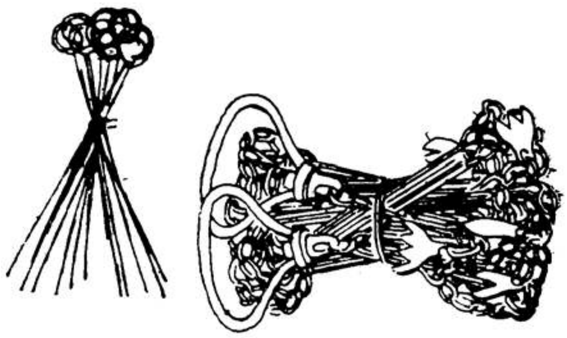


Fig. 2.1. A Chain and arrows.

1. **Chain** (Fig. 2.1). A chain is generally divided into 100 or 150 links. The links are composed of pieces of galvanised mild steel wire 4 mm in diameter. The ends of each link are bent into loops and connected together by means of three oval shaped rings which afford

the flexibility to the chain. The joints of the links are usually open but in good quality chains, these are welded so that true lengths of the chain, does not alter due to stretching. The ends of the chain are provided with brass handles with swivel joints so that the 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 a 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 at both the ends of a link. (Fig. 2.2). The end links also include the length of handles. Metallic tags of different patterns are fixed at various important points of a chain *i.e.,* 5 m, 10 m, 15 m, 20 m, for quick and easy reading of the chain.

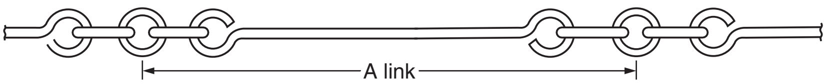


Fig. 2.2. A link.

Classification of Chains. The chains are generally of the following types:

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

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

(*c*) **Metric chain.** It is 20 m or 30 m long and is divided into 100 or 150 links respectively. Each link measures 20 cm.

Metric chains are shown in Figs. 2.3 and 2.4.

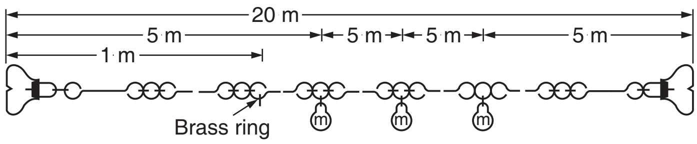


Fig. 2.3. A 20 metre chain

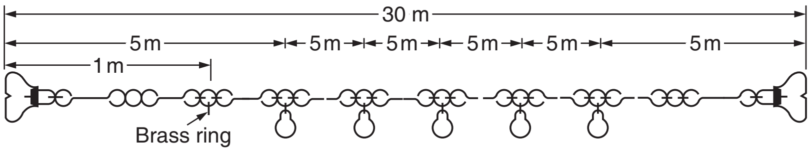


Fig. 2.4. A 30 metre chain.

According to I.S. 1492–1956 the surveying chains are calibrated into metres and its further smaller sub-divisions. Metric surveying chains are available in lengths of 20 metres and 30 metres. Small brass rings are provided at every metre length except at 5 m, 10 m, 15 m, 20 m, 25 m, where tallies are fixed, to facilitate the reading of the fractions of the chain.

The handles of the chain are provided with grooves so that the arrow can be held at the correct position. The tallies used for marking 5 m, 10 m, etc. are marked with letter M to distinguish the metric chain from a non-metric chain. The length of the chain whether 20 m or 30 m, is indicated on the handle, for easy identification.

Suitability of Chain. The chains are suitable for the following reasons:

(1) Its length alters due to continued use. Hence, it is suitable only for ordinary work.

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

(3) Being heavier, a chain sags considerably when suspended at its ends.

(4) It is only suitable for rough usage.

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

(6) It can be read easily.

Testing a Chain. During its use, links of a chain get bent and consequently the length of the chain decreases. On the other hand, the length of a chain increases due to the following factors:

(*i*) Wearing and tearing of 600 wearing surfaces.

(*ii*) Stretching of the links and the joints.

(*iii*) Opening out of small rings due to prolonged usage.

(*iv*) Rough handling through hedges, fences, etc.

This necessitates checking of the length of the chain before commencing the day's work and at frequent intervals afterwards. If the chain is not tested, the measurements will become unreliable. Before checking a chain, the surveyor should ensure that its links are not bent, openings are not too wide and there is no mud attached to them.

When a tension of 8 kg is applied at the ends of a chain and compared against a certified steel band standardised at 20°C, every metre length should be accurate within ± 2 mm and the overall length of the chain should be within the following limits:

20 metre chain ± 5 mm.

30 metre chain ± 8 mm.

Testing and adjusting a chain in the field. The length of a chain may be tested and adjusted as under:

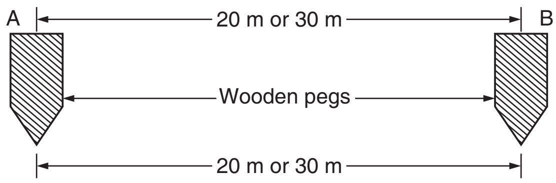


Fig. 2.5. A Adjustment of a chain length.

Two wooden pegs are driven at requisite distance apart *i.e.* 20 m or 30 m and nails are inserted onto their tops to mark the exact distance between points as shown in Fig. 2.5.

The overall length of the chain should be compared against the fixed points (nails) and the difference, if any, should be noted.

On comparison, if a chain is found to be longer than its standard length, it may be adjusted by:

(*i*) Closing the opened joints of the rings.

(*ii*) Reshaping the elongated rings.

(*iii*) Removing one or more small circular rings.

(*iv*) Replacing the worn-out rings.

On the other hand, if the chain is found to be too short, it may be adjusted by:

(*i*) Straightening the bent links.

(*ii*) Flattening the circular rings.

(*iii*) Replacing one or more small circular rings by bigger ones.

(*iv*) Inserting additional circular rings.

However, in both the cases, adjustment must be done symmetrically so that the measurements made by different portions of the chain, do not differ considerably.

2. Tapes. Depending upon the material, the tapes are classified as under:

(*i*) Cloth or linen tape (*ii*) Metallic tape

(*iii*) Steel tape. (*iv*) Invar tape.

(1) **Cloth or Linen Tape** (Fig. 2.6). Linen tapes are closely woven linen and varnished to resist moisture. They are generally 10 metres to 30 metres in length and 12 mm to 15 mm in width. One end of the tape is provided with a ring whose length is included in the total length of the tape. Cloth tapes are generally used for measuring offset measurements. They are seldom used for making accurate measurements due to the following reasons:

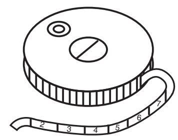


Fig. 2.6. A cloth or lines tape.

(*i*) It is easily affected by moisture and thus gets 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.

(2) **Metallic Tape.** (Fig. 2.7). A linen tape reinforced with brass or copper wires to prevent stretching or twisting of fibres 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 also available in different lengths but the tapes of 20 m and 30 m lengths are more common. These are supplied in leather cases, with a winding device. Each metre is divided into decimetres, and each decimetre is further sub-divided into centimetres.

Fig. 2.7. A metallic tape.

A metallic tape can be used for measuring accurate distances but it is commonly used for taking offset distances in chain surveying.

(3) **Steel Tape.** (Fig. 2.8). Steel tapes are available with different accuracy of graduations. A steel tape of lowest degree of accuracy is generally superior to a metallic or cloth tape for linear measurements. Steel tapes which consist of a light strip of width 6 to 10 mm are

accurately graduated. Steel tapes are available in different lengths but 10 m, 20 m, 30 m and 50 m, steel tapes are usually used for survey measurements. At the end of the tape, a brass ring is provided The length of the metal ring is included in the length of the tape. It is wound in a leather metal case, having a suitable winding device.

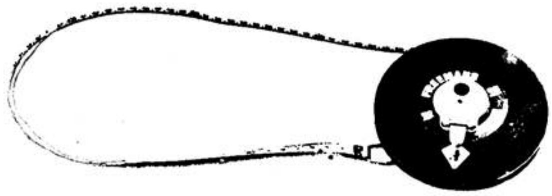


Fig. 2.8. A steel tape.

As steel tapes are delicate, they are generally not used in a terrain with vegetation or on rocky grounds.

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

2.3 INSTRUMENTS USED FOR MAKING STATIONS

The following instruments are required for making the survey stations:

1. **Chain Pins (arrows)** (Fig. 2.9).

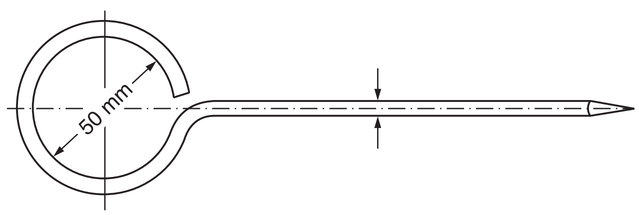


Fig. 2.9. An arrow.

An arrow is made of steel wire 4 mm (8 S.W.G.) in diameter. The length of the arrow may vary from 25 cm to 50 cm but the length in common use is 40 cm. One end of the arrow is bent into a loop of a circle of 50 mm diameter and the other end is made pointed sharp. 10 arrows generally accompany a chain. A chain pin (or arrow) is used to locate the end of the chain length.

2. **Pegs.** Wooden pegs usually 2.5 cm square and 15 cm deep are used to mark the positions of the survey stations.

3. **Iron Pegs.** When surveying in a comparatively harder ground, either iron pegs or loop wire nails, are generally used in place of wooden pegs.

4. **Ranging Rods.** Ranging rods are used for marking the positions of stations while ranging a line. It is made of well seasoned straight grained timber of teak or deodar and is generally available in 2 m or 3 m length having a 3 cm nominal diameter. It is divided into equal parts, each part measures 0.2 m. Its lower end is provided with a cross shoe of 15 cm length. It is generally painted alternatively black and white throughout its length.

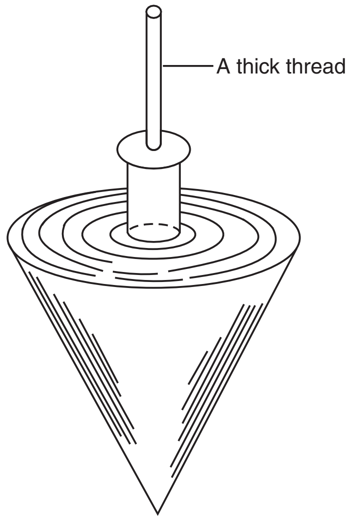


Fig. 2.10. A plumb bob.

5. **Ranging Poles.** The ranging poles are similar to ranging rods excepting that these are of heavier section of length 4 m to 6 m. These are used for ranging very long lines in undulating ground.

6. **Offset Rods.** The offset rod is also similar to a ranging rod. It is generally 3 m long and is divided into equal parts of 0.2 m. The top of the offset rod is provided with an open hook for pulling or pushing a chain through obstructions *i.e.*, hedges, bushes, etc. Two narrow vertical slots passing through the centre of the section at right angles to one another are provided at the eye level. It is used for aligning the offset line and measuring short offsets.

7. **Plumb Bob.** (Fig. 2.10). It is used to transfer the end points of the chain onto ground while measuring distances in a hilly terrain. It is also used for testing the verticality of ranging poles, ranging rods on levelling staves.

2.4 RANGING A LINE

The process of marking 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, is called *ranging*, When the line is short or its end station is clearly visible, the chain may be laid in true alignment. If the line is long or its end station is not visible due to intervening undulating ground, it is required to mark a number of points with ranging rods such as *a*, *b*, *c*, *d*, etc. along the chain line prior to chaining the distance between *A* and *B* (Fig. 2.11).

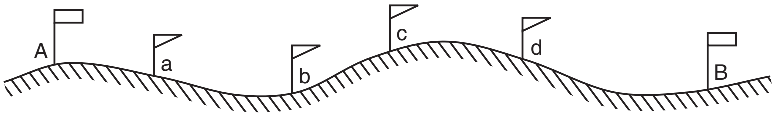


Fig. 2.11. Ranging a line.

Ranging may be done either by eye estimation or by using a line ranger or a theodolite.* Theodolites are generally used for important work only.

Classification of Ranging. Ranging may be classified as:

(*i*) Direct ranging (*ii*) Indirect ranging.

1. **Direct Ranging.** The process of establishing a number of intermediate ranging rods along the chain line by direct observation from either end station, is known as '*Direct Ranging*'.

Following steps are taken in direct ranging.:

(*i*) Erect ranging rods or poles vertically behind each end of the line.

(*ii*) Stand about 2 m behind the ranging rod at the beginning of the line.

(*iii*) Direct the assistant to hold a ranging rod vertically at arm's length at the point where the intermediate station is to be established.

(*iv*) Direct the assistant to move the rod to the right or left, until the three ranging rods appear to be exactly in a straight line.

(*v*) Stoop down and check the position of the rod by sighting over their lower ends in order to avoid error due to non-verticality of the ranging rods.

(*vi*) After ascertaining that the three ranging rods are in a straight line, signal the assistant to fix the ranging rod.

*Theodolite, discussed later.

Code of Signals

The following codes of signals are used to direct the assistant while ranging a line:

S.No.	*Code*	*Meaning*
1.	Rapid sweeps with right hand	Move considerably to the right.
2.	Rapid sweeps with left hand	Move considerably to the left.
3.	Slow sweeps with right hand	Move slowly to the right.
4.	Slow sweeps with left hand	Move slowly to the left.
5.	Right arm extended	Continue to move to the right
6.	Left arm extended	Continue to move to the left.
7.	Right arm up and moved to the right	Plumb the rod to the right.
8.	Left arm up and moved to the left	Plumb the rod to the left.
9.	Both arms above head and then brought down	Correct.
10.	Both arms extended forward and depressed briskly.	Fix.

Note: The following points may be noted:

(*i*) The signals should be made clearly without any confusion.

(*ii*) When the assistant is at a great distance, the surveyor should use his handkerchief for signalling.

2. **Indirect Ranging** (Fig. 2.12). When end stations are not intervisible and the intermediate ranging rods are placed in line by interpolation or by reciprocal ranging or by running an auxiliary line (or random line), the process is known as *Indirect Ranging*.

Indirect ranging is resorted to when the end stations of a line are not intervisible due to raised ground.

Intervening a raised ground. Let *A* and *B* two end stations intervened by raised ground. The ranging of intermediate points, may be done as done as discussed below:

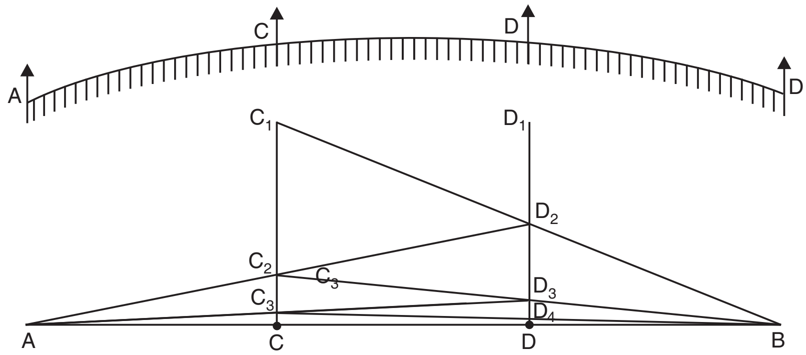


Fig. 2.12. Indirect ranging.

Procedure:

The following steps are involved:

(*i*) Fix two ranging rods *A* and *B* at the ends of the chain line.

(*ii*) Send two assistants with ranging rods to take positions at *C* and *D* as nearly on the line as can be judged.

(*iii*) Ensure that the assistant at *C* can see the ranging rods held at *B* and *D* and the assistant at *D* can see the ranging rods held at *A* and *C*.

(*iv*) Direct the assistants to proceed to line themselves alternately.

(*v*) Assistant at *C* should direct the assistant at *D* to be in line with *B*, and then the assistant at *D* should direct the assistant at *C* to be in line with *A*.

(*vi*) By directing each other, successively the two assistants go on changing their positions until both are exactly on the line *AB*.

(*vii*) Erect the ranging rods at *C* and *D* which serve as intermediate stations for ranging other points.

The above method may also be used for ranging a line across a valley.

2.5 CHAINING A LINE

For a chaining operation, two chainmen are required. The chainman at the forward end of the chain is called a *leader* while the other at the rear end, is called a *follower*.

The duties of the *leader* and *follower* are tabulated under:

Leader	*Follower*
1. To drag the chain forward.	1. To direct the leader to be in line with the ranging rod at the end station.
2. To insert an arrow at the end of every chain.	2. To carry the rear end of the chain ensuring that it is dragged above the ground.
3. To obey the instructions of the follower.	3. To pick up the arrows inserted by the leader.

2.6 UNFOLDING A CHAIN

Unfolding a chain must be done with great care. After removing the leather strap, both the handles should be held in the left hand and the chain should be thrown well forward with the right hand. The leader, then should take one handle of the chain and move forward until the chain is extended to its full length. The chain is then examined to see if there are any kinks or bent links. This operation is called *unfolding the chain*.

2.7 METHOD OF CHAINING

To chain a line, the follower holds the handle of the chain in contact with the peg at the beginning of the line and direct the leader to be in line with the ranging rod-fixed at the end of the chain line. The leader, taking 10 arrows in one hand and the other handle of the chain in other hand, walks along the line dragging the chain. At the end of the chain, the leader holds a ranging rod vertically in contact with the outside of the handle at arm's length and faces the follower. Using the code of signals, the follower directs the leader to move the rod to the right or left as required by the follower until it is exactly in line. The leader then holds the handle in both hands, stands in the line and straightens the chain by jerking it and stretches over the mark. He then holds the arrow against the end of the handle and inserts it vertically into the ground to mark the end of a chain length. If the ground is hard, the end of chain length may be marked by a cross (+) scratched with an arrow or made by chalk. An arrow is laid at the cross.

The leader, then holding the ranging rod and the remaining nine arrows, starts off dragging the chain a little off the line so that the arrow placed in position, is not disturbed.

The follower, holding the rear handle comes to the last fixed arrow and calls '*chain*' to give a warning to the leader that he has already reached a chain length and that he should stop moving forward. The process as explained in earlier paragraph is again repeated. When the tenth chain length is measured, the follower is left with no arrow. The follower then asks him to wait. He hands over all the ten arrows to the leader. The surveyor records the transfer of arrows in his field work. To measure a fractional length of a chain, the leader should drag the chain beyond the station and the follower holds the chain handle against the last arrow. The leader after stretching the chain comes to the station mark and counts the odd links.

2.8 FOLDING THE CHAIN

After the day's work the chain should be folded into 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 links 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 which somewhat resembles a sheaf of corn, it is tied with the leather strap.

2.9 ERROR IN MEASUREMENT DUE TO INCORRECT CHAIN LENGTH

Due to usage of a chain over a rough ground, its oval shaped rings get elongated and thus the length of the chain gets increased. On the other hand, sometimes, some of the links get bent and consequently the length of the chain gets decreased. It may therefore, be noted that length obtained by chaining with a faulty chain, is either too long or too short than the length which would be obtained by chaining with the chain of standard length. The general rule applicable may be stated:

"*If the chain is too long, the measured distance will be less and if the chain is too short, the measured distance will be mo*re."

Let L be true length of the chain

L' be faulty length of the chain

The true length of the line

$$= \frac{L'}{L} \times \text{measured length of the line.}$$

Remember. Product of the correct length and correct chain length = Product of the incorrect length and incorrect chain length *i.e.,*

''Birds of the same feather, flock together''.

Example 2.1. *The length of a survey line measured with a 30 m chain was found to be 631.5 m. When the chain was compared with a standard chain, it was found to be 0.10 m too long. Find the true length of the survey line.*

Solution. True length of the line

$$= \frac{L'}{L} \times \text{measured length of the line.}$$

Here $L' = 30.10$ m, $L = 30$ m and

Measured length of survey line

$= 631.5$ m

$\therefore$ True length of the survey line

$$= \frac{30.10}{30} \times 631.5 = 633.605 \text{ m.} \quad \textbf{Ans.}$$

2.10 COMMON MISTAKES IN CHAINING

While chaining a line, inexperienced chainmen generally commit the following mistakes:

1. Displacement of the arrows.
2. Failure to observe the zero point of the tape.
3. Adding or omitting a full chain length.
4. Reading from the wrong end of the chain
5. Reading numbers wrongly.
6. Reading wrong metre marks.
7. Calling numbers wrongly.
8. Wrong recording in the field book.

EXERCISE 2

1. State whether the following statements are true or false:

(*i*) A metallic tape is made of an alloy of nickel and steel.

(*ii*) Error due to faulty length of a chain is cumulative.

(*iii*) Line ranger is used for ranging when end stations are not intervisible.

(*iv*) It is better to move up than step down while measuring the distance along sloping ground.

(*v*) Cumulative errors though large, as compared to compensating errors, can be corrected but not the compensating errors.

2. Describe different types of chains commonly used in surveying, stating the special advantages of each.

3. Describe different types of tapes commonly used in surveying stating the advantages of each.

4. Give a complete list of instruments and equipments required for measuring a distance between two points on the surface of the earth.

5. Explain, in detail, how a chain is tested and adjusted in the field.

6. Describe how you would range a survey line between two points which are not intervisible due to an intervening raised ground.

7. What are the different kinds of ranging? Describe with sketches the method used for ranging across a high ground.

8. Explain the following:

(*i*) The leader and the follower,

9. The length of a line measured with a 20-metre chain was found to be 375 metres. The true length of the line was known to be 374.5 metres. Find the error in the chain.

10. The length of a line found to be 600 metres when measured with a 20-metre chain. If the chain in 15 cm too short, find out the correct length of the line.

Answers

9. 2.7 cm **10.** 595.5 m.

CHAPTER 3

Chain Surveying

3.1 CHAIN SURVEYING

Chain surveying is one of the methods of land surveying.

The method of surveying in which sides of various triangles are measured directly in the field without resorting to angular measurements, is called *chain surveying*.

3.2 SUITABILITY OF CHAIN SURVEYING

Chain surveying is most suitable in the following cases:

(1) When the ground is fairly level and open with simple details.
(2) When large scale plans are required, such as those for a factory site.
(3) When the area is comparatively small in extent.

3.3 UNSUITABILITY OF CHAIN SURVEYING

Chain surveying is unsuitable in the following cases:

(1) for large areas.
(2) for areas crowded with details.
(3) for wooded terrains.
(4) for undulating areas.

3.4 PRINCIPLE OF CHAIN SURVEYING

The principle of chain surveying is to divide the area into a number of triangles of suitable sides. As a triangle is the only a simple plane geometrical figure which can be plotted with the lengths of its sides alone, a network of triangles is preferred to in chain surveying (Fig. 3.1).

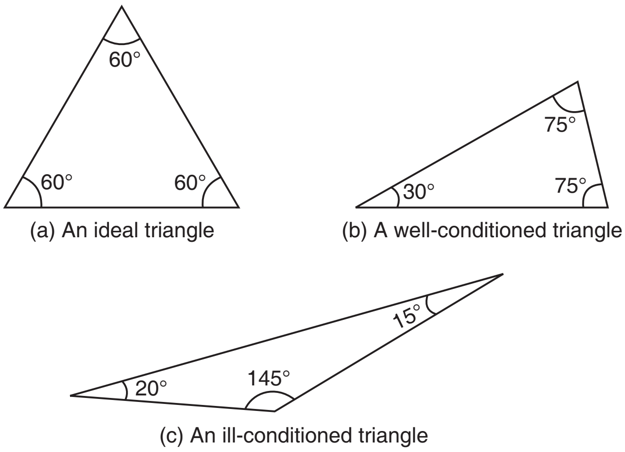


Fig. 3.1 Different types of triangles

3.5 SHAPE, SIZE AND ARRANGEMENT OF TRIANGLES

When the position of a point is fixed by the intersection of two arcs, its displacement due to errors in their radii is minimum, if the arcs intersect at 90°. In chain surveying all the three sides of a triangle are liable to error. Hence, all the sides of the triangle, should preferably be equal, having each angle nearly 60°. An equilateral triangle is also more accurately plottable than an obtuse angled triangle. Hence, to ensure minimum distortion due to errors in measurement and plotting, the best shaped triangle is an equilateral triangle. Due to varied configuration of the ground, it is not always possible to have equilateral triangles. Attempts should, therefore, be made to have triangles which are very nearly equilateral. Such triangles are known as *well-conditioned* or *well-shaped triangles*. A well conditioned triangle should not contain angles smaller than 30° and greater than 120°.

The triangles having angles smaller than 30° or greater than 120°, are known as *ill-conditioned triangles*.

The exact arrangement of the triangles to be adopted in the field, depends upon the shape, topography of the ground and the natural or artificial obstacles met with.

3.6 TECHNICAL TERMS USED IN CHAIN SURVEYING

The following technical terms are generally used in chain surveying:

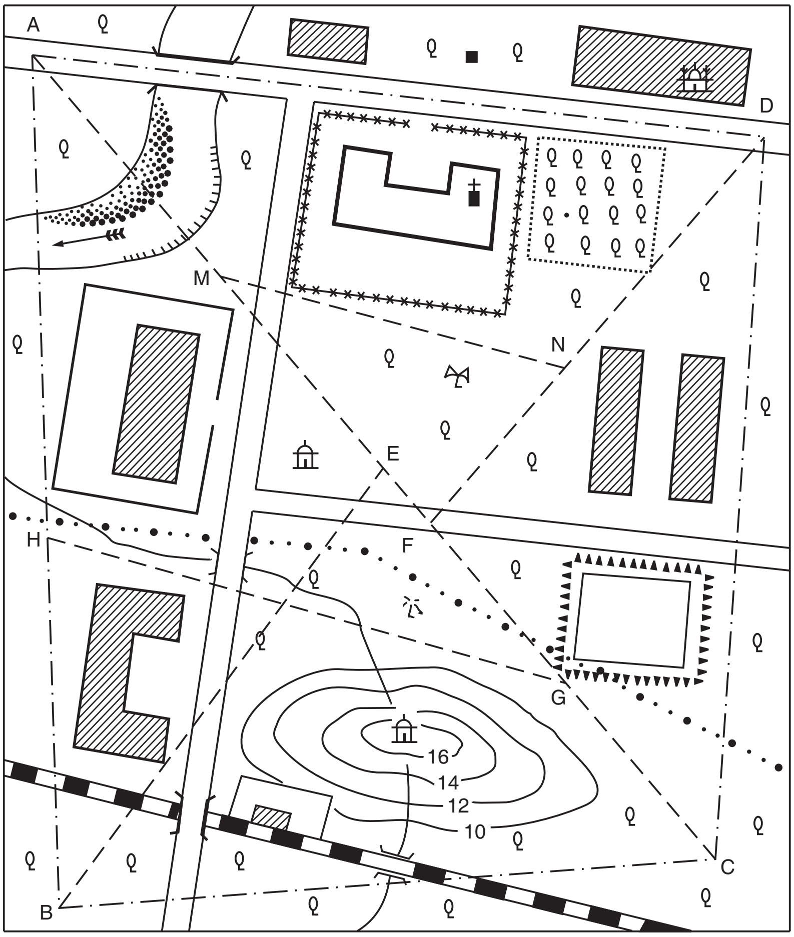


Fig. 3.2. Layout of a Chain Survey

(1) **Main Survey Station.** The point where two sides of a main triangle meet is called, *a main survey station*. Main survey station is a point of importance at the beginning and at the end of a chain line.

(2) **Subsidiary Survey Station** (or tie station). The stations which are selected on the main survey lines for running auxiliary lines, are called *subsidiary stations*.

(3) **Main Survey Lines.** The chain lines joining the main survey stations, are known as *main survey lines*.

(4) **Auxiliary, Subsidiary or Tie Lines.** The chain lines joining the subsidiary survey stations are known as *auxiliary, subsidiary* or more commonly as *tie lines*. These are provided to locate the interior details which are far away from the main lines.

(5) **Base Line.** The longest of the main survey lines, is called a *base line*. Various survey stations are plotted with reference to the base line. It lies roughly in the centre of the area along the longer side.

(6) **Check Lines.** The lines which are run in the field to check the accuracy of the field work are called *check lines.* If the measured length of a check line agrees with the length scaled off from the plan, the survey is accurate.

In Fig. 3.2, the following lines are shown:

Base line; *AC*

Main surveying lines: *AB, BC, CD* and *AD*

Subsidiary or tie lines: *BE* and *FD*

Check lines: *HC* and *MN*

Main surveying stations: *A*, *B*, *C* and *D*

Subsidiary stations: *E* and *F*.

3.7 SELECTION OF STATIONS

The following points should be kept in mind while selecting survey stations:

(1) Main survey stations at the ends of chain lines, should be intervisible.

(2) Survey lines should be minimum possible.

(3) The main principle of surveying *viz., working from the whole to the part and not from the part to the whole*, should be strictly observed.

(4) Survey stations should form well conditioned triangles.

(5) Every triangle should be provided with a check line.

(6) Tie lines should be provided to avoid too long offsets.

(7) Obstacles to ranging and chaining, if any, should be avoided.

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

(9) To avoid trespassing, main survey lines should remain within the boundaries of the property to be surveyed.

(10) Chain lines should lie preferably over level ground.

(11) Chain lines should be laid on one side of the road to avoid interruption of chaining by passing traffic.

3.8 SELECTION AND MEASUREMENT OF A BASE LINE

In chain surveying, the base line is the most important line as it fixes the directions of all other chain lines. The following points are kept in view during its selection and measurement:

(1) It should be laid preferably on a level ground.

(2) It should be run through the centre of the length of the area.

(3) It should be correctly measured horizontally.

(4) It should be measured twice or thrice and the mean distance accepted, as its correct length.

(5) Great care should be taken, to ensure straightness of the base lines.

(6) If convenient, two base lines perpendicularly bisecting each other should be laid out.

3.9 OFFSETS

In chain surveying, the positions of details i.e. boundaries, culverts, roads, stream bends, etc. are located with respect to the chain line by measuring their distances right or left of the chain line. Such lateral measurements are called *offsets*. There are two types of offsets *i.e.*,

(*i*) Perpendicular offsets. (*ii*) Oblique offsets

1. Perpendicular offsets (Fig. 3.3 *a*). When the lateral measurements for fixing detail points, are made perpendicular to a chain line, the offsets are known as *perpendicular or right angled offsets*. *EN* is a perpendicular offset on the right side of the chain line *AB*.

2. Oblique offsets. (Fig. 3.3 *b*). When the lateral measurements for fixing detail points, are made at any angle to the chain line, the offsets are known as *oblique offsets*. *CM* and *DM* are oblique offsets, on the right side of the chain line *AB*.

Depending upon the length, offsets are further classified as short offset and long offset.

The offsets having their length less than 15 m, are called *short offsets*. The offsets having their length more than 15 m, are called *long offsets*.

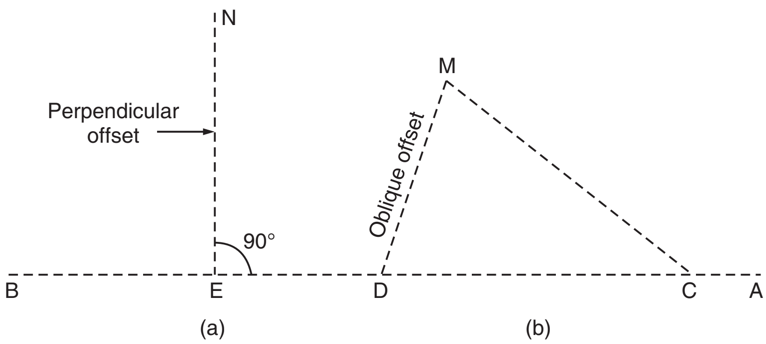


Fig. 3.3. Perpendicular and oblique offsets.

3.10 MEASUREMENT OF PERPENDICULAR OFFSETS

The offsets are generally measured either with a metallic or steel tape, depending upon the accuracy aimed at in surveying.

For every offset, the following two measurements, are involved:

1. **The distance along the chain line**. The total distance of the foot of an offset from the starting station of a chain line, is called *chainage* (Fig. 3.4).

2. **The length of the offset.** The distance of the detail point from the foot of its offset is called *length of the offset*.

While surveying in the direction from *A* to *B*, *AB* is a chain line and *PC* is a perpendicular offset. The distance *AC* from *A* is known as the *chainage* and distance *PC* from *C* is the length of the offset.

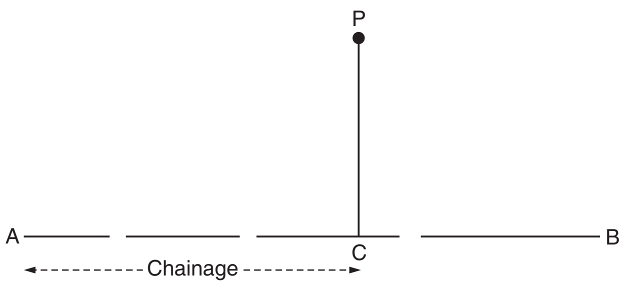


Fig. 3.4. Measurement of a perpendicular offset.

For short offsets, perpendicular directions are generally judged by eye only. When offsets are long and a better accuracy is required, the right angles are set out with either an optical square or a cross-staff.

Long offsets should be avoided to minimise error due to incorrect length of the tape or incorrect direction. Moreover, short offsets are measured more quickly and accurately as compared to long offsets.

3.11 LOCATING CORNERS, POINTS OF INTERSECTION AND BUILDINGS

For locating corners, point of intersections and buildings, proceed as under.

1. The corners of a field are located with perpendicular offsets and their accuracy is checked by providing check ties. Similarly, every point of intersection must also be provided with a set of check ties in addition to the perpendicular offset (Fig. 3.5).

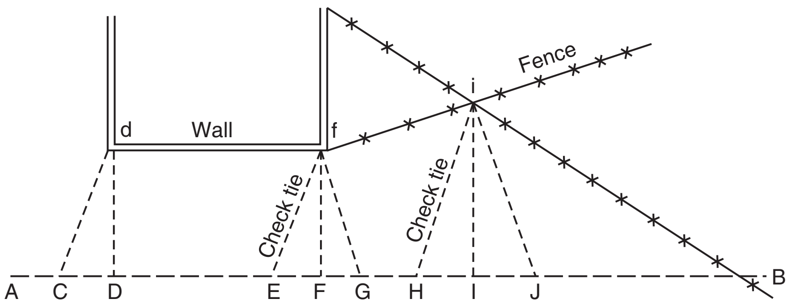


Fig. 3.5. Offse and check ties.

2. For locating buildings, their corners must be accurately surveyed by taking normal offsets and check ties. In addition, measurements of the periphery of the building, may also be recorded. Layouts of the offsets and check ties for locating building corners, are illustrated in (Fig. 3.6).

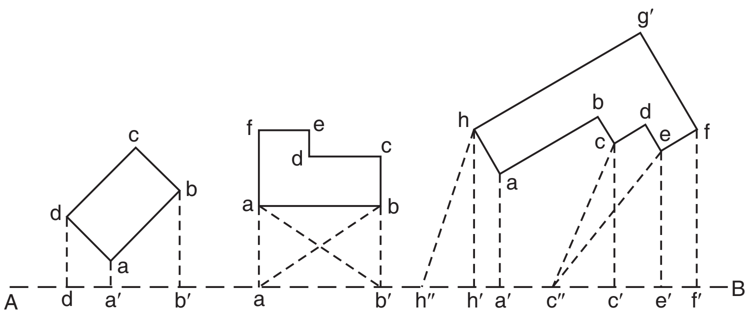


Fig. 3.6. Offsets to building corners.

3.12 FIELD BOOK

The book in which chainages, offset measurements and sketches of detail points are recorded, is generally called a *field book*. It is an oblong book of size about 20 cm × 12 cm and opens lengthwise. Two types of field books are in general use *i.e.*, single line field book and double line field book. In single line field books, a red line is ruled down the middle of each page which represents the chain line or survey line. In double-line field books, two blue lines about 1.5 cm to 2 cm apart are ruled down in the middle of each page to represent the chain line. The chainages are written on the line in single line field books and between the two lines in double-line field books. Offsets are written opposite their chainages to the right or left according to their positions whether on the right or on the left of the line. The single line field book is generally used for large scale survey with detailed dimension work. Double line field book is commonly used for ordinary work. The space on either side of the single line or the column is utilised for drawing sketches and symbols of the detail points located from the chain line.

3.13 BOOKING FIELD NOTES

In a field book, field notes are entered from the bottom of the page upward. At the beginning of a chain line, the following informations are recorded in the field book:

(1) The name or number of the chain line.

(2) The name or number of the survey station.

(3) The symbol Δ denoting the station mark.

(4) The direction of survey lines starting off from or ending at the station.

(5) The initial chainage which is generally zero, is enclosed in the symbol Δ.

All distances along the chain line *i.e.*, chainages are entered on the centre line or in the central column and offsets are written opposite them on the right or left of the column according to their ground positions with respect to the chain line. Close to the offsets, their sketches are drawn to guide the draughtsman to draw them correctly in office. When any linear detail such as a road, a foot path, a fence, a boundary line, etc. intersects the chain line, chainages of its points of intersection are entered in the column and the direction of the detail sketched. At the end of the chain line, chainage should be enclosed in the symbol Δ and the name or number of the station and chain line, should be nearly written.

Tie or subsidiary stations along a chain line should be indicated by a circle or an oval round their chainages. At the commencement of a tie line in the field book, position of the tie station should be described *e.g.* Tie station T_1 on *AB* at 360 m from A. Similarly at the end of the tie line, it should be described *e.g.*, Tie station T_2 on *BC* at 86.5 m from *B*. *A* specimen field book for chain survey, is shown on next page.

3.14 INSTRUCTIONS FOR BOOKING THE FIELD NOTES

The following points should be kept in mind while booking the chain survey measurements in a field book:

(1) Each chain line or a tie line, should be recorded on a separate page.

(2) The recorder should always face the direction of chaining while booking the field notes.

(3) All the measurements should be recorded as soon as these are taken and nothing should be left to memory.

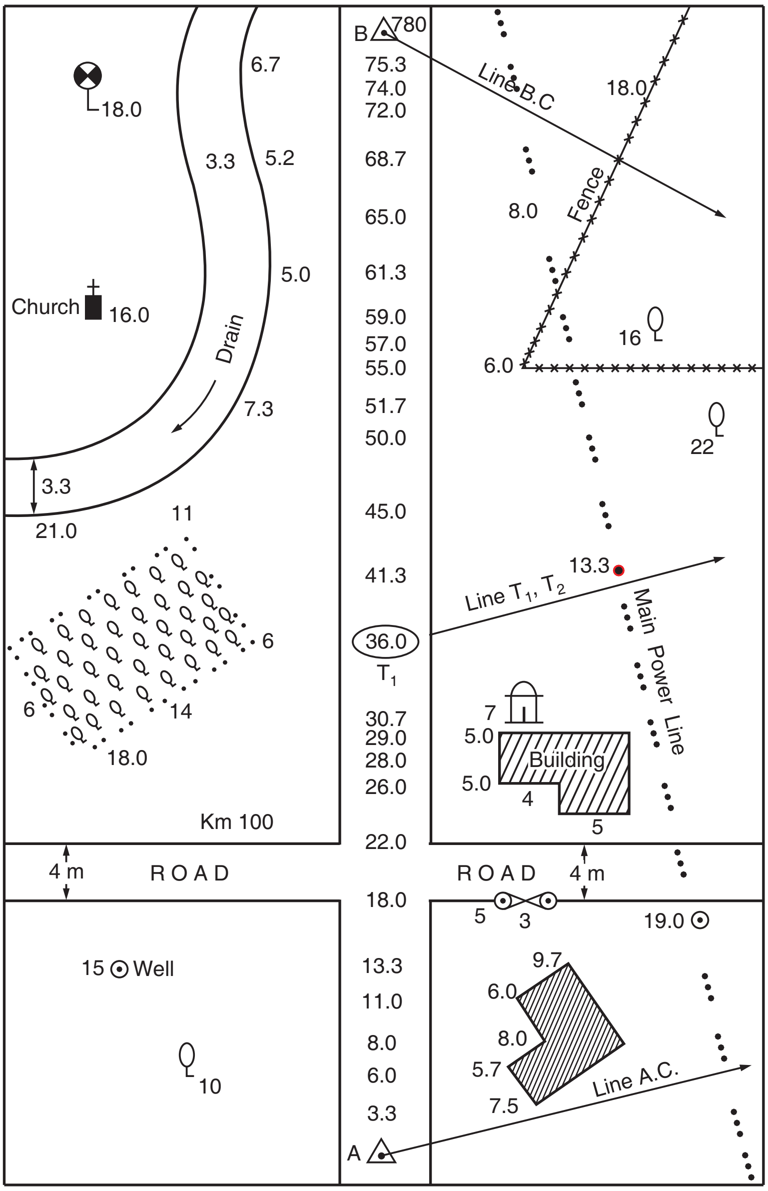
(4) The notes should be complete, neat and accurate with all information necessary for plotting the survey by a draughtsman in office.

(5) Numerals should be neatly and legibly written without any overwriting.

(6) Sketches of the details, should be neat and clear.

(7) A good quality of pencil should be used for recording the entries.

A field book specimen for chain survey



(8) The field book is an important document and should be kept clean. Wrong entries should be scored out and correct ones written over the wrong ones. If an entire page is to be discarded, it should be

crossed, and marked “cancelled”. A reference to the page on which correct readings are recorded, is made on this page.

(9) The complete record of the chain survey should include:

(*i*) A general lay-out plan of the lines.

(*ii*) The details of the lines.

(*iii*) The date of the survey.

(*iv*) A page index of the lines.

(*v*) Names of the surveyor and his assistants.

3.15 FIELD WORK

Field work of chain surveying is carried out in the following steps:

(*i*) Reconnaissance.

(*ii*) Marking stations.

(*iii*) Running survey lines.

1. **Reconnaissance.** It is always useful and often absolutely necessary for the surveyor to make a prelimtinary inspection ofthe area before commencing his actual detail survey, for the purpose of fixing the survey stations and forming a general plan for the net work of the chain lines. Such preliminary inspection of the area, is generally known as *reconnaissance* or *reconnoitre*. On arriving at the survey site, the surveyor should, therefore, walk over the entire area to examine the ground to decide upon the best layout of the chain lines. During reconnaissance, the surveyor should ensure that the survey stations are intervisible, there is no difficulty in chaining and the angles of the chain triangle are not acute. A fairly accurate key plan should be prepared to show the boundaries, main features, positions of chainlines and stations which should be lettered and numbered. Directions in which chain lines are to be measured, are marked with arrow heads.

2. **Marking stations.** On completion of successful reconnaissance, all survey stations should be marked in such a way that these are easily located during the progress of survey even after sometime, if necessary to revise a faulty work. In soft ground, wooden pegs are driven, leaving a small portion projected above the ground. In case of roads or hard surface ground, nails of spikes may be driven flush with the pavement. Sometimes, in hard ground, a portion may be dug and filled with cement mortar, etc. For marking a permanent station, a stone of any standard shape may be embedded in the ground and fixed with cement mortar.

A brief description of each survey station is given and reference sketches are drawn in the field book. The sketch, showing at least measurements to three permanent and definite points such as gates, pillars, light posts, corners of buildings, etc., is known as a *reference* or *location sketch*. (Fig. 3.7).

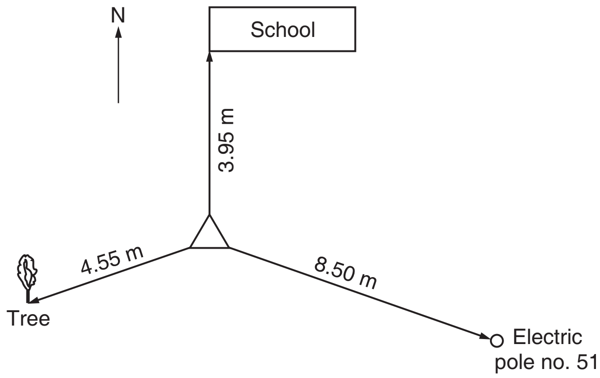


Fig. 3.7. Reference sketch.

Reference sketches are very useful to locate the positions of stations, in case their marks are displaced or points lost or if required at a later date. Measurement to the reference point must be taken correct to 5 mm. Reference sketches should be drawn by facing the north direction and a north line should be drawn. Measurements to references are written along arrows drawn to indicate their positions, which are technically known as '*ties*'.

3. **Running survey lines.** On completion of preliminary work, survey lines are run as detailed below:

(*i*) Ranging is done between the end stations of the base line.

(*ii*) A chain is stretched in true alignment keeping one end of the. chain at the starting station.

(*iii*) An arrow is fixed at the other end of the chain while it is kept laying on the ground.

(*iv*) The surveyor walks along the chain line and takes offsets to adjacent detail points on the right and left sides of the chain line.

(*v*) Chainages and offsets are recorded in the field book.

(*vi*) Process of chaining and offsetting, is repeated until the end of the base line is reached.

(*vii*) Other lines similarly are completed.

3.16 INSTRUMENTS FOR SETTING-OUT RIGHT ANGLES

For setting out right angles in chain surveying, the following instruments are generally used:

(*i*) Cross staffs (*ii*) Optical square

(*iii*) Prism squares.

1. **Cross staffs.** Cross staffs are generally of two types, *i.e.*,

1. Open cross staffs
2. French cross staffs

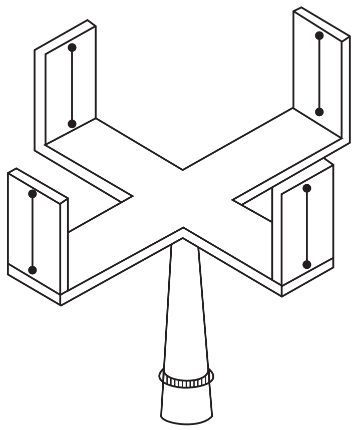


Fig. 3.8 An open cross staff.

(1) **Open cross staff** (Fig. 3.8). It is the simplest type of cross staff which is commonly used. It consists of a head and a leg. The head is a wooden block octagonal or round, about 15 cm side or 20 cm diameter and 4 cm deep. The wooden block is provided with two cuts 1 cm deep at right angles to each other, establishing two lines of sight. A better form of the open cross staff consists of four metal arms with vertical slits for slighting through at right angles to each other. The head block is fixed to the top of an iron shoed wooden staff (about 2.5 cm in diameter and 1.2 m to 1.5 m long) which is driven into the ground.

Uses of an open cross staff. An open cross staff may be used for the following purposes:

A. Finding the foot of a perpendicular offset.

Following steps are involved:

(*i*) The cross staff is held vertically on the chain line where the perpendicular from an object is expected to meet.

(*ii*) Turn the cross staff until one pair of opposite slits is directed to a ranging rod fixed at the forward end of the chain line.

(*iii*) Look through the other pair of slits and see if the point to which the offset is taken, is bisected.

(*iv*) If not, the cross staff is moved forward or backward on the chain line until the line of sight through the pair of slits at right angles to the chain line, bisects the desired point.

(*v*) Care should be taken to hold the cross staff vertically while viewing through the slits.

(2) **French cross staff** (Fig. 3.9). It consists of an octagonal brass tube with slits on all the eight faces. It has alternate vertical sight slits and on opposite vertical faces windows are provided. Vertical fine wires are stretched on each of the four sides. These are used for setting out right angles. Vertical sight slits on the centres of opposite slits, make angles of 45° with each other. With this arrangement, it is sometimes possible to set out 45° angles also.

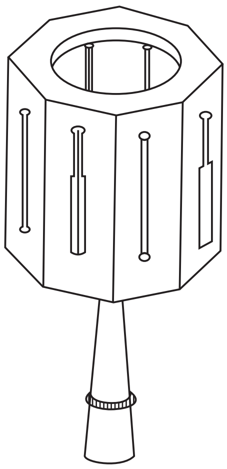


Fig. 3.9 A French cross staff.

The base of French cross staff is fitted on the pointed staff with a socket. It is inferior to the open cross staff because the sights are too close *i.e.*, 8 cm apart.

2. **Optical squares.** There are two types of optical squares *i.e.*

(*i*) Optical Squares (*ii*) Indian Optical Squares.

(1) **Optical Square** (Fig. 3.10). It consists of a circular metal box about 5 cm in diameter and 1.25 cm deep. The periphery is formed of two cylinders, one capable of sliding over the other so that the eye and object openings can be closed to protect the mirrors from dust. Two plain mirrors A and B are placed inclined at an angle of 45°. Upper half depth of mirror B, known as *horizon glass* is a plain glass and lower half is silvered. Mirror *A* known as *Index glass* is completely silvered. To an eye placed at *E*, the ranging rod placed at *Q* is visible through the transparent half of horizon mirror *B*, and at the same time the

image of the ranging rod placed at *P* at right angles to the line *EQ*, is seen after reflection at *B* in its silvered part.

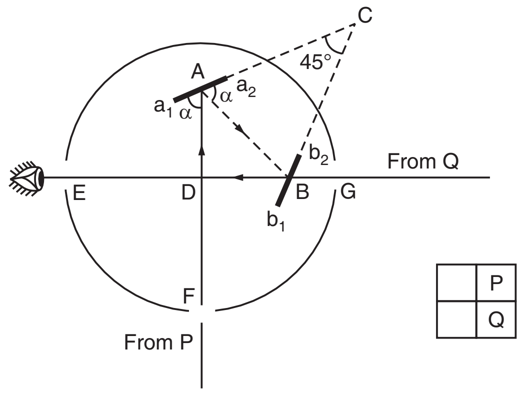


Fig. 3. 10

Three openings are cut alike in the rims of box and cover:

(*i*) a pin hole for the eye (or *sight hole*),

(*ii*) a small rectangular slot for *horizon sight*, diametrically opposite to the sight hole, and

(*iii*) a large rectangular slot for object sight, at right angles to the line joining the pin hole and horizon sight hole.

(2) **Indian Optical Square** (Fig. 3.11). It is a brass wedge shaped hollow box of 5 cm sides and 3 cm deep with a handle 7.2 cm long. Two mirrors m_1 and m_2 are fixed inclined at an angle of 45° to the inner sides of the box Two rectangular openings *a b* and *c d*, are provided in the sides above these mirrors for sighting. *MNOP* is the open face of the square which is kept towards the object to which offsets are to be taken.

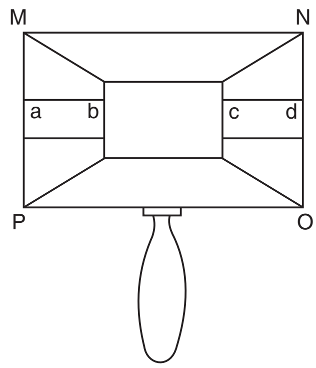


Fig. 3.11 An Indian optical square.

Use of an Indian optical square (Fig. 3.12). An optical square may be used for the following purposes:

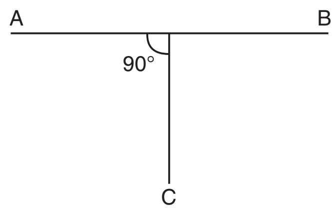


Fig. 3.12

A. Taking offsets to objects from the Chain line. Following steps are involved:

(*i*) Hold the instrument in a hand quite erect.

(*ii*) Stand on the chain line *AB* and turn its open face *MNQP* towards the object *C*.

(*iii*) Sight the ranging rod at the forward station B looking through the opening in the direction of a c or d b according as the object C is on the right or on the left.

(iv) Walk along the chain line AB forward or backward until the image of the object C, appears exactly in coincidence with the ranging rod at B.

(*v*) The plumb line suspended from the handle gives the required point on the chain line.

3.17 CALCULATION OF THE AREA OF A CROSS STAFF SURVEY

The area of a right angled triangle is calculated from the formula *i.e., the area of a right angled triangle is equal to its base multiplied by half the perpendicular*.

The area of a trapezoid is calculated from the formula *i.e., the area of a trapezoid is equal to the base multiplied by half the sum of two perpendiculars*.

3.18 PLOTTING A CROSS STAFF SURVEY

On completion of field observations and measurements, the survey is plotted to a convenient scale as explained in the following solved examples:

Example 3.1. *Plot the following details of a field and calculate its area in hectares, all measurements being in metres.*

	U	
T 80	*Δ100*	
	74	
	54	*42 S*
R 50	*26*	
	14	*30 Q*
	Δ	
	P	

Solution. (Fig. 3.13).

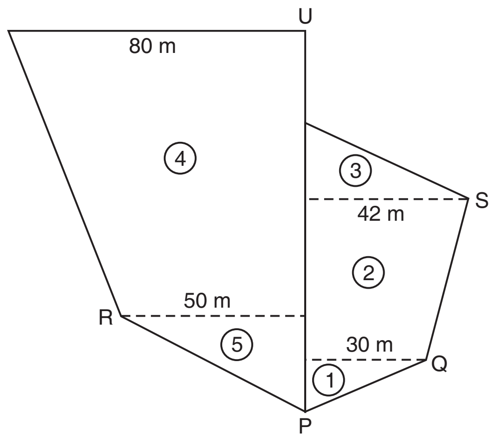


Fig. 3.13

Area of figure (1) $= \frac{1}{2}$ (14 × 30) = 210 sq m

Area of figure (2) $= \frac{(54-14)(30+42)}{2}$ = 1440 sq m

Area of figure (3) $= \frac{1}{2}$ (20 × 42) = 420 sq m

Area of figure (4) $= \frac{1}{2}$ [(100 – 26) (50+ 80)] = 4810 sq m

Area of figure (5) $= \frac{1}{2}$ (26 × 50) = 650 sq m

Total area = 7530 sq m

$\therefore$ The area of the field $= \dfrac{7530}{10{,}000}$ = **0.753 hectare. Ans.**

Example 3.2. *Draw a rough sketch of the following cross-staff survey of a field ABCDEFG and calculate its area in hectares. All distances are given in metres.*

	750	*D*
	650	*105 E*
C 90	*490*	
	300	*125 F*
B 80	*180*	
	100	*25 G*
	0	*A*

Solution. (Fig. 3.14).

Area of figure (1) $= \dfrac{1}{2} \times 100 \times 25$

$= 1{,}250$ sq. m

Area of figure (2) $= \dfrac{1}{2}$ (200 x 150)

$= 15{,}000$ sq m

Area of figure (3) $= \dfrac{1}{2}$ (350 × 230)

$= 40{,}250$ sq m

Area of figure (4) $= \dfrac{1}{2}$ (100 x 105)

$= 5{,}250$ sq m

Area of figure (5) $= \dfrac{1}{2}$ (260 x 90)

$= 11{,}700$ sq m

Area of figure (6) $= \dfrac{1}{2}$(310 x 170)

$= 26{,}350$ sq m

Area of figure (7) $= \dfrac{1}{2}$ (180 x 80)

$= 7{,}200$ sq m

$= 1{,}07{,}000$ sq m.

Total Area = **10.7 hectares. Ans.**

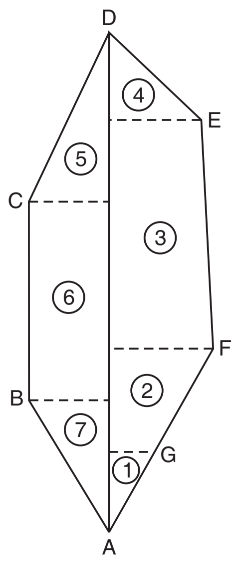
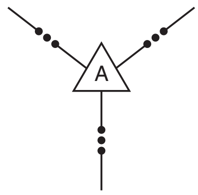
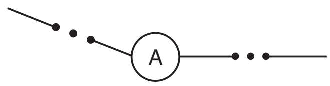
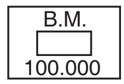
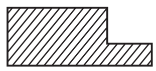
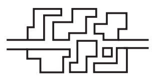
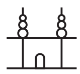
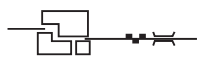
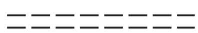
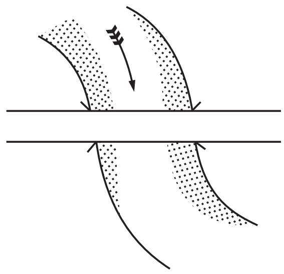
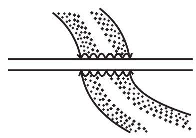
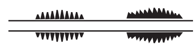
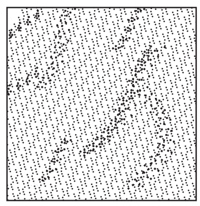
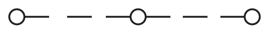
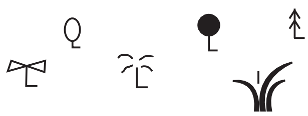
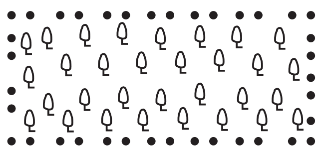


Fig. 3.14

Some of the conventional signs in common use, are shown here under:

Sl. No.	*Name of detail*	*Conventional Sign*
1.	Chain line	
2.	Triangulation station	
3.	Traverse station	
4.	Bench mark	
5.	Building	
6.	Township	
7.	Temple	
8.	Mosque	
9.	Lined well	
10.	Foot path	
11.	Cart track with a bridge	
12.	Unmetalled road	

(*Contd.*)

13.	Metalled road with a bridge	
14.	Metalled road with a boat bridge	
15.	Road in cuttings	
16.	Road on embankment	
17.	Sand dunes	
18.	Pipe line	
19.	Telephone/telegraph line	
20.	Main power line	
21.	Power line	
22.	Trees	
23.	Orchard	

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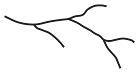
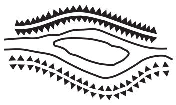
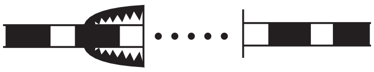
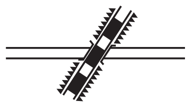
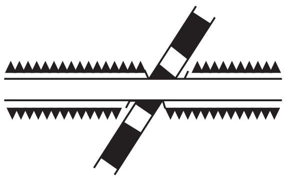
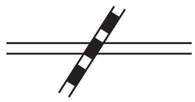
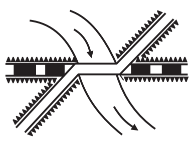
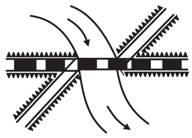
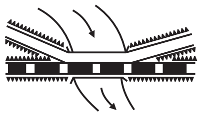
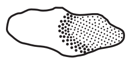
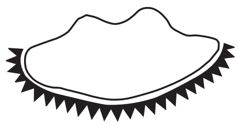
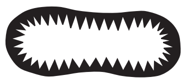
24.	Swamp or Marsh	
25.	Steam single line	
26.	River double line with embankments	
27.	Railway, single line with station	
28.	Railway, other gauge	
29.	Railway bridge	
30.	Railway tunnel with or without cuttings	
31.	Railway over road	
32.	Road over railway	
33.	Level crossing	
34.	Bridge carrying railway below road	

(*Contd.*)

35.	Bridge carrying railway over road	
36.	Bridge carrying road and railway	
37.	Ropeway with terminus	
38.	Lake as surveyed	
39.	Lake as surveyed with embankment	
40.	Quarry	

3.19 PLOTTING A CHAIN SURVEY

On successful completion of field observations and measurements, plotting of a chain survey, is done in the office by draughtsman. Plotting includes preparation of plans and computation of their areas with the help of field notes. The plotting may be done in the following steps:

1. **Selection of scale.** Scale of a map is very important. It is to be decided before field work is commenced. The choice of the scale is governed by the size of sheet, extent of area and purpose of the survey.

2. **Plotting of Survey Lines.** The longest line generally known as *base line* is first drawn in the appropriate position on the sheet. The positions of intermediate survey stations are carefully scaled and marked with fine pencil dots. Other chain lines forming triangles with base line are plotted by describing short intersecting arcs with the lengths of their sides as radii. The accuracyof plotting of these triangles can be checked by fitting in the check lines. The whole framework must be plotted and checked before plotting of the details of chain lines, is commenced.

3. **Plotting of Details.** Plotting of offsets may be done in two ways:

(*i*) In the first method chainages of the offsets, are marked along the survey line and the lengths are plotted at right angles. In plotting short offsets, perpendicularity of offsets may be estimated by eye. But for long offsets, pencil lines are drawn perpendicular to survey lines by set squares.

(*ii*) In the second method, a short scale called an offset scale is used. An ordinary scale is laid parallel to the chain line such that zero of the offset scale coincides with the chain line. The chainages can be read on an ordinary scale. The lengths of offsets are read on offset scale. The offset scale is slided along the ordinary scale which is held by weights. The various offset lengths are pricked off rapidly. If the offset scale is graduated such that its zero division is at the centre of its length, the ordinary scale is laid down parallel to the chain line and at a distance equal to half the length of the offset scale so that zero of the offset scale coincides with the chain line. The offset scale may then be slided to various chainages. The offsets are marked on both the sides of the chain line.

The plotted points are joined by straight or curved lines as the case may be. It should be remembered that changes in direction of the boundary occur only at the ends of offsets.

3.20 COMMON MISTAKES IN PLOTTING DETAILS

The following mistakes are commonly committed in plotting details:

1. Plotting offsets from wrong points
2. Plotting offsets on wrong side of survey line
3. Omitting offsets
4. Scaling chainages from wrong end of the chain line
5. Joining wrong detail points.

EXERCISE 3

1. Explain the principle used in Chain surveying. What are the limitations of chain surveying? Explain briefly the situations where it can be suitably employed.
2. Explain the principle of Chain Survey. When does it become inconvenient?

3. Which type of area is best suited for Chain Survey? Give reasons.

4. What are the instruments used in Chain Surveying? How is the Chain Survey executed in the field?

5. What is Reconnaissance? State its importance in Chain Surveying.

6. Draw a neat sketch of an optical square and give in details its principle, construction and working.

7. Attempt the following:
(*a*) Advantages of working from the whole to the part.
(*b*) Continuing a chain line when it crosses a river at an oblique angle.
(*c*) What do you understand by well-conditioned triangles and why are they used?

8. Describe with the help of neat sketches:
(*i*) An optical square
(*ii*) An Engineer's chain
(*iii*) A Cross-staff.

9. Describe in detail how you would range and chain a line between two points which are not intervisible because of an intervening raised ground.

10. (*a*) Describe the principle, construction and working of an optical square.
(*b*) What are the different kinds of ranging across a high ground?

11. (*a*) Define surveying. Explain the principle of Chain Surveying.
(*b*) Give a list of sources of error in chain survey and say which of these are cumulative and which are compensating.

12. (*i*) Enumerate the instruments required for making a chain survey.
(*ii*) Describe a field book and show how the field measurements are entered in it.

13. Define the following terms:
(*i*) Swing offset (*ii*) Oblique offset (*iii*) Random line (*iv*) Reference sketch (*v*) Key plan (*vi*) Base line (*vii*) Check line (*viii*) Tie line (*ix*) Well-conditioned triangle (*x*) Tie station.

14. (*a*) What are the conventional Signs?
(*b*) Give the conventional signs for the following:
(*i*) Metalled road in cutting, a culvert, Bridge, road on embankments.
(*ii*) Railway double line, Railway bridge, a single line railway track.
(*iii*) Compound wall, Hedge, compound wall with gate, Grassy land, Marshy land, lake.
(*iv*) Canal, Pond Canal with lock.

15. What are offsets? How are they taken and recorded? Why is it desirable to take short offsets.

CHAPTER 4

Compass Surveying

4.1 INTRODUCTION

The branch of surveying in which directions of survey lines are determined by a compass and their lengths by chaining or taping directly on the surface of the earth, is called *compass surveying*. Method of chain surveying is preferred to, if the area to be surveyed is small in extent and higher accuracy is not aimed at. On the other hand, if the area is comparatively large, with undulations, compass surveying is usually adopted. Before recommending a compass survey for any area, it must be ascertained that area is not magnetically disturbed.

4.2 SURVEYING COMPASSES

Surveying compasses are of the following two types:

1. Prismatic Compass   2. Surveyor's Compass.

1. Prismatic Compass. It is most suitable type of surveying compass which consists of a circular box about 100 mm in diameter. Prismatic compass can be used as a hand instrument or on a tripod. Prismatic compass can be accurately centered over ground station marks.

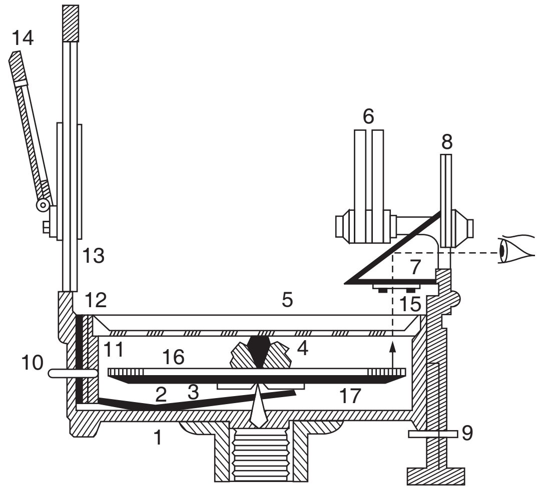


Fig. 4.1 Prismatic Compass.

1. Compass box	2. Lifting lever	3. Needle
4. Agate cap	5. Glass cover	6. Sun glass
7. Prism	8. Eye vane	9. Focussing stud
10. Brake pin	11. Spring brake	12. Lifting pin
13. Object vane	14. Adjustable mirror	15. Prism cap
16. Graduated ring	17. Pivot.	

The main parts of a prismatic compass, are shown in Fig. 4.1. A broad magnetic needle attached to an aluminium circular ring is graduated to degrees and half degrees. The graduations run clockwise, having zero at South, 90° at West, 180° at North and 270° at East. The graduations are written inverted. When the needle is balanced on the pivot, it orients itself in the magnetic meridian. The North and South ends of the aluminium ring lie in N-S direction (Fig. 4.2).

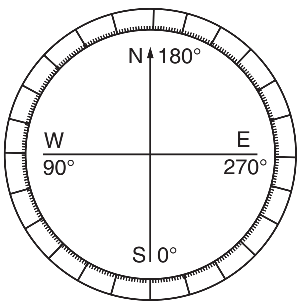


Fig. 4.2 A graduated ring.

The object vane carries a vertical hair attached to a suitable frame. The object vane is sometimes provided with a hinged mirror which can be moved upward and downward by a screw. The mirror can be inclined at any desired angle so that objects too high or too low, can be sighted.

Sight vane or eye slit consists of vertical slit cut into the upper assembly of a prism. The object vane and sight vane, are hinged to the box diagonally opposite at the top.

When an object is sighted out of magnetic meridian, sight vane rotates with respect to N-S ends of ring through the angle which the line makes with the magnetic meridian. When the line of sight falls along the magnetic meridian, the South end of the ring comes vertically below the horizontal face of the sighting prism and the 0° or 360° reading is seen through the prism. The inverted figures of the graduations are reflected by the hypotenusal side of the eye prism. This arrangement makes the eye to read figures erect and magnified manifold. A glass lid is fitted over the box to protect the needle from dust.

When not in use, the object vane may be folded on the glass lid. It presses against a bent lever which lifts the needle off the pivot and holds it against the glass lid. Oscillations of the needle can be dampen to facilitate the reading of the graduated ring by a braking pin placed at the base of the object vane. When bright objects are sighted, a dark glass is fitted in the eye vane to diminish the intensity of light.

2. Surveyor's Compass (Fig. 4.3). Surveyor's compass is similar in construction to the prismatic compass with a few modifications stated below:

(1) The graduated ring is directly attached to the circular box and not with the magnetic needle.

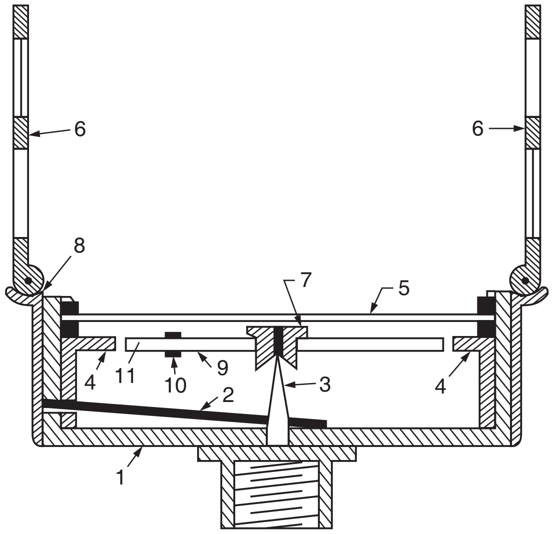
(2) The edge bar magnetic needle floats freely over the pivot.

(3) The eye vane consists of a simple metal vane with a fine sight hole.

(4) No mirror is attached to the object vane for sighting object at higher elevation or lower depression.

(5) Readings are taken against the north end of the needle.

(6) The ring is graduated in quadrental system having 0° at North and South ends; 90° at East and West ends.



1. Box
2. Lifting lever
3. Pivot
4. Circular graduated arc
5. Glass top
6. Sight vanes
7. Jewel bearing
8. Lifting pin
9. Magnetic needle
10. Rider
11. Metal pin

Fig. 4.3 Surveyor's Compass.

4.3 ADJUSTMENT OF A SURVEYING COMPASS

The adjustments of a surveying compass are:

(*a*) **Temporary Adjustments.** The adjustments which are required to be made at every set up of the instrument, are known as t*emporary or Station Adjustments*. These include:

(*i*) Centering, (*ii*) Levelling.

(*iii*) Focussing the prism (only in the case of prismatic compass).

(*b*) **Permanent Adjustments.** The adjustments which are made only if the fundamental relations between the various parts of a compass, are disturbed due to careless handling or otherwise, are called *permanent adjustments*.

1. **Temporary Adjustments.** Temporary adjustments of a compass, are made at every station, as discussed below:

(1) **Centering.** The process of centering the instrument, *i.e.* making the pivot exactly vertical over the ground station mark, is called *centering*. The compass is fixed on the top of a tripod and by adjusting the legs of the tripod, centering is achieved. A plumb bob may be hung

from the centre of the circular box, to check the compass centering. If no plumb is provided, the centering may be judged by dropping a small pebble freely from the centre of the bottom of the circular box. If the compass is centered perfectly, is the pebble falls exactly over the ground station mark.

(2) **Levelling.** The process of holding the compass in such a way that its graduated ring swings freely, is called *levelling*. The levelling is done by eye judgement. Generally the compass is provided with a ball and socket arrangement attached to the tripod for achieving quick levelling of the instrument. In surveyor's compass two plate levels at right angles to each other, are sometimes provided. The ball and socket arrangement is adjusted till the two bubbles remain central in both the plate levels.

(3) **Focussing the prism.** (Only applicable to prismatic compass). The process of moving up or down the prism for obtaining the figures and graduations sharp and clear, is called *focussing the prism*.

2. **Permanent Adjustments.** The following fundamental relationships between different parts of a compass, are established by making permanent adjustments:

(1) When plate bubbles, if provided, are at the centres of their run, the vertical axis of the compass should be truly vertical.

(2) When the instrument is perfectly levelled, sight vanes should be vertical.

(3) The ends of the needle and the centre of the pivot should lie in the same vertical plane.

(4) The centre of the pivot should coincide with the geometrical centre of the graduated ring.

4.4 COMPARISON BETWEEN A SURVEYOR'S COMPASS AND PRISMATIC COMPASS

S. No.	*Item*	*Surveyor's Compass*	*Prismatic Compass*
1.	Magnetic needle	(*i*) The needle is of edge bar type and also acts as an index.	(*i*) The needle is broad needle type but does not act as an index.
2.	Gradauated ring	(*i*) The graduated ring is attached to the box and not to the needle. This rotates along with the ring of sight.	(*i*) The graduated ring is attached with the needle. This does not rotate alongwith the line of sight.
		(*ii*) The graduations are in Q.B. system, having 0° at North and South and 90° at East and West. East and West are interchangeable.	(*i*) The graduations are in W.C.B. system having 00, at South, 90° at West, 180° at North and 270° at East end.
		(*iii*) The graduations are engraved erect.	(*iii*) The graduations are engraved inverted.
3.	Sighting vanes	(*i*) The object vane consist of a metal vane with a vertical hair.	(*i*) The object vane consists of a metal vane with a vertical hair.
		(*ii*) The eye vane consists of a small vane with fine slit.	(*ii*) The eye vane consists of a metal vane with a slit.
4.	Reading system	(*i*) The readings are taken directly by seeing through the top of the glass.	(*ii*) The readings are taken with the help of a prism provided at the eye slit.
		(*ii*) Sighting and reading cannot be done simultaneously from one position of the observer.	(*ii*) Sighting and reading can be done simultaneously from one position of the observer.
5.	Tripod	(*i*) The instrument cannot be used without a tripod.	(*i*) Tripod may or may not be provided. The instrument may be used even by holding in hand.

4.5 MERIDIAN AND BEARING

The direction of a survey line may be defined in two ways:

(*i*) Relatively to each other

(*ii*) Relatively to some reference direction.

In the first case, directions are expressed in terms of the angles between two consecutive line. In second case, these are expressed in terms of bearings.

Bearing. The horizontal angle between the reference meridian and the survey line measured in a clockwise direction, is called *bearing*.

Meridian. The fixed direction on the surface of the earth with reference to which, bearings of survey lines are expressed, is called a *meridian*.

The meridians of reference directions employed in surveying may be one of the following:

(*i*) True meridian

(*ii*) Magnetic meridian

(*iii*) Any arbitrary direction.

1. **True Meridian.** The lines of intersection of the earth surface by a plane containing North pole, South pole and the given place of observation, is called *true meridian or geographical meridian*. It represents true north-south direction at the place.

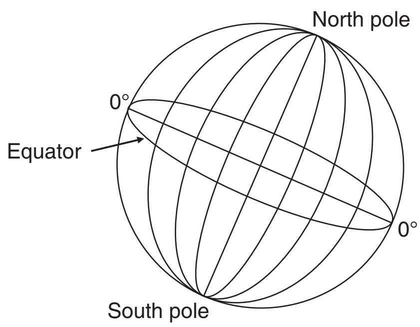


Fig. 4.4

Geographical meridians at different places are not parallel to each other. They converge to a point in northern and southern hemispheres as shown in Fig. 4.7.

Equatorial circumference of the earth surface is divided into 360º. The true meridian of Greenwich has been assumed Internationally as 0º. The meridians on its eastern sides are known as *East of Greenwich* and on the western side of zero meridian as *West of Greenwich*. 180° meridian on the globe is just opposite to Greenwich meridian.

Determination of the true meridian at any place may be made by making astronomical observations to heavenly bodies. *i.e.* the sun and stars.

The true meridian at any place is not variable. In engineering surveys it is very useful to save time in laying the lines during construction. Due to convergence of true meridians, it is only adopted for large scale surveys in the area, of limited extent.

It may be mentioned here that the maps prepared by the National Survey Department of any country, such a Survey of India are based on true meridians.

True Bearing (Fig. 4.5). The horizontal angle (θ) between the true meridian and a line measured in a clockwise direction, is called *true bearing of the line*.

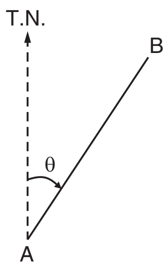


Fig. 4.5

2. **Magnetic Meridian.** The geometrical longitudinal axis of a freely suspended and properly balanced magnetic needle, unaffected by local attractive forces, defines the magnetic north-south line which is called the *magnetic meridian*. It does not coincide with the true meridian except in certain localities.

Magnetic Bearing. (Fig. 4.6). The horizontal angle (θ) which a line makes with the magnetic meridian, is called *magnetic bearing*. It is not constant at a point but varies with laps of time.

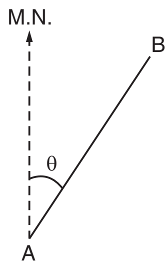


Fig. 4.6. Magnetic Bearing

3. Arbitrary Meridian. The convenient direction assumed as a meridian for measuring bearings of survey lines, is known as *arbitrary meridian*. Arbitrary meridians are generally assumed for survey of small plots of land. An arbitrary meridian has the merit of being invariable and its direction can be easily recovered at a future date, if and when required. If the angle between the true meridian and the assumed arbitrary meridian, is established later, the arbitrary meridian may be converted into true meridian by applying the desired correction.

4.6 DESIGNATION OF BEARINGS

Bearings of survey lines are designated in the following systems:

(*i*) The whole circle bearing system (W.C.B.)

(*ii*) The quadrantal bearing system (Q.B.)

1. **The Whole Circle Bearing System.** The whole circle bearing system is also sometimes known as *Azimuthal system*. In this system, the bearing of a line is measured from the true north or magnetic north in clockwise direction. The value of a bearing may vary from 0º to 360°, utilising the whole circle of graduations. Prismatic compass is graduated on whole circle bearing system. The system of measuring bearings from the north direction, is adopted in India and United Kingdom.

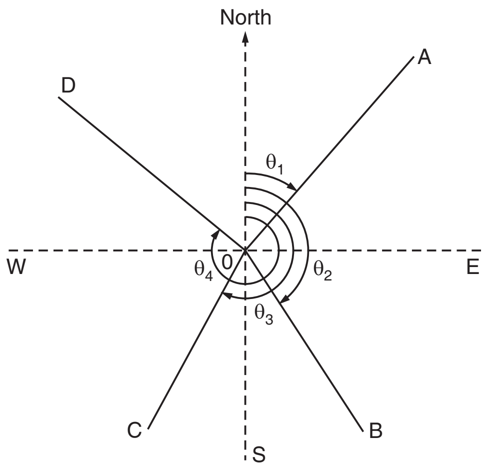


Fig. 4.7. Whole circle bearings of lines.

Referring to Fig. 4.7, W.C.B. of lines *OA*, *OB*, *OC*, and *OD* are θ_1, θ_2, θ_3, and θ_4 respectively.

Note. In some countries, W.C.B. of survey lines are reckoned from the South. These bearings differ by 180º in magnitude as compared to those expressed from the North.

2. **The quadrantal bearing system.** In quadrantal bearing system, bearings of survey lines are measured eastward or westward from North and South whichever is nearer. In this system, both north and south directions are used as reference meridians and bearings are reckoned either clockwise or anticlockwise, depending upon the position of the line. The quadrant in which a line lies is mentioned, to

specify the location of the line. Surveyor's compass is graduated in quadrantal bearing system.

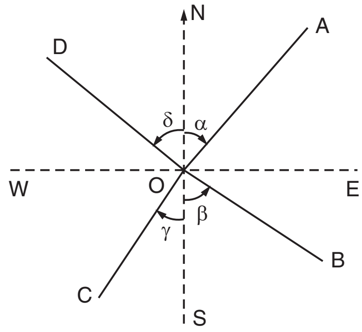


Fig. 4.8. Quadrantal bearings.

Bearings designated by quadrantal bearing system, are some time called *Reduced Bearings*.

Referring to Fig. 4.8, Q.B. of lines *OA*, *OB*, *OC* and *OD* are designated as $N\alpha^o E$, $S\beta^o E$, $S\gamma^o W$, $N\delta^o W$, respectively.

Thus, in quadrantal bearing system, reference meridian is prefixed and the direction of measurement whether eastward or westward, is affixed to the numerical value of the bearing. The numerical value of a quadrantal bearing, may vary from 0º to 90º.

4.7 CONVERSION OF BEARINGS FROM ONE SYSTEM TO THE OTHER

The conversion of bearings from one system to another may be easily done by drawing a diagram. In Fig. 4.9 suppose W.C.B. of any line is 175°. The *Q.B.* of the line AB = 180° – 175° = 5°. The line is in *SE* quadrant. It is also nearer to the South direction. Hence, the *Q.B.* of the line, is designated as *S* 5° *E*.

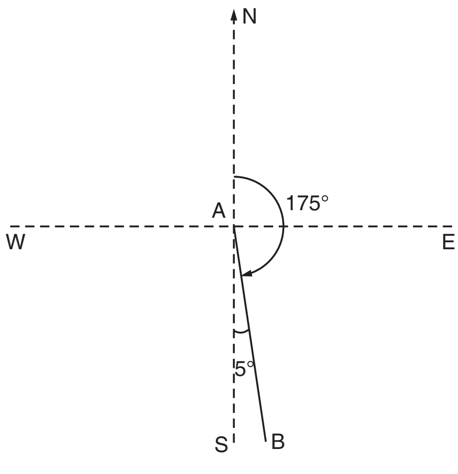


Fig. 4.9

For converting W.C. bearings into reduced bearings or Q.B., the rules are stated in Table 4.1.

Table 4.1. Conversion of W.C.B. into Q.B.

Case	*W.CB. between*	*Rule for Q.B.*	*Quadrant*
I	0° and 90°	W.C.B.	N.E
II	90° and 180°	180° – W.C.B.	S.E.
III	180° and 270°	W.C.B. – 180°	S.W.
IV	260° and 360°	360° – W.C.B.	N.W.

Note. When a line lies exactly either along North, South East or West, the W.C.B. of the line is converted in the quadrantal system as follows:

W.C.B. of a line = 0° then, Q.B. of the line is *N*

W.C.B. of a line = 90° then, Q.B. of the line is E 90°

W.C.B. of a line = 180° then, Q.B. of the line is *S*

W.C.B. of a line 270° then, Q.B. of the line is *W* 90°

The rules for the conversion of Q.B. into W.C.B. are stated in Table 4.2.

Table 4.2. Conversion of Q.B. into W.C.B.

Case	*RB.*	*Rule for W.C. B.*	*W.C.B. between*
I	N α° E	R.B.	0°and 90°
II	S β° E	180° – R.B.	90° and 180°
II	S γ° W	180° + R.B.	180° and 270°
IV	N δ° W	360° – R.B.	270° and 360°

4.8 FORE AND BACK BEARINGS

Every line may be defined by two bearings, one observed at either end of the line. Both the bearings expressed in W.C.B. system differ each other by 180°. The bearing of a line in the direction of the progress of survey, is called *Fore or Forward Bearing* (*F.B.*) while the bearing in the opposite direction of the progress of survey, is known as Reverse or *Back bearing* (*B.B.*).

In Fig. 4.10 the bearing of the line *AB* in the direction from *A* to *B* is a fore bearing (α) whereas the bearing of the line *AB* in the direction from *B* to *A*, is a back bearing (β).

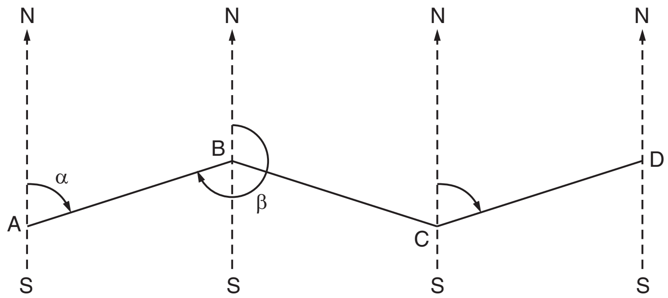


Fig. 4.10. Direction of survey is West to East.

4.9 RELATIONSHIP BETWEEN FORE BEARING AND BACK BEARING

(1) **W.C.B. system:** (Fig. 4.11)

Let the fore bearing of a line $AB = \alpha°$.

The back bearing of $BA = \beta$

or $\beta = 180° + \angle SBA = 180° + \angle BAN' = \alpha + 180°$

$\therefore$ Back bearing = Fore bearing + 180° ...(*i*)

Now, consider the bearing of BA as a fore bearing $= \beta$

Then, $\alpha = 180° - \angle S'AB = 180° - \angle ABN$

$= 180° - (360° - \beta) = \beta - 180°$

or Back bearing = Fore bearing – 180° ...(*ii*)

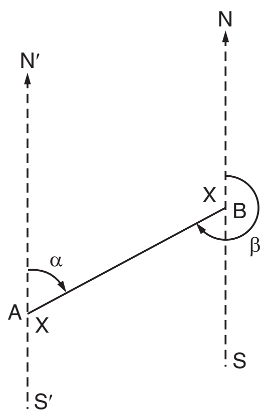


Fig. 4.11

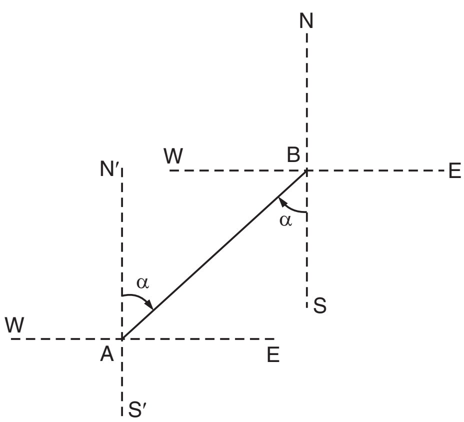


Fig. 4.12

Equations (*i*) and (*ii*) may be combined into one equation i e. Back bearing = Fore bearing ± 180°, using +ve sign if the fore bearing is less than 180° and – ve sign if the fore bearing is greater than 180°

(2) **Q.B. system:** (Fig. 4.12)

Let the fore bearing of a line $AB = N\alpha°E$

Back bearing of line $AB = \angle SBA = \angle BAN' = \alpha°$

or Back bearing of $AB = S\alpha°W$

To convert the fore bearing of a line into its back bearing in Q.B. system, replace N by *S*, *S* by *N*, *E* by *W* and *W* by *E*, without changing the numerical value of the bearing.

4.10 CALCULATION OF INCLUDED ANGLES FROM BEARINGS

Knowing the bearings of two adjacent lines, their included angles may be easily calculated as under:

I. Given W.C.B. of the lines (Fig. 4.13)

Let W.C.B. of the line $AB = \alpha°$

W.C.B. of the line $AC = \beta°$

∴ The included angle BAC

$= \angle NAC - \angle NAB$

$= \beta - \alpha$

= Bearing of *AC*— Bearing of *AB*

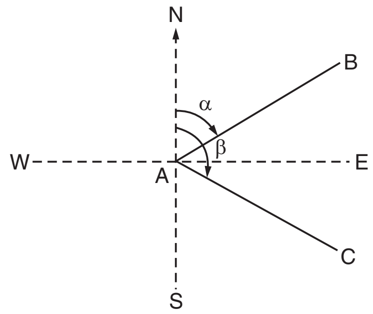


Fig. 4.13

Note. The difference of bearings of two adjacent lines is the included angle measured clockwise from the line whose bearing is less.

II. Given Q.B. of the lines (Fig. 4.14)

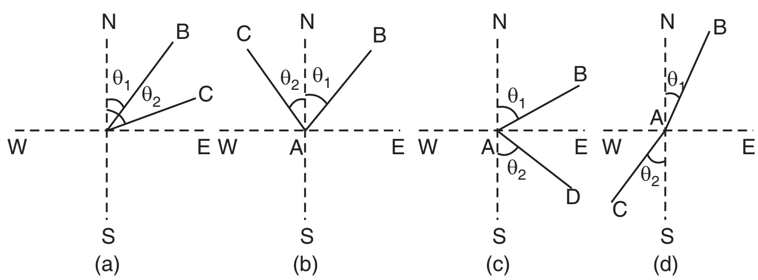


Fig. 4.14

A diagram may be drawn and bearings of the lines plotted in their respective quadrants. The included angle is calculated from one of the undermentioned formulae.

(1) If the bearings have been measured to the same side of the common meridian, the included angle $\alpha = \theta_2 - \theta_1$ i.e. the difference of the bearings. This is true for all the quadrants. [Fig. 4.14 (*a*)].

(2) If the bearings have been measured to the opposite side of the common meridian, the included angle $\alpha = \theta_1 + \theta_2$ i.e. the sum of the bearings. [Fig. 4.14 (*b*)].

(3) If the bearings have been measured to the same side of different meridians, the included angle $\alpha = 180° - (\theta_1 + \theta_2)$ *i.e.* the difference of 180° and the sum of the bearings. [Fig. 4.14 (*c*)].

(4) If the bearings have been measured to the opposite side of different meridians, the included angle $\alpha = 180° - (\theta_1 - \theta_2)$ *i.e.* the difference of 180° and the difference of the bearings. [Fig. 4.14 (*d*)].

4.11 CALCULATION OF BEARINGS FROM INCLUDED ANGLES

Knowing the bearing of a line and the included angles between the successive lines, the bearings of remaining lines, may be calculated as under:

Let the given observed bearing of the line *AB* be θ_1

$\alpha, \beta, \delta, \phi$ .... etc., the included angles measured clockwise between adjacent lines.

$\theta_2, \theta_3, \theta_4$ .... etc., the bearings of successive lines.

The bearing of *BC* $\theta_2 = \theta_1 + \alpha - 180°$ ...(*i*)

The bearing of *CD* $\theta_3 = \theta_2 + \beta - 180°$ ...(*ii*)

The bearing of *DE* $\theta_4 = \theta_3 + \gamma - 180°$ ...(*iii*)

The bearing of *EF* $\theta_5 = \theta_4 + \delta - 180°$ ...(*iv*)

The bearing of *FG* $\theta_6 = \theta_5 + \phi - 180°$ ...(*v*)

From Fig. 4.15, it is evident that each of $(\theta_1 + \alpha)$, $(\theta_2 + \beta)$ and $(\theta_3 + \gamma)$ is more than 180°; $(\theta_4 + \delta)$ is less than 180° and $(\theta_5 + \phi)$ is greater than 540°. Hence, in order to calculate the bearing of the next line, the following statements may be made:

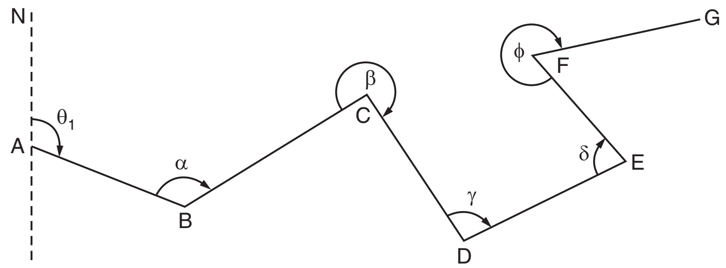


Fig. 4.15

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

more than 180°, deduct 180°,

more than 540° deduct 540°,

less than 180°, add 180°, to get the bearing of the next line".

Note: The following points may be noted.

(*i*) In a closed traverse run in anticlockwise direction, the observed included angles are interior angles.

(*ii*) In a closed traverse run in clockwise direction, the observed included angles, are exterior angles.

Example 4.1. *(a) Convert the following whole circle bearings into quadrantal bearings:*

(i) 12° 45′ *(ii) 160° 10′*

(iiii) 210° 30′ *(iv) 285° 50′.*

(b) Convert the following quadrantal bearings into whole circle bearings:

(i) N 30° 30′ E *(ii) S 70° 42′ E*

(iii) S 36° 35′ W *(iv) N 85° 10′ W.*

Solution. (*a*) Refer to Fig. 4.10 and Table 4.1.

(*i*) Reduced Bearing = W.C.B. = 12° 45′ = *N* 12° 45′ *E*

(*ii*) Reduced Bearing = 180° – W.C.B. = 180° – 160° 10′

= 19° 50′= S 19° 50′ E

(*iii*) Reduced Bearing = W.C.B. – 180° = 210° 30′ – 180°

Reduced Bearing = 30° 30′ = *S* 30° 30′ *W*

(*iv*) Reduced Bearing = 360° – W.C.B. = 360° – 285° 50′

= 74° 10′ = N74° 10′ *W*.

(*b*) Refer to Fig. 4.14 and Table 4.2

(*i*) W.C.B. R.B. = 30° 30′.

(*ii*) W.C.B. = 180° – R.B. = 180° – 70° 42′ = 109° 18′

(*iii*) W.C.B. = 180° + R.B. = 180° + 36° 35′ = 216° 35′

(*iv*) W.C.B. = 360° = R.B. = 360° – 85° 10′ = 274° 50′.

Example 4.2. *The whole-circle-bearing of a line is (i) 180° (ii) 270°. What will be its reduced bearing in each case?*

Solution. (Fig. 4.16)

(*i*) RB. = 180° – W.C.B.
= 180° – 180°
= 0°
= S. **Ans.**

(*ii*) R.B. – 360° – W.C.B.
= 360° – 270°
= 90°
= W 90°. **Ans.**

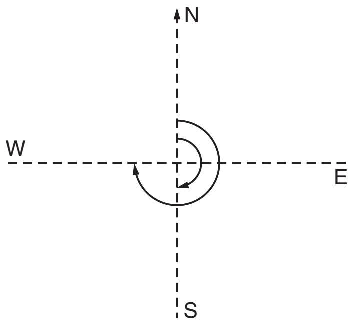


Fig. 4.16

Example 4.3. *The fore bearings of traverse sides are as follow: AB 85° 10′; BC 155° 30′; CD 265° 5¢ and 355° 30′. Find their back bearings.*

Solution.

Back bearing of a line	= fore bearing of the line ± 180°.	
∴ Back bearing of *AB*	= 85° 10′ + 180° = 265° 10′.	**Ans.**
Back bearing of *BC*	= 155° 30′ + 180° = 335° 30′.	**Ans.**
Back bearing of *CD*	= 265° 05′ – 180° = 85° 05′.	**Ans.**
and Back bearing of *DE*	= 355° 30′ – 180° = 175° 30′.	**Ans.**

Example 4.4. *The fore bearings of AB and BC are respectively N 30° 10′ E and S 20° 20′ W. Find their back bearings.*

Solution. The back bearing of a line in Q.B. system is obtained by replacing *N* by *S* and *E* by *W* and *vice versa*.

∴ Back bearing of *AB* = S 30° 10′ W. Ans.

and Back bearing of *BC* = N 20° 20′ E. Ans.

Example 4.5. *Find the included angles between lines AB and AC if their whole circle bearings are:*

(*i*) *AB*	*75° 30′*	*AC*	*108° 50′*
(*ii*) *AB*	*185° 50′*	*AC*	*269° 25′*
(*iii*) *AB*	*60° 10′*	*AC*	*245° 10′*
(*iv*) *AB*	*70° 20′*	*AC*	*285° 40′.*

Solution. (*i*) Bearing of *AB* = 75° 30′

Bearing of *AC* = 108° 50′

$\therefore$ Included angle *BAC* = Bearing of *AC* – Bearing of *AB*
= 108° 50′ – 75° 30′ = 33° 20′. **Ans.**

(*ii*) Bearing of *AB* = 185° 50′
Bearing of *AC* = 269° 25′

$\therefore$ Included angle *BAC* = 269° 25′ – 185° 50′ = 83° 35′ **Ans.**

(*iii*) Bearing of *AB* = 60° 10′
Bearing of *AC* = 245° 10′

Difference in bearings = 245° 10′ – 60° 10′ = 185° 0′

As it is more than 180°, deduct it from 360°

$\therefore$ Included angle *BAC* = 360° – 185° 0′ = 175° 0′. **Ans.**

(*iv*) Bearing of *AB* = 70° 20′
Bearing of *AC* = 285° 40′

$\therefore$ Difference in bearings = 285° 40′ 70° 20′ = 215° 20′

As it is more than 180°, deduct it from 360°

$\therefore$ Included angle *BAC* = 360° – 215° 20′ = 144° 40′. **Ans.**

Example 4.6. *Find the included angle between lines AB and AG, if their reduced bearings are:*

	AB	*AC*
(i)	*N 40° 10′ E*	*N 89° 45′ E*
(ii)	*N10° 50′ E*	*S 40° 40′ E*
(iii)	*S 35° 45′ W*	*N 45° 20′ E*
(iv)	*N 30° 25′ E*	*N 30° 25′ W*

Solution. Refer to the formulae stated in Table 4.1.

(*i*) (Fig. 4.17).

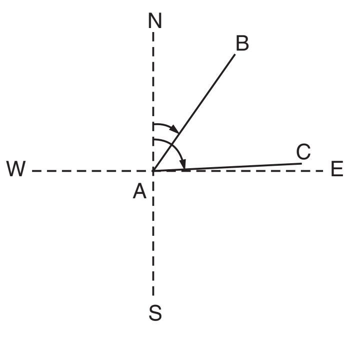


Fig. 4.17

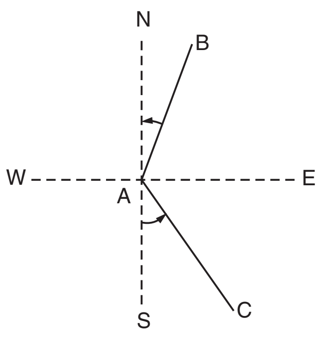


Fig. 4.18

Bearing of AB = N 40° 10′ E; Bearing of AC = N 89° 45′ E

∵ Bearings are measured on the same side of the north meridian, and both lie in NE quadrant.

∴ Included angle BAC = difference in the bearings

= 89° 45′ – 40° 10′ = 49° 35′ **Ans.**

(*ii*) (Fig. 4.18).

Bearing of AB = N 10° 50′ E; Bearing of AC = S 40° 40′ E

The bearings are measured on the same side of N-S meridian, and lie in adjacent quadrants.

∴ Included angle BAC

= 180° – sum of the bearings

= 180° – (10° 50′ + 40° 40′) = 128° 30′. **Ans.**

(*iii*) (Fig. 4.19)

Bearing of AB = S 35° 45′ W

Bearing of AC = N 45° 20′ E

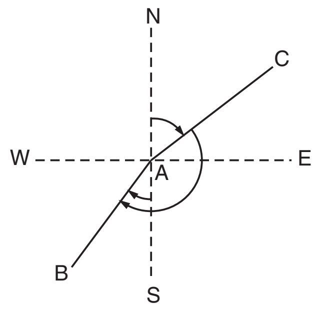


Fig. 4.19

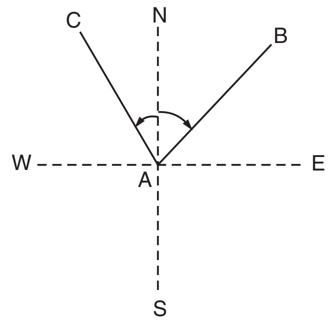


Fig. 4.20

The bearings are measured on opposite sides of the meridian and lie in opposite quadrants.

∴ Included angle CAB = 180° – (difference in bearings)

= 180° – (45° 20′ – 35° 45′)

= **170° 25′.** **Ans.**

(*iv*) (Fig. 4.20)

Bearing of AB = N 30° 25′ E

Bearing of AC' = N 30° 25′. W

The bearings are measured on the opposite side of common meridian and lie in adjacent quadrants.

∴ The included angle CAB = sum of the bearings

= 30° 25′ + 30° 25′ = 60° 50′ . **Ans.**

Example 4.7. *The bearings of the sides of a closed traverse ABCDEA are as follows:*

Side	*F.B.*	*RB.*
AB	*170° 15′*	*287° 15′*
BC	*22° 00′*	*202° 00′*
CD	*281° 30′*	*101° 30′*
DE	*181° 15′*	*1° 15′*
EA	*124° 45′*	*304° 45′*

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

Solution. (Fig. 4.21).

(*i*) The include angle *A* = The difference of bearings of *AB* and *AE*.

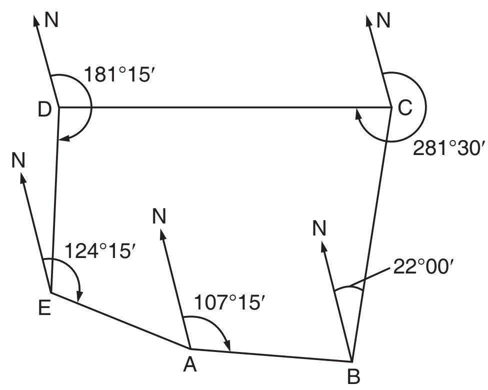


Fig. 4.21

As the bearing of AB is less than that of AE, add 360º.

∴ Included angle *A*

= (107° 15′ + 360°) – 304° 45′

= 162° 30′. **Ans.**

(*ii*) The included angle at *B*.

The difference of bearings of *BC* and *BA*

= (22° 00′ + 360°) – 287° 15′

∴ Included angle *B* = 94° 45′. **Ans.**

(*iii*) The included angle at *C*:

The difference of bearings of *CD* and

CB = 281° 30′– 202° 00′ = 79° 30′

∴ Included angle C = 79° 30′. **Ans.**

(iv) The included angle at D :

The difference of bearings of DE and DC = 181° 15′ – 101° 30′

= 79° 45′

∴ Included angle D = 79° 45′. **Ans.**

(v) The included angle at E :

The difference of bearings of EA and

ED = 124° 45′ – 1° 15′ = 123° 30′

∴ Included angle E = 123° 30′. **Ans.**

Check: Sum of the included angles of the pentagon

= (2 x 5 – 4) = 6 right angles.

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

= 162° 30′ + 94° 45′ + 79° 30′ + 79° 45′ + 123° 30′

= 540° 00′ or 6 right angles.

4.12 LOCAL ATTRACTION

North end of a freely suspended magnetic needle always points to the magnetic north, provided it is not influenced by any other external forces except the earth's magnetic field. It is a common experience that the magnetic needle gets deflected from its normal position, if placed near magnetic rocks, iron ores, cables carrying current or iron electric poles. Such a disturbing force is known as *'local attraction'*. Magnetic bearings are, therefore, not reliable unless these are checked against the presence of local attraction at each station and its elimination.

1. 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 of the stations.

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

It may be noted that local attraction at any station affects all the magnetic bearings by an equal amount and hence, the included angles deduced from the affected bearings are always correct. In case, the fore and back bearings of neither line of a traverse differs by the permissible error of reading, the mean value of the bearings of the line least affected, may be accepted. 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.

2. Method of elimination of local attraction by the included angles. The following steps are followed:

(*i*) Compute the included angle at each station from the observed bearings, in case of a closed traverse.

(*ii*) Starting from the unaffected line, run down the correct bear ings of the successive sides.

The complete procedure for detecting the local attraction is explained in Solved Example 4.8.

Example 4.8. *Below are the bearings observed in a traverse survey conducted with a prismatic compass at a place where local attraction was suspected.*

Line	*Fore bearing*	*Back bearing*
PQ	*124 ° 30′*	*304° 30′*
QR	*68° 15′*	*246° 00′*
RS	*310° 30′*	*135° 15′*
SP	*200° 15′*	*17° 45°*

At what stations do you suspect local attraction? Find the corrected bearings of the lines and also calculate the included angles.

Solution. (Fig. 4.22)

On examining the fore and back bearings of the traverse lines, it is seen that the fore and back bearings of line *PQ* differ exactly by 180° . Hence, stations *P* and *Q* are free from local attraction. The stations affected by local attraction may, therefore, be *R* and *S*.

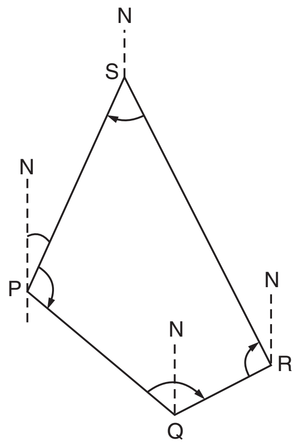


Fig. 4.22

Calculation of included angles

Angle at *P*:

Difference of bearings of *PQ* and *PS*

124° 30′ – 17° 45′ = 106° 45′

∴ Included angle at P = 106° 45′

Angle at *Q*

Difference of bearings of *QP* and *QR*

304° 30′ – 68° 15′ = 236° 15′

∴ Included angle at *Q* = 360° – 236° 15′ = 123° 45′.

Angle at R:

Difference of bearings of RS and RQ

$$310° 30' - 246° 00' = 64° 30'$$

$\therefore$ Included angle at R = 64° 30′

Angle at S:

Difference of bearings of SP and SR

$$200° 15' - 135° 15' = 65° 00'$$

$\therefore$ Included angle at S = 65° 00′

Check: The sum of interior angles

$$106° 45' + 123° 45' + 64° 30' + 65° 00' = 360° \text{ O.K.}$$

Calculation of bearings:

Bearing of the line PQ	= 124° 30′	(given)
Add included angle Q	+ 123° 45′	
Sum	= 248° 15′	
Sum is more than 180°, subtract	– 180° 00′	
$\therefore$ Bearing of the QR	= 68° 15′	
Add included angle R	+ 64° 30′	
Sum	= 132° 45′	
Sum is less than 180°, add	+ 180° 00′	
$\therefore$ Bearing of the line RS	= 312° 45′	
Add included angles S	+ 65° 00′	
Sum	= 377° 45′	
Sum is more than 180°, subtract	– 180° 00′	
$\therefore$ Bearing of the line SP	= 197° 45′	
Add included angle P	+ 106° 45′	
Sum	= 304° 30′	
Sum is more than 180°, subtract	– 180° 00′	
$\therefore$ Bearing of the line PQ	= 124° 30′	**Checked**

Corrected bearings	Included angles	
PQ = 124° 30′	P = 106° 45′	**Ans.**
QR = 68° 15′	Q = 123° 45′	
RS = 312° 45′	R = 64° 80′	
SP = 197° 45′	S = 65° 00′	

3. Method of elimination of local attraction by applying corrections to bearings

The following steps are followed:

(*i*) Calculate the magnitude and direction of the error due to local attraction at each affected station.

(*ii*) Run down the bearings, starting from the bearing unaffected by local attraction.

Complete procedure for eliminating the local attraction is explained in solved Example 4.9.

Example 4.9. *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:*

Line	*Fore bearing*	*Back bearing*
AB	*74° 20′*	*256° 0′*
BC	*107° 20′*	*286° 20′*
CD	*224° 50′*	*44° 50′*
DA	*205° 40′*	*126° 00′*

Solution. As the fore and back bearings of the line *CD* differ exactly by 180°, these bearings may be accepted as correct.

Back bearing of *BC*	= 286° 20′ (Correct)
Subtract 180°	– 180° 00′
∴ Correct fore bearing of *BC*	= 106° 20′
Observed bearing of *BC*	= 107° 20′
∴ Error due to local attraction at *B*	= Observed bearing – correct bearing
	= 107° 20′ – 106° 20′
	= + 1° 0′
Correction at *B*	= – 1° 0′
∴ Back bearing of *AB*	= Observed bearing + correction
	= 256° 0′ – 1° 0′
	= 255° 0′
Subtract 180°	– 180° 0′
Correct fore bearing of *AB*	= 75° 0′

∴ Error due to Local attraction at A = Observed bearing – correct bearing

$$= 74° 20' – 75° 0'$$
$$= – 0° 40'$$

Correction at A $= + 0° 40'$

∴ Back bearing of AD = Observed bearing + correction

$$= 126° 0' + 0° 40'$$
$$= 126° 40'$$

∴ Fore bearing of line AD = $126° 40' + 180°$

$$= 306° 40'$$

Result :

Line	*F.B.*	*B.B.*
AB	75° 0′	255° 0′
BC	106° 20′	286° 20′
CD	224° 50′	44° 50′
DA	306° 40′	126° 40′

4.13 MAGNETIC DECLINATION

The horizontal angle between true north and the magnetic north at a place at the time of observation, is *called magnetic declination* (Fig. 4.23).

The angle of convergence between the true north and magnet 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 or 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.* When both true and magnetic meridians coincide magnetic declination is zero.

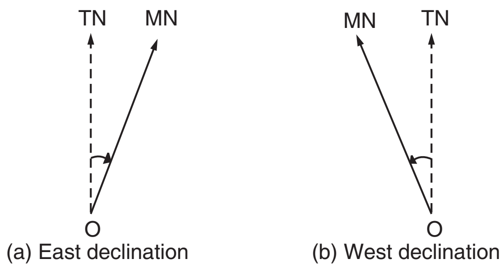


Fig 4.23. Magnetic declination

The imaginary lines joining the places of equal declination either positive or negative, on the surface of the earth, are called *"Isogonic lines"*. As the earth magnetism is not regular and the intensity of its magnetic field also varies, the isogonic lines do not form complete circles but these follow irregular paths. The isogonic lines having zero declination, are known as '*Agonic lines*'.

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

4.14 CALCULATION OF TRUE BEARING OR MAGNETIC BEARING

If we know magnetic bearing of a line and magnetic declination at that place, the true bearing of the line, may be calculated from the formula.

True bearing = magnetic bearing ± magnetic declination, use +ve sign if declination is west and – ve sign, if it is east.

If we know true bearing of a line and magnetic declination at that place the magnetic bearing of the line may be calculated from the formula:

Magnetic bearing = true bearing ± magnetic declination, use +ve sign for eastern declination and –ve sign for western declination.

Example 4.10. *The true and magnetic bearings of a line AB are 78° 45′ and 75° 30′ respectively. Calculate the magnetic declination at the place.*

Solution. (Fig. 4.24)

Magnetic declination

= True bearing – Magnetic bearing

= 78° 45 – 75′ 30′

= 3° 15′

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

∴ Magnetic declination

= 3° 15′ East. **Ans.**

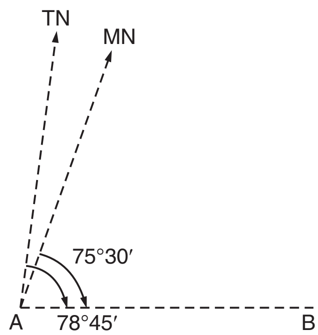


Fig. 4.24

Example 4.11. *The true and magnetic bearings of a line AB are 120° 4′ and 123° 15′ respectively. Calculate the magnetic declination at the station A.*

Solution. (Fig 4.25)

Magnetic declination = true bearing – magnetic bearing

= 120° 45′ – 123° 15′ = – 2° 30′

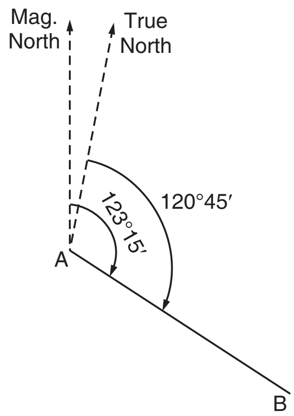


Fig 4.25

As the sign is negative, magnetic declination is west.

∴ Magnetic declination = 2° 30′ west. **Ans.**

Example 4.12. *Calculate the true bearing of a line CD if the magnetic bearing is S 50° 45′ W and the declination is 3° 45′ E.*

Solution. (Fig. 4.26)

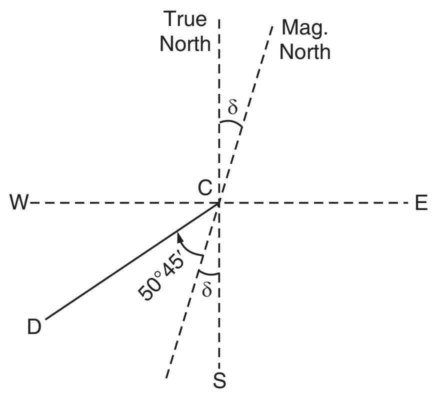


Fig. 4.26

As the declination is east, magnetic meridian will be east of true meridian.

$\therefore$ True bearing = magnetic bearing + declination
$= S\ 50° 45' W + 3° 45'$
$= S\ 54° 30' W$. **Ans.**

4.15 SOURCES OF ERROR IN COMPASS TRAVERSING

The errors in compass traversing, may be broadly classified as under:

(*i*) Instrumental Errors

(*ii*) Observational Errors.

1. **Instrumental Errors.** The instrumental errors which are caused by defective parts of the instrument, are:

(1) **Sluggish magnetic needle.** The needle of the compass gets sluggish due to dullness of the pivot. The pivot gets dull when the magnetic needle is unnecessarily allowed to swing even when not in use.

(2) **The eccentricity of the pivot.** If the pivot is not at the centre of the graduated ring, readings will be erroneous.

(3) **Non-verticality of the sight vanes.** The sight vane, object vane and the pivot may not be in the same vertical plane.

(4) Non-horizontality of the graduated ring.

(5) Un-equal divisions of the graduated ring.

(6) The line of sight may not be passing through the centre of the graduated circle.

(7) Non-coincidence of the magnetic and geometrical axes of the needle.

2. **Observational Errors.** The observational errors include the following:

(1) Incorrect bisection of the ranging rods.

(2) Incorrect reading of compass.

(3) Incorrect recording of readings.

(4) Presence of magnetic substances in the vicinity of stations.

(5) Magnetic changes in the atmosphere.

4.16 PRECAUTIONS TO BE TAKEN IN COMPASS SURVEY

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

(1) Set up and level the compass carefully.

(2) Stop the vibrations of the needle by gently pressing the brake-pin so that it may come to rest soon.

(3) Always look along the needle and not across it, to avoid parallax.

(4) 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.

(5) 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.

(6) Stations should be selected such that these are away from the sources causing local attraction.

(7) Surveyor should never carry iron articles, such as a bunch of keys, which may cause local attraction.

(8) Fore and back bearings of each line should be taken to guard against the local attraction. If the compass cannot be set at the end of a line, the bearings may be taken from any intermediate point along that line.

(9) Two sets of readings should be taken at each station for important details by displacing the magnetic needle after taking one reading.

(10) Avoid taking a reading in wrong direction viz. 25° to 20° instead 20° to 25° and so.

(11) 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.

(12) Object vane and eye vane must be straightened before making observations.

EXERCISE 4

1. Explain, with the help of neat sketches the graduations of a prismatic compass and a surveyor's compass.

2. (*a*) Draw a neat sketch of a prismatic compass and name the parts thereon.

3. Tabulate the differences of a prismatic compass and a surveyor's compass.

4. What are the sources of error in compass survey and what precautions are taken to eliminate them.

5. Define the following terms:

(*i*) Meridian (*ii*) True meridian

(*iii*) Magnetic Meridian (*iv*) Convergency of meridians.

10. Define the following terms:

(*i*) Bearing (*ii*) True meridian

(*iii*) Magnetic bearing (*iv*) Fore and back bearings.

7. Explain a bearing. What are different systems of designation of bearings. Explain each system with neat sketches.

8. What is local attraction ? How is it detected and removed ?

9. Write short notes on:

(*i*) Fore and back bearings.

(*ii*) Reduced bearings and whole circle bearings.

(*iii*) True bearing and magnetic bearing.

(*iv*) Magnetic declination and convergency of meridians,

(*v*) Local attraction and dip of needle.

(*vi*) Isogonic lines and agonic line

10. Convert the following whole circle bearings to quadrantal bearings.

(*a*) 87° 30′ (*b*) 120° 05′

(*c*) 210° 10′ (*d*) 266° 36′

(*e*) 310° 10′ (*f*) 359° 15′

11. Convert the following quadrantal bearings to the whole circle bearings:

(*a*) N 30° 30′ E (*b*) S 20° 45′ E

(*c*) S 10° 45′ W (*d*) N 50° 45′ W.

12. (*i*)Write the back bearings of the following forebearings

(*a*) 30° 05′ (*b*) 120° 25′

(*c*) 225° 15′ (*d*) 310° 36′

(*ii*) Write the fore bearings of the following back bearings:

(*a*) 67° 15′ (*b*) 136° 36′

(*e*) 189° 20′ (*d*) 7° 07′.

13. The bearings of the sides of a triangle *ABC* are as under:

$AB = 45° 15'$

$BC = 150°50'$

$CA = 270° 00'$

Calculate the interior angles of the triangle.

14. The bearings of lines *AB* and *AC* are 30° 45′ and 127° 35′ respectively. Calculate the acute angle *BAC*.

15. The bearings of the sides of a traverse ABCDEA are as follows:

Side	*F.B*	*B.B*
AB	150° 10′	285 10′
BC	20° 20′	200° 20′
CD	275° 35′	95 35′
DE	179° 45′	359° 45′
EA	120° 50′	300° 50′

Compute the interior angles of the travèrse and exercise the geometric checks.

Answers

10. (*a*) N 87° 30′ E (*b*) S159° 55′ E
(*c*) S 30° 10′ W (*d*) S 86° 36′ W
(*e*) N 49° 50′ W (*f*) N 0° 45′ W.

11. (*a*) 30° 30′ (*b*) 159° 15′
(*c*) 190° 45′ (*d*) 309° 15′.

12. (*i*) (*a*) 210° 15′ (*b*) 300° 25′
(*c*) 45° 45′ (*d*) 130° 36′.
(*ii*) (*a*) 247° 15′ (*b*) 316° 36′
(*c*) 9° 20′ (*d*) 187° 07′.

13. *A* = 44° 45′; *B* = 74° 25′; *C* = 60° 50′.

14. Angle *ABC* = 89° 50′.

15. *A* = 164° 20′; *B* = 90° 10′; *C* = 75° 15′, *D* = 84° 10′; *E* = 121° 05′.

CHAPTER 5

Levelling

5.1 LEVELLING

The art of determining relative altitudes of points on the surface of the earth or beneath the surface of the earth, is called *levelling*. This branch of surveying deals with measurements in vertical planes.

5.2 THE LEVEL

The instrument which is used for levelling, is known as a *level*. It consists essentially of the following parts:

1. **Telescope.** It is an optical instrument used for magnifying and viewing the images of distant objects. The telescopes which are fitted in levels, are generally of two types:

(*i*) External focussing telescope

(*ii*) Internal focussing telescope.

The external focussing telescopes were genetally used in olden type level whereas internal focussing telescopes, are being used in modern survey instruments.

A surveying telescope is similar to Keplar's telescope. It consists of two convex lenses fitted in a tube. The lens fitted near the eye, is called the *eye piece* and the other fitted at the end nearer to the object, is called *the objective.*

(1) **Cross Hairs of the Eye Piece.** The cross hairs are placed in front of the eye piece within its focal distance, where an inverted image of the object is produced by the objective. The cross hairs also get magnified by the eyepiece along with the inverted image of the objects.

(2) **Focussing of telescope.** The operation of obtaining a clear image of the object in the plane of cross hairs, is known as *focussing*.

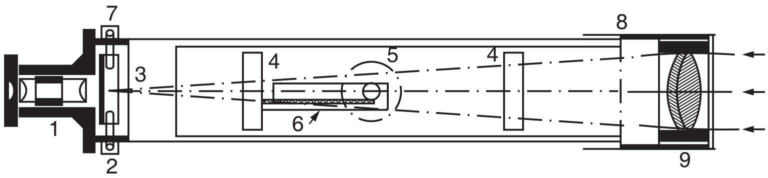
Focussing of a telescope is achieved in two steps:

Focussing the eyepiece. In this operation, the cross hairs are made to appear clearly visible, with the help of the eye piece unit which may be moved in or out. By doing so the cross hairs are brought in the plane of distant vision. The focussing of the eye piece, depends on the observer's eye sight.

Focussing the objective. In this operation, the image of the object is brought in the plane of the cross hairs which are clearly visible. The focussing of a telescope can be done externally or internally. The telescopes are also classified as external focussing tele scopes and internal focussing telescopes.

(3) **External focussing Telescope.** The telescope in which focussing is achieved by the external movement of either objective or eye-piece, is known as *an external focussing telescope.*

In an external focussing telescope, the body is formed by two tubes at the ends of each, objective and eye piece are fitted. One of the tubes is made to slide axially within the other by means of rack and pinion arrangement attached to the focussing screw of the telescope. (Fig. 5.1).

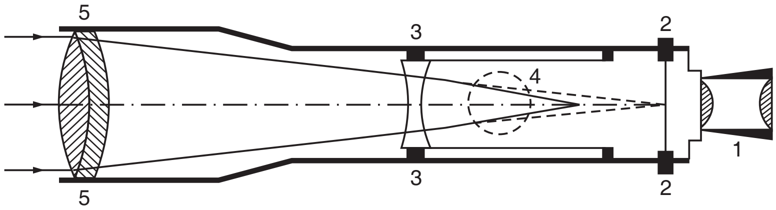


1. Eye piece	2. Diaphragm screws	3. Diagram
4. Stops	5. Focussing screw	6. Rack
7. Pinion	8. Ray shade	9. Obective glass

Fig. 5.1. An external focussing telescope.

(4) **Internal Focussing Telescope.** The telescope in which focussing is achieved internally with a concave lens, is known as *internal focussing telescope.*

In an initial focussing telescope, the objective and eye piece are kept at a fixed distance, and focussing is achieved by a double concave lens mounted in a short tube capable of sliding axially to and fro between the eye piece and the objective with a rack and pinion arrangement attached to the focussing screw. (Fig. 5.2).



1. Eye piece 2. Diaphragm screws 3. Focussing lens
4. Focussing screw 5. Object glass.

Fig. 5 2. An internal focussing telescope.

(5) **Parallax.** When the image of an object formed by the objective does not lie in the plane of the cross hairs, any movement of the eye causes an apparent movement of the image with respect to cross hairs. This shift of the image, is called *'parallax'*.

(6) **Objective** (Fig. 5.3). An objective is a lens on which rays from an object are incident. It is invariably a compound lens consisting of:

(*i*) The front double convex lens made of crown glass.

(*ii*) The back convex lens made of flint glass.

The two lenses when cemented together with balsm at their common surface, are generally known as *achromatic lens*. In such lenses the spherical and chromatic aberrations known as optical serious defects are practically eliminated.

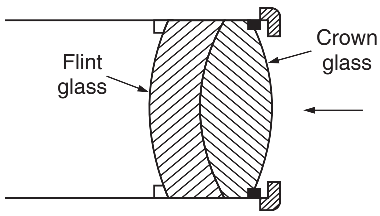


Fig. 5.3. An objective.

(7) **Eye-piece.** It is an assembly through which image of the object formed by the objective and magnified by itself, is viewed. It consists of two plano-convex lenses of equal focal length placed in a small tube such that their spherical surfaces face each other and are separated by a distance equal to the focal length of either.

Ramsden's eye-piece is used in most of the surveying instruments. It is also sometimes called as *positive or non-erecting eye-piece* as it does not change the inverted image formed by the objective. This eye-piece does not satisfy the conditions of minimum spherical aberration but the curvature of the lenses are arranged so as to remedy the defects as far as possible.

(8) **Diaphragm.** A frame carrying cross hairs usually made of either silk thread, spider thread or platinum wire and placed at the plane at which vertical image of the object is formed by the objective, is known as a *diaphragm*.

A few typical arrangements of cross hairs used in diaphragms of level telescopes, are shown in Fig. 5.4.

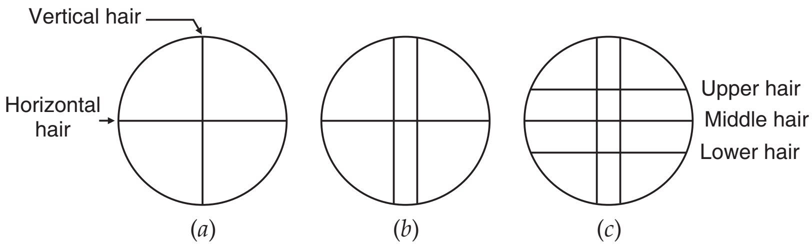


Fig. 5.4. Type of diaphragms.

The vertical hair of the diaphragm enables the surveyor to check the verticality of the levelling staff along sideways whereas the horizontal hairs are used to read the staff graduations. With the diaphragms shown in Fig. 5.4 (*a*) and 5.4 (*b*) only one reading of the staff is possible but with the third one [5.4 (*c*)] three readings of a staff for each setting of the instrument are possible. The mean of the three readings should generally agree with the middle wire reading.

2. **Level Tube.** A level tube also known as *bubble tube*, consists of a glass tube placed in a brass tube which is sealed with plaster of Paris. The whole of the interior surface the upper half is accurately ground so that its longitudinal section, is an arc of a circle. The level tube is nearly filled with either ether, alcohol or a mixture of both. The remaining space is occupied by an air bubble. The centre of the air bubble always rests at the highest point of the tube. (Fig. 5.5).

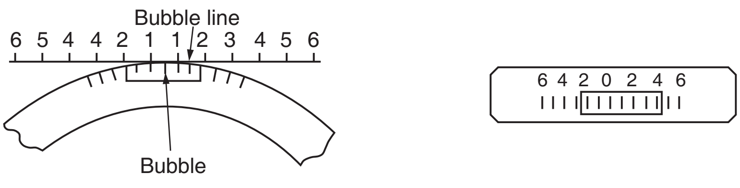


Fig. 5.5. A level tube.

The outer surface of the level tube is graduated in both directions from the centre. The exact centering of the bubble can be ascertained by observing the number of divisions of its ends from the centre. *The* *line tangential to the circular arc at its highest point, i.e., the middle of the tube or the zero of the graduations, is called the axis of the level tube.* When the bubble is central, the axis of the bubble becomes horizontal. The length of the bubble changes with a change in temperature. With a rise in temperature the liquid expands and thus the bubble shortens and consequently, its sensitiveness is reduced.

The level tube is attached on the top of the telescope by means of Capstan-headed nuts.

3. **A Levelling Head.** The levelling head generally consists of two parallel plates with three or four foot screws. The upper plate is known as *tribarch* and the lower plate as *trivet* which can be screwed on to the tripod. A levelling head has to perform the following three distinct functions:

(1) To support the telescope.

(2) To attach the level to the tripod.

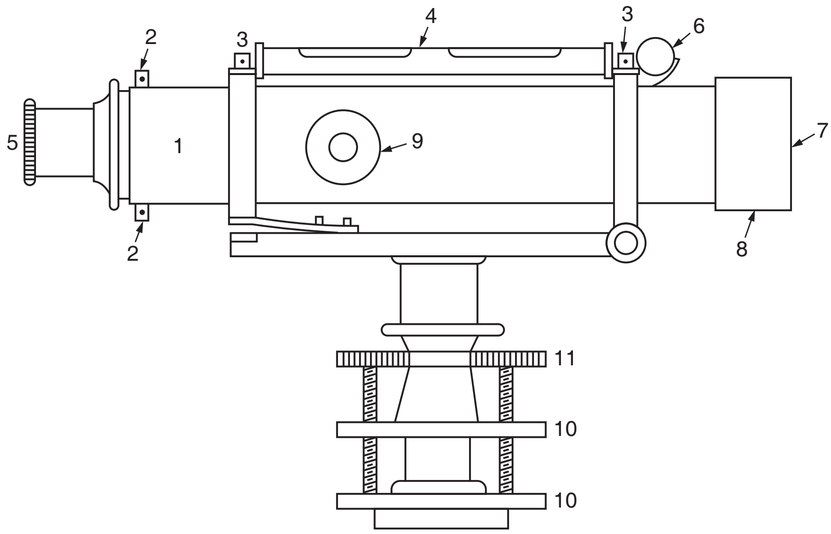
(3) To provide a means for levelling the instrument.

4. **Tripod.** When in use, the level is supported on a tripod which consists of three solid or framed legs. At the lower ends, the legs are provided with pointed iron shoes. The tripod head carries a its upper surface, an external screw to which the foot-plate (trivet) of the levelling head can be screwed.

5.3 THE DUMPY LEVEL

The dumpy level designed by Gravatt, consists of a telescope rigidly fixed to its support. It can neither be rotated about its longitudinal axis, nor it can be removed from its supports. A long bubble tube is attached on the top of the telescope. (Fig. 5.6).

The fact that a dumpy level in its original design, was comparatively shorter than a Wye Level of the same magnifying power, originated the name dumpy level. Dumpy literally means short and thick. Its levelling head generally consists of two parallel plates with either three or four foot screws. The upper plate is known as *tribarch* and the lower plate known as *trivet*, is screwed on to the tripod before setting up.



1. Telescope.
2. Diaphragm adjusting screws.
3. Bubble tube adjusting screws.
4. Longitudinal bubble.
5. Eye-piece.
6. Transverse bubble tube.
7. Objective end.
8. Ray shade.
9. Focussing screw.
10. Levelling head.
11. Foot screws.

Fig. 5.6. A Dumpy level.

5.4 ADVANTAGES AND DISADVANTAGES OF DUMPY LEVEL

These include the following:

(*i*) It is simple in construction with a few movable parts.

(*ii*) It requires fewer permanent adjustments.

(*iii*) Adjustments once carried out remain for a longer good period.

5.5 LEVELLING STAFF

A straight, rectangular wooden rod graduated into metres/feet and their smaller divisions, is called a *levelling staff.* The bottom of the levelling staff represents the zero reading. The reading given by the line of sight on a levelling staff held vertically is the height of the line of collimation from the point on which staff is held.

Classification of Levelling Staves. The levelling staves may be divided into two classes:

(*i*) Self reading staff.(*ii*) Target staff.

1. **Self reading staff.** A staff on which readings are directly read by the observer through the telescope, is known as *self-reading staff* Self reading staffs are of three types as discussed below:

(1) **Solid staff.** These are usually 3 m long in one length. Due to the absence of a hinge or socket on these staffs, greater accuracy in reading is achievable but on the other hand it is inconvenient to carry them in the field. Use of a solid staff is generally restricted to only precise levelling work.

(2) **Folding or hinged staff** (Fig. 5.7). A folding staff is made of well-seasoned timber. It is 4 m long and consists of two portions, each being 2 m hinged together. The width and thickness of the staff, is kept 75 mm and 18 mm respectively.

The foot of the staff is provided with a brass cap to avoid wear and tear due to usage. Sometimes, a plummet is also provided to ascertain the verticality of the staff by the staffman.

Each metre length is sub-divided into decimetres and each decimetre is further divided into 20 equal divisions of 5 mm width. Decimetre numerals 1 to 9 of each metre length, are marked in black and metre numerals in red. The graduations are marked inverted so that they appear erect when viewed through the telescope.

The staff may be folded together so that:

(*i*) One 2 m piece is capable of folding on the other when not in use.

(*ii*) Two pieces are detachable from one another so that one half may be used while working in plain areas.

(*iii*) When the two portions are locked together, the entire length should behave as a rigid rod.

(3) **Telescopic or Sopwith type staff.** (Fig. 5.8). It consists of three pieces. Top piece is solid 1.25 m long whereas central piece 1.25 m and lower piece 1.5 m are hollow. The top portion slides into the central piece and central piece slides into the lower portion. The total length of the staff when fully extended is 4 m. The upper two pieces are held by brass spring catches.

The smallest division of this type of levelling staff is also 5 mm. The metre numerals which are shown on the left are marked in red. The decimetre numerals 1 to 9 are shown on the right and marked in black. The decimetre numeral 10 of each metre length is omitted and letter *M* is marked to indicate the end of the metre length. Graduations are marked erect and when viewed through the telescope these appear inverted. While using a telescopic staff it may be ensured that the three parts are fully extended in length when using the full length *i.e.*, 4 m.

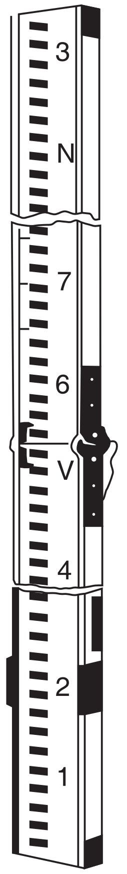


Fig. 5.7. Folding staff.

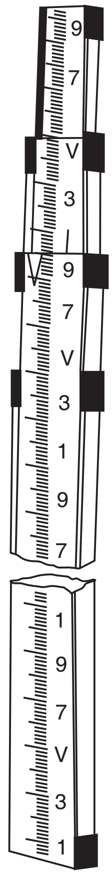


Fig. 5.8. Telescopic staff.

5.6 TECHNICAL TERMS USED IN LEVELLING

The following technical terms are generally used in levelling:

1. **Level surface.** The surface which is parallel to the mean spheroidal surface of the earth, is known as *level surface*. Every point on this surface is equidistant from the centre of the earth. It is also

normal to the plumb line at every point. The surface of still water in a lake represents a level surface.

2. **Level line.** A line lying in the level surface, is known as a *level line*. Every point of a level line, is equidistant from the centre of the earth. The cross-section of still water of a lake, represents a level line.

3. **Horizontal surface.** A surface tangential to the level surface at any point, is known as a *horizontal surface*. It is perpendicular to the plumb line at the tangent point.

4. **Horizontal line.** A line lying in the horizontal surface, is known as a *horizontal line.* It is a straight line tangential to the level line.

5. **Vertical line.** A line perpendicular to the level line is called a *vertical line*. The plumb line at any place, is called the *vertical line*.

6. **Vertical plane.** The plane which contains the vertical line at a place, is called a *vertical plane*.

7. **Vertical angle.** The angle between an inclined line and a horizontal line at a place, in vertical line is called *vertical angle*.

8. **Datum surface.** The imaginary level surface with reference to which vertical distances of the points (above or below) are measured, is called *datum surface.*

9. **Mean sea level datum.** The mean sea level datum obtained by making hourly observations of the tides at sea coast over a period of 19 years, is known as *mean sea level*. The M.S.L. datum adopted by the Survey of India for determining the elevations of different points in India is that of Bombay after partition.

10. **Reduced level (R.L.).** The height or depth of a point above or below the assumed datum, is called *reduced level* (R.L.). It is also known as elevation of the point. Elevations of the points below the datum surface, are known as *negative elevations*.

11. **Line of sight.** The line passing through the optical centre of the objective, traversing the eye-piece and entering the eye, is known as *line of sight.*

12. **Line of collimation.** The line passing through the optical centre of the objective and the point of intersection of the cross hairs stretched in front of the eye piece and its continuation, is called *line of collimation.*

13. **Optical centre of a lens.** The point in a lens through which rays pass without any lateral displacement, is called *optical centre*. It is so situated in the lens that it distances from the curved surfaces are directly proportional to the radii.

14. **Axis of the telescope.** The line joining the optical centre of the objective and the centre of the eye piece, is called *axis of the telescope.*

15. **Bench mark. (B.M.)** A relatively permanent and fixed reference point of known elevation above the assumed datum, is called a *bench mark.*

5.7 SPECIAL TERMS AND THEIR ABBREVIATIONS USED IN LEVELLING

1. **Instrument station.** The point where level is set up for observations, is called *instrument station*.

2. **Station.** The point where levelling staff is held, is called *station*, It is the point whose elevation is to be determined or the one that is to be established at a given elevation.

3. **Height of instrument (H .I.).** The elevation of the line of sight with respect to the assumed datum, is known as *height of instrument.* In levelling it does not mean the height of the telescope point above the ground level where the level is set up.

4. **Back sight (B.S).** The first sight taken on a levelling staff held at a point of known elevation, is called *back sight*. It ascertains the amount by which the line of sight is above or below the elevation of the point. Back sight enables the surveyor to obtain the height of the instrument.

5. **Fore sight (F.S.).** The sight taken on a levelling staff held at point of unknown elevation to ascertion the amount by which the point is above or below the line of sight, is called a *fore sight*. Fore sight enables the surveyor to obtain the elevation of the point. It is also generally known as *minus sight* as the fore sight reading is always subtracted from the height of the instrument (except when the staff is held inverted) to obtain the elevation.

6. **Change point.** The point on which both the fore sight and back sight, are taken during the operation of levelling, is called a *change point.* Two sights are taken from two different instrument stations, a fore sight to ascertain the elevation of the point while a back sight is taken on the same point to establish the height of the instrument for the new setting of the level. The change point is always selected on a relatively permanent point.

7. **Intermediate sight.** The fore sight taken on a levelling staff held at a point between two turning points, to determine the elevation of that point, is known as *intermediate sight.* It may be noted that for

one setting of a level, there will be only a back sight and a fore sight but there can be any number of intermediate sights..

5.8 TEMPORARY ADJUSTMENTS OF LEVEL

The adjustments which are made for every setting of a level, are called *Temporary Adjustments*. These include:

(*i*) Setting up the level.

(*ii*) Levelling up the level.

(*iii*) Elimination. of parallax of telescope.

1. **Setting up the Level.** This operation includes fixing the instrument on the tripod and also levelling the instrument approximately by leg adjustment. To achieve this, release the clamp, hold the instrument in the right hand and fix it on the tripod by turning round the levelling head with the left hand. The tripod legs are so adjusted that the telescope is at a convenient height and the tribarch is approximately levelled, Modern levels are provided with a small circular bubble on the tribarch for achieving approximate levelling of the instrument.

2. **Levelling Up the level.** After setting up the level, accurate levelling is done with the help of foot screws, and by using plate levels. The object of levelling up the instrument is to make its vertical axis truly vertical.

The following steps are followed: (Fig. 5.9).

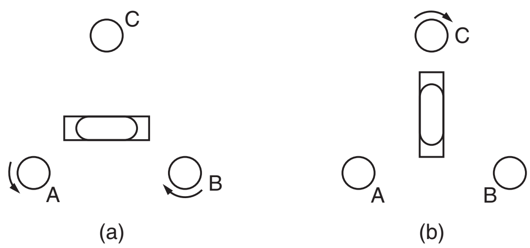


Fig. 5.9. Levelling up with three screws.

(1) Loosen the clamp and turn the instrument until the longitudinal axis of the plate level is parallel to a line joining any two levelling screws. Such as *A* and *B* in Fig. 5.10 (*a*).

(2) Holding these two foot screws with the thumb and first finger of each hand, turn them uniformly so that the thumbs move either towards each other or away from each other until the plate bubble is central. The bubble moves in the same direction as left thumb.

(3) Rotate the upper plate through 90°, *i.e.*, until the axis of the plate level coincides a line joining the third foot screw *C* and the midpoint of the first two screws *A* and *B* Fig. 5.10 (*b*).

(4) Hold the third screw with the thumb and first finger of the right hand and turn it until the plate bubble is central.

(5) Rotate the upper plate through 90° to its original position [5.10 (*a*)] and repeat step 2 till the bubble is central.

(6) Rotate again through 90° and repeat step 4.

(7) Repeat the steps 2 and 4 till the bubble remains central in both the positions.

(8) Rotate the instrument through 180°. The bubble should remain central if the instrument is in adjustment. The vertical axis of the level will now be truly vertical. If not, the instrument needs permanent adjustment.

Note. It is very essential to keep the same quarter of the circle of changing directions and not to swing through the remaining three quarters of the circle for repeating steps 2 and 4.

3. **Elimination of parallax of telescope.** If the image formed by the objective does not lie in the plane of the cross hairs, there will be a shift in the image due to shift of the eye. Such displacement of the image is termed as *parallax*. For an accurate sighting, the parallax is eliminated in two steps: (*i*) focussing the eye-piece for distinct vision of the cross hair (*ii*) focussing the objective so that the image is formed in the plane of cross hairs.

(1) **Focussing the eye-piece.** Following steps are involved:

(*i*) Direct the telescope either towards the sky or hold a sheet of white paper in front of the objective.

(*ii*) Move the eye piece in or out till the cross hairs appear distinct.

In some levels, the eye-piece is graduated and numbered. Once

the eye-piece is focussed, the observer may note this position to save much of his time at other settings.

(2) **Focussing the objective.** Following steps are involved:

(*i*) Direct the telescope towards the levelling staff.

(*ii*) Turn the focussing screw till the image appears clear and sharp.

(*iii*) The image so formed must be in the plane of cross hairs.

5.9 BENCH MARKS

The point whose elevation above known datum, is known is called a *bench mark*.

Depending upon the permanency and precision, bench marks may be divided into the following types:

1. **G.T.S. Bench Marks.** These bench marks are established by the Survey of India with greatest precision at an interval of about 100 km all over the country. Their elevations refer to the means sea level datum obtained by hourly observations of the tides over a period of 19 years, at Bombay port. G.TS. bench marks falling in the belts of the area bounded by 1° Latitude and 1° longitude, are published in levelling pamphlets. These are also depicted on topo sheets published by the Survey of India and their elevations correct to two places of decimal of a metre, are entered.

2. **Permanent Bench Marks.** These bench marks are established between G.T.S. bench marks by the Survey of India or other government agencies such as P.W.D., on clearly defined and permanent natural or cultural detail points such as isolated rocks culverts, kilometre stones, railway platforms, gate pillars of inspection houses etc. The permanent bench marks established by the Survey of India contain the inscripts G.T.S.

•

B.M

Their elevations are also published in the levelling pamphlet of the area. P.W.D. bench marks are marked on a plane surface by a rectangle. Below or above the rectangle, the letters B.M. along with R.L. of the bench mark are also cut and filled in Japan black. Such Bench Marks are used for reference and checking purpose, For irrigation projects, G.T.S. or other permanent bench marks are referred to, to decide the required slope of the bed of canals so that water flows freely under gravity.

3. **Arbitrary Bench Marks.** These are the reference points whose elevations are arbitrarily assumed for small levelling operations. Their elevations do not refer to any fixed datum as in the case of. G.T.S. or permanent bench marks.

4. **Temporary Bench Marks.** These are the reference points on which a day's work is closed and from where levelling is continued next day in the absence of a permanent B.M. Their elevations are referred to as the reduced levels. Such bench marks should be carefully

established on permanent detail-points such as kilometre stones, parapets, floor of verandahs, roots of old trees, etc. Their correct descriptions should invariably be written in level books.

5.10 CLASSIFICATION OF LEVELLING

Levelling may be classified into two main types *i.e.*,

(*i*) Simple Levelling, (*ii*) Differential Levelling.

1. **Simple Levelling.** The operation of levelling for determining the difference in elevation, if not too great, between two points visible from a single position of the level, is known as *Simple levelling.*

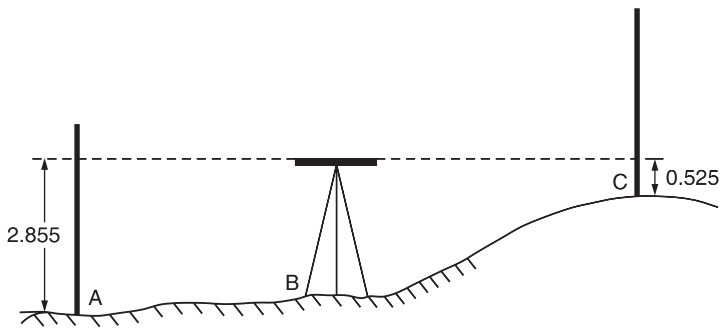


Fig. 5.10. Simple levelling

Suppose *A* and *C* are two points whose difference in elevations, is required with a level set up at *B*. To eliminate the effect of the earth curvature and instrumental errors, it is always advisable to ensure that their distances from the level are kept equal but not necessarily on the line joining them. (Fig. 5.10).

Procedure. Following steps are involved:

(*i*) Level the instrument correctly.

(*ii*) Direct the telescope towards the staff held vertically on *A*. Focuss it carefully to obtain clear graduations.

(*iii*) Take the reading' of the central horizontal hair of the diaphragm where it appears to cut the staff, ensuring that the bubble is central.

(*iv*) Send the staff to the next point *C*.

(*v*) Direct the telescope towards *C* and focuss it again.

(*vi*) Check up the bubble if central. If not, bring it to the central position by the foot screw *nearest to the telescope* or the micrometer screw in case of a tilting level.

(*vii*) Take the reading of the central horizontal cross hair.

Illustration. Let the respective readings on staff *A* and staff *C* be 2.855 m and 0.525 m respectively. The difference of elevation between *A* and *C*.

$$= 2.855 - 0.525 = 2.330 \text{ m}$$

If R.L. of *A* is 500.000 m, the R.L. of *C*, may be calculated as under:

R.L. of the point *A* = 500.000 m.

R.L. of the line of sight = 500.000 + 2.855 = 502.855 m.

R.L. of the point *C* = 502.855 – 0.525 = 502.330 m.

2. **Differential Levelling** (Fig. 5.11). The method of levelling for determining the difference in elevation of two points either too far apart or obstructed by an intervening ground, is known as *Differential levelling*. In this method, the level is set up at a number of points and the difference in elevation of successive points, is determined as in the case of simple levelling. This levelling process is also known as *Fly, Compound or Continuous levelling.*

Let us suppose that *A* and *B* are two points which are far apart and the difference in their elevations, is to be determined by differential levelling.

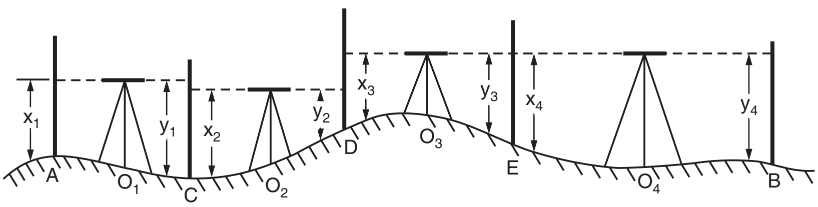


Fig. 5.11. Differential Levelling.

Procedure. Following steps are involved:

(*i*) Set up the level at O_1 ensuring that the line of sight intersects the staff held at *A*. Level it correctly.

(*ii*) With the bubble central, take the back staff reading on the staff held vertically at *A*.

(*iii*) Select a point *C* equidistant from the instrument position O_1, and take the fore staff reading on the staff held vertically at *C*.

(*iv*) Shift the instrument to O_2, set up and level it correctly.

(*v*) With the bubble central, take the back staff reading on the staff held vertically at *C* again.

(*vi*) Select a point *D* equidistant from the instrument position O_2 and take the fore staff reading on the staff held vertically at *D*.

(*vii*) Repeat the process until the fore staff reading is taken on the staff held on the last point *B*.

Note. The following points may be noted:

(*i*) The points where, two readings are taken at the successive points *C*, *D*, *E* etc., are called *change points*.

(*ii*) The level must be set up on firm ground otherwise it may sink during the interval of reading the back and fore sights.

(*iii*) The bubble must always be brought to the centre of its run before staff reading is taken.

(*iv*) The staff from the change point must not be removed till a back sight is taken from the next instrument station by simply turning around to face the telescope.

5.11 BOOKING AND REDUCTION OF THE LEVELS

Booking and reduction of the levels may be done by following two methods:

(*i*) Rise and fall method.

(*ii*) Height of collimation method.

1. **Rise and Fall Method.** In this method, the difference of level between two consecutive points for each setting of the instrument, is obtained by comparing their staff readings. The difference between their staff readings indicates *a rise* if the back staff reading is more than the fore sight and *a fall* if it is less than the fore sight. The rise and fall worked out for all the points give the vertical distance of each point relative to the preceding one. If the R.L. of the back staff point is known, then R.L. of the following point may be obtained by adding its rise or subtracting its fail from the R.L. of preceding point as the case may be.

The specimen page of a level book illustrating the method of booking staff readings and calculating R.Ls. of stations by the Rise and Fall method is shown in Table 5.1.

Table 5.1. Rise and fall method of reduction of levels

Stn. No.	*B.S.*	*I.S.*	*F.S.*	*Rise*	*Fall*	*R.L.*	*Remarks*
1	0.585					100.00	B.M.
2	1.855		2.955		2.370	97.300	C.P.
3		1.265		0.590		98.220	
4		2.995			1.660	96.560	
5	2.350		0.350	2.575		99.135	C.P.
6		2.855			0.505	98.630	
7	2.585		1.655	1.200		99.830	C.P.
8			2.435	0.250		100.000	B.M.
Total	7.475		7.395	4.615	4.535		

Arithmetic checks:

$$\Sigma B.S. - \Sigma F.S. = \Sigma Rise - \Sigma Fall = \text{Last R.L.} - \text{First R.L.}$$

i.e., $7.475 - 7.395 = 4.615 - 4.535 = 100.080 - 100.000 = 0.080.$

2. **Height of Collimation Method.** In this method, height of the instrument (H.I.) is calculated for each setting of the instrument by adding the back sight (B.S.) to the elevation of the B.M. The reduced level of the first station is obtained by subtracting its fore sight from the instrument height (H.I.). For the second setting of the instrument, the height of the instrument is calculated by adding the back sight taken on the first station to its reduced level. The reduced level of the last point is obtained by subtracting the foresight of the last point from the height of instrument at the last setting.

If an intermediate sight is observed to an intermediate station, its reduced level is obtained by subtracting its foresight from the height of the instrument for its setting.

The specimen page of a level field book illustrating the method of booking staff readings and calculating R.Ls. of the stations by the height of collimation method, is shown in Table 5.2.

Table 5.2 Height of Instrument Method of Reduction of Levels

Stn. No.	*B.S.*	*I.S.*	*F.S.*	*H.I.*	*R.L.*	*Remarks*
1	0.585			100.585	100.00	B.M.
2	1.855		2.955	99.485	97.630	C.P.
3		2.265			98.220	
4		2.925			96.560	
5	2.350		0.350	101.485	99.135	C.P.
6		2.855			98.630	
7	2.685		1.655	101.515	99.830	C.P.
8			2.435		100.080	B.M.
Total	7.475		7.395			

Arithmetic checks:

$$\Sigma B.S. - \Sigma F.S. = \text{Last R.L.} - \text{First R.L.}$$

i.e., $7.475 - 7.395 = 100.080 - 100.000 = 0.080$

Table 5.3. Comparison of Line of Collimination Method with Rise and Fall Method

Height of collimation method	*Rise and fall method*
1. It is more rapid and saves a considerable time and labour.	1. It is laborious as the staff reading of each station is compared, to get a rise or fall.
2. It is well adopted for reduction of levels for construction work such as longitudinal or cross-section levelling operation.	2. It is well adopted for determining the difference in levels of two points where precision is required.
3. There is no check on reduction of R.Ls. of intermediate stations.	3. There is a complete check on the reduction of R.Ls. of intermediate stations.
4. There are only two arithmetical checks *i.e.*, the difference between the sum of the back sights and the sum of the fore sights must be equal to be the difference in R.L. of the last station and first station.	4. There are three arithmetical checks *i.e.*, the difference between the sum of the back sights and the sum of foresights must be equal to the difference between the sum of the rises and the sum of falls as well as it must

	also be equal to the difference in R.Ls. of the last station and first station.
5. Errors if any, in intermediate sights are not detected.	5. Errors in intermediate sights are noticed as these are used for finding out the rises and falls.

Example 5.1. *The following consecutive readings were taken with a dumpy level:*

0.795, 1.655, 2.890, 3.015, 0.655, 0.625, 0.955,0.255, 1.635, 0.860, 2.375.

The instrument was shifted after the fourth and the eighth readings. The first reading was taken on bench mark whose R.L. is 550.605 metres.

Rule out a page of a level field book and enter the above readings. Calculate the reduced levels of the stations by the rise and fall method and apply arithmetical checks.

Solution.

Sl. No.	*B.S.*	*I.S.*	*F.S.*	*Rise*	*Fall*	*R.L.*	*Remarks*
1	0.795					550.605	B.M.
2		1.655			0.860	549.745	
3		2.890			1.235	548.510	
4.	0.655		3.015		0.125	548.385	C.P.
5		0.625		0.030		548.415	
6		0.955			0.330	548.085	
7	1.635		0.255	0.700		548.785	C.P.
8		0.860		0.775		549.560	
9			2.375		1.515	548.045	
Total	3.085		5.645	1.505	4.065		

Arithmetic checks:

(1) ΣB.S. – ΣF.S. = 3.085 – 5.645 = – 2.560

(2) Σ Rise – Σ Fall = 1.505 – 4.065 = – 2.560

(3) R.L. last point – R.L. of first point

= 548.045 – 550.605 = – 2.560

Explanation. While entering the readings in the level book, keep in mind the following points :

(*i*) First reading taken on the B.M. must be a back sight reading.

(*ii*) Fourth and eight readings must be the fore sight readings.

(*iii*) Fifth and ninth readings must be back sight readings.

(*iv*) Last reading must always be a fore sight reading.

Example 5.2. *The following consecutive readings were taken with a level and a 4 metre levelling staff on a continuously sloping ground.*

0.755, 1.545, 2.335, 3.545, 3.665, 0.525, 1.275, 2.650, 2.895, 3.565, 0.345, 1.525, 1.850, 2.675, 3.775. The first reading was taken on a bench mark whose R.L. is 200 metres.

Rule out a page of level book and enter the above readings. Calculate the reduced levels of the stations by the line of collimation method and apply normal and apply normal arithmetical checks.

Solution.

Sl. No.	*B.S.*	*I.S.*	*F.S.*	*Ht. of line of collimation*	*R.L.*	*Remarks*
1	0.755			200.755	200.000	B.M.
2		1.545		200.755	199.210	
3		2.335		200.755	198.420	
4		3.545		200.755	197.210	
5	0.525		3.655	197.625	197.100	C.P.
6		1.275		197.625	196.350	
7		2.650		197.625	194.975	
8		2.895		197.625	194.730	
9	0.345		3.565	194.405	194.060	C.P.
10		1.525		194.405	192.880	
11		1.850		194.405	192.555	
12		2.675		194.405	191.730	
13			3.775	194.405	190.630	B.M.
Total	1.625		10.995			

Arithmetic checks :

1. ΣB.S. – ΣF.S. = 1.625 – 10.996 = – 9.370
2. R.L. of last point – R.L. of first point
   = 190.630 – 200.000 = – 9.370.

5.12 ERRORS IN LEVELLING

Errors in levelling may be categorised into following three heads:

(*i*) Personal errors.

(*ii*) Errors due to natural causes.

(*iii*) Instrumental errors.

1. **Personal errors.** Personal errors include:

(*a*) Error in sighting.

(*b*) Error in manipulation.

(*c*) Error in reading the staff.

(*d*) Error in recording and computation.

(*A*) **Error in sighting.** This error is caused when it is difficult to see the exact coincidence of cross hairs and the staff graduations. This may be either due to long sights or due to coarseness of the cross hairs and the staff. Sometimes, atmospheric conditions also cause an error in sighting. This error is accidental and may be classified as compensative.

(*B*) **Error in manipulation.** These include the errors due to the following reasons:

(*i*) **Carelessly setting up the level.** The instrument should be set up on firm ground and carefully levelled. Neither the telescope nor the tripod should be touched while taking readings.

(*ii*) **Imperfect focussing of eyepiece and objective.** To eliminate this error, the eyepiece must be moved in or out till the cross hairs are distinctly visible against a white paper. The parallax should also be completely removed by properly focussing the object glass before taking every reading.

(*iii*) **The bubble not being central at the time of taking readings.** When the bubble is exactly central, the vertical axis is truly vertical and the horizontal axis of the telescope becomes horizontal. If the bubble is not central, the horizontal axis of the telescope gets inclined which affects the staff readings. The error is more for long sights and less for short sights. To avoid this error, the observer should develop a habit to check the bubble before and after taking each reading.

(*C*) **Error in reading the staff.** These errors are generally committed by a beginner and are :

(*i*) Reading the staff upwards, instead of downwards.

(*ii*) Reading against the top or bottom hair instead of the central hair.

(*iii*) Concentrating the attention on the decimal part of the readings, and reading the whole metre wrongly.

(*iv*) Reading the inverted staff as a vertical held staff.

(*D*) **Errors in recording and computation.** The common errors in recording include:

(*i*) Entering a reading in the wrong column *i.e.*, B.S. reading in I.S. or F.S. columns or vice versa.

(*ii*) Recording the readings with digits interchanged *i.e.*, 2.654 instead 2.456.

(*iii*) Omitting an entry.

(*iv*) Mistaking the numerical value of a reading called by the level man.

(*v*) Entering the inverted staff reading without a minus sign.

(*vi*) Adding the foresight reading instead of subtracting it and or subtracting a backsight reading instead of adding.

2. **Errors due to natural causes.** These include the following errors:

(1) **Errors due to curvature.** The curvature of the earth surface lowers the elevations of the stations and its amount is directly proportional to the square of the horizontal distance between the staff position and the point of the observation. The correction of the curvature has to be subtracted from the observed staff reading to get correct reading. In case of ordinary levelling, error due to curvature is a negligible quantity (only 0.003 m for a sight of 300 m length).

(2) **Errors due to refraction.** The effect of refraction on the observed readings, is opposite to that of the curvature. Refraction raises the elevations of the stations and the error is also directly proportional to the square of the horizontal distance of the station from the level. This is negligible for short sights and is generally ignored in ordinary levelling.

(3) **Errors due to wind and sun.** Due to strong wind it is always difficult to hold the staff vertical. Due to non-verticality of the staff, the observed readings are erroneous. Similarly, the wind also causes vibrations in the instrument and the bubble of the level tube does not remain central. In strong wind it is always advisable to suspend the work, to protect the level by an umberella and also the staff readings may be kept small.

The sun causes a considerable trouble if it shines on the object glass. It is recommended always to protect the objective by an umbrella. The effect of sun is also to change the length of staff due to change in temperature. But, in ordinary levelling, the change in length is negligible.

3. **Instrument errors.** Instrumental errors are caused due to:

(1) **Imperfect adjustment of the level.** In a perfectly adjusted level, the line of collimation remains horizontal when the bubble of the level tube occupies the central position. The non-adjustment of the level, makes the line of collimation either inclined upward or downward and the observed readings are either more or less. Such errors get compensated if the backsight and foresight distances are kept equal as in the case of fly-levelling. But, in the case of intermediate sights, the distances considerably differ and the readings are thrown into error by different amounts. In case of levelling on steep slopes, the back sights are either longer or shorter than foresights, the error becomes cummulative, It may be noted that error due to non-adjustment of the level, is very common and of serious nature. The level must always be carefully tested and adjusted before it is used. Care should also be taken to ensure that backsight and foresight distances are equal.

(2) **Defective level tube.** If the bubble of the level tube is sluggish, it will remain central even though the bubble axis is not horizontal. On the other hand if it is too sensitive, considerable time is spent to bring the bubble central. Irregularity of curvature of the tube is also a serious defect. The effect of defective level tube also gets neutralized if the sights are equal.

(3) **Shaky tripod.** A shaky tripod causes an instability of the instrument. It wastes a considerable time to make accurate observations. Every bolt and nut of the tripod and the screws of the foot shoes should be tightened before observations are made. Best check to test the stability of the tripod is to twist one of its legs after reading a staff and release it. Observe if the reading remains the same as before, if not, the tripod is not stable.

(4) **Incorrect graduations of the staff.** If the graduations of a staff are not perfect, this error is caused, But in ordinary levelling the error may be negligible, because the readings are generally made only correct to 0.005 m. In case of precise levelling the graduations should be compared against invar tape under magnification.

EXERCISE 5

1. Draw a neat diagrammatic sketch of a Dumpy level and describe its different parts there on.
2. (*i*) What are the different types of levels used in levelling?
   (*ii*) Explain the essential differences between them. Which instrument would you prefer and why?
3. Define the following:
   Level line, Level surface, Horizontal line, Horizontal surface, Line of collimation, Axis of telescope, Foresight, Back sight, Intermediate sight, Bench mark, Mean sea level, Height of instrument and Reduced level.
4. Differentiate between the following with neat sketches:
   (*i*) Foresight and back sight
   (*ii*) G.T.S. Bench mark and temporary bench mark
   (*iii*) Level surface and horizontal surface
   (*iv*) Line of collimation and height of instrument
   (*v*) Line of collimation and optical axis of a surveying telescope.
5. Describe the temporary adjustments of a level.
6. Describe fully the methods of reduction of levels and discuss their merits and demerits.
7. Describe a level field book for rise and fall method and explain how field notes are booked and accuracy of the reduction of levels is checked.
8. What are the common difficulties generally faced in levelling? How will you overcome each of them?
9. What are the sources of error in levelling? What precautions you will take to avoid them?
10. Compare 'line of collimation' with rise and fall' method for reducing levels.
11. The back sight reading at *A* is 3.565 m and the fore sight reading at *B* is 2.865 m. Find the difference in level of *A* and *B*.
12. The back sight reading on a levelling staff held vertically on a bench mark whose R.L. is 100.000 m was 2.965 m and the fore sight on the staff hold vertical on a rail was 0.895 m. Find the reduced level of the rail.
13. The following readings are successively taken with an instrument in levelling work:
    0.359, 0.489, 0.622, 1.758, 1.895, 2.350, 1.780, 0.345, 0.687, 1.230.
    The position of the instrument was changed after taking 4th and 7th readings.

Draw out a page of a level field book and enter the above readings properly. Assume R.L. of the first point as 85.000 m. Calculate R.L. of all the points and apply usual checks.

14. The following consecutive readings were taken with a dumpy level and a 4 m staff on a continuously sloping ground along a straight line at a common interval of 30 m.

Answers

11. 0.699 m **12.** 102.070 m

13. S*BS* – S*FS* = SRise – S Fall = 2.169, R.L. of last point = 82.831 m

14. R.L. of last point 7.955, Gradientm 48.56.

CHAPTER 6

Theodolite

6.1 INTRODUCTION

An instrument used for measuring horizontal and vertical angles accurately, is known as a *theodolite*. Theodolite is also used for prolongation of survey lines, finding difference in elevations and setting out engineering works requiring higher precision *i.e.* ranging the highway and railway curves, aligning tunnels, etc.

6.2 CLASSIFICATION OF THEODOLITES

Theodolites are primarily classified as (*i*) Transit theodolite (*ii*) Non-transit theodolite.

(1) **Transit Theodolite.** The theodolite whose telescope can be transited, is called a transit *theodolite i.e.*, it can be revolved through a complete revolution about its horizontal axis in a vertical plane.

(2) **Non-Transit Theodolite.** The theodolite whose telescope cannot be transited, is called a *non-theodolite. i.e.,* it cannot be revolved through a complete revolution about its horizontal axis in vertical plane.

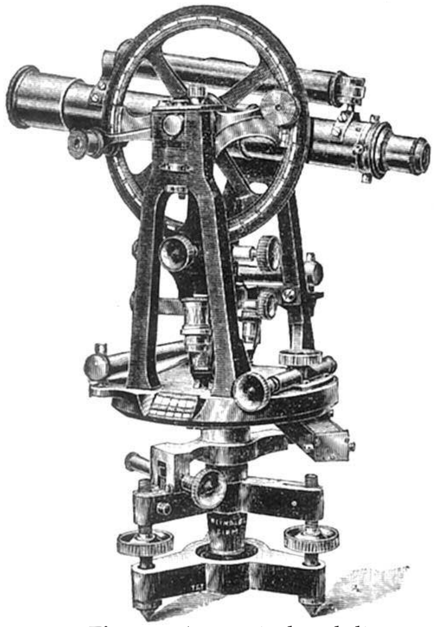


Fig. 6.1. A transit theodolite
(By courtesy kay bee and company).

Non-transit theodolites are much inferior as compared to transit theodolites. These have become almost obsolete now-a-days.

Theodolites are also classified as

(1) Vernier theodolites. (2) Glass arc theodolites.

(1) **Vernier theodolites.** In this type of theodolites, verniers are provided for reading horizontal and vertical graduated circles.

(2) **Glass arc theodolites.** In this type of theodolites, micrometres are provided for reading horizontal and vertical graduated circles.

6.3 PARTS OF A TRANSIT THEODOLITE (Fig. 6.2)

A transit theodolite consists of the following essential parts:

1. **Levelling Head.** It consists of two parts *i.e.* upper tribarch and lower tribarch.

(1) **The upper tribarch.** It has three arms. Each arm carries a levelling screw. Levelling screws are provided for supporting and levelling the instrument. The boss of the upper tribarch is pierced with a female axis in which lower male vertical axis operates.

(2) **The lower tribarch.** It has a circular hole through which a plumb bob may be suspended for centering the instrument quickly and accurately.

The three distinct functions of a levelling head are:

(*i*) To support the main part of the instrument.

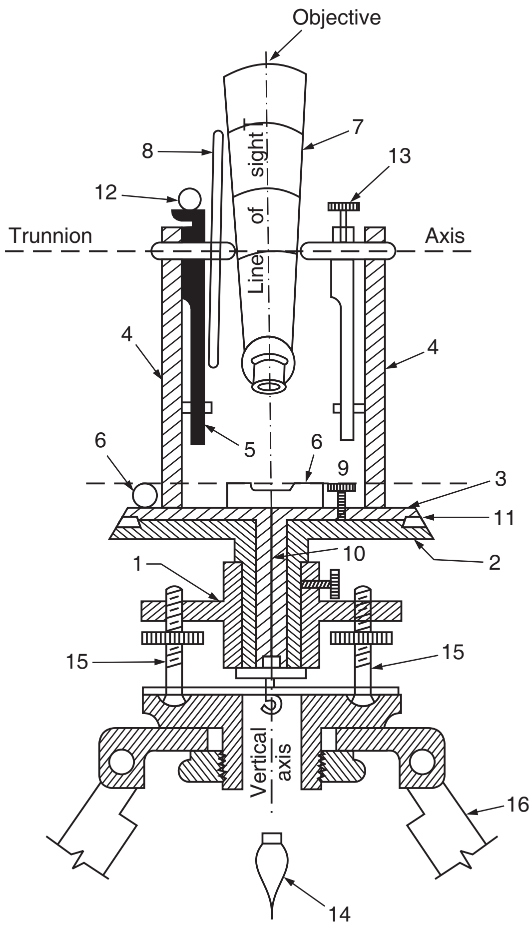
(*ii*) To attach the theodolite to the tripod.

(*iii*) To provide a means for levelling the theodolite.

2. **Lower plate (or scale plate).** The lower plate which is attached to the outer spindle, carries a horizontal graduated circle at its bevelled edge. It is therefore sometimes known as the *scale plate*. It is divided into 360°. Each degree is further divided into ten minutes or twenty minutes arc intervals. Scale plate can be clamped at any position by a clamping screw and a corresponding slow motion can be made with a tangential screw or slow motion screw.

When the lower clamp is tightened, the lower plate is fixed to the upper tribarch of the levelling head. The size of the theodolite is determined by the size of the diameter of this lower plate.

3. **Upper plate (or vernier plate).** The upper plate or vernier plate is attached to the inner spindle axis. Two verniers are screwed to the upper plate diametrically opposite. This plate is so constructed



1. Levelling head
2. Lower plate or scale 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 screw
16. Tripod

Fig. 6.2. Parts of a theodolite.

that it overlaps and protects the lower plate containing the horizontal circle completely except at the parts exposed just below the verniers.

The verniers are fitted with magnifiers. The upper plate supports the Y_s or A_s which provide the bearings to the pivots of the telescope. It carries an upper clamp screw and a corresponding tangent screw for accurately fixing it to the lower plate. On clamping the upper clamp and unclamping the lower clamp, the instrument may be rotated on its outer spindle without any relative motion between the two plates. On the other hand, if the lower clamp screw is tightened and upper clamp screw is unclamped, the instrument may be rotated about its inner spindle with a relative motion between the verniers and the graduated scale of the lower plate. This property is utilised for measuring the angle between two settings of the instrument. It may be ensured that the clamping screws are properly tightened before using the tangent screws for finer setting.

4. **The Standards (or A Frame).** Two standards resembling the English letter *A* are firmly attached to the upper plate. The tops of these standards form the bearings of the pivots of the telescope. The standards are made sufficiently high to allow the rotation of the telescope on its horizontal axis in vertical plane. The T-frame and the arm of vertical circle clamp, are also attached to the standards.

5. **T-frame or Index-Bar.** It is T-shaped and is centered on the horizontal axis of the telescope in the frame of the vertical circle. The two verniers *C* and *D* are provided on it at the ends of the horizontal arm, called the '*Index Arm*'. A vertical leg known as clipping arm is provided with a fork and two clipping screws at its lower extremity. The index and clipping arms together, are known as *T-frame*. At the top of this frame, is attached a bubble tube which is called the *altitude bubble tube*.

6. **Plate levels.** The upper plate carries two plate levels placed at right angles to each other. One of the plate bubbles is kept parallel to the trunnion axis. The plate levels can be centered with the help of the foot screws. In some theodolites only one plate level is provided.

7. **Telescope.** The telescopes may be classified as under:

(*i*) The external focussing telescope.

(*ii*) The internal focussing telescope.

(1) **The External Focussing Telescope.** This type of telescope consists of an outer and an inner tube. The outer tube is attached to the pivot by a thick metal band and the inner tube slides in the outer tube by means of a rack and pinion turned by a large milled-headscrew. With this arrangement, the telescope can be focussed to varying distances. The object glass is usually fixed at the end of the inner tube.

(2) **The Internal Focussing Telescope.** In this type of telescope, an internal focussing lens (double concave) is introduced between the objective and eye piece both of which are mounted at the ends of the tube. Focussing may be achieved by moving the internal focussing lens with a focussing screw.

The interior of the telescope tube is painted dull black to prevent reflection from the internal surface.

8. **Vertical circle.** A vertical circle is attached to the telescope. It is graduated in various ways by the manufacturers. Vertical circle is used for measuring vertical angles *i.e.* angles of elevation and depression.

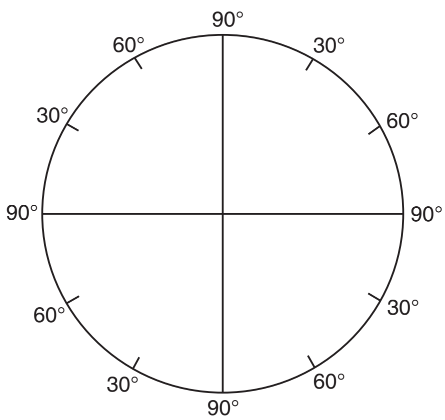


Fig. 6.3. Graduations of a vertical circle.

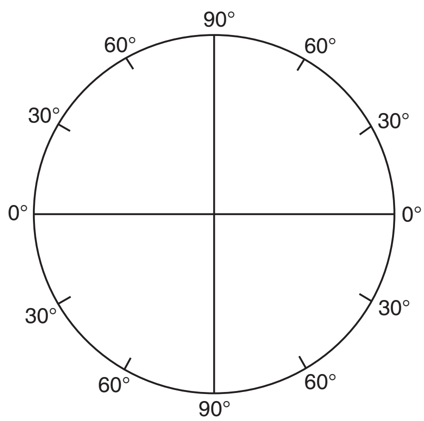


Fig. 6.4. Graduations of a vertical circle.

9. **Tripod.** While working, the theodolite is supported on a tripod which consists of three solid or framed legs. The lower ends of each leg are provided with pointed iron shoes. The tripod head carries at its upper surface an external screw to which foot plate of the levelling head may be screwed.

10. **The Plumb bob.** A plumb bob is suspended from the hook fitted to the bottom of the main vertical axis to centre the theodolite exactly over the ground station mark.

6.4 DEFINITIONS AND OTHER TECHNICAL TERMS

Following terms are used while making observations with a theodolite.

1. **Vertical axis.** The axis about which a 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 a *trunnion axis*, or *transverse axis*.

3. **Line of collimation.** The line which passes through the intersection of the cross hairs of the eye-piece and optical centre of the objective and its continuation, is called *line of collimation*. The line of collimation is also sometimes known as the *line of sight*. The angle between the line of collimation and the horizontal plane containing the horizontal axis, is called *error of collimation*.

4. **Axis of telescope.** The line joining the optical centre of the objective to the centre of eye piece, is called *axis* of *telescope*.

5. **Axis of the level tube**. The straight line which is tangential to longitudinal curve of the level tube at its centre is called *axis of the level tube*. When the bubble of the level tube is central, the axis of the level tube becomes horizontal (Fig. 6.5).

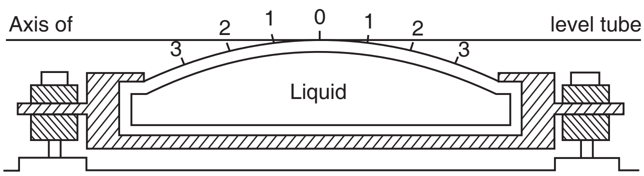


Fig. 6.5. A cross-section of a level tube.

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 180º 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.* right or left. When the telescope is rotated in clockwise (right) direction, it is known as *right swing*. If it is rotated in 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 observation*'.

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 front left to right and *vice versa*.

12. **A 'measure'.** It is the determination of the number of degrees, minutes and seconds, or grades contained in an angle.

13. **A 'set'.** A 'set' of horizontal observations of any angle consists of two horizontal measures, one on the face left and the other on the face right.

14. **Telescope normal.** A telescope is said to be normal when its vertical circle is on the left and the bubble of the telescope is up.

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

The fundamental lines of a transit are:

(1) The vertical axis.

(2) The axes of plate bubbles.

(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 the telescope bubble or altitude bubble.

6.5 TEMPORARY ADJUSTMENTS OF THEODOLITE

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:

(*i*) Setting up the theodolite over the station.

(*ii*) Levelling of the theodolite.

(*iii*) Elimination of the parallex.

1. **Setting up.** The operation of setting up a theodohite includes:

(*a*) Centering the theodolite over the ground mark.

(*b*) Approximate levelling with the help of tripod legs.

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:

(1) Suspend the plumb bob with a string attached to the hook fitted to the bottom of the instrument to define the vertical axis.

(2) Place the theodolite over the station mark by spreading the legs well apart so that telescope is at convenient height.

(3) The centering may be done by moving the legs radially and circumferentially till the plumb bob hangs within 1 cm horizontally of the station mark.

(4) By unclamping the centre-shifting arrangement, finer centering may be made.

Approximate levelling with the hep 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 levelling is done. The approximate levelling may be done either with reference to a small circular bubble provided on the tribarch or by eye judgement.

2. **Levelling of a theodolite.** The operation of making the vertical axis of a theodolite truly vertical, is known as *levelling of the theodolite*.

After having levelled approximately and centred accurately, accurate levelling is done with the help of plate levels.

Levelling with a three Screw head. Following steps are involved (Fig. 6.6).

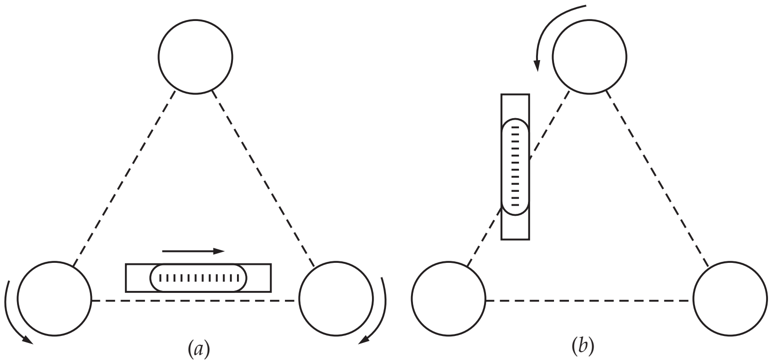


Fig. 6.6. Levelling of a theodolite with a three-screw head.

(1) Turn the horizontal plate until the longitudinal axis of the plate level is approximately parallel to a line joining any two levelling screws Fig. 6.7 (*a*).

(2) Bring the bubble to the centre 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 two foot-screws to the approximate mean of the two positions.

(5) Turn the theodolite through 90° in azimuth so that the plate level becomes perpendicular to the previous position Fig. 6.6 (*b*).

(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.

Note. If a 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 the shorter plate level central. Repeat the process till both the plate bubbles occupy the central positions of their run for all the positions of the instrument.

3. **Elimination of Parallax.** An apparent change in the position of the object caused by the change in position of the observer's eye, is known as *parallax*.

In a telescope, parallax is caused when the image formed by the objective is not situated in the plane of the cross-hairs. Unless parallax is removed, accurate bisection and sighting of objects become difficult.

Elimination of parallax may be done by focussing the eyepiece for distinct vision of cross-hairs and focussing the objective to bring the image of the object in the plane of the cross-hairs as discussed below.

Focussing the eye-piece. To focus the eye-piece for distinct vision of the cross-hairs, either hold a white paper in front of the objective or sight the telescope towards the sky. Move the eye piece in or out till the cross-hairs are seen sharp and distinct.

Focussing the objective. After cross-hairs have been properly focussed, direct the telescope on a well defined distant object and intersect it with vertical wire. Focus the objective till a sharp image is seen. Removal of the parallax may be checked by moving the eye slowly to one side. If the object still appears intersected, their is no parallax.

If, on moving the eye laterally, the image of the object appears to move in the same direction as, the observer's eye and the image of the object are on the opposite side of the vertical wire. The image of the object and the eye are brought nearer to eliminate the parallax. This parallax is called far *parallax*.

If on the other hand the image appears to move in reverse direction to the movement of the observer's eye and the image of the object are on the same side of the vertical wire. The parallax is then called *near parallax*. It may be removed by increasing the distance between the image and the eye.

6.6 USES OF A THEODOLITE

Theodolites are commonly used for

(*i*) Measurement of horizontal angles.

(*ii*) Measurement of vertical angles.

(*iii*) Prolongation of straight line.

1. **Measurement of horizontal angles with a theodolite** (Fig. 6.8)

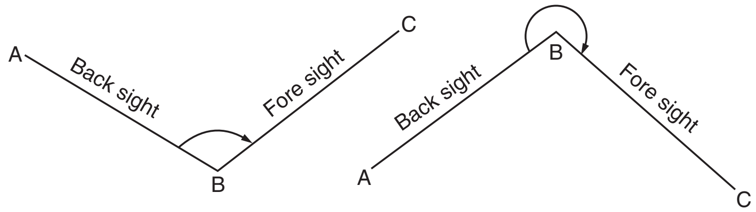


Fig. 6.7. Measurement of horizontal angles.

Procedure. To measure a horizontal angle *ABC* between sides *BA* and *BC*, the following procedure is followed:

(1) Set up, centre and level the theodolite over the ground point *B*.

(2) Loosen the upper plate, set the vernier to read zero and clamp the upper plate.

(3) Loosen the lower plate and swing the telescope until the left point *A* is sighted. Tighten the lower clamp. Accurate bisection of the arrow held on station *A* is done by using the lower tangent screw. Read both the verniers and take the mean of the readings.

(4) Unclamp the upper plate and swing the telescope in clockwise direction until point *C* is brought in the field of view. Tighten the upper clamp and bisect the mark at *C* accurately, using the upper tangent screw.

(5) Read both the verniers and take the mean of the readings. The difference of the mean of the readings to *C* and to *A*, is the required angle A*BC*.

(6) Change the face of the instrument and repeat the whole procedure. The measure of the angle is again obtained by taking the difference of the mean of the readings to *C* and *A* on face right.

(7) The mean of two measures of the angle *ABC* on two faces, is the required value of the angle *ABC*.

Note. The following points may be noted:

(*i*) To eliminate the error due to imperfect adjustment, observations on both the faces should be made for precise work.

(*ii*) To eliminate the error of eccentricity of the graduated circle and verniers, both verniers should be read.

6.7 PROLONGATION OF A STRAIGHT LINE WITH A THEODOLITE

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

First Method (Fig. 6.8)

Fig. 6.8. Prolongation of a line.

Procedure

(1) Set up the theodolite at *A*, centre and level it accurately.

(2) Intersect an arrow centred over the mark at *B*.

(3) Establish a point *C* in the line of sight at a convenient distance.

(4) Shift the instrument to *B*.

(5) Centre the theodolite over *B*, level it and sight *C*. Establish another point *D*.

(6) Proceed in a similar manner until the desired point *F* is established.

Second Method (Fig. 6.9)

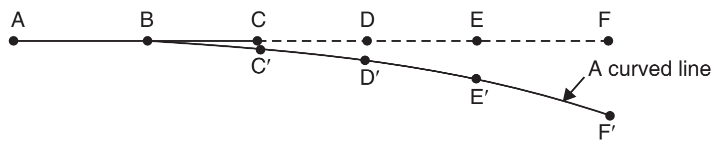


Fig. 6.9. Prolongation of a line.

Procedure:

(1) Set up the theodolite at *B* and centre it carefully.

(2) Intersect *A* accurately and clamp both the plates.

(3) Plunge the telescope and establish a point *C* in the line of sight.

(4) Shift the instrument to *C* and centre it carefully.

(5) Intersect *B* and clamp both the plates.

(6) Plunge the telescope and establish a point *D* is the line of sight.

(7) Continue the process till the last point *F* is established.

Note. The following points may be noted:

(*i*) If the instrument is in perfect adjustment, the points, *B*, *C*, *D*, *E* and *F* will lie in a straight line.

(*ii*) If the line of sight is not perpendicular to the horizontal axis, the established point *C′*, *D′*, *E′* and *F′* would lie on a curve.

Third Method (Fig. 6.10)

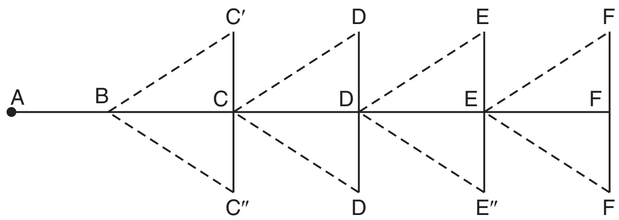


Fig. 6.10. Prolongation of a line.

Procedure. Following steps are involved:

(1) Setup the theodolite at *B* and centre it carefully.

(2) Intersect *A* on face left and clamp both the plates.

(3) Plunge the telescope and establish a point *C′*.

(4) Change the face and intersect *A* again.

(5) Plunge the telescope and establish a point *C″* at the same distance as *C′* from *B*.

(6) If the instrument is in adjustment, the point *C′* and *C″* will coincide.

(7) If not, establish a point *C* midway between *C′* and *C″*.

(8) Shift the instrument to *C* and repeat the process to establish a point *D*.

(9) Repeat the process until the required point *F* is established.

Note. This is the most accurate method of prolongation of a line.

6.8 PRECAUTIONS TO BE TAKEN WHILE USING A THEODOLITE

The following precautions must be taken by the observer:

(1) Turn the theodolite by the standards and not by the telescope, ensuring even and slow movement.

(2) Make a few revolutions of the theodolite in azimuth and that of telescope in altitude before making observations.

(3) Keep the theodolite clean. Lenses should be dusted with a brush.

(4) Lift the theodolite by its standards.

(5) Donot force the foot screws and tangent screws too hard.

(6) Ensure that movement of the observer does not affect the levelling of the instrument.

(7) Bisect the signal when it is properly fixed.

(8) Clamp the vertical axis tightly while observing the horizontal angles.

(9) Ensure that the telescope does not overshoot the signal mark. In case it does, rotate the theodolite round till the mark comes again. The final intersection of the mark should be made with tangent screw and last motion of the screw should be against the spring on swing right and in the direction of spring on swing left.

(10) Do not force the instrument in its carrying case to avoid damaged to its parts.

EXERCISE 6

1. Explain bow you will use a theodolite as a level.
2. How will you do the temporary adjustments of a theodolite?
3. Describe temporary adjustments of a theodolite. How will you measure the horizontal angle by it?
4. What is meant by face left and face right of a theodolite? How would you change face? What instrumental errors are eliminated by face right and face left observations?
5. Describe with the aid of a sketch, the function of an internal focussing lens in a surveying telescope. Also, state the advantages of internal focussing as compared to external focussing.

CHAPTER 7

Plane Table Surveying

7.1 INTRODUCTION

Planetabling is one of the methods of surveying in which field observations and plotting proceed simultaneously. For correct representation of various features on the surface of the earth by plane tabling, surveyor must be a good artist.

7.2 PRINCIPLE OF PLANETABLING

The principle of planetabling is based on the fact that the lines joining the points on the planetable, are made to lie parallel to their corresponding lines joining the ground points while working at each station. The principle of plane tabling can be best understood by considering the graphical reduction of a triangle to given dimensions. The base of the triangle is plotted on the desired scale and the base angles are plotted directly by turning the alidade at each end. The intersection of rays gives the desired location of the vertex of the triangle. *The planetabling, may be defined as graphical construction of straight lines, angles and triangles for plotting the ground detail points.*

7.3 INSTRUMENTS USED IN PLANE TABLING

The instruments required for plane tabling are as discussed under:

1. **The Plane Table** (Fig. 7.1). It consists of a wooden table mounted on a light wooden tripod in such a way that the table top (board) may be rotated about its vertical axis and can be clamped in any position. The table top is levelled by adjusting the legs of the tripod.

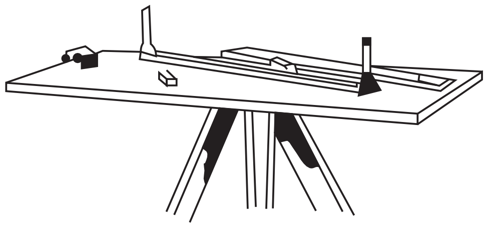


Fig. 7.1. A plane table with an alidade.

The table measures 750 mm × 600 mm and the legs of the tripod are usually 1200 mm long. The instrument is made entirely of well-seasoned wood except for the metal plate, bolts, nuts and screws, which are of brass and the shoes of the legs, which are of iron.

Qualities of a good planetable. A good plane table should possess the following qualities:

(1) The table-top should be truly flat and free from knots.

(2) The butterfly nuts which clamp the legs to the clamping head, should not be free.

(3) The clamping assembly should fit the plate at the bottom of the plane table.

(4) Annular ring should be properly fixed with the plane-table.

(5) There should not be any movement of the table top when properly clamped.

2. **The Alidade** (Fig. 7.2)

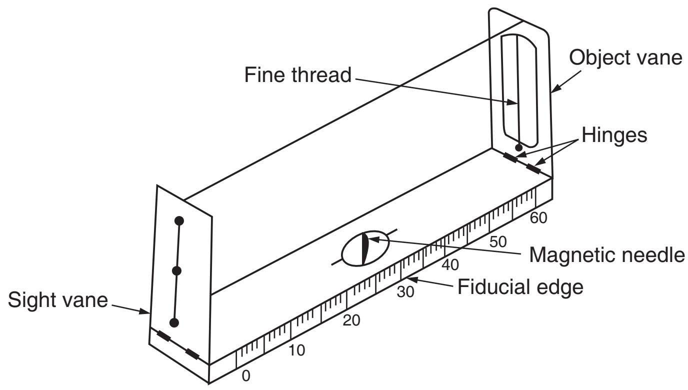


Fig. 7.2. An alidade.

It generally consists of a metal or wooden rule with two vanes at the ends. Vanes are hinged and can be folded on the rule when the alidade is not in use. One of the vanes known as *sight vane* is provided with a narrow slit with three holes, one at the top, one at the bottom and one in the middle. The other vane which is known as *object vane*, is open and carries a hair or a fine thread or a thin wire stretched between the top and bottom of the slit. With the help of the slit, a definite line of sight may be established parallel to the ruling edge of the alidade. The alidade can be rotated about the point which represents the instrument station on the sheet so that line of sight passes through the station sighted. The length of the ruling edge of the alidade should not be shorter than the longest side of the plane table. The two vanes should be perpendicular to the surface of the table. The working edge of the alidade is known as *fiducial edge*. A plane alidade can be used only when the elevations or depressions of the objects are low. If elevations of the objects are more than what can be accommodated by the line of sight, the alidade can be used by stretching a thin thread tightly between the tops of the sight and object vanes.

3. **Spirit Level** (Fig. 7.3). It consists of a small metal tube which contains a small bubble. The spirit level may also be circular but its base must be flat so that it can be laid on the table. The table is truly level when the bubble remains central all over the table.

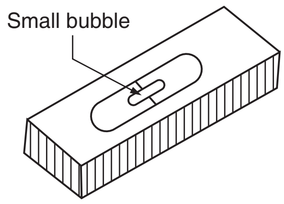


Fig. 7.3. A spirit level.

4. **The Magnetic Compass** (Fig. 7.4). A box compass consists of a magnetic needle pivoted at its centre freely. It is used for orienting the plane table to magnetic north. The edges of the box compass are straight and the bottom is perfectly flat. The dip can be temporarily adjusted by tying a rider round the elevated end. The magnetic needle should be fairly sensitive and play freely. When a compass work

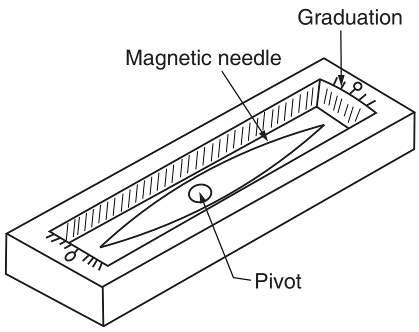


Fig. 7.4. A compass.

unsatisfactorily due to worn out *agate*, a new agate should be replaced. At the first plane table station, the longer edges of the compass are placed parallel to the sides of the plane table. The plane table is then rotated till the needle points N-S direction. A line drawn along the longer edge represents the magnetic north. In case ground control points are already plotted on the sheet. The table is set with respect to the ground control points by any one of the methods described in article 5.4.3. The box compass is rotated till its needle rests in N-S direction. A line along the edge of the compass is drawn, which defines the magnetic north.

5. **Plumbing Fork** (Fig. 7.5). The plumbing fork consists of a hair pin-shaped brass frame, having two equal arms of equal lengths. One end has a pointer while a plumb bob is attached to the other end. It is used in large scale survey for accurate centering of the station location on the table over its ground position. It is used for transferring the location of the instrument station on the sheet on the ground.

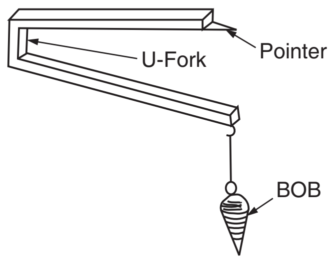


Fig. 7.5. A plumbing fork.

The fork is placed with its upper arm lying on the top of the table and the lower arm below it. The table is said to be centered when the plumb bob hangs freely over ground mark. The pointed end of the fork points the required location on the plane table. On small scale survey exact centering of the point, is not required. The centre of the table may be used as the location of the ground station.

6. **Drawing paper.** The drawing paper used for plane-tabling must be of superior quality so that it can stand erasing. It may have minimum effect of distortion due to climatic changes. Changes in humidity of the atmosphere alter the dimensions of paper in different directions differently. Sometimes drawing paper is mounted on a zinc sheet to avoid shrinkage and expansion due to atmospheric humidity.

7.4 WORKING OPERATIONS

Following three operations are carried out at each plane table station

(*i*) Fixing the plane table on the tripod.

(*ii*) Setting up the plane table.

(*iii*) Sighting the ground stations and intersected points.

1. **Fixing the plane table on the tripod.** In this operation, leather scrap of the tripod, is unfolded and legs of the tripod are well spread. The tripod is held so that its top height is roughly 1.2 m about the ground level. The bolt is removed from the brass annular ring and table top is placed on the top of the tripod so that it fits well with clamping assembly of the tripod. The bolt with a washer is then tightened.

2. **Setting up the plane table.** The setting up operation consists of the following:

(*i*) Levelling the plane table

(*ii*) Centering the plane table

(*iii*) Orientation of the plane table.

(1) **Levelling.** In this operation, the table top is made truly horizontal. For rough and small scale work, levelling can be done by eye estimation whereas for accurate and large scale work, levelling is achieved with an ordinary spirit level. The levelling is specially important in hilly terrain where some of the control points are situated at higher level and some other at lower level. The dislevelment of the plan table, throws the location of the point considerably out of its true location.

Procedure. Following steps are involved:

(*i*) Set up the planetable at the convenient height (nearly 1.2 metres) by spreading the legs to keep the table approximately, levelled, ensuring that location of the occupied station, is also roughly centered over its ground position.

(*ii*) Rotate the plane table about its vertical axis till its longer edge is parallel to the line joining the shoes of any two legs of the tripod. Place the third leg pointing towards the observer in between the surveyor's legs.

(*iii*) Place a spirit level on the plane table such that its longitudinal axis is parallel to longer edges of the table. With the help of the third leg, by moving it right or left, bring the bubble of the spirit level central.

(*iv*) Next place the spirit level perpendicular to its previous position. With the help of the third leg, by moving it forward or backward, bring the bubble of the spirit level central.

(*v*) Rotate the table top through 180°. Check if the bubble remains central in all positions.

(*vi*) Repeat the above procedure if found, necessary.

(2) **Centering.** In this operation, the location of the plane table station on the paper, is brought exactly vertical above the ground station position. For rough and small scale work, exact centering of the station, is not necessary and centre of the table may be centered over the ground position.

Procedure. Place one end of the U-Fork touching the plotted location and the plumb bob hanging from the other end below the table, points towards the ground point. In case it does not, shift the plane table bodily such that the plumb bob is exactly over the ground station without disturbing levelling. Before centering is done, the table should be roughly oriented otherwise centering might be disturbed when orientation is done.

(3) **Orientation.** In this operation, the plane table is set at a station such that its edges make a fixed angle with a fixed direction. The fixed direction is known as the *meridian*. In case, the table is not correctly oriented at each station, the locations of detail points obtained by any one of the methods of planetabling *i.e.* Radiation, Resection or Intersection described in article No. 7.5, will not represent their correct relative positions. The *main principle of planetabling is based on the fact that the lines joining the locations of the ground stations on the sheet, are made parallel to their respective ground lines.* This is achieved by the process of orientation which involves rotation of the table about its vertical axis in azimuth. The operation of orientation is sometimes called "*Setting the plane table*". As already discussed, the process of orientation disturbs the centering and *vice versa*. For accurate and large scale work, centering must be checked before orientation. Some times, both the process of centering and orientation, are repeated till the two required conditions are satisfied.

Orientation of a planetable may be done by the following methods:

(*i*) Orientation with a magnetic compass.

(*ii*) Orientation with a back ray.

Method 1. Orientation with a Magnetic compass. In case true north is not known at the plane table station, magnetic north is sometimes used as reference *i.e. meridian*. At the starting station, the table is set such that the centre area falls on it. Place a box magnetic compass such that its magnetic needle rests in N-S directions. Draw a pencil line along the longer edge of the box. On subsequent stations, after levelling and centering the table over the ground mark, the magnetic compass is laid along the drawn magnetic north. The table is then rotated until the needle rests in N-S direction. Clamp the table.

The table is correctly oriented in magnetic meridian if the plane table station is free from local attraction.

Method 2. Orientation with a Back Ray. In this method, a ray is drawn from the plotted location of the instrument station to the next forward station. Its extremeties are marked on both the edges of the alidade. On arrival at the forward station, the alidade is laid along the ray drawn from the previous station. The table is rotated until the line of sight intersects the previous station. This operation is termed "*setting by the back ray*". This method is independent of the defects of magnetic compass and local attraction. It is essential that the same edge of the alidade is used for drawing lines. It may also be ensured that the line *i.e.* back ray remains vertically above the ground position of the forward station.

3. **Sighting the ground Station.** In this operation, the table is accurately centred and levelled, over the ground station, The fiducial edge or working edge of the alidade is kept touching the plotted location of the instrument station. The ground control point is sighted through its sight vane so that station, thread of the object vane and the slit hole of the sight vane, all are in a straight line. The sighting operation is required for sighting all the stations or details whose locations are either known or are to be surveyed on the plane table.

7.5 METHODS OF PLANE TABLE SURVEYING

Plane table surveying may be carried out by one of the following methods:

1. **Radiation Method.** In this method, a plane table is set up at any commanding station. Detail points are plotted on their radiating lines drawn from the location of the instrument station, after reducing their respective ground distances on the desired scale of survey.

This method is suitable for the survey of small areas which can be commanded from a single station. Particularly, preparation of plans on large scales, is conveniently done by this method. This method is rarely used to survey a complete project. It is generally combined with other methods for surveying details within a chain length from the instrument station.

Procedure. The following steps are followed to locate the points from the instrument station (Fig. 7.6).

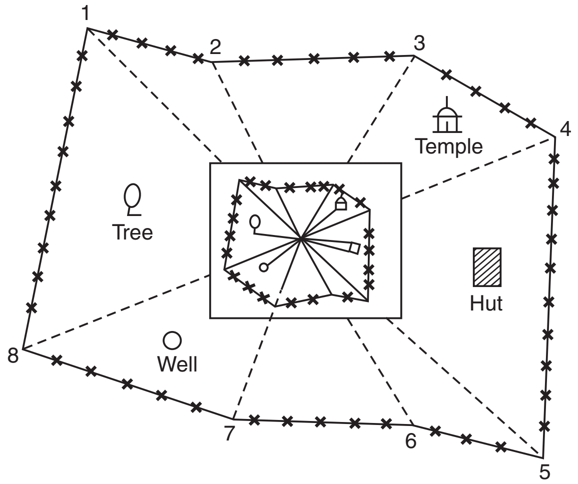


Fig. 7.6. Radiation Method.

(*i*) Set up the plane table at the station, centre and level it accurately.

(*ii*) Choose a location of the station *A* on the drawing paper at a convenient place, considering the general lay out of the area.

(*iii*) Transfer the point on to the ground by means of a plumbing U-fork for setting up the table on subsequent days.

(*iv*) Clamp the plane table tightly and draw the magnetic north with the help of a magnetic compass.

(*v*) Pivoting the alidade about *a*, the location of instrument station, sight the detail points *B, C, D, E*, etc. in turn and draw rays along the fiducial edge of the alidade.

(*vi*) Measure the *ground* distances by direct chaining and plot them on their respective lines drawn, on the desired scale. If the ground is sloping, slope correction is applied and equivalent horizontal distances are plotted.

(*vii*) Conventional symbols are drawn for different details and inked up.

2. **Intersection Method.** In this method, either coordinates of at least two accessible and intervisible points must be known or the distance between them is measured directly in the field. These points are plotted on the required convenient scale. The locations of other detail points are determined by drawing rays from each end station after proper orientation of the table. The intersection of rays gives the location of detail point. It is thus evident that it is very essential to

have at least two points whose locations are plotted before the survey may be started. *The line joining the locations of the given stations is known as the base line.* In this method, no other linear measurement is required except that of the base line. The point of intersection of the rays drawn from the ends of the base line, forms the vertex of the triangle and two rays represent the remaining two sides. The position of the vertex is determined by completing the triangle graphically. This is why this method is also known as "*Graphic triangulation*'.

Procedure. Following steps are followed to locate points by the method of intersection

(*i*) Select two points *A* and *B* on a fairly levelled ground at sufficiently large distance apart. Measure the distance *AB* directly. In case their independent coordinates are available, these can be plotted to get the locations of the two stations.

(*ii*) Plot the base line *AB* on the plane table on the desired scale in a convenient position, keeping in view the general layout of the area to be surveyed.

(*iii*) Set up the instrument on the station *A* such that its plotted location is centered over the ground point. The line *ab* is also kept approximately coincident with the ground line *AB*.

(*iv*) Level the table and plate the fiducial edge of the alidade along the line-*ab*.

(*v*) Rotate the plane table until the point *B* is sighted.

(*vi*) Check up whether the location of *A* is vertically over the ground point. If not, centre the table and repeat the step No. 5.

(*vii*) With the help of a magnetic compass, mark the north direction on the drawing sheet.

(*viii*) Pivoting the alidade about *a*, sight the points 1, 2, 3, etc. in turn and draw their corresponding rays, a_1, a_2, a_3, etc.

(*ix*) Shift the plane table to the station *B* and centre it over the ground mark.

(*x*) Placing the alidade along line *ba*, rotate the table till the station *A* is sighted. Clamp it.

(*xi*) Pivoting the alidade about *b*, sight the points 1, 2, 3, etc. Draw their corresponding rays b_1, b_2, b_3, etc. to intersect the rays previously drawn from the station *A*. The intersections of the corresponding rays, give the required locations of the points, 1, 2, 3, etc. (Fig. 7.7).

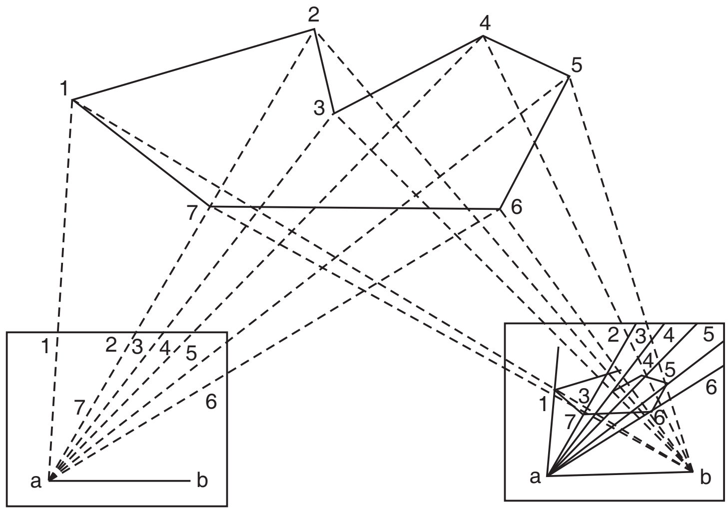


Fig. 7.7. Intersection method.

Suitability. The method of intersection is suitable when distances between detail points are either too large or can not be measured accurately due to undulations. The method is generally used for surveying the detail points. Whenever this method is used for locating other points to be used at subsequent plane table stations, the points should be got by way of intersection of at least three rays. It may be noted that the angles of intersection of different rays should not be acute to obtain accurate locations of the points. Triangles should be well conditioned. The angle of intersections of rays, should not preferably be less than 30° and not more than 120°. As no linear measurements are required in this method it can be suitably employed for surveying mountaneous regions. It may not be out of place to mention here than mapping of large areas is mostly done by the method of intersection by the department of the Survey of India. As accumulation of error is limited only to the scale of plotting of the base line, graphic triangulation can be extended to cover a large area without introducing any appreciable error.

3. **Resection Method.** The process of determining the location of-the station occupied by the plane table, by means of drawing rays from stations whose locations have already been plotted on the sheet, is called *resection*. This method which is also generally known as *Interpolation Method* or *Fixing Method* consists of drawing rays from known points whose locations are already available on the sheet. The intersection of these rays will be at a point if the orientation of the

table was correct before rays are drawn. It is seldom possible to get an accurate orientation even with a magnetic compass. The problem, therefore, lies in orienting the table at the unknown occupied station. It may be solved by one of the following methods:

(*i*) Resection by three points

(*ii*) Resection by two points.

1. Three Point Method of Resection. *"Finding the location of the station occupied by a planetable on the sheet, by means of sighting to three well defined points whose locations have previously been plotted on the sheet, is known as three point problem"*.

In this method, the plane table is set up with the help of three known points without visiting them. Let *a*, *b*, *c*, represent the locations of *A, B, C,* three ground stations and *P* represents the instrument position, the location of which is to be determined. The table is said to be oriented when rays drawn from three points *A, B* and *C* intersect at a point, and they do not. form any triangle. The point of intersection of three rays is the required location of the instrument station *P*.

The orientation of the plane table can be achieved by mechanical (or tracing paper) method as explained under:

Mechanical (or Tracing Paper) Method

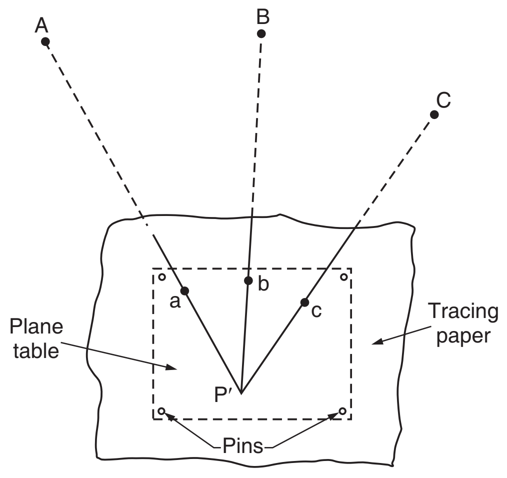


Fig. 7.8. Tracing paper method.

Let *A, B*, and *C* be three known points whose location on the sheet are respectively, *a*, *b* and *c*. The instrument station *P* is represented by *p*. (Fig. 7.8).

Procedure. Following steps are followed:

(*i*) Set up the plane table on the station *P*. Orient it roughly with the help of a magnetic compass or by eye judgement.

(*ii*) Fix a tracing paper large enough to include the locations of all the four points on the sheet, Mark a points *p′* on the tracing paper to represent the instrument position.

(*iii*) Pivoting the alidade about *p′*, sight *A, B, C* in turn and draw rays, *p′A, p′B* and *p′C* on the tracing paper.

(*iv*) Now, remove the tracing paper. Move it on the sheet in such a way that lines *p′A*, *p′B* and *p′C* are made to pass through the plotted locations *a*, *b* and *c* of the ground stations respectively.

(*v*) Prick through the point *p′* to get the location *p* of the station *P* on the sheet.

(*vi*) Align the alidade along the longest ray *pa* (assuming *A* to be the farthest point). Rotate the table until the point *A* is sighted.

(*vii*) Pivoting the alidade about the locations *b* and *c*, draw rays from stations *B* and *C*. These rays should also pass through the point *p.* This provides a check on the orientation of the table.

It may be noted that accuracy of the work depends upon the accuracy with which lines are drawn from the assumed position of the instrument station on the tracing paper and also upon the fineness of the lines drawn. This method may be used only to survey detail points. It should not be used for providing control points.

(2) **The Two Point Problem**

Statement. *"Finding the location of the station occupied by the table on the sheet by means of sighting to two well defined points whose locations have previously been plotted on the sheet, is known as the Two point problem".*

Let there be two points *A* and *B* whose locations have been plotted as *a* and *b* on the sheet. Let *C* be the instrument position whose location is required.

Procedure. (Fig. 7.9). Following steps are followed:

(*i*) Choose an auxiliary point *D* such that *CD* is approximately parallel and roughly equal to *AB* by eye judgement.

(*ii*) Orient the table over the point *C* such that locations, *a*, and *b* lie parallel to their ground positions *A* and *B*. This can be done by eye judgement. Clamp the table.

(*iii*) Pivoting the alidade about *a* and *b*, draw rays to intersect at *c* (1st Position), The degree of accuracy of the location *c* thus obtained, depends upon the approximation that has been made in the orientation.

Transfer the point *c* on the ground with a U-fork and fix a wooden peg.

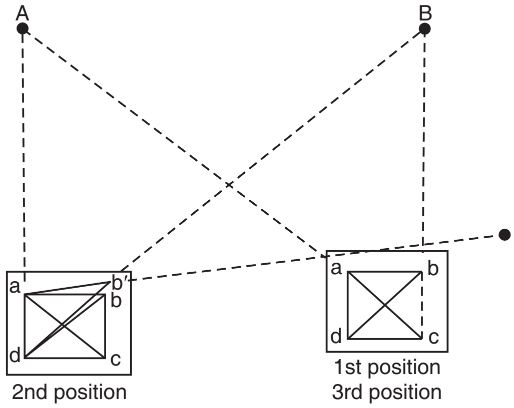


Fig. 7.9. Two point problem.

(*iv*) Pivoting the alidade about *c*, sight the station *D* and draw a ray. Also draw ray at the extremties of the alidade.

(*v*) Shift the table to station *D* and orient it accurately with the back ray method, ensuring that the ray *cd* passes through a point vertically above the ground make *D*. (2nd Position)

(*vi*) Pivoting the alidade about *a* and *b*, draw rays from *A* and *B* which will intersect on the line drawn from *C* if the orientation was correct.

(*vii*) If not, pivoting the alidade about the point of intersection of the rays drawn from *C* and obtained from *A* sight the station *B*. Draw a ray to cut *cb* (or produced) at *b*′.

(*viii*) Align the alidade along the line *ab*′ and fix a point *E* in the line of sight, at a great distance.

(*ix*) Align the adidade along *ab* and rotate the table until the point *E* is again sighted. Clamp the table.

(*x*) Pivoting the alidade about *a* and *b*, draw rays from *A* and *B* to intersect at *d*. Pivoting the alidade about *d* draw a ray towards *C*.

(*xi*) Shift the table to *C* (1st Position). Orient it with a back ray method. Pivoting the alidade about *a* and *b*, draw rays from *A* and *B*. The rays intersect on the line drawn from *D*, to give the correct location of the station *C*.

Suitability. The accuracy of the interpolation by two point problem depends upon the selection of the station *D* which should lie on a line approximately parallel to *AB*. *CD* should also be at equal to distance as *AB*. The resection by two point problem does not give much accurate result. Moreover, labour and time are wasted to set the table at two stations to achieve the orientation.

7.6 ADVANTAGES AND DISADVANTAGES OF PLANE TABLING

Advantages. Following are the advantages:

1. Less number of control points are required as extension of planimetric control is provided by plane tabling itself while the survey proceeds.

2. Depiction of irregular details and contours can be done accurately as the entire area remains in view during survey.

3. As the numerical values of angles as well as linear measurements are not observed, the errors and mistakes due to reading, recording and plotting, are eliminated.

4. As plotting is done in the field itself, chances of omissions of important measurements, are avoided.

5. The principles of intersection and resection are conveniently employed to avoid computation.

6. Checking of plotted details can be done easily.

7. The amount of office work is practically reduced to nil.

8. It is less costly as compared to other methods of surveying.

Disadvantages. Following are the disadvantages:

1. The plane table and its accessories are cumbersome which are required to be carried in the field.

2. Considerable time is required for a surveyor, to gain proficiency in plane tabling.

3. The time required to survey the area in the field, is comparatively more.

4 The method can only be used in open country with clear visibility.

5. The rainy season and cold wind affect the progress of survey considerably.

EXERCISE 7

1. (*a*) Enumerate the instruments used in plane tabling.

(*b*) Describe the construction and requirement of each instrument used in plane tabling with neat diagrams.

(*c*) Describe the qualities of a good planetable.

(*d*) How will you check the accuracy of a plain alidade.

2. (*a*) Narrate the working operations of plane tabling at each station and describe each briefly.

(*b*) Describe the method of orientation with a back ray.

3. (*a*) Define the following terms:

(*i*) Radiation (*ii*) Intersection

(*iii*) Resection.

(*b*) What is the basic difference in the above three methods?

4. Explain the methods used for plane tabling. Under what conditions is each of these preferred to?

5. (*a*) Discuss the advantages and disadvantages of plane table surveying over other methods of surveying.

(*b*) What is three point problem and how it is solved by the tracing paper method?

6. (*a*) Describe two methods of orienting a plane table.

(*b*) State the various sources of error in plane table surveying.

7. State a two point problem and show how it may be solved.

8. What is meant by two-point problem ? Explain by sketches how you would solve it in the field. (No compass is available).

9. Enumerate the various sources of error in plane tabling. How will you guard against them?

CHAPTER 8

Contouring

8.1 INTRODUCTION

Contouring is one of the methods of depicting topography of the terrain. It is the most efficient method for depicting relief in mountaineous regions.

8.2 CONTOUR, CONTOUR INTERVAL AND HORIZONTAL EQUIVALENT

1. **Contour.** An imaginary line, on the ground, joining the points of equal elevation above the assumed datum, is called a *contour*. It is a plan projection of the plane passing through the points of equal height on the surface of the earth. Concept of a contour can be made clearly by surveying the boundary of still water in a pond. If the level of the water surface is 100 m, then the periphery of water represents a contour of 100 metres. Now imagine that water level is lowered by 5 metres, the new periphery of water will then represent a contour of 95 m. (Fig. 8.1).

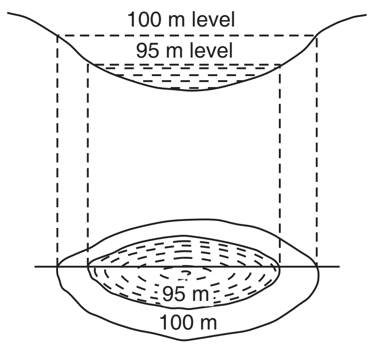


Fig. 8.1. Contours.

2. **Contour Interval.** The vertical distance between any two consecutive contours, is called *contour interval*. It is kept the same on a map to depict correct topography of the terrain.

3. **Horizontal Equivalent.** The least-horizontal distance between two consecutive contours, is called *horizontal equivalent*. It is different for different contours and is dependent on the slope of the ground surface. It is comparatively less in hills than in plains.

8.3 CHARACTERISTICS OF CONTOURS

The following characteristics of contours are kept in view while preparing or reading a contour map.

(1) Two contours of different elevations do not cross each other except in the case of an overhanging cliff.

(2) Contour of different elevations do not unite to form one contour except in the case of a vertical cliff.

(3) Contours drawn closer depict a steep slope and, if drawn far apart, represent a gentle slope.

(4) Contours equally spaced depict a uniform slope. When contours are parallel, equidistant and straight, these represent an inclined plane surface.

(5) Contour at any point is perpendicular to the line of the steepest slope at the point.

(6) A contour line must close itself but need not be necessarily within the limits of the map itself.

(7) A set of ring contours with higher values inside, depict a hill whereas a set of ring contours with lower values inside, depict a pond or a depression without an outlet.

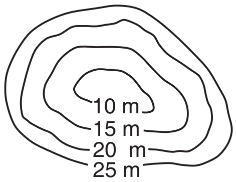
(8) When contours cross a ridge or *V*-shaped valley, they form sharp *V*-shapes across them. Contours represent a ridge line, if the concavity of higher value contour lines is towards the next lower value contour and on the other hand contours represent a valley if the concavity of the lower value contour lies towards the higher value contour.

(9) The same contour must appear on both sides of a ridge or a valley.

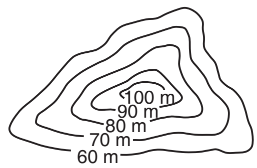
(10) Contours do not have sharp turnings.

8.4 CONTOURS OF NATURAL FEATURES

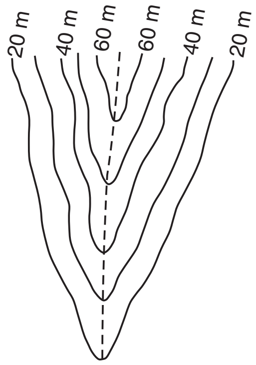
Keeping in view, the characteristics of contours enumerated above, different natural features may be shown by contours as under:



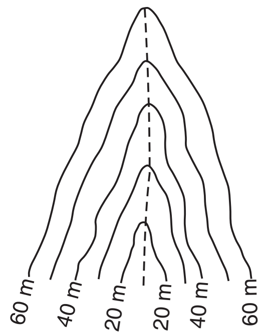
(*a*) A pond or a depression without an outlet



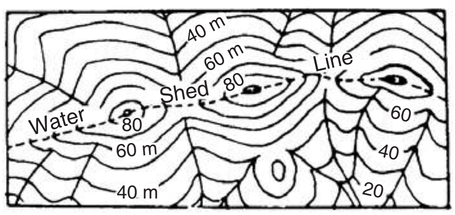
(*b*) A hill or a mound



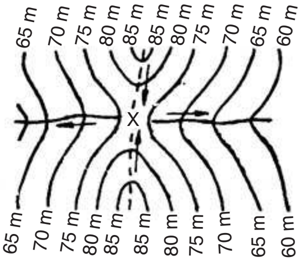
(*c*) A ridge line



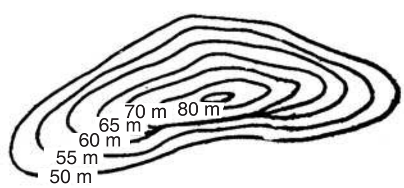
(*d*) A valley



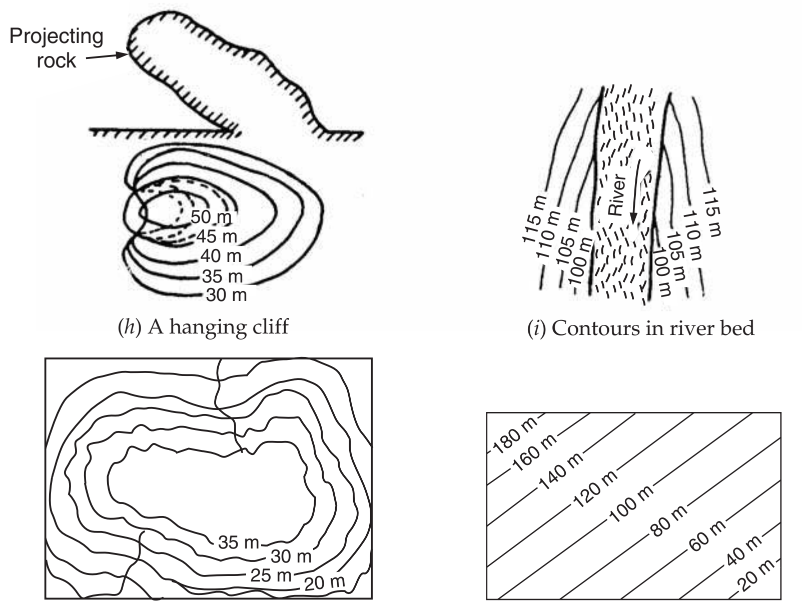
(*e*) A water shed line



(*f*) A pass on a saddle



(*g*) A vertical cliff



(*h*) A hanging cliff

(*i*) Contours in river bed

(*j*) A plateau

(*k*) An inclined plane

Fig. 8.2. Contours of natural features.

8.5 MEHTHOD OF CONTOUR INTERPOLATION

The following methods are generally used:

(*i*) Interpolation by square method.

(*ii*) Intepolation by cross section method.

1. **Interpolation by square method.** In this method, the entire area is divided into a number of squares, the sides of which may vary from 5m to 25 m, depending upon the nature of the ground, the contour interval and the scale of the plan. The squares may not be of the same size throughout but may vary according to the requirements of the map. The corners of the squares are marked on the ground and spot levels of these points are given with a level by normal method of levelling. Special care is to be taken to give spot levels to the salient features of the ground such as hill tops, deepest point of the depressions, etc. and their measurements from the respective corners of the squares are noted.

The squares are plotted on the desired scale of the plan and reduced levels of the corners as well as that of the salient features are entered. The contours of desired values are then interpolated as shown in Fig. 8.3.

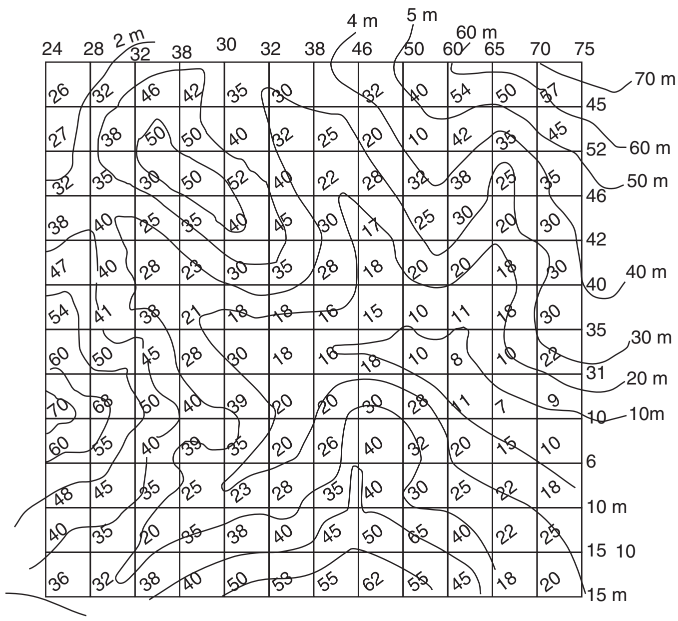


Fig. 8.3. Locating contours by the method of squares.

Suitability. This method is suitable in low undulations without any vegetative covers.

2. **By Cross-Section Method.** In this method, cross-sections perpendicular to the centre line of the area are set out. The spacing of the cross-section depends upon the contour interval, scale of the plan and the characteristics of the ground. In general, spacing of cross-sections at 20 m in hilly country and 100 m in flat country are adopted. Points of

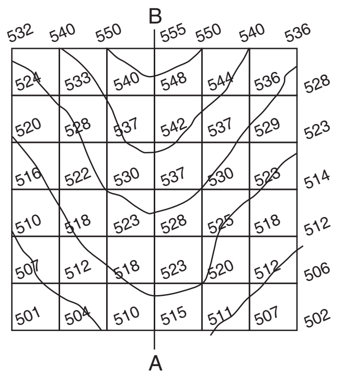


Fig. 8.4. Locating contours by method of cross-section

salient features along the centre line and on cross-sections are also located. The layout of the cross-sections need not necessarily be at right angles to the centre line. These may be inclined at suitable angles to the centre if found necessary. First plot the centre line and cross-sections on the desired scale and line and cross-sections the desired scale and enter their reduced levels. The contours are then interpolated with respect to these levels. (Fig. 8.4).

8.6 USES OF CONTOUR MAPS (FIG. 8.5)

Contour maps are used for the following:

(1) To study the general character of the tract of the country without visiting the ground. With the knowledge of the characteristics of the contours, it is easier to visualise whether the country is flat, undulating or mountaineous.

(2) To decide the most economical and suitable sites for engineering works such as canals, sewers, reservoirs, roads, railways etc.

(3) To determine the catchment area of the drainage basin and hence the capacity of the proposed reservoir.

(4) To compute the earth work required for filling or cutting along the linear alignment of projects such as canals, roads, etc.

(5) To ascertain the intervisibility of points.

(6) To trace a contour gradient for the road alignment.

(7) To draw longitudinal sections and cross-sections to ascertain the nature of the ground.

(8) To calculate the water capacities of reservoirs.

(9) To decide the best positions of the guns, the line of march and camping grounds by the army commanders during wars.

8.7 TOPOGRAPHICAL MAP

A portion of a topographical map is shown in Fig. 8.5.

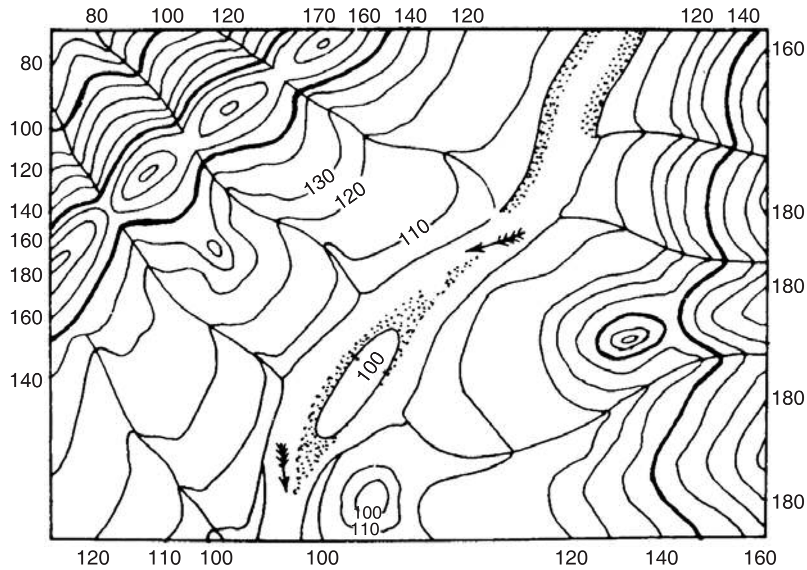


Fig. 8.5. A topographical map.

EXERCISES

1. Define contour. What do you understand by contour interval and on what factors does it depend?

2. What is meant by "Contour interval"? Name the factors that govern the selection of the contour interval and describe how these affect the choice.

3. Show with neat sketches, the characteristic features of contour lines of the following:

(*i*) An overhanging cliff ; (*ii*) A pond ; (*iii*) A depression; (*iv*) A ridge line ; (*v*) Area having flat slope ; (*vi*) A valley ; (*vii*) A saddle or pass; (*viii*) A vertical cliff ; (*ix*) A plateau ; (*x*) A plain with a knoll.

4. (*a*) Define contours and give characteristics of contours.

(*b*) Describe the method of plotting contours by taking spot levels in the field.