INSTITUTE OF TEXTILE TECHNOLOGY

CHOUDWAR

SUB-LAND SURVEY-I

BRANCH-CIVIL ENGG.

SEM-4th

PREPARED BY

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INTRODUCTION TO SURVEYING, LINEAR MEASUREMENTS

CH-1.

SURVEYING :—

"SURVEYING" is the art of determining the relative positions of different objects on the surface of the earth by measuring the horizontal distances between them, and by preparing a map to any suitable scale. Thus, the measurements are taken only in the horizontal plane.

AIMS AND OBJECTIVES :—

The aim of surveying is to prepare a map to show the relative positions of the objects on the surface of the earth. The map is drawn to some suitable scale. It shows the natural features of a country, such as towns, villages, roads, railways, rivers etc. Maps may also include details of different engineering works, such as roads, railways, irrigation canals, etc.

USES OF SURVEYING :—

Surveying may be used for the following various applications

- → To prepare a topographical map which shows the hills, valleys, rivers, villages, towns, forests, etc of a country.
- → To prepare a cadastral map showing the boundaries of fields, houses and other properties.
- → To prepare an engineering map which shows the details of engineering works such as roads, railways, reservoirs, irrigation canals, etc.
- → To prepare a military map showing the road and railway communications with different parts of a country. Such a map also shows the different strategic points important for the defence of a country.
- → To prepare a contour map to determine the capacity of a reservoir and to find the best possible route for roads, railways, etc.
- → To prepare a geological map showing areas including underground resources.
- → To prepare an archeological map including places where ancient relics exist.

PRINCIPLES OF SURVEY :-

The general principles of surveying are :-

1) To work from the whole to the part, and
2) To locate a new station by the least two measurements (linear or angular) from fixed reference points.

1) According to the first principle, the whole area is first enclosed by main stations (i.e., controlling stations) and main survey lines (i.e., controlling lines). The area is then divided into a number of parts by forming well conditioned triangles. A nearly equilateral triangles is considered to be well-conditioned triangles. The main survey lines are measured very accurately with a standard chain.

→ Then the sides of the triangles are measured. The purpose of this process of working is to prevent accumulation of error. During this procedure, if there is any error in the measurement of any side of a triangle, then it will not affect the whole work. The error can always be detected and eliminated.

2) According to the second principle, the new stations should always be fixed by atleast two measurements (linear or angular) from fixed reference points. linear measurements refers to horizontal distances measured by chain or tape. Angular measurements refer to the magnetic bearing or horizontal angle taken by a prismatic compass or theodolite.

→ In chain surveying, the positions of main stations and directions of main survey lines are fixed by tie lines and check lines.

* PLANE SURVEYING AND GEODETIC SURVEYING :-

Surveying is primarily classified as

1) Plane Surveying.
2) Geodetic Surveying.

1) Plane Surveying :- We know that the shape of the earth is spheroidal. Thus, the surface is obviously curved. But in plane surveying, the curvature of the earth is not taken into consideration. This is because plane surveying is carried out over a small area. So, the surface of the earth is considered as plane. In such surveying, a line joining any two points is considered to be straight. The triangle formed by any three points is considered as a plane triangle

and the angles of the triangle are assumed to be plane angles. Plane Surveying is conducted by State agencies like the Irrigation Department, Railway Department, etc. Plane Surveying is done on an area of less than 250 km².

2> Geodetic Surveying :— In geodetic surveying, the curvature of the earth is taken into consideration. It is extended over a large area. The line joining any two points is considered as a curve line. The triangle formed by any three points is considered to be spherical and the angles of the triangle are assumed to be spherical angles.

→ Geodetic Surveying is conducted by the Survey of India Department, and is carried out over an area exceeding 250 km².

* CLASSIFICATION OF SURVEY :— (SECONDARY)

Surveys may be classified under the uses or purpose of the resulting maps.

(A) Classification based upon the nature of the field survey:—

(1) Land Surveying :—

(i) Topographical Surveys :— This consists of horizontal and vertical location of certain points by linear and angular measurements and is made to determine the natural features of a country such as rivers, streams, lakes, woods, hills, etc, and such artificial features as roads, railways, canals, towns and villages.

(ii) Cadastral Surveys :— Cadastral Surveys are made incident to the fixing of property lines, the calculation of land area, or the transfer of land property from one owner to another. They are also made to fix the boundaries of municipalities and of state and Federal jurisdictions.

(iii) City Surveying :— They are made in connection with the construction of streets, water supply systems, sewers and other works.

(2) Marine or Hydrographic Survey :—

Marine or Hydrographic Survey deals with bodies of water for purpose of navigation, water supply, harbour works or for the determination of mean sea level. The work consists in measurement of discharge of streams, making topographic survey of shores and banks, taking and locating soundings to determine the depth of water and observing the fluctuations of the ocean tide.

(3) Astronomical Survey :—

The astronomical survey offers the surveyors means of determining the absolute location of any point or the absolute location and direction of any line on the surface of the earth. This consists in observations to the heavenly bodies such as the sun or any fixed star.

(B) Classification based on the object of survey :—

(1) Engineering Survey :—

This is undertaken for the determination of quantities or to afford sufficient data for the designing of engineering works such as roads and reservoirs, or those connected with sewage disposal or water supply

(2) Military Survey :—

This is used for determining points of strategic importance.

(3) Mine Survey :—

This is used for the exploring mineral health

(4) Geological Survey :—

This is used for determining different strata in the earth's crust

(5) Archaeological Survey :—

This is used for unearthing relics of antiquity.

(C) Classification based on Instrument Used :—

An alternative classification may be based upon the instruments or methods employed, the chief types being :—

(1) Chain Surveying
(2) Theodolite Survey
(3) Traverse Survey
(4) Triangulation Survey
(5) Tacheometric Survey
(6) Plane table Survey
(7) Photogrammetric Survey
(8) Aerial Survey

* PRECISION AND ACCURACY OF MEASUREMENT :—

→ Precision is the degree of perfection used in the instruments, the methods and the observation

→ Accuracy is the degree of perfection obtained. Accuracy depends on (1) Precise instruments
(2) Precise methods , and
(3) Good planning

The use of precise instruments simplify the work, save time and provide economy. The use of precise methods eliminate or try to reduce the effect of all types of errors. Good planning, which includes proper choice and arrangement of survey control and the proper choice of instruments and methods for each operati-on, saves time and reduces the possibility of errors.

* INSTRUMENTS USED FOR MEASUREMENT OF DISTANCE :—

There are various methods of making linear measurements and their relative merit depends upon the degree of precision requi-red.

They can be divided into three heads

(1) Direct measurements.
(2) Measurements by optical means. (Tacheometric Surveying)
(3) Electromagnetic methods (Electromagnetic Distance Measurement)

(1) Direct measurements :—

The various methods of measuring the distances directly are as follows :—

(a) By pacing or stepping.
(b) Measuring with passometer.
(c) Measuring with perambulator.
(d) Measuring by speedometer.
(e) By chaining.

(a) By pacing or stepping :— For rough and speedy work, distances are measured by pacing, i.e., by counting the walking steps of a man. The walking step of a man is considered as 2.5 ft or 80 cm. This method is generally employed in the reconnaisance survey of any project.

(b) By passometer :— A small instrument just like a stop watch, the passometer is used for counting the number of steps automatically by some mechanical device. It offers an improvement over the normal pacing method when a very long distance is to be measured and when it becomes very tedious to count and extremely difficult to remember the number of steps.

(c) By speedometer :— ~~It is a wheel fitted with a fork and handle. The~~

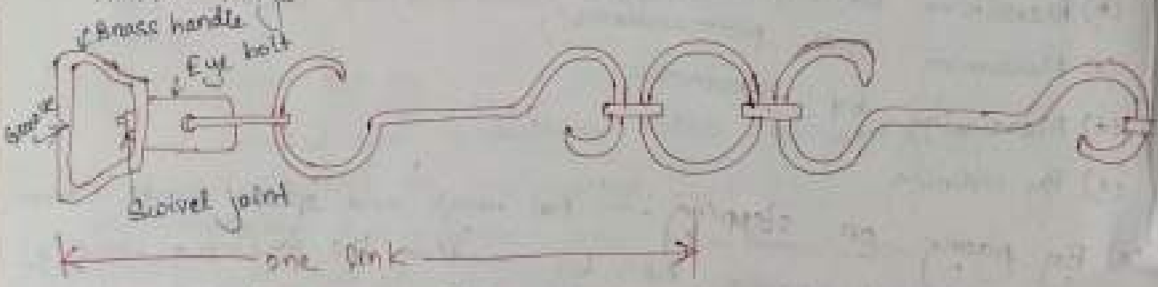
It is used in automobiles for recording distances.

(d) By perambulator :— It is a wheel fitted with a fork and handle. The wheel is graduated and shows a distance per revolution. There is a dial which records the number of revolutions. Thus the distance can be ascertained.

(e) By chaining :— This is an accurate and common method of measuring distance. In this method, the distances are directly measured in the field by chain or tape.

* CHAINS :—

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



(chain)

(Tallies)

* Types of chain :—

The following are the different types of chains :—

(a) Metric chain → 20m and 30m.
(b) Steel Band → 20m or 30m.
(c) Engineer's Chain → 100 ft.
(d) Gunter's chain → 66 ft.
(e) Revenue chain → 33 ft.

* ERRORS AND MISTAKES IN LINEAR MEASUREMENT :—

True Error :— The difference between a measurement and the true value of the quantity measured.

Mistake :— Mistakes are errors which arise from inattention, inexperience, carelessness and poor judgment or confusion in the mind of the observer. If a mistake is undetected it produces a serious effect upon the final result. Hence every value to be recorded in the ~~field~~ field must be checked by some independent field observation.

* SOURCES OF ERRORS IN MEASUREMENT :—

While dealing with any kind of measurements, it is important to identify the possible sources of error. It helps to maintain the accuracy of physical measurement in both fields and labs. There are mainly three possible sources of errors.

→ **Instrumental :—** The imperfection or faulty adjustment of the measurement instrument may cause errors.

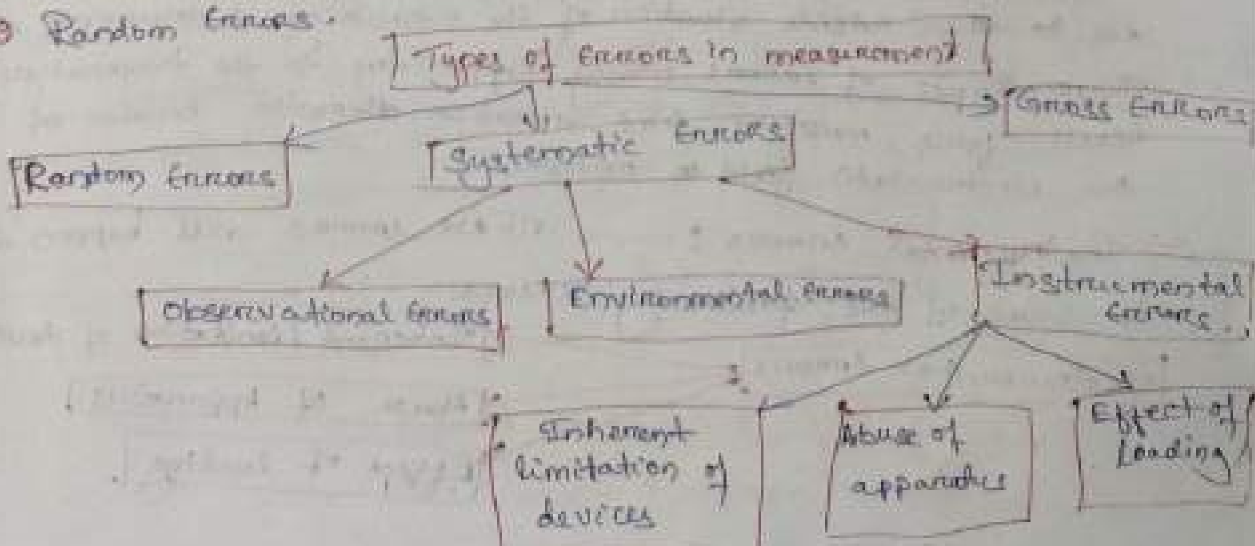
→ **Personal :—** Error may also arise due to imperfection of human sight in observation and of touch in manipulating instruments. These man made error are known as personal error.

→ **Natural :—** Variation of natural phenomena is also a possible source of error. Variation of temperature, humidity, gravity, wind, refraction, magnetic declination etc. are most common natural phenomena which cause measurement errors. If they are not properly observed while taking measurements, the results will be incorrect.

* CLASSIFICATION OF ERRORS AND MISTAKE IN MEASUREMENT

The errors in measurement may happen from the various sources which are generally categorized into the following types. These are :—

① Systematic Errors.
② Gross Errors.
③ Random Errors.



(1) Gross Errors :—

It can be defined as physical errors in analysis apparatus or calculating and recording measurement outcomes. In general, these type of errors will happen throughout the experiments, wherever the researcher might study or record a worth different from the real one, possibly due to reduced view. With human concern types of errors well predictable, although they can be estimated and corrected.

→ These types of errors can be prohibited by the following actions:

* Careful reading as well as a recording of information.
* Taking numerous readings of the instrument by different operators.

(2) Random Errors :—

This type of error is constantly there in a measurement, which is occurred by essentially random oscillation in the apparatus measurement analysis or in the experimenter's understanding of the apparatus reading.

These types of errors shows up as dissimilar outcomes for apparently the similar frequent measurement, which can be expected by contrasting numerous measurements, with condensed by averaging numerous measurements.

(3) Systematic Errors :—

These types of errors are generally categorized into three types

Systematic errors →
- Observational Errors
- Environmental Errors
- Instrumental Errors.

(a) Observational Errors :— These errors may occur due to the fault study of the instrument reading, and the sources of these errors are many.

(b) Environmental Errors :— Environmental errors will happen due to the outside situation of the measuring instruments. These types of errors mostly happen due to the temperature, result, force, moisture, dirt, vibration otherwise because of the electrostatic field or magnetic

(c) Instrumental Errors :— These errors will happen due to some of the following reasons.

Instrumental Errors →
- Inherent limitation of devices
- Abuse of Apparatus.
- Effect of loading.

(i) An inherent limitation of devices :— These errors are integral in devices due to their features namely mechanical arrangement. These may happen due to the instrument operation as well as the operation or computation of the instrument. These types of errors will make the mistake to study very low otherwise very high.

(ii) Abuse of Apparatus :— The error in the instrument happens due to the machinist's fault. A superior device, used by an unintelligent method by may provide a vast result.

(iii) Effect of Loading :— The most frequent types of this error will occur due to the measurement work in the device. For instance type of this error will occur due to the measurement work in the device.

Remedies :—

→ Double check all measurements for accuracy. For example, double-enter all inputs on two worksheets & compare them.

→ Double check your formulas are correct.

→ Make sure observers and measurement takers are well trained.

→ Make the measurement with the instrument that has the highest precision.

→ Take the measurements under controlled conditions.

→ Use multiple measures for the same construct. For example, if you are testing for depression, use two different questionnaries.

Corrections to measured Length :—

→ Correction applied to incorrect length :—

If the chain or tape is too long the measured distance will be less. The error will therefore, be 'negative' and the correction is 'positive'.

Similarly, if the chain or tape is too short, the measured distance will be more, the error will be 'positive' and the correction is 'negative'.

$$\text{True Length of Line (TL)} = \left(\frac{L'}{L}\right) \times \text{measured Length (ML)}$$

where,

L = standard or true length of chain.

L' = True length ± error.

$= L \pm e$

e = error in chain or tape, i.e., when it is too long or too short.

* Correction of incorrect chain :—

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

* Correction to volume :—

True volume $= \left(\frac{L'}{L}\right)^3 \times$ measured volume.

* Hypotenusal allowance :—

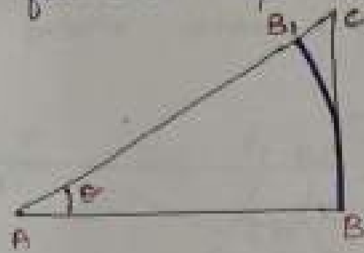
Hypotenusal allowance per tape $= L(\sec\theta - 1)$

where, L = Length of tape.

θ = slope of the ground.

2) APPLYING HYPOTENUSAL ALLOWANCE :-

In this method, the slope of the ground first found out by using the clinometer or Abney level. Hypotenusal allowance then made for each tape length.



$AB = AB_1 = 20m = 100$ links

$AC = AB \sec\theta = 150 \sec\theta$

$B_1C = AC - AB_1$

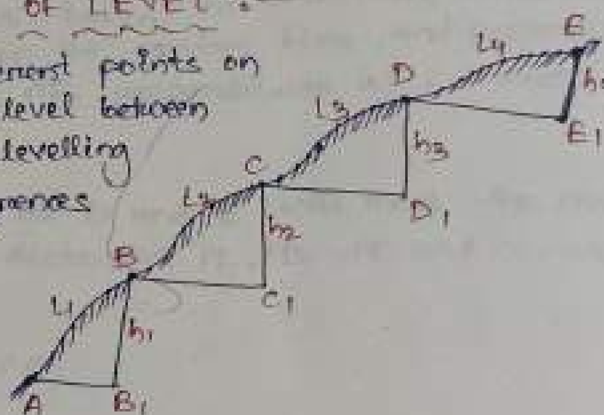
$= 100\sec\theta - 100$

$= 100(\sec\theta - 1)$

The amount $100(\sec\theta - 1)$ is said to be the hypotenusal allowance.

While chaining along the slope, one chain would be actually located at B_1. But the arrow should be placed at C after making hypotenusal allowance. The next chain length will start from C. The same principle is followed until the end of the line is reached.

3) KNOWING THE DIFFERENCE OF LEVEL :-



Suppose A, B, C, D, & E are different points on sloping ground. The difference of level between these points is determined by a levelling instrument. Let the respective differences be h_1, h_2, h_3, h_4. Then the sloping distances AB, BC, CD, & DE are measured. Let the distances $L_1, L_2, L_3, \& L_4$ respectively.

The required horizontal distances are given by

$AB_1 = \sqrt{L_1^2 - h_1^2}$, $BC_1 = \sqrt{L_2^2 - h_2^2}$, $CD_1 = \sqrt{L_3^2 - h_3^2}$

$DE_1 = \sqrt{L_4^2 - h_4^2}$

Total horizontal distance $= AB_1 + BC_1 + CD_1 + DE_1$

* OBSTACLE IN CHAINING :

A chain line may be interrupted in the following situations:

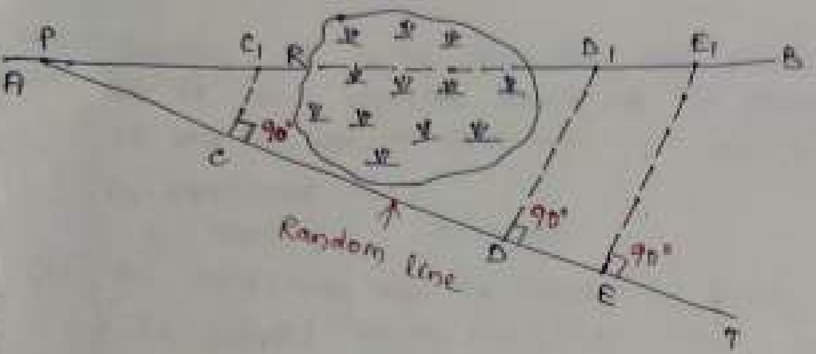
(1) When chaining is free, but vision is obstructed.
(2) When chaining is obstructed, but vision is free, and
(3) When chaining and vision are both obstructed.

(1) When chaining is free, but vision is obstructed :-

Such a problem arises when a rising ground or a jungle area interrupts the chain line. Here the end stations are not intervisible. There may be two cases.

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

Case II :— The end stations are not visible from intermediate points when a jungle area comes across the chain line.



let AB be the actual chain line which cannot be ranged and extended because of interruption by a jungle. let the chain line be extended upto R. A point P is selected on the chain line and a random line PT is taken in a suitable direction. Points C, D and E are selected on the random line, and perpendiculars are projected from them. The perpendicular at C meets the chain line at C_1.

Theoretically, the perpendiculars at D and E will meet the chain line at D_1 and E_1. Now the distances PC, PD, PE and CC_1 are measured.

from triangles PDD_1 and PCC_1

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

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

Again, from triangles PEE_1 and PCC_1

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

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

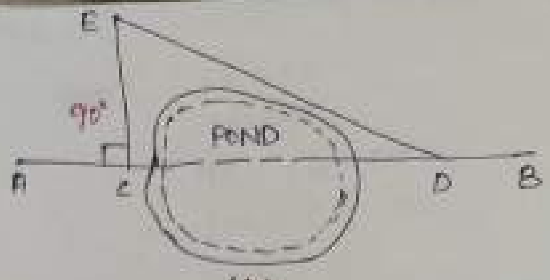
Distance $PE_1 = \sqrt{PE^2 + EE_1^2}$

(2) Chaining obstructed but vision free :—

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

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

(a)



(b)

(a) - Suppose AB is the chain line. Two points C and D are selected on it on opposite banks of the pond. Equal perpendiculars CE and DF are errected at C and D. The distance EF is measured.

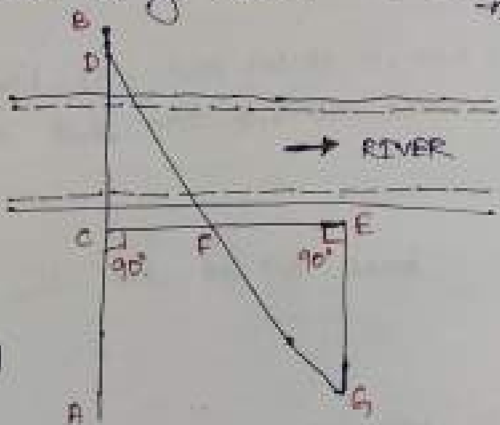
Here, $CD = EF$

(b) - The pond may also be crossed by forming a triangle. A point C is selected on the chain line. The perpendicular CE is set out at C, and a line ED is suitably taken. The distances CE and ED are measured.

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

Case-II :— Sometimes it is not possible to go around the obstruction.

(a) Imagine a small river comes across the chain line. Suppose AB is the chain line. Two points C and D are selected on this line on opposite banks of the river. At C a perpendicular CE is errected and bisected at F. A perpendicular is set out at E and a point G is so selected on it that D, F and G are in the same straight line.



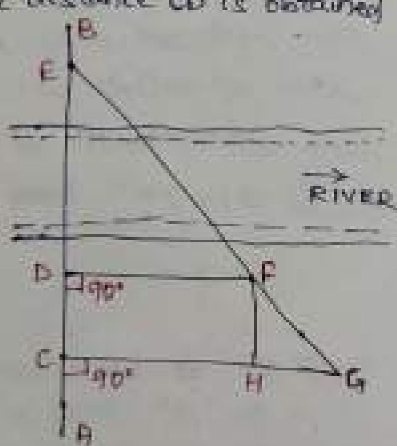
From triangles DCF and GEF,

$GE = CD$

This distance GE is measured, and thus the distance CD is obtained directly.

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

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



Now, from triangles DEF and HFG,

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

where, $FH = CD$
$CH = DF$
$HG = CG - CH$
$\therefore HG = CG - DF$

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

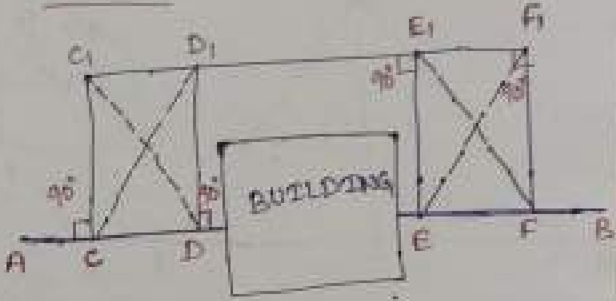
$$\boxed{ED = \frac{CD}{CG - DF} \times DF}$$

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

(3) Chaining and vision both obstructed :—

Such a problem arises when a building comes across the chain line.

Suppose AB is the chainline. Two points C and D are selected on it at one side of the building. Equal perpendiculars CC_1 and DD_1 are erected. The line C_1D_1 is extended until the building is crossed. On the extended line, two points E_1 and F_1 are selected. Then perpendiculars E_1E and F_1F are so erected that:



$$\boxed{E_1E = F_1F = D_1D = C_1C}$$

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

Here, $\boxed{DE = D_1E_1}$

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

SETTING PERPENDICULAR WITH CHAIN AND TAPE :—

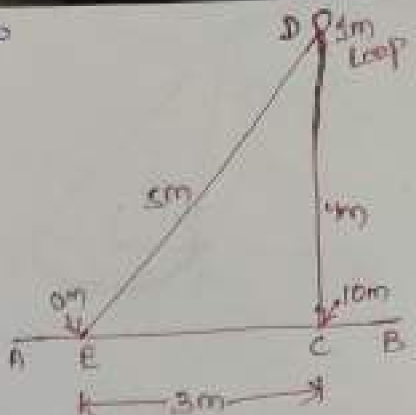
(A) To Erect a perpendicular to a chain line from a point on it–

The method of establishing perpendiculars with the tape are based on familiar geometric constructions. The following are some of the methods most commonly used. The illustrations given are for a 10m tape. However, a 20m tape may also be used.

(i) The 3-4-5 method :—

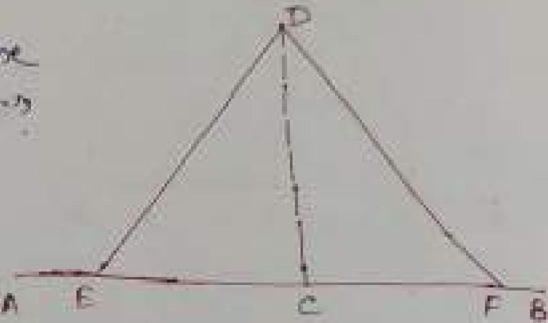
Let it be required to erect a perpendicular to the chain line at a point 'c' in it. Establish a point "E" at a distance 3m from "c". Put the "0" end of the tape (10m long) at E and the 10m end at "c".

The 5m and 6m marks are brought together to form a loop of 1m. The tape is now stretched tight by fastening the ends E and C. The point "D" is thus established. Angle DCE will be 90°. One person can set out a right angle by this method.



(ii) Second Method :—

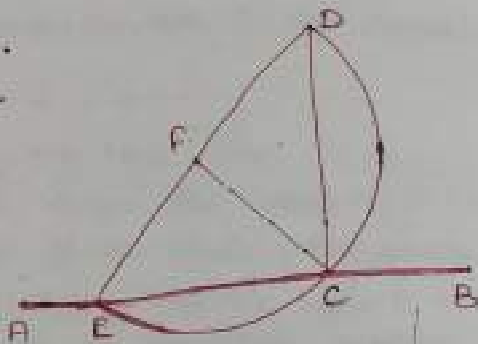
Select "E" and "F" equidistant from "C". Hold the zero end of the tape at "E", and 10m end at "F". Pick up 5m mark, stretch the tape tight and establish "D". Join DC.



(iii) Third Method :—

Select any point 'F' outside the chain. Preferably at '5m' distance from 'C'. Hold the 5m mark at 'F' and zero mark at 'C', and with 'F' as centre draw an arc to cut the line at 'E'. Join 'EF' and produce it to 'D' such that

EF = FD = 5m



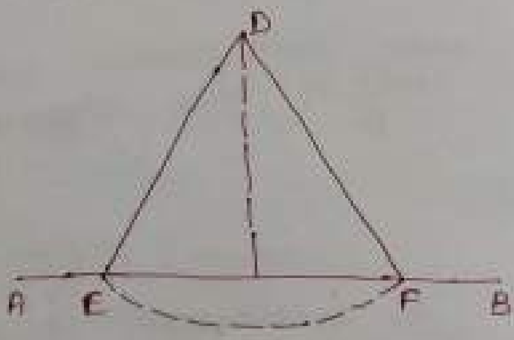
Thus, point 'D' will lie at the 10m mark of tape laid along EF with its zero end at 'E'. Join 'DC'.

(B) To drop a perpendicular to a chainline from a point outside it :—

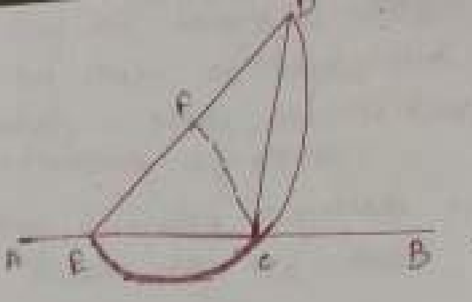
Let it be required to drop a perpendicular to a chain line AB a point 'D' outside it.

(i) First method :—

Select any point 'E' on the line. With 'D' as centre and DE as radius, draw an arc to cut the chainline in 'F'. Bisect EF at 'C'. CD will be perpendicular to 'AB'.

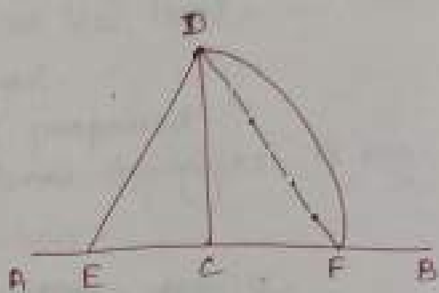


(ii) Second Method :—



Select any point 'E' on the line. Join ED and bisect it at 'F'. With 'F' as centre and 'EF' or 'FD' as radius, draw an arc to cut the chain line in 'C'. CD will be perpendicular to the chain line.

(iii) Third Method :—



Select any point 'E' on the line. With 'E' as centre and 'ED' as radius, drawn an arc to cut the chain line in 'F'. Measure 'FD' and 'FE'. Obtain the point 'C' on the line by making

$$FC = \frac{FD^2}{2FE}$$

Join 'C' and 'D'. CD will be perpendicular to the chainline.

* ERROR DUE TO INCORRECT RANGING :—

If the chain is stretched out of the line, the measured distance will always be more and hence the error will be positive. For each and every stretch of the chain, the error due to bad ranging will be cumulative and the effect will be too great a result. The error is not very serious in ordinary work if only the length is required. But if offsetting is to be done, the error is very serious.

* PURPOSE OF CHAIN SURVEYING :—

→ Chain Surveying is the type of surveying in which only linear measurements are made in the field.

→ In chain surveying, only measurements are taken in the field, and the rest work, such as plotting, calculation, etc are done in the office.

→ This is most suitably adapted to small plane areas with very few details. If carefully done, it gives quite accurate results.

* PRINCIPLE OF CHAIN SURVEYING :—

→ The principle of chain surveying is triangulation. This means that the area to be surveyed is divided into a number of small triangles which should be well conditioned.

Problem 1 The distance between two points, measured with a 20 m chain, was recorded as 327 m. It was afterwards found that the chain was 3 cm too long. What was the true distance between the points?

Solution Given data:

True length of chain, $L = 20$ m

Error in chain, $e = 3$ cm $= 0.03$ m, too long

$L' = L + e = 20 + 0.03 = 20.03$ m

Measured length = 327 m

$$\text{True length of line} = \frac{L'}{L} \times \text{ML}$$

$$= \frac{20.03}{20} \times 327 = 327.49 \text{ m}$$

Problem 2 The distance between two stations was 1,200 m when measured with a 20 m chain. The same distance when measured with 30 m chain was found to be 1,195 m. If the 20 m chain was 0.05 m too long, what was the error in the 30 m chain?

Solution Let us consider the 20 m chain.

$L = 20$ m $\qquad L' = 20 + 0.05 = 20.05$ m

Measured length = 1,200 m

$$\text{True length of line} = \frac{20.05}{20} \times 1{,}200 = 1{,}203 \text{ m}$$

Let us now consider the 30 m chain.

$L = 30$ m $\qquad L' = ?$

True length of line 1,203 m (as obtained from 20 m chain)
Measured length = 1,195 m.

From the relation

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

$$1{,}203 = \frac{L'}{30} \times 1{,}195$$

$$L' = \frac{1{,}203 \times 30}{1{,}195} = 30.20\,\text{m}$$

Now, L' is greater than L. So, the chain is too long.

Amount of error, $e = 30.20 - 30 = +\,0.20$ m

Problem 3 A line was measured by a 20 m chain which was accurate before starting the day's work. After chaining 900 m, the chain was found to be 6 cm too long. After chaining a total distance of 1,575 m, the chain was found to be 14 cm too long. Find the true distance of the line.

Solution First part:

$$L = 20 \text{ m}$$

$$L' = 20 + \frac{0 + 0.06}{2} \text{ (considering mean elongation)}$$

$$= 20.03 \text{ m}$$

$$ML = 900 \text{ m}$$

$$TL = ?$$

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

$$= \frac{20.03}{20} \times 900 = 901.35 \text{ m}$$

Second part:

$$L = 20 \text{ m}$$

$$L' = 20 + \frac{0.06 + 0.14}{2} = 20.1 \text{ m}$$

$$ML = 1{,}575 - 900 = 675 \text{ m}$$

$$TL = \frac{20.1}{20} \times 675 = 678.375 \text{ m}$$

True distance = 901.350 + 678.375 = 1,579.725 m

Problem 4 On a map drawn to a scale of 50 m to 1 cm, a surveyor measured the distance between two stations as 3,500 m. But it was found that by mistake he had used a scale of 100 m to 1 cm. Find the true distance between the stations.

Solution First method:

As the surveyor used the scale of 100 m to 1 cm,

$$\text{Distance between stations on map} = \frac{3500}{100} = 35 \text{ cm}$$

As the actual scale of map is 50 m to 1 cm.

True distance on the ground = 35 × 50 = 1,750 m

Second method:

$$\text{True distance} = \frac{\text{RF of wrong scale}}{\text{RF of correct scale}} \times \text{measured length}$$

$$\text{True distance} = \frac{\frac{1}{100 \times 100}}{\frac{1}{50 \times 100}} \times 3{,}500$$

$$= \frac{50 \times 100}{100 \times 100} \times 3{,}500$$

∴ True distance = 50 × 35 = 1,750 m

Problem 5 An old map was plotted to a scale of 40 m to 1 cm. Over the years, this map has been shrinking, and a line originally 20 cm long is only 19.5 cm long at present. Again the 20 m chain was 5 cm too long. If the present area of the map measured by planimeter is 125.50 cm^2, find the true area of the land surveyed.

Solution According to the given conditions,

19.5 cm on the map was originally 20 cm.

Therefore, 1 cm on the map was originally $= \frac{20}{19.5}$ cm, and

1 cm^2 on the map was originally $= \frac{(20)^2}{(19.5)^2}$ cm^2

125.50 cm^2 was originally $= \frac{(20)^2}{(19.5)^2} \times 125.50 = 132.0184$ cm^2

Scale of map was 1 cm = 40 m

⇒ 1 cm^2 = 1,600 m^2

Area on the ground = 1,600 × 132.0184

= 211,229.44 m^2

Since the chain was 0.05 m too long,

$$\text{True area} = \frac{(20.05)^2}{(20)^2} \times 211{,}229.44 = 212{,}286.90 \text{ m}^2$$

= 21.2286 hectares

(1 hectare = 10,000 m^2)

Problem 6 A steel tape was exactly 30 m long at 20°C when supported throughout its length under a pull of 10 kg. A line was measured with this tape under a pull of 15 kg and at a mean temperature of 32°C and found to be 780 m long. The cross-sectional area of the tape = 0.03 cm^2, and its total weight = 0.693 kg. α for steel = 11×10^{-6} per °C and E for steel = 2.1×10^6 kg/cm². Compute the true length of the line if the tape was supported during measurement (i) at every 30 m (ii) at every 15 m.

Solution Given data:

$L = 30$ m, $A = 0.03$ cm²

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

$P_0 = 10$ kg, $E = 2.1 \times 10^6$ kg/cm²

$P_m = 15$ kg, $W = 0.693$ kg

$T_m = 32$°C, ML = 780 m

(a) When supported at every 30 m:

Total correction per tape length is to be found out first. Here, $n = 1$.

(i) Temperature correction, $C_t = \alpha(T_m - T_0)\, L$

$$= 11 \times 10^{-6}\,(32 - 20) \times 30$$

$$= 0.00396 \text{ m (+ve)}$$

(ii) Pull correction, $C_p = \dfrac{(P_m - P_0)L}{A \times E}$

$$= \frac{(15 - 10) \times 30}{0.03 \times 2.1 \times 10^6} = 0.00238 \text{ m (+ve)}$$

(iii) Sag correction, $C_s = \dfrac{LW^2}{24 n^2 P_m^2}$

$$= \frac{30 \times (0.693)^2}{24 \times (15)^2} = 0.00267 \text{ m (– ve)}$$

Total correction = + 0.00396 + 0.00238 – 0.00267

= + 0.00367 m (too long)

so $L' = L + e = 30.00367$ m

$$\text{True length} = \frac{L'}{L} \times \text{ML}$$

$$= \frac{30.00367}{30} \times 780 = 780.094 \text{ m}$$

(b) When supported at every 15 m:

Here, span $n = 2$

Let us find out the correction per tape length.

(i) Temperature correction = 0.00396 m (+ve) as before

(ii) Pull correction = 0.00238 m (+ve) as before

(iii) Sag correction = $\dfrac{LW^2}{24n^2 P_m^2}$

$$= \frac{30 \times (0.693)^2}{24 \times 2^2 \times (15)^2} = 0.00067 \text{ m } (-\text{ve})$$

$$\text{Total correction} = +\,0.00396 + 0.00238 - 0.00067$$

$$= +\,0.00567 \text{ m (too long)}$$

so $$L' = L + e = 30.00567$$

$$\text{True length} = \frac{30.00567}{30} \times 780 = 780.147 \text{ m}$$

Problem 7 A 20-m steel tape was standardised on flat ground, at a temperature of 20°C and under a pull of 15 kg. The tape was used in catenary at a temperature of 30°C and under a pull of P kg. The cross-sectional area of the tape is 0.22 cm², and its total weight is 400 g. The Young's modulus and coefficient of linear expansion of steel are 2.1×10^6 kg/cm² and 11×10^{-6} per °C respectively. Find the correct horizontal distance if P is equal to 10 kg.

Solution Given data:

$L = 20$ m	$A = 0.02$ cm²
$T_0 = 20$°C	$\alpha = 11 \times 10^{-6}$ per °C
$P_0 = 15$ kg	$E = 2.1 \times 10^{-6}$ kg/cm²
$T_m = 30$°C	$W = 400$ g $= 0.4$ kg
$P = 10$ kg	$n = 1$

Here, applied pull $P = 10$ kg.

(a) Temperature correction, $C_t = \alpha(T_m - T_0)\, L$

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

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

$$= 0.00220 \text{ m } (+\text{ve})$$

(b) Pull correction, $C_p = \dfrac{(P - P_0)\, L}{A \times E}$

$$= \frac{(10 - 15)\,20}{0.02 \times 2.1 \times 10^6}$$

$$= -\frac{5 \times 20}{0.02 \times 2.1 \times 10^6}$$

$$= -\,0.00238 \text{ m } (-\text{ve})$$

(c) Sag correction, $C_s = \dfrac{LW^2}{24 n^2 P^2}$ $(n = 1)$

$$= \frac{20 \times (0.4)^2}{24 \times (10)^2} = 0.00133 \text{ m } (-\text{ve})$$

Total correction = + 0.00220 − 0.00238 − 0.00133 = − 0.00151 m

Correct horizontal distance = 20 − 0.00151 = 19.99849 m

Problem 8 A 30 m steel tape was standardised at a temperature of 20°C and under a pull 5 kg. The tape was used in catenary at a temperature of 25°C and under a pull of P kg. The cross-sectional area of the tape is 0.02 cm², its weight per unit length is 22 g/m, Young's modulus = 2×10^{-6} kg/cm², $\alpha = 11 \times 10^{-6}$ per °C. Find the correct horizontal distance, if P is equal to (i) 5 kg, and (ii) 11 kg.

Solution Given data:

$L = 30$ m	$A = 0.02$ cm²
$T_0 = 20°C$	$E = 2 \times 10^6$ kg/cm²
$P_0 = 5$ kg	$\alpha = 11 \times 10^{-6}$ per °C
$T_m = 25°C$	$W = 22$ g/m
P = (i) 5 kg (ii) 11 kg	Total weight $W = 22 \times 30$
$n = 1$	= 660 g
	= 0.66 kg

(a) When applied pull $P = 5$ kg:

(i) Temperature correction $C_t = \alpha(T_m - T_0) L$

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

$$= 0.00165 \text{ m (+ve)}$$

(ii) Pull correction $= \dfrac{(P - P_0) L}{AE}$

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

(iii) Sag correction, $C_s = \dfrac{LW^2}{24n^2p^2} = \dfrac{30 \times (0.66)^2}{24 \times (5)^2}$ $(n = 1)$

$$= + 0.02178 \text{ m (–ve)}$$

Total correction = + 0.00165 – 0.02178 = – 0.02013 m

Correct horizontal distance = 30 – 0.02013 = 29.97987 m

(b) When applied pull $P = 11$ kg:

(i) Temperature correction C_t = 0.00165 m (+ve) as before.

(ii) Pull correction, $C_p = \dfrac{(P - P_0) L}{AE}$

$$= \frac{(11 - 5) \times 30}{0.02 \times 2 \times 10^6} = 0.0045 \text{ m (+ ve)}$$

(iii) Sag correction, $C_s = \dfrac{LW^2}{24n^2p^2}$ $(n = 1)$

$$= \frac{30 \times (0.66)^2}{24 \times (11)^2}$$

$$= 0.00449 \text{ m (– ve)}$$

Total correction = + 0.00165 + 0.00450 – 0.00449

= + 0.00166 m

Correct horizontal distance = 30 + 0.00166

= 30.00166 m

Problem 9 A steel tape was exactly 20 m long at 20°C when supported throughout its length under a pull of 5 kg. A line measured with this tape under a pull of 16 kg and at a mean temperature of 32°C, was found to be 680 m long. Assuming the tape is supported at every 20 m, find the true length of the line. Given that: (i) Cross-sectional area of tape = 0.03 cm², (ii) $E = 2.1 \times 10^6$ kg/cm², (iii) $\alpha = 11 \times 10^{-6}$ per °C, and (iv) weight of tape = 10 g/cc. (WBSC 1982)

Solution Given data:

$L = 20$ m

$T_0 = 20$°C

$P_0 = 5$ kg

$T_m = 32$°C

$P_m = 16$ kg

ML = 680 m

$n = 1$

$A = 0.03$ cm²

$\alpha = 11 \times 10^{-6}$ per °C

$E = 2.1 \times 10^6$ cm²

Given weight = 10 g/cc

Total $W = 0.03 \times 20 \times 100 \times 10$

$= 600$ g $= 0.6$ kg

Correction per tape length:

(a) Temperature correction, $C_t = \alpha (T_m - T_0) L$

$= 11 \times 10^{-6} (32 - 20) \times 20$

$= 0.00264$ m (+ve)

(b) Pull correction, $C_p = \dfrac{(P_m - P_0) L}{AE}$

$$= \frac{(16 - 5) \times 20}{0.03 \times 2.1 \times 10^6} = 0.00349 \text{ m} \quad (+\text{ve})$$

(c) Sag correction $C_s = \dfrac{L(W)^2}{24 P_m^2}$ $(n = 1)$

$$= \frac{20 \times (0.6)^2}{24 \times (16)^2} = 0.00117 \text{ m} \quad (-\text{ve})$$

Total correction = + 0.00264 + 0.00349 – 0.00117

= + 0.00496 m

Actual length of tape, $L' = 20.00496$ m

True length of line $= \dfrac{L'}{L} \times \text{ML}$

$$= \frac{20.00496}{20} \times 680 = 680.169 \text{ m}$$

Problem 10 A 30 m steel tape was standardised at a temperature of 20°C and under a pull of 10 kg. The tape was used in catenary to fix a distance of 28 m between two points at 40°C and under a pull of 5 kg. Given that the cross-sectional area of the tape = 0.02 cm^2, total weight 470 g, Young's modulus of steel = 2.1 × 10^6 kg/cm^2, and coefficient of linear expansion = 11 × 10^{-6} per °C, (a) find the correct distance between the points, and (b) find the value of pull for which the measured distance would be equal to the correct distance.

Solution Given data:

Distance between two points = 28 m

So, here L = 28 m (span length)

$T_0 = 20°C$

$P_0 = 10$ kg

$T_m = 40°C$

$P_m = 5$ kg

$A = 0.02\ cm^2$

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

$\alpha = 11 \times 10^{-6}$ per °C

Total weight = 470 g

$$\text{Weight for 28 m} = \frac{470 \times 28}{30}$$

W = 439 g = 0.439 kg

(a) We have to first find the correct distance.

(i) Temperature correction $= 11 \times 10^{-6} (40 - 20) \times 28$

$= 0.00616$ m (+ve)

(ii) Pull correction $= \dfrac{(5-10)\,28}{0.02 \times 2.1 \times 10^6} = -\,0.00333$ m (–ve)

(iii) Sag correction $= \dfrac{28\,(0.439)^2}{24 \times (5)^2} = 0.00899$ m (–ve)

Total correction = + 0.00616 – 0.00333 – 0.00899 = – 0.00616 m

Correct distance = 28 – 0.00616 = 27.99384 m

(b) Now we have to find the normal tension at which the measured distance would be equal to the correct distance. This condition will be satisfied when

Pull correction = sag correction

Let, Normal tension = P_n

Then,
$$\frac{(P_n - P_0)\,L}{AE} = \frac{L W^2}{24 P_n^2}$$

or
$$\frac{(P_n - 10)}{0.02 \times 2.1 \times 10^6} = \frac{(0.439)^2}{24 P_n^2}$$

or
$$24 P_n^3 - 240 P_n^2 - 8{,}095 = 0$$

or
$$P_n^3 - 10 P_n^2 - 3373 = 0$$

Now, the value of P_n has to be found out by trial.

Method of Trials: Putting the assumed values of P_n in the equation,

$$P_n^3 - 10P_n^2 - 337.3 = 0$$

We get the following results.

When $P_n = 10$,
$1000 - 1000 - 337.3 = - 337.3$ which is not acceptable.

When $P_n = 11$,
$1331 - 1210 - 337.3 = - 216.3$ which is not acceptable.

When $P_n = 12$,
$1728 - 1440 - 337.3 = - 49.3$ which is not acceptable

When $P_n = 12.25$,
$1838.27 - 1500.63 - 337.3 = + 0.34$ which may be accepted, as the equation is nearly satisfied.

So, the required pull, at which the measured distance will be equal to correct distance, is 12.25 kg.

→ In chain surveying the sides of the triangles are measured directly on the field by chain or tape, and no angular measurements are taken. Here, the tie lines and check lines control the accuracy of work.

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

* Chain surveying is recommended when :–

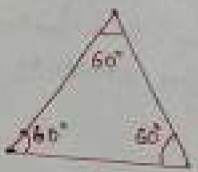
(1) The ground surface is more or less level.
(2) A small area is to be surveyed.
(3) A small - scale map is to be prepared.
(4) The formation of well - conditioned triangles is easy.

* Chain surveying is unsuitable when :–

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

* WELL - CONDITIONED AND ILL - CONDITIONED TRIANGLES :–

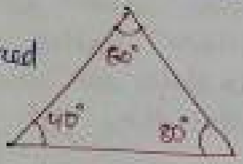
→ A triangle is said to be well - conditioned when no angle in it is less than 30° or greater than 120°.



(Ideal Triangle).

An equilateral triangle is considered to be the best - conditioned or ideal triangle.

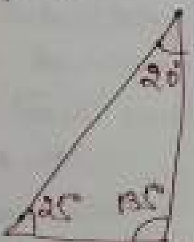
→ Well - conditioned triangles are preferred their apex points are very sharp and can be located by a single 'dot'. In such a case, there is no possibility of relative displacement of the plotted point.



(well - conditioned)

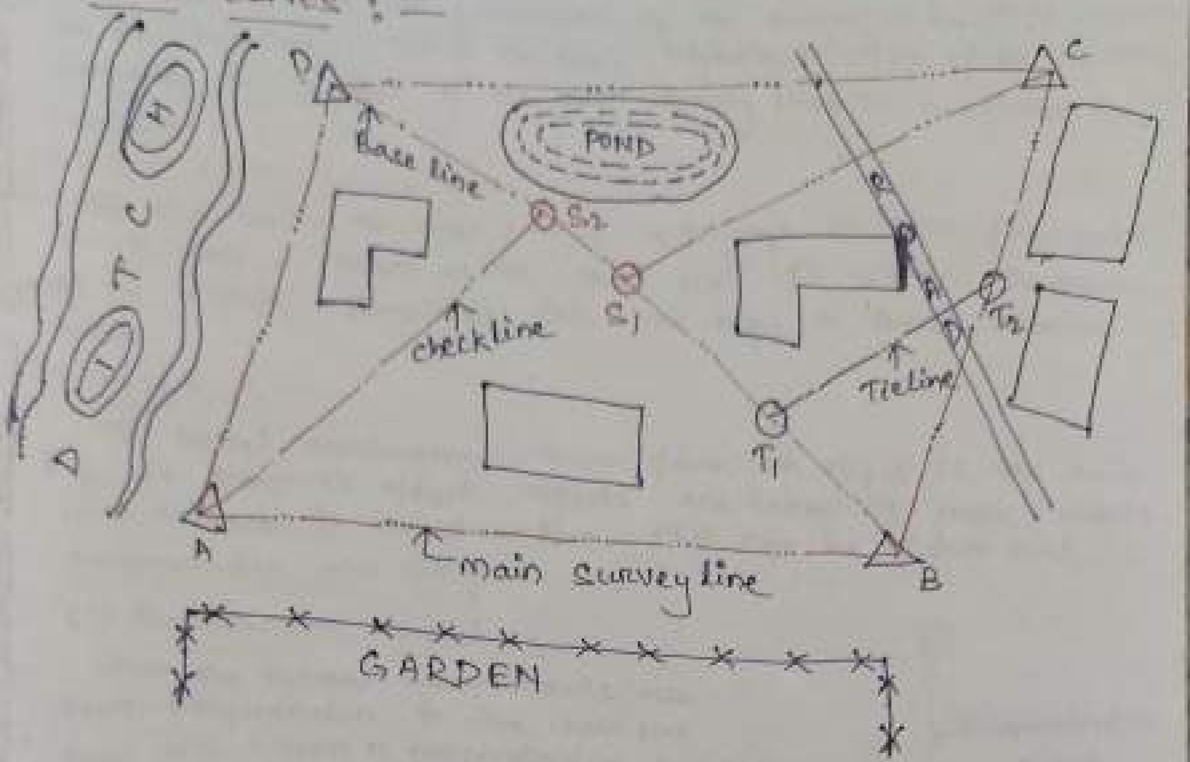
→ A triangle in which an angle is less than 30° or more than 120° is said to be ill - conditioned.

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



(Ill - conditioned triangle)

* SELECTION OF SURVEY STATION, BASE LINE, TIE LINES, CHECK LINES :—



Index Sketch

(A) SURVEY STATIONS :—

Survey stations are the points at the beginning and the end of a chain line. They may also occur at any convenient points on the chain line. Such stations may be :-

① Main stations.

② Subsidiary stations.

③ Tie stations.

① **Main stations :—** Stations taken along the boundary of an area as controlling points are known as 'main stations'. The lines joining the main stations are called 'main survey lines'. The main survey lines should cover the whole area to be surveyed. The main stations are denoted by '△' with letters A, B, C, D etc. The chain lines are denoted by —··—··—

② **Subsidiary stations :—** Stations which are the main survey lines or any other survey lines are known as subsidiary stations. These stations are taken to run subsidiary lines for dividing the area into triangles, for checking the accuracy of triangles and for locating interior details. These stations are denoted by '⊙' with letters S_1, S_2, S_3, etc.

③ **Tie stations :—** These are also subsidiary stations taken on the main survey lines. Lines joining the tie stations are known as tie lines. Tie lines are mainly taken to fix the directions of adjacent sides of the chain survey map. These are also taken to form 'chain angles' in chain traversing. Sometimes tie lines are taken to locate interior details.

(B) BASE LINE :—

The line on which the framework of the survey is built is known as the "baseline". It is the most important line of the survey. Generally, the longest of the main survey lines is considered the base line.

(c) CHECK LINE :—

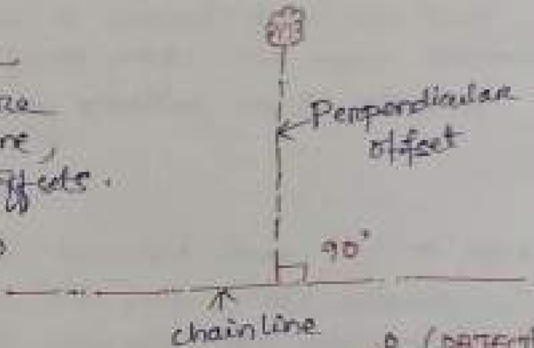
The line joining the apex point of a triangle to some fixed point on its base is known as the "check line". It is taken the accuracy of the triangle. Sometimes this line helps to locate interior details.

(D) OFFSET :—

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

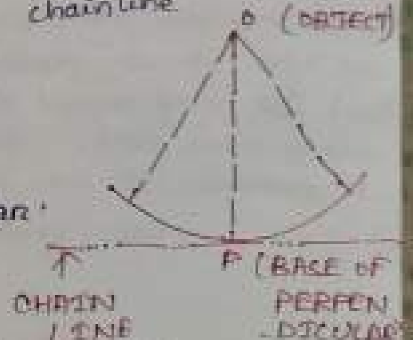
(i) Perpendicular offset :—

When the lateral measurements are taken perpendicular to the chain line, they are known as perpendicular offsets.



→ Perpendicular offsets may be taken in the following ways:—

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

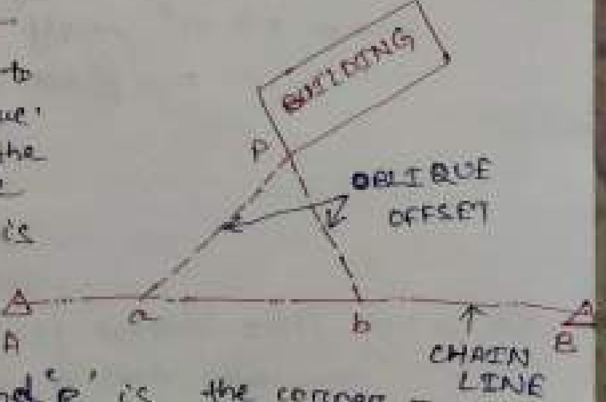


* By setting a right angle in the ratio 3:4:5.
* By setting a right angle with the help of builder's square or tri-square.
* By setting a right angle by cross-staff or optical square.

(ii) Oblique offsets :—

Any offset not perpendicular to the chain line is said to be oblique.

→ Oblique offsets are taken when the objects are at a long distance from the chain line or when it is not possible to set up a right angle due to some difficulties.



→ Suppose AB is a chain line and 'P' is the corner of a building. Two points 'a' and 'b' are taken on the chainline. The chainages of 'a' and 'b' are noted. The distances 'ap' and 'bp' are measured and noted in the field book. The 'ap' and 'bp' are the oblique offsets. When the triangle 'abp' is plotted, the apex point 'p' will represent the position of the corner of the building.

* FIELD BOOK :—

The notebook in which field measurements are noted is known as the 'field book'. The size of the field book is 20cm x 12cm and it opens lengthwise. Field books may be of two types :—

1) Single-line, and 2) Double-line.

1) Single-line field book :—

In this type of field book, a single red line is drawn through the middle of each page. This line represents the chain line, and the chainages are written on it. The offsets are recorded, with sketches, to the left or right of the chainline.

→ The recording of the field book is started from the last page and continued towards the first page. The main stations are marked by "Δ" and subsidiary stations or tie stations are by "⊙".

2.) Double-line field book :—

In this type of field book, two red lines, 1.5cm apart are drawn through the middle of each page. This column represents the chainline, and the chainages are written in it.

→ The offsets are recorded, with sketches, to the left or right of this column. The recording is begun from the last page and continued towards the first. The main stations are marked by 'Δ' and subsidiary or tie stations by '⊙'. This type of field book is commonly used.

* INSTRUMENTS FOR SETTING OFFSET :—

The instruments used for setting offset are :—

(i) Cross Staff , (ii) Optical square.

(i) Cross Staff :— The simplest instrument used for setting out right angles is a cross staff. It consists of either a frame or box with two pairs of vertical slits and is mounted on a pole shod for fixing in the ground.

→ The common forms of cross staff are

(a) Open cross staff.
(b) French cross staff.
(c) Adjustable cross staff.

(a) Open Cross Staff :—

→ It is provided with two pairs of vertical slits giving two lines of sights at right angles to each other.

→ The cross staff is set up at a point on the line from which the right angle is to run, and is then turned until one line of sight passes through the ranging pole at the end of the survey line.

(b) French Cross Staff :—

→ It consists of a hollow octagonal box. Vertical sighting slits are cut in the middle of each face, such that the lines between the centres of opposite slits make angles of 45° with each other. It is possible to set out angles of either 45° or 90° with this instrument.

(c) Adjustable Cross-Staff :—

The adjustable cross staff consists of two cylinders of equal diameter placed one on top of the other. Both are provided with sighting slits. The upper box carries a vernier and can be rotated relatively to the lower by a circular rack and pinion arrangement actuated by a milled headed screw. The lower box is graduated to degrees and sub-divisions.

(ii) Optical Square :—

Optical square is more convenient and accurate instrument than the cross staff for setting out a line at right angles to another line.

It consists of a circular box with three slits. In line with two openings, a glass silvered at the top and unsilvered at the bottom, is fixed facing one opening.

* ERRORS IN CHAIN SURVEYING :—

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

(1) Compensating.

(2) Cumulative.

(1) Compensating Errors :—

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

(a) Incorrect holding of the chain.

(b) Horizontality and verticality of steps not being properly maintained during the stepping operation.

(c) Fractional parts of the chain or tape not being uniform throughout its length, and.

(d) Inaccurate measurement of right angles with chain and tape.

(2) Cumulative Errors :—

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

Positive Errors :— When the measured length is more than the actual length, (i.e., when the chain is too short), the error is positive. Such errors occur due to :—

- The length of chain or tape being shorter than the standard length.
- Slope correction not being applied.
- Correction for sag not being made.
- Measurement being taken with faulty alignment.
- Measurement being taken in high winds with the tape in suspension.

Negative Errors :— When the measured length of the line is less than the actual length (i.e., when the chain is too long). These errors occur when the length of the chain or tape is greater than the standard length due to the following reasons :—

- The opening of ring joints.
- The applied pull being much greater than the standard pull.
- Wearing of connecting rings.
- Elongation of the links due to heavy pull.

* PRECAUTIONS AGAINST ERRORS AND MISTAKES :-

- The point where the arrow is fixed on the ground should be marked with a cross (X).
- The zero end of the chain or tape should be properly held.
- Care should be taken so that the chain is properly extended.
- Ranging should be done accurately.
- No measurement should be taken with the chain in suspension.
- In stepping operations, horizontality and verticality should be properly maintained.
- Measurements should not be taken with the tape in suspension in high winds.
- The chainman should call the measurement loudly and distinctly and the surveyor should repeat them while booking.
- While noting the measurement from the chain, the teeth of the tally should be verified with respect to the correct end.

ANGULAR MEASUREMENT AND COMPASS SURVEYING

CHAPTER-3

* MEASUREMENT OF ANGLE WITH CHAIN, TAPE & COMPASS :—

In chain surveying, the area to be surveyed is divided into a number of triangles. This method is suitable when the areas are involved, methods of chain surveying alone are not sufficient and convenient. In such cases, it becomes essential to use some sort of instrument which enables angles or directions of the survey lines to be observed. In measurement of angles the following instruments are used.

(a) Instruments for the direct measurement of directions :—

(i) Surveyor's Compass.

(ii) Prismatic Compass.

(b) Instruments for measurements of angles.

(i) Sextant.

(ii) Theodolite.

→ When triangulation of an area is not possible, traversing is adopted. Traversing is that type of surveying in which a number of connected survey lines ~~from~~ form the framework and the directions and lengths of the surveyline are measured with the help of an angle (or direction) measuring instrument and a tape (or chain) respectively. When the lines form a circuit which ~~extends~~ ends at the starting point, it is known as closed traverse. If the circuit ends elsewhere, it is said to be an open traverse.

→ The magnetic bearings of the survey lines are measured by a compass and the lengths of the lines are measured either with a chain or with a tape. The direction of magnetic meridian is established at each traverse station independently. The methods of taking the details are almost the same as for chain surveying.

* COMPASS :—

Magnetic Compass gives directly the magnetic bearings of lines. The bearings may either be measured in the W.C.B system or in Q.B system depending upon the compass used. The bearings so measured are entirely independent on any other measurement.

The most essential features of a magnetic compass are :—

(a) Magnetic needle, to establish the magnetic meridian.

(b) A line of sight, to the sight the other end of the line.

(c) A graduated circle, either attached to the box.

(d) A compass box to house the above parts.

* TYPES OF COMPASS :—

The types of compass :—

(1) The Prismatic Compass.
(2) The Surveyor's Compass.

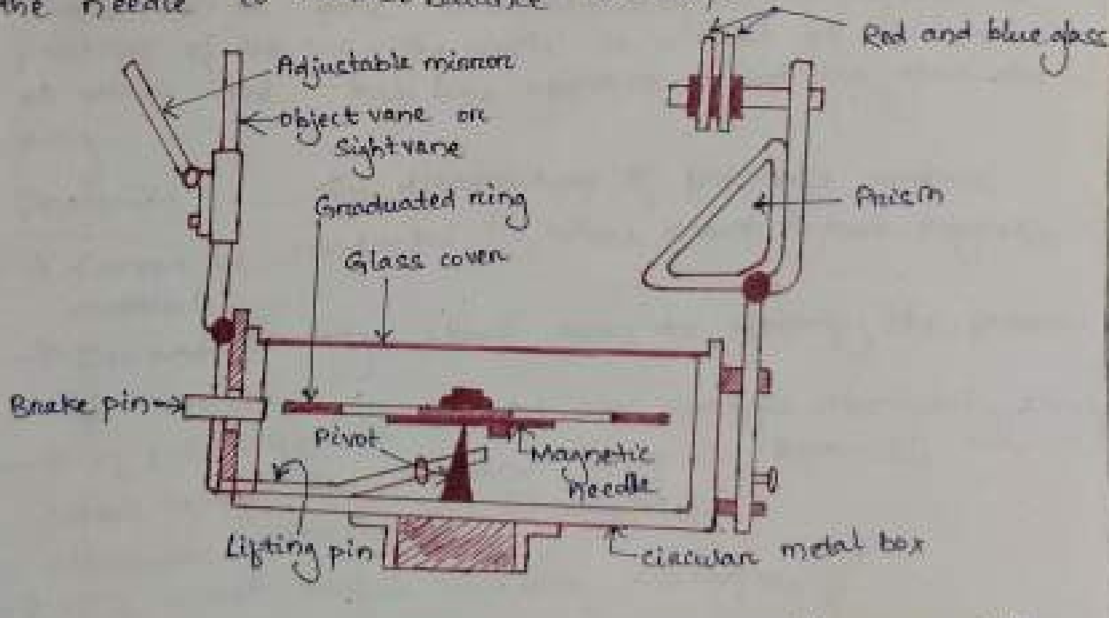
(1) The Prismatic Compass :—

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

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

(b) Magnetic Needle and Graduated Ring :— The magnetic needle is made of a broad, magnetised iron bar. The bar is pointed at both ends. The magnetic needle is attached to a graduated aluminium ring.

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



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

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

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

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

(g) Lifting Pin :— A lifting pin is provided just below the sight vane. When the sight vane is folded, it presses the lifting pin. The lifting pin then lifts the magnetic needle out of the pivot point to prevent damage to the pivot head.

(h) ~~Glass~~ Glass Cover :— A glass cover is provided on top of the box to protect the aluminium ring from dust.

(*) MERITS AND DEMERITS OF PRISMATIC COMPASS :—

* Merits :— The greatest advantage of prismatic compass is that both sighting the object as well as reading circle can be done simultaneously without changing the position of the eye. The circle is read at the reading at which the hair line appears to cut the graduated ring.

* Demerits :— The disadvantage of prismatic compass.

→ Cannot be conducted in areas known to have magnetic materials.

→ Demand knowledge about rocks to identify the presence of iron ore.

→ It is difficult to hold the compass absolutely steady when taking the bearing hence it is impossible to eliminate the errors completely.

→ It is not useful over long distances.

(2) The Surveyor's Compass :—

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

(a) → There is no prism on it. Readings are taken with naked eye.

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

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

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

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

(f) → No mirror is attached to the objectvane.

* MERITS OF SURVEYOR'S COMPASS :—

The advantage of surveyor's compass :—

→ Easy to handle.

→ Portable & Light weight.

→ It has less angle or position errors.

* TESTING & ADJUSTMENT OF COMPASS :—

* Permanent Adjustments of Surveyor's Compass :—

Permanent adjustments are those adjustments which are done only when the fundamental relations between the parts are disturbed. They are, therefore, not required to be repeated at every setup of the instrument. These consist of :

(i) Adjustment of levels.
(ii) Adjustment of sight vanes.
(iii) Adjustment of needle.
(iv) Adjustment of pivot point.

(i) Adjustment of levels :—

* Object :— To make the levels, when they are fitted, perpendicular to the vertical axis.

* Test :— Keep the bubble tube parallel to two foot screws & centre the bubble. Rotate the instrument through 90° about the vertical axis, till it comes over the third foot screw and centre the bubble. Repeat till it remain central in any of these positions, turn the instrument through 180° about vertical axis. If the bubble remains central, it is in adjustment. If not,

* Adjustment :— Bring the bubble half way by footscrews & half by adjusting the screws of the bubble tube.

(ii) Adjustment of Sight Vanes :—

* Object :— To bring the sight vanes into a vertical plane when the instrument is levelled.

* Test :— Level the instrument properly. Suspend a plumbline at some distance and look at it, first through one of the sight vanes and then through the other.

* Adjustment :— If the vertical hair in the object vane or the slit in the eye vane and either file the higher side of the bed .

(iii) Adjustment of Needle :—
The needle is adjusted for : (a) Sensitivity , (b) Balancing the needle, (c) straightening vertically , & (d) straightening horizontally.

(iv) Adjustment of the pivot :—
* Object :— To bring the pivot point exactly in the centre of the graduated circle .
* Test and Adjustment :— (1) Bring the North end of the needle against the North 0° mark of the graduated circle. If it doesnot read 0°, correct the error by bending the pivot pin slightly in a direction at right angles to the line between the North and South.

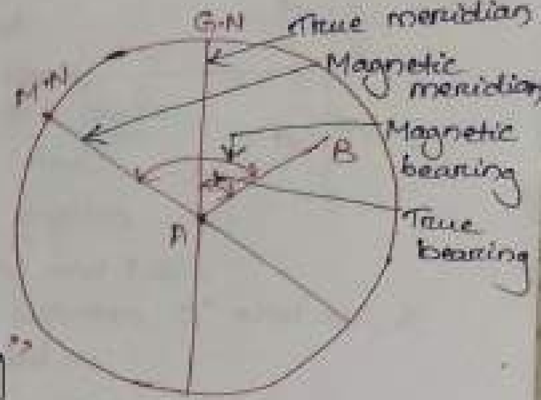
* Permanent Adjustments of prismatic compass :—
The permanent adjustments of prismatic compass are done only when fundamental relation between the parts are disturbed .

(i) Adjustment of Needle :—
The needle is adjusted for (a) sensitivity , (b) Balancing the needle
(a) Sensitivity :— The needle may loose its sensitivity either by the loss of its magnetism or by the pivot becoming blunt. To test it, level the instrument and lower the needle on its pivot. If it comes to rest quickly, it shows the sign of sluggishness. Remagnetise the needle, if necessary.
(b) Balancing the Needle :— Due to the effect of the dip, the needle may not the balanced on its pivot. To test it, level the instrument and lower the needle on its pivot.

(ii) Adjustment of pivot :—
* Object :— To bring the pivot point exactly in the centre of the graduated circle.
* Test and Adjustment :— (1) Bring the North end of the needle against the North 0° mark of the graduated circle. Note the reading of the southend of the needle. If it doesnot read 0°, correct the error by bending the pivot pin slightly in a direction at right angles to the line between the North and South zeros.
(2) Bring the North end of the needle exactly against 90° mark, and note the reading against the South end. If it doesnot read 90°, correct the error by bending the pivot pin in a direction at right angles to the line between the two 90° marks.

* The sight vanes are generally not adjustable.
* The needle cannot be straightened.

* DESIGNATION OF ANGLES :—



① CONCEPT OF MERIDIAN :— The line or plane passing through the geographical north pole, geographical south pole and any point on the surface of the earth, is known as 'true meridian' or 'geographical meridian'.

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

② MAGNETIC MERIDIAN :— When a magnetic needle is suspended freely and balanced properly, unaffected by magnetic substances, it indicates a direction. This direction is known as the magnetic ~~bea~~ meridian.

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

③ ARBITRARY MERIDIAN :— Sometimes for the survey ~~bea~~ of a small area, a convenient direction is assumed as a meridian, known as the "arbitrary meridian".

The angle between the arbitrary meridian and a line is known as the "arbitrary bearing" of the line.

④ GRID MERIDIAN :— Sometimes, for preparing a map some state agencies assume several lines parallel to the true meridian for a particular zone. These lines are termed as "grid lines" and the central line the "grid meridian". The bearing of a line with respect to the grid meridian is known as the "grid bearing of the line".

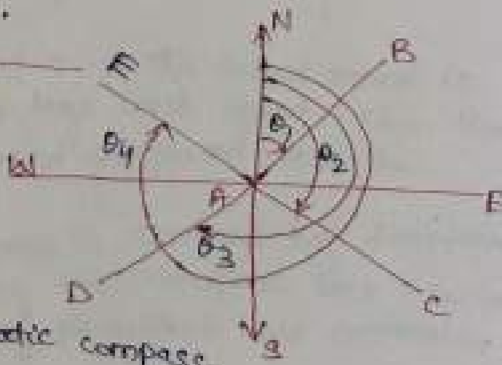
* DESIGNATION OF MAGNETIC BEARING :—

Magnetic bearings are designated by two systems :—

(i) Whole Circle Bearing (W.C.B), &

(ii) Quadrantal Bearing (Q.B).

(i) Whole Circle Bearing (WCB) :—

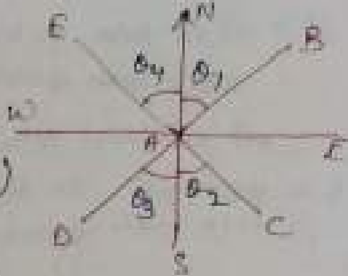


The magnetic bearing of a line measured clockwise from the north pole towards the line, is known as WCB, of that line. Such bearing may have any value between 0° and 360°.

The WCB of a line is obtained by prismatic compass.

* WCB of AB = θ_1
* WCB of AC = θ_2
* WCB of AD = θ_3
* WCB of AE = θ_4

(ii) Quadrantal Bearing :— (QB)



The magnetic bearing of a line measured clockwise or counterclockwise from the North pole or Southpole (whichever is nearer the line) towards the East or West, is known as the 'quadrantal bearing' of the line. This system consists of four quadrants - NE, SE, SW and NW. The value of a quadrantal bearing lies between 0° and 90°, & quadrants should always be mentioned.

Example :— QB of AB = Nθ_1E , QB of AD = Sθ_3W.
QB of AC = Sθ_2E , QB of AE = Nθ_4W.

* Reduced Bearing (RB) :— When the WCB of a line is converted to QB, it is termed the reduced bearing (RB). Thus, the RB is similar to the QB. Its value lies between 0° and 90°, but the quadrants should be mentioned for proper designation.

* Conversion of WCB to RB :—

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

* SUITABILITY OF APPLICATION (WCB & QB) :—

→ The whole circle bearing is mostly used in prismatic compass.
→ The Quadrantal bearing is used in surveyor's compass.
→ The WCB are extensively used to find out the bearing of the traversing and included angles between them, waypoints and direction.

* USE OF COMPASS :— (PRISMATIC COMPASS)

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

(1) Fixing the compass with tripod stand :— The tripod stand is placed at the required station with its legs well apart. Then the prismatic compass is held by the left hand and placed over the threaded top of the stand.

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

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

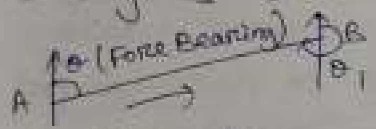
(4) Adjustment of prism :— The prism is moved up and down till the figures on the graduated ring are seen sharp and clear.

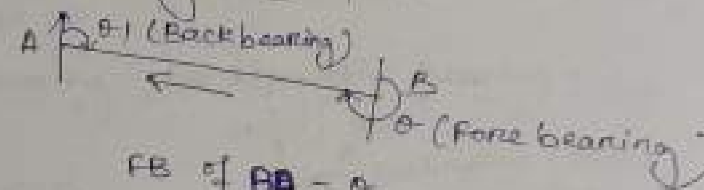
(5) Observation of Bearing :— After centering and levelling the compass box over the station, the ranging rod at the required station is bisected perfectly by sighting through the silt of the prism and horsehair at the sight vane.

* CONCEPT OF FORE BEARING & BACK BEARING :—

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

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





FB of AB = θ
BB of AB = θ1

FB of BA = θ
BB of BA = θ1

→ In the WCB system, the difference between the FB & BB should be exactly 180°.

$$BB = FB \pm 180°$$

Use the positive sign, when FB is less than 180°, and the negative sign when it is more than 180°.

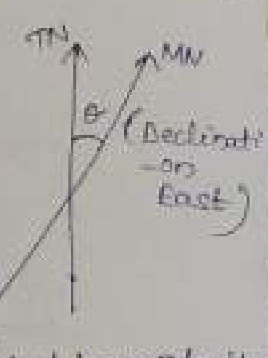
→ In the QB system, the FB and BB are numerically equal but the quadrants are just opposite.

Example :— FB of AB is N 30° E ; then its BB is S 30° W.

* EFFECTS OF EARTH'S MAGNETISM :—

The Earth acts as a powerful magnet and like any magnet, forms a field of magnetic force which exerts a directive influence on a magnetised bar of steel or iron.

→ MAGNETIC DIP :— The lines of force of earth's magnetic field run generally from south to North. Near the equator, they are parallel to the earth's surface. The angle which these lines of force make with the surface of the earth is called the angle of dip or the dip of the needle.



→ MAGNETIC DECLINATION :—

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

→ When the north end of the magnetic needle is pointed towards the west side of the true meridian the position is termed 'Declination West' (θW)

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

* VARIATION OF MAGNETIC DECLINATION :—

The magnetic declination at a place is not constant. It varies due to :—

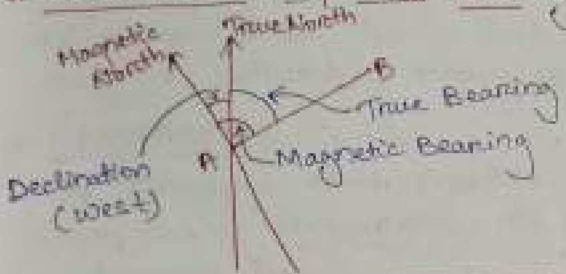
(a) Secular Variation :— The magnetic meridian behaves like a pendulum with respect to the true meridian. After every 100 years or so, it swings from one direction to the opposite direction, and hence the declination varies.

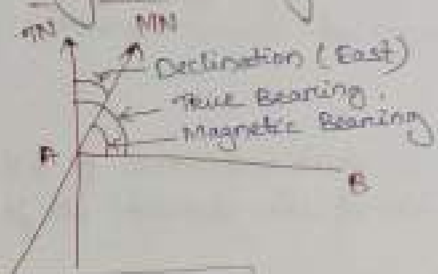
(b) Annual Variation :— The magnetic declination varies due to the rotation of the earth, with its axis inclined, in an elliptical path around the sun during a year. The amount of variation is about 1 to 2 minutes.

(c) Diurnal Variation :— The magnetic declination varies due to the rotation of the earth on its own axis in 24 hours. The amount of variation is found to be about 3 to 12 minutes.

(d) Irregular Variation :— The magnetic declination is found to vary suddenly due to some natural causes, such as earthquakes, volcanic erruptions and so on.

* Determination of True Bearing and Magnetic Bearing :—





(a) True Bearing = magnetic bearing ± declination

Note, Use the positive sign when declination east, and the negative sign when declination west

(b) Magnetic Bearing = True Bearing ± declination

Note, Use the positive sign when declination west, and the negative sign when declination east.

* SOURCES OF ERROR IN COMPASS SURVEYING :—

The following are the kind of error which may occur while taking readings with a compass :—

1) Instrumental Errors :—

→ The needle may not be perfectly straight and might not be balanced properly.

→ The pivot point may be eccentric

→ The graduations of the ring may not be uniform.

→ The sight vane may not be vertical.

→ The horse hair may not be straight and vertical

2) Personal Errors :—

→ The centring may not be done perfectly over the station.

→ The graduated ring may not be levelled.

→ The object might not be bisected properly.

→ The readings may be taken or entered carelessly.

→ The observer may be carrying magnetic substances.

3) Other Sources of Error :—

→ There may be local attraction due to the presence of magnetic substances near the station.

→ The magnetic field could vary on account of some natural causes.
→ The magnetic declination might vary.

* Remedies to be taken in compass Surveying :—

→ The centring should be done perfectly.
→ To stop the rotation of the graduated ring, the brake pin should be pressed very gently and not suddenly.
→ Readings should be taken along the line of sight and not from any side.
→ The stations should be not be selected near magnetic substances.
→ The observer should not carry magnetic substances.
→ The glass cover should not be dusted with a handkerchief.

* PRINCIPLE OF COMPASS SURVEYING :—

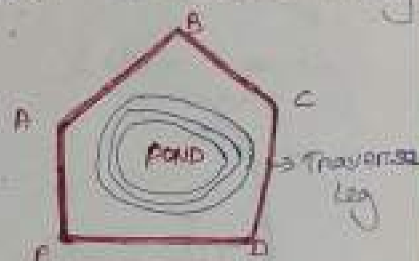
The principle of compass surveying is traversing, which involves a series of connected lines. The magnetic bearings of the lines are measured by prismatic compass and the distances of the lines are measured by chain. Such survey doesnot require the formation of a network of triangles.

* TRAVERSING :—

The surveying which involves a series of connected lines is known as traversing. The sides of the traverse are known as traverse legs.

→ In traversing, the lengths of the lines are measured by chain and the directions are fixed by compass or theodolite or by forming angles with chain and tape.

→ A traverse may be of two types — "closed" and "open".



(closed traverse)

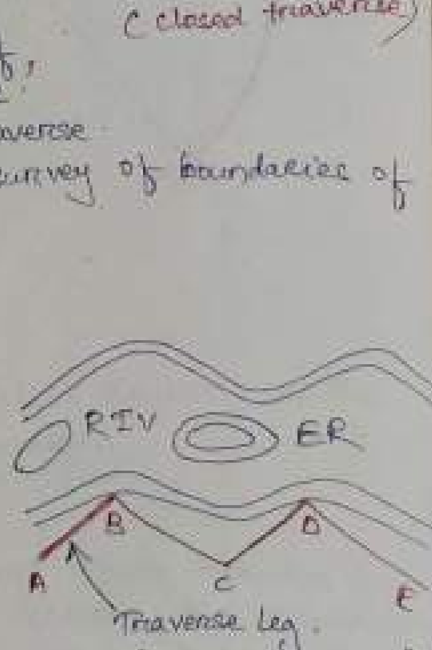
(1) CLOSED TRAVERSE :—

When a series of connected lines forms a closed circuit, i.e., when the finishing point coincides with the starting point of a survey, it is called a 'closed traverse'.

→ Here ABCDEA represents a closed traverse.
→ Closed traverse is suitable for the survey of boundaries of ponds, forests, estates, etc.

(2) OPEN TRAVERSE :—

→ When a sequence of connected lines extends along a general direction and does not return to the starting point, it is known as 'open traverse' or 'unclosed traverse'.



(Open traverse)

→ Here, ABCDE represents an open traverse.
→ Open traverse is suitable for the survey of roads, rivers, coast lines, etc.

* METHODS OF TRAVERSING :—

Traverse survey may be conducted by the following methods :—

1) Chain Traversing (by chain angle)
2) Compass Traversing (by free needle).
3) Theodolite Traversing (by fast needle)
4) Plane Table Traversing (by plane table).

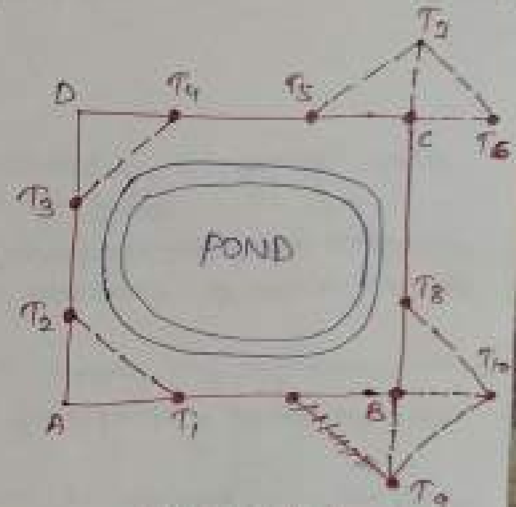
1) CHAIN TRAVERSING :—

Chain Traversing is mainly conducted when it is not possible to adopt triangulation. In this method, the angles between adjacent sides are fixed by chain angles. The entire survey is conducted by chain and tape only and no angular measurements are taken. When it is not possible to form triangles as for example, in a pond, chain traversing is conducted. The formation of chain angles is explained below.

(a) **First Method** :—

Suppose a chain angle is to be found to fix the directions of sides AB and AD. The stations T_1 and T_2 are fixed on lines AB and AD. The distances AT_1, AT_2 and T_1T_2 are measured.

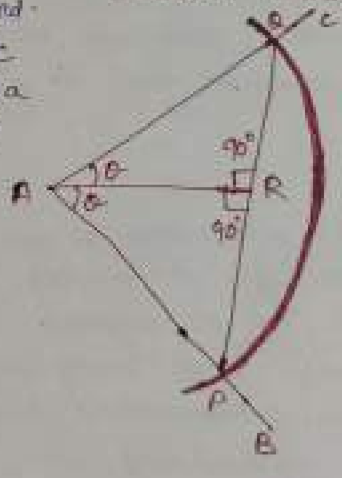
Then the angle $\angle T_1\text{-}A\text{-}T_2$ is said to be the chain angle. So, the chain angle is fixed by the tie line T_1T_2.



(Chain Traversing)

(b) **Second Method** :—

Sometimes the chain angle is fixed by chord. Suppose the angle between the lines AB and AC is to be fixed. Taking 'A' as the centre and a radius equal to one tape length (15m), an arc intersecting the lines AB and AC at points P and Q, respectively, is drawn. The chord PQ is measured and bisected at R.



Let, $\angle PAR = \theta$, Then $\angle BAC = 2\theta$

Here, $AP = AQ = 15m$

In triangle PAR,

$$\sin\theta = \frac{PR}{AP} = \frac{2PR}{2AP} = \frac{PQ}{30}$$

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

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

2) COMPASS TRAVERSING :—

In this method, the fore and back bearings of the traverse legs are measured by prismatic compass and the sides of the traverse by chain or tape. Then the observed bearings are verified and necessary corrections for local attraction are applied. In this method, closing error may occur when the traverse is plotted. This error is adjusted graphically by using 'Bowditch's rule'.

(3) THEODOLITE TRAVERSING :—

In such traversing, the horizontal angles between the traverse legs are measured by theodolite. The lengths of the legs are measured by chain or by employing the stadia method. The magnetic bearing of the starting leg is measured by theodolite. Then the magnetic bearings of the other sides are calculated. The independent coordinates of all the traverse stations are then foundout. This method is very accurate.

(4) PLANE TABLE TRAVERSING :—

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

* LOCAL ATTRACTION :—

A magnetic meridian at a place is established by a magnetic needle which is uninfluenced by other attracting forces.

→ The magnetic needle may be attracted and prevented from indicating the true magnetic meridian when it is in proximity to certain magnetic substances.

→ Local attraction is a term used to denote any influence, such as the above, which prevents the needle from pointing to the magnetic North in a given locality.

→ Some sources of local attraction are :- magnetite in the ground, wire carrying electric current, steel structures, railroad rails, underground iron pipes, keys, steel-boxed spectacles, metal buttons, axes, chains, steel tape, etc., which may be lying on the ground nearby.

→ Detection of Local Attraction :—

The local attraction at a particular place can be detected by observing the fore and back bearings of each line and finding its difference. If the difference between fore and back bearing is 180°, it may be taken the both the stations are free from local attraction, provided there are no observational and instrumental errors.

→ If the difference is other than 180°, the fore bearing should be measured again to find out whether the discrepancy is due to avoidable attraction from the articles on persons, chains, tapes etc. If the difference still remains, the local attraction exists at one or both the stations.

⇒ Elimination of Local Attraction :—

If there is local attraction at a station, all the bearings measured at that place will be incorrect and the amount of error will be equal in all the bearings. There are two methods for eliminating the effects of local attraction.

* First Method :— In this method, the bearings of the lines are

calculated on the basis of the bearing of that line which has a difference of 180° in its fore and back bearings. It is, assumed that there are no observational and other instrumental errors. The amount and direction of error due to local attraction at each of the affected station is found. If, however, there is no such line in which the two bearings differ by 180°, the corrections should be made from the mean value of the bearing of that line in which there is least discrepancy between the back sight and fore sight readings.

If the bearings are expressed in quadrantal system, the corrections must be applied in proper direction. In 1st and 3rd quadrants, the numerical value of bearings increase in clockwise direction while they increase in anti-clockwise direction in 2nd and 4th quadrants. Positive corrections are applied clockwise and negative corrections counter-clockwise.

* Second Method :—

This is more a general method and is based on the fact that though the bearings measured at a station may be incorrect due to local attraction, the ~~traction~~ included angle calculated from the bearings will be correct since the amount of error is the same for all the bearings measured at the station. The included angles between the lines are calculated at all the stations. If the traverse is a closed one, the sum of the internal included angles must be $(2n-4) \times 90°$. If there is any discrepancy in these observational and instrumental errors also exist. Such error is distributed equally to all the angles. Proceeding now with line, the bearings of which differ by 180°, the bearings of all other lines are calculated.

* Special Case :—

Special case of local attraction may arise when find no line which has a difference of 180° in its fore and back bearings is closet to 180°. The mean value of the bearing of that line is found by applying half the correction to both the fore and back bearings of that line, thus obtaining the modified fore and back bearings of that line differing exactly by 180°. Proceeding with the modified bearings of that line, corrected bearings of other lines are found.

* SOURCES OF ERRORS IN COMPASS SURVEYING :—

The errors may be classified as :—

(a) Instrumental Errors.
(b) Personal Errors.
(c) Errors due to natural causes.

(a) Instrumental Errors :—

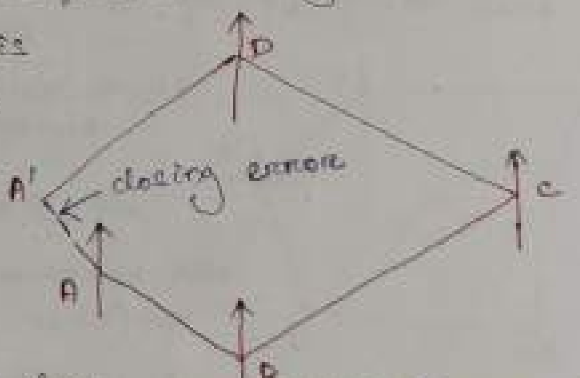
They are those which arise due to the faulty adjustments of the instruments. They may be due to the following reasons :—

1) The needle not being perfectly straight.
2) Pivot being bent.
3) Sluggish needle.
4) Blunt pivot point.
5) Improper balancing weight
6) Plane of sight not being vertical.
7) Line of sight not passing through the centres of the sight.

(b) Personal Errors :—
They may be done to the following reasons :—
→ Inaccurate levelling of the compass box.
→ Inaccurate centering.
→ Inaccurate bisection of signals.
→ Carelessness in reading and recording.

(c) Natural Errors :—
→ Variation in declination.
→ Local attraction due to proximity of local attraction forces
→ Magnetic changes in the atmosphere due to clouds and storms.
→ Irregular Variations due to magnetic storms etc.

* PLOTTING OF COMPASS TRAVERSING :—
The following are the various methods of plotting compass traverse.
(a) By parallel meridian through each station :— The starting point 'A' is suitably selected on the paper and a line representing the north line. The bearing of the line AB is plotted by protractor and its length is plotted to any suitable scale.
→ At station B, the northline is drawn parallel to the northline which are drawn at A. Then the bearing of the line BC is plotted and its length marked according to the previous scale.

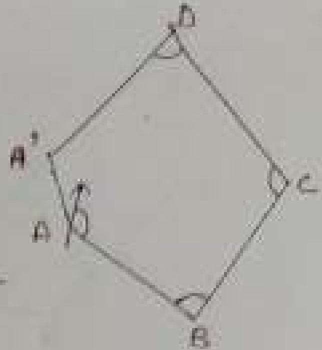


→ Similarly, all the traverse legs are plotted. In case of closed traverse, there may be a closing error which should be adjusted graphically.

(b) By considering included angles :—
The starting station 'A' is suitably selected on the sheet. 'A' line representing the north line is drawn through station 'A'. The bearing of the line "AB" is plotted by protractor and the distance "AB" is ~~plotted~~ ~~by pro~~ marked to a suitable scale. At station 'B' the angle ∠B is plotted and the distance 'BC' marked according to the ~~pro~~ previous scale. Angle ∠C is plotted at station 'C' and

the distance 'CD' is marked.

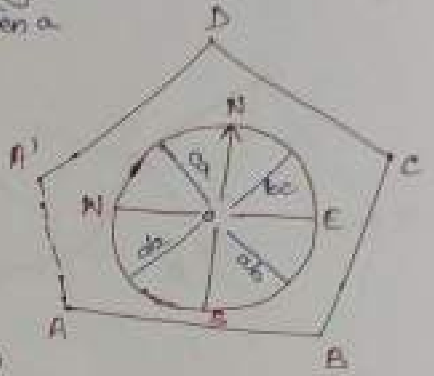
This process is continued until all the lines have been plotted. In this case also there may be a closing error which has to be adjusted graphically.



(C) By considering the central meridian :—

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

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



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

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

* CHECK OF CLOSING ERROR IN CLOSED & OPEN TRAVERSE :

→ Closed Traverse :—

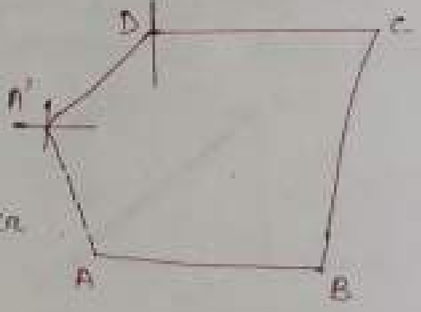
The errors involved in traversing are two kinds: linear and angular. The following are the checks for the angular work:-

(1) Traverse by included angles :—

(a) The sum of measured interior angles should be equal to (2N-4) right angles,

where, N = number of sides of the traverse.

(b) If the exterior angles are measured, their sum should be equal to (2N+4) 90°



(2) Traverse by deflection angles :—

The algebric sum of the deflection angles should be equal to 360°, taking the right-hand deflection angles as positive and left-hand angles as negative.

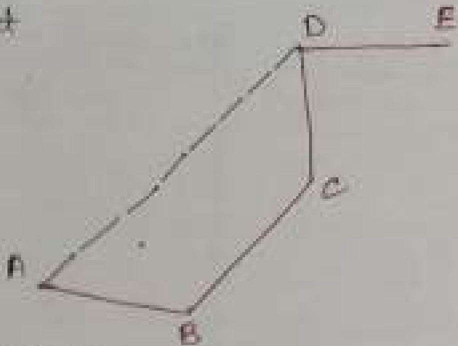
(3) Traverse by direct observation of bearings :—

The fore bearing of the last line should be equal to its back bearing ± 180° measured at the initial station.

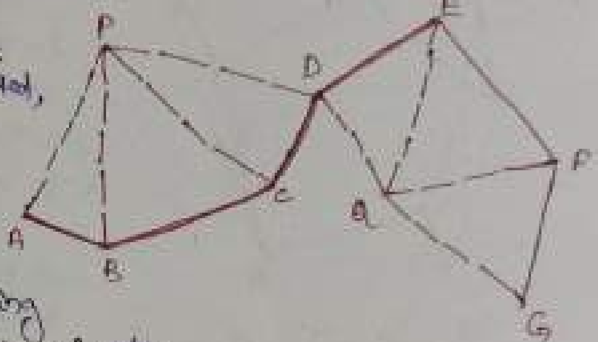
→ Checks in Open Traverse :—

No direct check of angular measurement is available. However, indirect checks can be made.

(i) The observation of bearing of 'AB' at station A, bearing of 'AD' can also be measured, if possible. Similarly, at 'D', bearing of 'DA' can be measured and check applied. If the two bearings differ by 180°, the work (upto 'D') may be accepted as correct. If there is small discrepancy, it can be adjusted before proceeding further.



(ii) Another method, which furnishes a check when the work is plotted, and consists in reading the bearings to any prominent point 'P' from each of the consecutive stations. The check in plotting consists in laying off the line 'AP', 'BP', 'CP', etc and nothing whether the lines pass through one point.



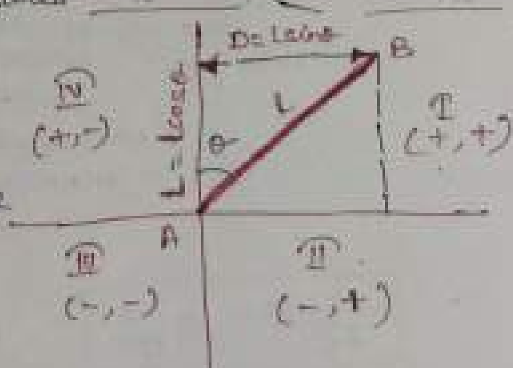
* LATITUDE AND DEPARTURE :—

The "latitude" of a survey line may be defined as its co-ordinate length measured parallel to an assumed meridian direction (i.e. true north or magnetic north or any other reference direction).

→ The "departure" of a survey line may be defined as its co-ordinate length measured at right angles to the meridian direction.

→ The "latitude" (L) of the line is positive when measured northward (or upward) and is termed as northing; the latitude is negative when measured southward (or downward) and is termed as "southing".

→ Similarly, the departure (D) of the line is positive when measured eastward and is termed as 'easting', the departure is negative when measured westward and is termed as 'westing'.



→ The latitude and departure of the line AB of length (l) and reduced bearing (θ) are given by :—

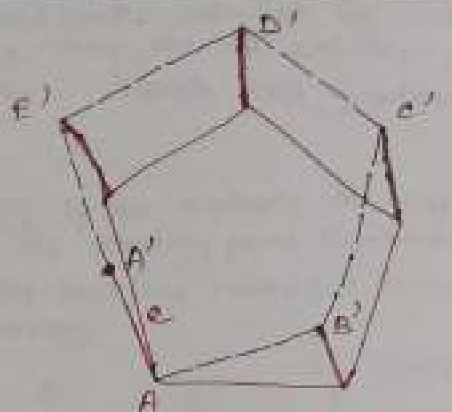
$L = \pm l \cos\theta$ and $D = \pm l \cdot \sin\theta$

→ The following table gives signs of latitude and departures :—

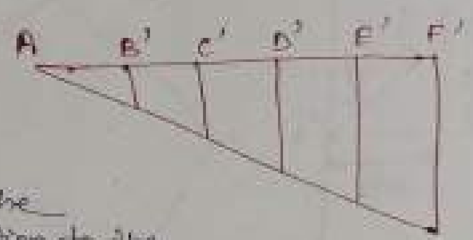
W.C.B	R.B and Quadrant	Sign of Latitude	Sign of Departure
0° to 90°	N θ E ; I	+	+
90° to 180°	S θ E ; II	—	+
180° to 270°	S θ W ; III	—	—
270° to 360°	N θ W ; IV	+	—

* BOWDITCH'S METHOD :—

The basis of this method is on the assumptions that the errors in linear measurements are proportional to $\sqrt{L}$ and the errors in angular measurements are inversely proportional to $\sqrt{L}$ where 'L' is the length of a line.



→ The Bowditch's rule, also termed as the compass rule, is mostly used to balance a traverse where linear and angular measurements are of equal precision.



→ The total error in latitude, and in the departure is distributed in proportion to the lengths of the sides.

→ The Bowditch Rule is :—

Correction to latitude (or departure) of any side = Total error in latitude (or departure) × $\frac{\text{Length of that side}}{\text{Perimeter of traverse}}$

Thus if :

C_L = correction to latitude of any side.
C_D = correction to departure of any side.
ΣL = Total error in latitude.
ΣD = Total error in departure.
Σl = Length of the perimeter.
l = length of any side.

we have,

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

and

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

* GALES TABLE TRAVERSE :—

Traverse computations are usually done in a tabular form, a more common form being Gales Traverse Table.

→ Adjust the interior angles, i.e., sum of interior angles to be equal to $(2N-4)\times 90°$ and exterior angles $(2N+4)\times 90°$.

→ In case of a compass traverse, the bearings are adjusted for local attraction if any.

→ Starting with observed bearings of one line, calculate the bearings of all other lines. Reduce all bearings to quadrantal system.

→ Calculate the consecutive co-ordinates (i.e., latitudes and departures)

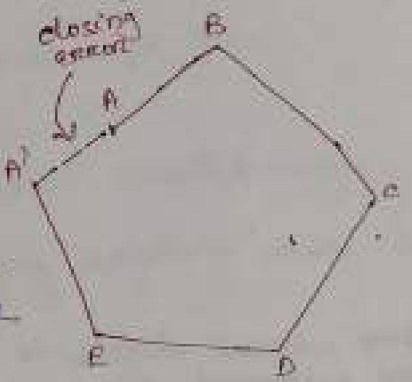
→ Calculate ΣL and ΣD.

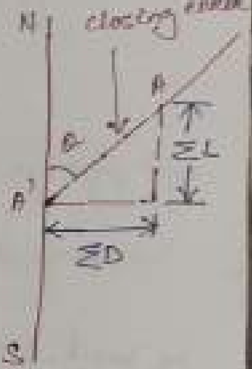
→ Apply necessary corrections to the latitudes and departures of the lines so that $\Sigma L = 0$ and $\Sigma D = 0$. The corrections may be applied either by transit rule or by compass rule depending upon the type of traverse.

→ Using the corrected consecutive co-ordinates, calculate the independent coordinates to the points so that they are all positive, the whole of the traverse thus lying in the North-East quadrant.

* CLOSING ERROR :—

When a closed traverse is plotted by any of the methods, the endpoints of the last line may not coincide with the starting point. The traverse ABCDEA is plotted starting at 'A' and the last line comes out to be 'EA''. The distance "AA'" is known as closing error.





→ The error can occur due to combination of errors in angular measurements and linear measurements.

Before a closed traverse is plotted, the following checks should be applied.

→ Verify that the sum of the interior angles is equal to $(2n-4)\times 90^\circ$.

→ Calculate the latitudes and departures of the lines.

→ Check that the sum of latitudes $\Sigma L = 0$ and the sum of departure $\Sigma D = 0$

→ The closing error $(e) = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$

→ The direction of the closing error is given by $\tan\theta = \frac{\Sigma D}{\Sigma L}$, "θ" being the angle made with the north or south direction. The quadrant of the error can be determined from the signs of ΣD and ΣL.

* Correcting angles :—

If the interior angles of the traverse are measured and they do not satisfy the geometric condition, that their sum should be equal to $(2n-4)\times 90^\circ$, then the angular error can be distributed equally amongst all the angles.

* Correcting Bearings :—

If the bearings of a traverse are measured and it is found that the forebearing of the last line and the observed back bearing of the same line ~~last line~~ do not agree, then the error can be distributed among the angles. The error in the bearing, 'e' can be distributed as follows :—

Error in the bearing of first line $= \frac{e}{N}$

Error in the bearing of second line $= \frac{2e}{N}$

Error in the bearing of last line = $\frac{Ne}{N} = e$.
This is equivalent to applying the correction by distributing it equally amongst the observed angles.

*** Remedies of errors in compass surveying :—**

The errors in compass surveying is adjusted by Bowditch's rule and graphical method.

—— x ——

PLANE TABLE SURVEYING　　CHAPTER-5

*** OBJECTIVES OF PLANE TABLE SURVEYING :—**

The main objectives of plane table surveying are :—

→ To carry out small scale surveying rapidly.
→ To run the survey lines between stations that have been previously fixed by other methods of surveying.
→ To locate the topographical details.
→ To survey industrial areas where compass surveying is not feasible.
→ For the preparation of small scale maps.

*** USE OF PLANE TABLE :—**

→ It is best fitted for small-scale surveying i.e., any types of fields.
→ It is also used in surveying industrial areas where compass survey fails to perform.
→ It is often used to fill in details between stations fixed by triangulation method or theodolite traversing method.

*** INSTRUMENTS & ACCESSORIES USED IN PLANE TABLE SURVEYING :—**

Plane tabling is a graphical method of survey in which the field observations and plotting proceed simultaneously. It is means of making a manuscript map in the field while the ground can be seen by the topographer and without intermediate steps of recording and transcribing field notes. It can be used to tie topography by existing control and to carry its own control systems by triangulation or traverse and by lines of levels.

→ The following instruments are used in plane table survey :— plane table, Alidade, Spirit level, Compass, plumbbob, Drawing paper.

(1) Plane Table :—

The plane-table is a drawing board of size 750mm x 600mm made of well-seasoned wood like teak, pine etc. The top surface of the table is well levelled. The bottom surface consists of a threaded circular plane for fixing the table on the tripod stand by a wing nut.

→ The plane table is meant for fixing a drawing sheet over it. The positions of the objects are located on this sheet by drawing rays and plotting to any suitable scale.

(2) Alidade :—

A plane table alidade is a straight edge with some form of sighting device. There are two types of alidade — plain and telescopic.

(a) Plain Alidade :— The plain alidade consists of a metal or wooden ruler of length about 50cm. One of its edges is bevelled and is known as the fiducial edge. It consists of two vanes at both ends which are hinged with the ruler. One is known as the object vane and carries a horse hair, the other is called the sight vane and is provided with a narrow slit.

(b) Telescopic Alidade :— The telescopic alidade consist of a telescope meant for inclined sighting distant objects clearly. This alidade has no vanes at the ends, but is provided with fiducial edge.

(3) The Spirit Level :—

The spirit level is small metal cube containing a small bubble of spirit. The bubble is visible on the top along a graduated glass tube.

(4) The Compass :—

There are two kinds of compass — (a) the trough compass,
(b) the circular box compass.

(a) The trough Compass — The trough compass is a Rectangular box made of non-magnetic metal containing a magnetic needle pivoted at the centre.

(b) The circular Box Compass — It carries a pivoted magnetic needle at the centre. The circular box is fitted on a square base plate.

(5) U-fork or plumbing fork with plumb-bob :—

The U-fork is a metal strip bent in the shape of 'U' (hair pin) having equal arm lengths. The top arm is pointed and the bottom arm carries a hook for suspending a plumb-bob.

→ This is meant for centering the table over a station.

(6) Drawing paper :—

Good-quality drawing sheet, tinted off-white to reduce strain on the eyes, should be used. The paper should be seasoned to reduce the effects of temperature changes and humidity. The paper can be fixed onto the board with the pins or tape.

* METHODS OF PLANE TABLE SURVEYING :—

The following are the four methods of plane tabling :

① Radiation. ,

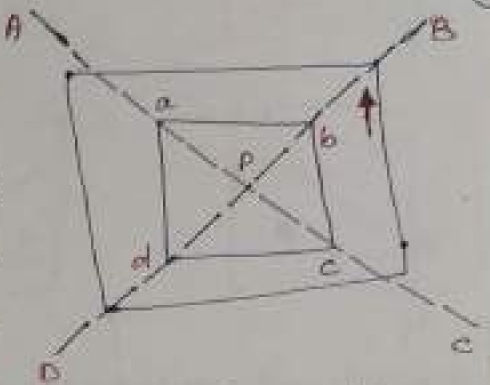
② Intersection.

③ Traversing

④ Resection.

1) **RADIATION** :—

This method is suitable for locating the objects from a single station. In this method, rays are drawn from the station to the objects, and the distances from the station to the objects are measured and plotted to any suitable scale along the respective rays.



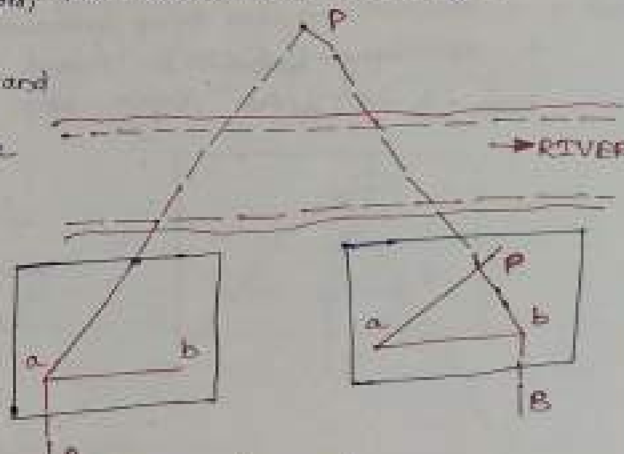
⇒ **Procedure** :—

(a) Suppose 'P' is a station on the ground from where the object A, B, C and D are visible.
(b) The plane table is set up over the station 'P'. A drawing sheet is fixed on the table, which is then levelled and centered.
(c) A point 'p' is selected on the sheet to represent the station 'P'.
(d) The northline is marked on the right-hand top corner of the sheet with compass.
(e) With the alidade touching 'p', the ranging rods at A, B, C, and D are bisected and the rays drawn.
(f) The distances PA, PB, PC and PD are measured and plotted to any suitable scale to obtain the points a, b, c, and d, representing the objects A, B, C and D on paper.

2) **INTERSECTION METHOD** :—

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

⇒ **Procedure** :—



→ Suppose A and B are two stations and P is an object on the far bank of a river. Now it is required to fix the position of 'P' on the sheet by the intersection of rays, drawn from A and B.

→ The table is set up at 'A', it is levelled and centred so that a point 'a' on the sheet is just over the station 'A'.

→ The north line is marked on the right-hand top corner. The table is then clamped.

→ With the alidade touching 'a', the object 'P' and the ranging rod at 'B' are bisected and rays are drawn.

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

→ The table is shifted and centred over B and levelled properly. Now the alidade is placed along the line 'ba' and orientation is done by back sighting. At the same time centring, levelling and orientation must be perfect.

→ With the alidade touching 'b', the object P is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point 'p'. The point 'p' is the required plotted position of 'P'.

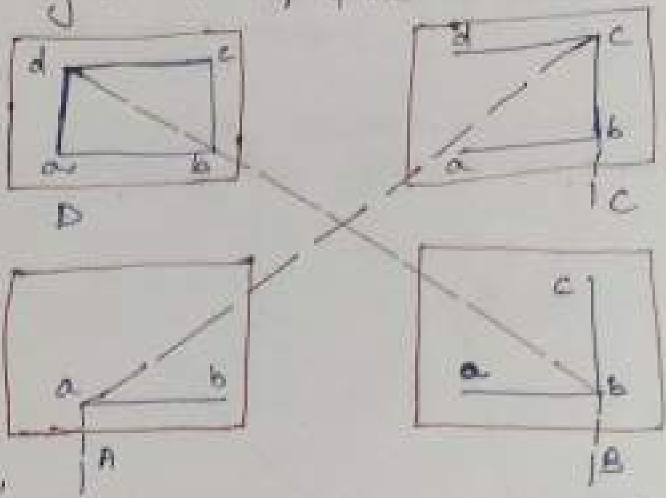
3) **TRAVERSING METHOD** :—

This method is suitable for connecting the traverse station. This is similar to compass or theodolite traversing. But here fielding and plotting are done simultaneously with the help of the radiation and intersection method.

⟹ Procedure :—

→ Suppose A, B, C and D are the traverse station.

→ The table with drawing sheet is setup at the station A, after that the table is centred, levelled. The northline is marked on the right-hand top corner of the sheet.



→ With the help of alidade touching point 'a' the ranging rod at 'B' is bisected and drawn. The distance AB is measured and then plotted to a suitable scale.

→ The table is shifted and centred over 'B'. Then it is levelled and oriented by backsighting.

→ With the alidade touching point 'b', the ranging rod at 'c' is bisected & a ray is drawn. The distance BC is measured and plotted to the same scale

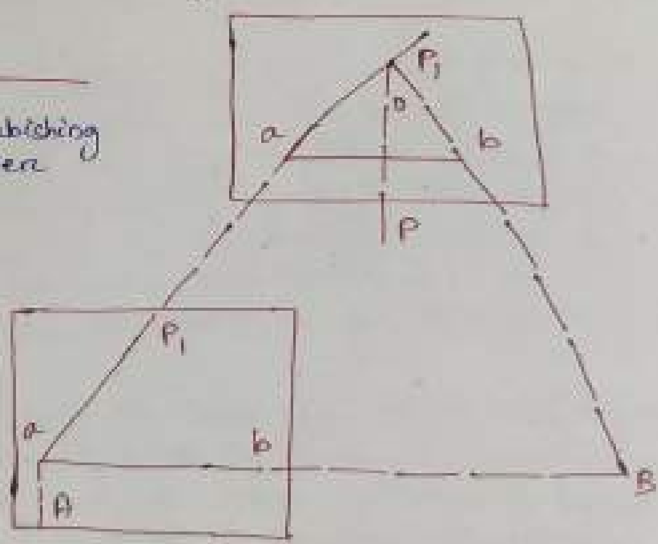
→ The table is shifted and set up at 'c' and the same procedure is repeated

→ In this manner, all stations of the traverse are connected.

→ At the end, the finishing and starting point may not coincide and there may be some closing error. The error is adjusted graphically by Bowditch's rule and the table is again setup at 'A', and adjustment is done.

4) RESECTION METHOD :—

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



⟹ Procedure :—

→ Suppose it is required to establish a station at position 'P'. Two points A and B on the ground are selected. The distance AB is measured and plotted to any suitable scale. This line AB is known as the 'base line'.

→ The table is set up at 'A', after that is levelled, centred and oriented by bisecting the ranging rod at B. The table is then clamped.

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

→ The table is shifted and centred in such a way that P_1 is just over 'P'. It is then oriented by backsighting the ranging rod at 'A'.

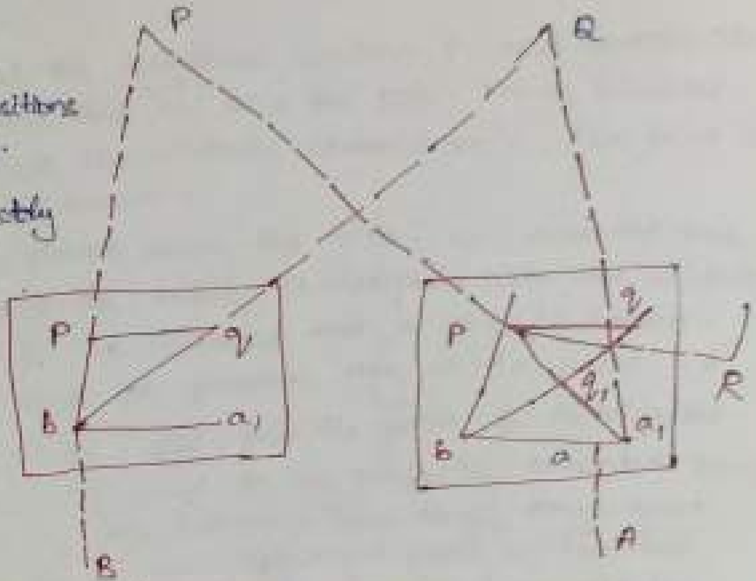
→ With the alidade touching point 'b', the ranging rod at 'B' is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point 'P'. This point represents the position of the station 'P' on the sheet. Then the actual position of the station 'P' is marked on the ground by U-fork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

* TWO-POINT PROBLEM :—

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

Procedure :—

→ Suppose P and Q are two well-defined points whose positions are plotted on map as 'p' and 'q'. It is required to locate a new station at 'A' by perfectly bisecting P and Q.



→ An auxillary station B is selected at a suitable position. The table is setup at 'B', and levelled and oriented by eye estimation. It is then clamped.

→ With the alidade touching 'p' and 'q', the points P and Q are bisected and rays are drawn. Suppose these rays intersect at 'b'.

→ With the alidade centred on 'b', the ranging rod at A is bisected and a ray is drawn. Then by eye estimation, a point a_1 is marked on this ray.

→ The table is shifted and centred on A, with a_1 just over A. It is levelled and oriented by backsighting. With the alidade touching 'p', the point 'P' is bisected and a ray is drawn. Suppose this ray intersects the line ba_1 at point a_1, as was assumed previously.

→ With the alidade centred on a_1 the point 'Q' is bisected and a ray is drawn. Suppose this ray intersects the ray bq at a point q_1. The triangle pqq_1 is known as the triangle of error, and is to be eliminated.

→ The alidade is placed along the line pq_1 and a ranging rod 'R' is fixed at some distance from the table. Then, the alidade is placed along the line 'pq' and the table is turned to bisect 'R'. At this position the table is said to be perfectly oriented.

→ Finally, with the alidade centred on 'p' and 'q', the points 'P' and 'Q' are bisected and rays are drawn. Suppose these rays intersect at a point "a". This would represent the exact position of the required station 'A'. Then the station 'A' is marked on the ground.

* THE THREE-POINT PROBLEM :—

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

The table is directly placed at the required position. The problem may be solved by three methods :— (a) the graphical or Bessel's method, (b) the mechanical method, and (c) the trial and error method.

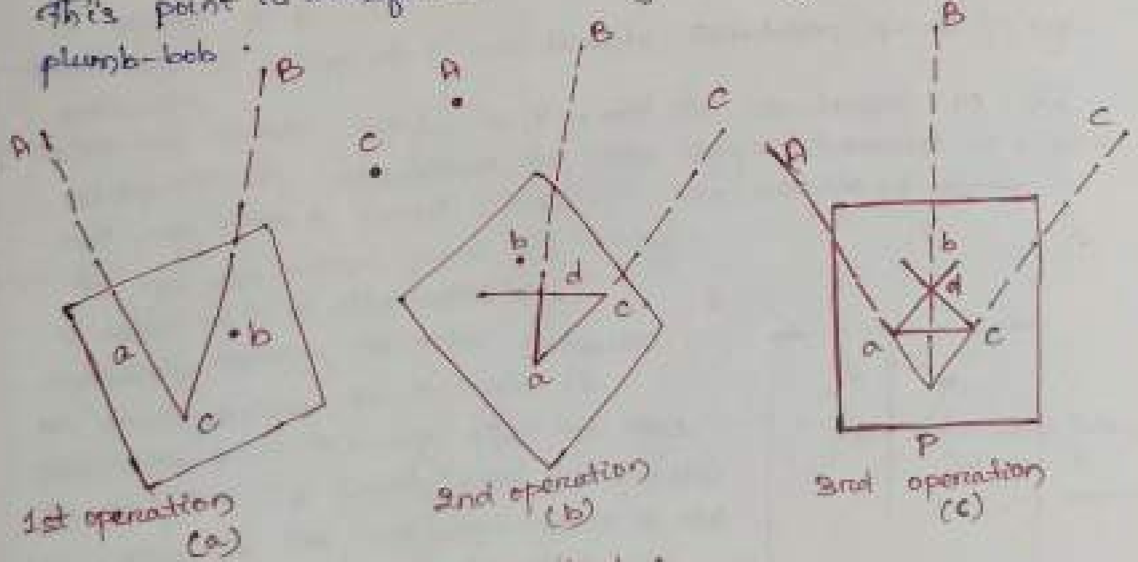
(a) The Graphical Method :—

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

The table is placed at the required station 'P' and levelled. The alidade is placed along the line 'ca' and the point 'A' is bisected. The table is clamped. With the alidade centred on 'c', the point 'B' is bisected and ray is drawn.

Again the alidade is placed along the line 'ac' and the point 'C' is bisected and the table is clamped. With the alidade touching 'a', the point 'B' is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point 'd'.

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



1st operation (a)

2nd operation (b)

3rd operation (c)

(b) The Mechanical Method :—

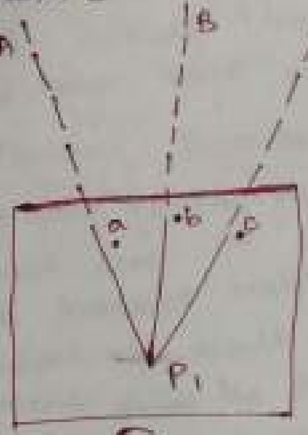
Suppose A, B and C are three well-defined points which have been plotted on the map as a, b, and c. It is required to locate a station at 'p'.

The table is placed at 'p' and levelled. A tracing paper is fixed on the map and a point 'p' is marked on it.

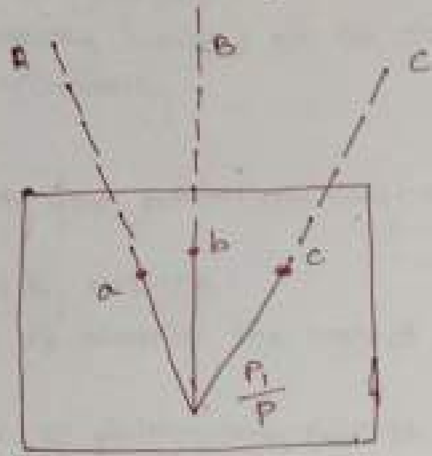
With the alidade centred on 'p' the points A, B, and C are bisected and rays are drawn. These rays may not pass through the points a, b and c as the orientation is done approximately.

Now the tracing paper is unfastened and moved over the in such a way that three rays simultaneously pass through the plotted positions a, b, and c. Then the point 'p' is pricked with a pin to give an impression 'p' on the map. 'p' is the required point on the map. The tracing paper is then removed.

→ Then the alidade is centred on 'p' and the rays are drawn towards A, B and C. These rays must pass through the points a, b, and c.



(a)



(b)

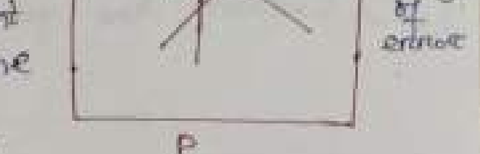
(C) The Method of Trial and Error :—

→ Suppose A, B, and C are three well-defined points which have been plotted as a, b, and c on the map. Now it is required to establish a station at P.

→ The table is set up at 'P' and levelled. Orientation is done by eye estimation.

→ With the alidade, rays Aa, Bb, and Cc are drawn. As the orientation is approximate, the rays may not intersect at a point, but may form a small triangle — the triangle of error.

→ To get the actual point, this triangle of error is to be eliminated. By repeatedly turning the table clockwise or anticlockwise, the triangle is eliminated in such a way that the rays Aa, Bb, and Cc finally meet at a point 'p'. This is the required point on the map. This point is transferred to the ground by U-fork and plumb-bob.



* ERRORS IN PLANE-TABLE :—

The following are the common errors in plane tabling :–

(A) Instrumental Error :—

→ The surface of the table may not be perfectly level.

→ The fiducial edge of the alidade might not be straight.

→ The vanes may not be vertical.

→ The horsehair may be loose and inclined.

→ The table may be loosely joined with the tripod stand.

(B) Personal Errors :—

- The levelling of the table may not be perfect.
- The table may not be centred properly.
- The orientation of the table may not be proper.
- The table might not be perfectly clamped.
- The objects may not be bisected perfectly.
- The alidade may not be correctly centred on the station point.
- The rays might not be drawn accurately.

(C) Plotting Errors :—

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

* PRECAUTIONS IN PLANE TABLE :—

- Before starting the work the equipments for survey work should be verified. Defective accessories should be replaced by perfect equipment.
- The centring should be perfect.
- The levelling should be proper.
- The orientation should be accurate.
- The alidade should be centred on the same side of the station-pin until the work is completed.
- While shifting the plane table from one station to another, the tripod stand should be kept vertical to avoid damage to the fixing arrangement.
- The pencil should have a sharp joint.
- Only the selected scale should be on the table.
- The stations on the ground are marked A, B, C, D, etc. while the stations points on the map are marked a, b, c, d etc.

———— X ————

* Note :—
The principle of plane table is parallelism.

MAP READING CADASTRAL MAPS & NOMECLATURE

CHAPTER-4

* STUDY OF DIRECTION :—

Direction is the most important thing. Direction is the way that we have to travel to get from one place or object to another place or object. It's usually measured in terms of compass directions :- north, south, east and west. North is directly up on standard maps; south is directly down; east is directly right; and west is directly left.

→ Like distance, direction is difficult to measure on maps because of the distortion produced by projection systems. However, this distortion is quite small on is usually measured relative to the location of North or South pole. Directions determined from these locations are said to be relative to True North or True South.

→ The magnetic poles can also be used to measure direction. However, these points on the Earth are located in spatially different spots from the geographic North and South pole. In the field, the direction of features is often determined by a magnetic compass which measures angles relative to Magnetic North. Using the declination diagram found on a Map, individuals can convert their field measures of magnetic direction into directions that are relative to either Grid or True North. Compass directions can be described by using either the azimuth system or the bearing system. The azimuth system calculates direction in degrees of a full circle. A full circle has 360° degrees. In the azimuth system, north has a direction of either 0° or 360°. East and west have an azimuth of 90° and 270°, respectively. Due south has an azimuth of 180°.

→ The bearing system divides direction into four quadrants of 90°. In this system, north and south are the dominant directions. Measurements are determined in degrees from one of these directions.

* SCALE :—

→ It is not always possible to represent the actual length of an object of an object on a drawing. So, it is required to reduce the object, in order to accomodate it on the drawing, in some proportion. The ratio by which the actual length of the object is reduced or increased is known as the "scale".

*** Full-size Scale :—** If the actual length of the object is shown on the drawing, the scale used is said to be a full-size scale.

*** Reducing Scale :—** If the actual length of an object is reduced in order to accomodate it on the drawing sheet the scale used is said to be a reducing scale

* **Increasing or Enlarging scale :**— If the actual length of an object is enlarged so as to bring out its details more clearly on the drawing, the scale used is said to be an "enlarging scale".

* **Representative Fraction (RF) :**— The ratio of the distance on the drawing to the corresponding actual length of the object is known as the "representative fraction", i.e.,

$$R.F = \frac{\text{distance on drawing of object}}{\text{corresponding actual distance of object}}$$

(both distances in same units) i.e., in "cm".

* For example,
  If a scale is 1cm = 10m, then

$$RF = \frac{1}{10 \times 100} = \frac{1}{1,000}$$

* **Types of Scale :**—
  Scales can be of the following four types :—
  (a) Plain.
  (b) Diagonal.
  (c) Comparative
  (d) Vernier

(a) **Plain Scale :**— This scale used to represent two successive units, such as 'kilometer', 'hectometers', metres, decimeters, metres 1/10th of metre and so on.

(b) **Diagonal Scale :**— This is a scale used to represent three successive units on one unit its fraction up to the second place of decimals, such as 'kilometer', 'hectometers', 'decameters', 'metres', decimetres, centimetres, and metres, 1/100th of a metre, and so on.

(c) **Vernier Scale :**— Vernier scales are sliding scales along a main scale and are used to measure parts of a main scale division. Vernier scale are designed on the assumption that the human eye can perceive accurately the coincidence of two lines, marking on the vernier scale and one on the main scale.

(d) **Comparative Scale :**— A comparative scale is a type of rating scale that is used to measure a survey respondent's preferences in terms of popular external benchmarks such as well-known service providers. It is also referred to as a comparative intensity scale.

(e) **Chord Scale :**— A scale of chords is prepared either to measure a given angle or to set off a given angle. This is done essentially by measuring or setting the chord forming the angle.

* **CONVERSION TABLE :**—
  * **Length :**—
  → 12 inch = 1 foot.
  → 3 feet = 1 yard.
  → 1 inch = 2.54 cm.
  → 1 foot = 0.3048 m
  → $5\frac{1}{2}$ yards = 1 rod or pole
  → 4 poles (66 ft) = 1 chain.

→ 10 chains = 1 furlong.
→ 8 furlongs = 1 mile.
→ 6 feet = 1 fathom.
→ 120 fathoms = 1 cable length.
→ 1 mile = 1,760 yards.
= 5,280 feet.
= 1.609 km.
→ 1 nautical mile = 6,080 feet.
→ 1 nautical mile = 1.152 miles.
→ 1 nautical mile = 1.852 km.
10 decametre = 1 hectametre.
10 hectametre = 1 kilometre.
1000 metres = 1 kilometre.

* Area :—
* $100 m^2$ = 1 are (a)
* 100 ares = 1 hectare.
* 100 hectare = $1 km^2$.
* 1 hectare = $10,000 m^2$.
* 640 acres = 1 square mile.
* 1 hectare = 2.47 acres.
* 484 square yards = 1 square chain.
* 10 square chain = 1 acre.
* 1 acre = 4,840 square yards.
= 3.025 bighas.
* 1 bigha = 1600 square yards.
* 20 kathas = 1 bigha.
* 16 chattak = 1 katha.
* 1 katha = 720 square feet.
* 1 chattak = 45 square feet.

* GRID REFERENCE AND GRID SQUARE :—

* Grid Reference :—
A grid of square helps the map-reader to locate a place. The vertical lines are called eastings. They are numbered - the numbers increase the east. The horizontal lines are called northings as the numbers increase in an northerly direction.
→ A grid reference system, also known as grid reference or grid system, is a geographic coordinate system that defines locations in maps using cartesian coordinates based on particular map projection. Grid lines on map illustrate the underlying coordinate system. Such coordinate lines

are numbered to provide a unique reference to each location on the map. Grid coordinates are normally eastings and northings.

→ Easting and northing are geographic cartesian coordinates for a point. Easting is the eastward measured distance (or the x-coordinate) and northing is the northward measured distance (or the y-coordinate). When using common projections such as the transverse Mercator projection, these are distances projected on an imaginary surface similar to a bent sheet of paper, and are not the same as distances measured on the curved surface of the Earth.

→ Easting and Northing coordinates are commonly measured in metres from the axes of some horizontal datum.

* **Grid Square :—**

Grid System vary, but the most common is a square grid with grid lines intersecting each other at right angles and numbered sequentially from the origin at the bottom left of the map. The grid numbered on the east-west (horizontal) axis are called Eastings, and the grid numbers on the north-south (vertical) axis are called Northings.

Numerical grid references consist of an even number of digits. Eastings are written before Northings. Thus in a 6 digit grid reference 123456, the easting component is 123 and the Northing component is 456, i.e., if the smallest unit is 100 metres, it refers to a point 12.3 km east and 45.6 km north from the origin.

→ Grids may be arbitrary, or can be based on specific distances, for example some maps use a one-kilometre square grid spacing.

→ A grid reference locates a unique square region on the map.

* **Note :—** A grid is a network of a series of vertical and horizontal lines constructed the perpendicular to each other. One series of lines runs from East to west and other from North to South.

* **Arbitrary grid :—** When grids of the maps are drawn on random data they are called arbitrary grid. It generally is drawn from field data and ~~they~~ then analyses.

* **Grid Square :—** At the time of drawing maps vertical and horizontal lines are drawn which intersect each other to create a grid. Each grid is termed as a grid square.

* CADASTRAL MAP PREPARATION METHODOLOGY :—

→ The word cadastral is derived from the french word "Cadastre" which means "register of territorial property".

→ Cadastral Maps are prepared to record the boundaries and ownership details of land properties such as Fields, Buildings, etc.

→ These maps are used to
→ assess the land tax and to indicate the ownership.
→ Village map is an example.

→ Cadastre — A parcel - based and up-to-date land information system containing a record of land that describes the geometry of parcels and linked to records such as nature of interests, ownership and value of the parcel and its improvement.

→ Cadastral Surveys document the boundaries of land ownerships, by the production of documents, diagrams, sketches, plans, charts, and maps. They were originally used ensure reliable facts for land valuation and taxation.

* Historically developed to collect taxes.
* Widely adopted to support land registration (legal purpose).
* A tool to improve land development (physical purpose).

→ Cadastral map consist of :—
* inventory of property parcels indicating parcel boundaries and unique parcel ~~code~~ identifier.
* register of interests (rights, restrictions, responsibilities). and interest holder (eg. owners).

Cadastral information on a computer system to determine its :—
* Cadastral ~~ide~~ identifier and links
* Location.
* Boundary Route.
* Land Use.
* Components distinguished with respect to a different function or landuse.
* Technical fittings (main connections).
* Land purpose assigned in the local spatial development plan.
* Destinct attributes of a particular real estate, especially its surface and value.

→ Mapping Requirements :—
* Basic scale shall be in the range of 1:1500 to 1:5000.
* Follow ~~case~~ cassine map projection.
* Datum need to be everest spheroid.
* Units shall be square meters.
* Units shall be standardized including symbols.
* Map contents need to be

→ Cadastral Survey Methods :—
(1) Chain / Tape Surveying.
(2) Plane Table Surveying.
(3) Total Station.
(4) Photogrammetry.
(5) Remote Sensing.
(6) GPS Method.

(1) Chain / Tape Surveying :—
→ It is primitive where only linear measurements are made.
→ It is suitable for survey of small areas with simple detail.

* Procedure :—
(1) Reconaissance, (2) marking of station, (3) Running of survey line, (4) Recording, (5) plotting.

(2) Plane–Table Surveying :—
→ It is graphical method in which field observation and plotting proceeds simultaneously.
→ It is simple and cheaper method.
→ This procedure is more accurate than chain surveying.

* Procedure :—
(1) Fixing, (2) Levelling, (3) centering, (4) Orientation, (5) sighting the points.

(3) Total – Station :—
It is electronic tacheometer, it is very accurate and applied for measuring fixed boundaries. It is mostly used as it provides better accuracy compared to other traditional method.

* Procedure :—
(i) Tripod setup, (ii) Mounting the total station,
(iii) centering & levelling, (iv) Instrument setup & observation.
(v) Field Book Recording.

(4) Photogrammetry :—
Under photogrammetry aerial and orthophotos are used to prepare the digital cadastral map.
→ It is less accurate than ground survey but is very fast and cheap.

(5) Remote Sensing :—
→ It is the latest method under which high resolution satellite images are initially acquired and then these images are convert-ed to orthophoto to extract the cadastral map.

* Procedure :—
(1) Image acquistion.
(2) Ortho image preparation.
(3) Extraction of cadastral data.

* UNIQUE IDENTIFICATION NUMBER OF PARCEL :—
→ The ULPIN (Unique Land Parcel Identification Number) is described as "Aadhaar for Land".

→ The ULPIN is a fourteen-digit Alpha Numeric ID.
→ The number will be used to identify every surveyed parcel of land.
→ The identification of number is to be launched based on latitude and longitude coordinates of the land parcel.
→ The ULPIN will help to develop a land bank.
→ The system will always to help to keep the land ~~records~~ records up-to-date.
→ The system will make sharing of land record data across the departments easier. This will standardize land data and will eventually bring in effective integration and interoperability across departments.

* POSITION OF EXISTING CONTROL POINTS AND ITS TYPES:-

A control point is a point on the ground or any permanent structure whose horizontal and vertical location/position is known. Control points are used as a starting point of all types of surveys.

→ Control points are known coordinates with a digital drawing that are physically marked out in the field.
→ These orient the tool to where it is on the field.
→ Usually clearly indicated on the drawing as "Xs" or intersection of gridlines.
→ Control establishment is an important exercise in mapping process. The mapping accuracy is directly based on the accuracy of control network. The control network is formed by a group of points whose position (x, y, z) are known to a high degree of accuracy.

⇒ Type of Control Point :—
Control Survey.
→ Horizontal Control Survey.
→ Vertical Control Survey.

① Horizontal Control Survey :—
Those control points whose coordinate (x, y) are known is called horizontal control point. These stations are established on the basis of the horizontal measurements like distances and directions. These positions can be referenced by parallels or plane coordinates axes because they are used as a framework for other surveys, these surveys must be precise and accurate. These surveys provide a network of points on the ground that ~~can~~ can be used as the control for any other surveying project, such as a boundary or construction survey. It helps in ~~finding~~ lost monuments or points can be replaced accurately, surveys can be coordinated, more than one network station can provide a check to the work, and a reduction in the cost of the project.

② Vertical Control Survey point :—
Those control points which are established on the vertical plane on the basis of distance is called vertical control point.

It determines elevation with respect to sea level. These surveys are also used as a benchmark upon which other surveys are based and high degree of accuracy is required. These surveys are useful for tidal boundary surveys, route surveys, construction surveys and topographic surveys. In a vertical control system atleast two permanent benchmarks should be used, but more may be required depending upon the needs and complexity of the project. These projects are needed for the construction of water and sewer systems, highways, bridges, drains and other major town or city infrastructure.

* ADJACENT BOUNDARIES AND FEATURES :—

Boundaries separate different regions of the Earth. Boundar-ies can be classified at many levels: they may be international (between countries), national (between states of a country), regional (between regions of a state), Local (between localities of a region or local government area) or as in the context of this paper - individual boundaries separating parcels of subdivided land.

→ Boundary lines (commonly called property lines) defines the extent of the legal limits of ownership of any parcel of land.

→ The following boundary features present conflicting evidence in the determination of a true boundary position, in order of priority :—

* Natural boundaries (eg., rivers, cliffs)
* Monumented lines (boundaries marked by survey or other defining marks, natural or artificial.
* Old occupations, long undisputed (example:- an old wall or fence.
* Abuttals (a described "bound" of the property. eg., a natural or artificial feature such as a street or road)

* TOPOLOGY CREATION AND VERIFICATION :—

Topology is a collection of rules that, coupled with a set of editing tools and techniques, enables the ~~geodat~~ geodatabase to more accurately model geometric relationship. ArcGIS implements topology through a set of ~~~~ rules that defines how features may share a geographic space and a set of editing tools that work with features that share geometry in an integrated fashion. A topology is stored in a geodatabase as one or more relationships that define how the features in one or more features classes share geometry.

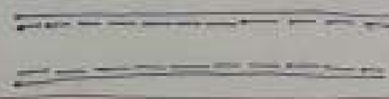
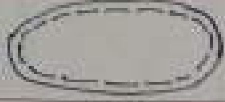
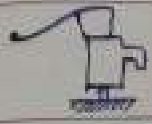
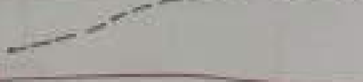
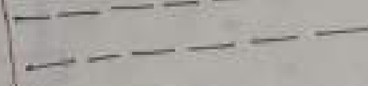
→ Topology is also used for analyzing spatial relationships in many situations ~~also~~ such as dissolving the boundaries between adjacent polygons with the same attribute values or traversing a network of the elements in a topology graph.

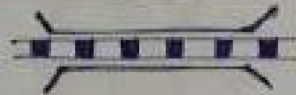
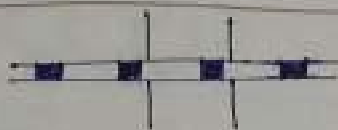
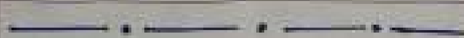
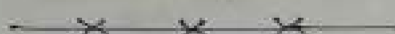
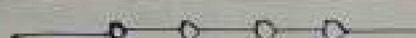
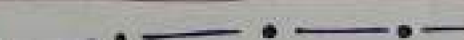
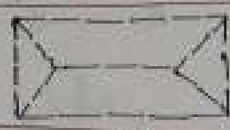
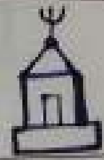
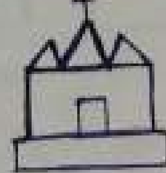
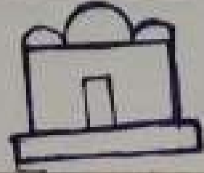
→ Features can share geometry within a topology.
- Here are some examples among adjacent features :—
  - * Area features can share boundaries (polygon Topography)
  - * Line features can share endpoints (edge topography)

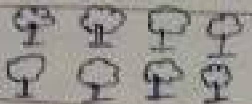
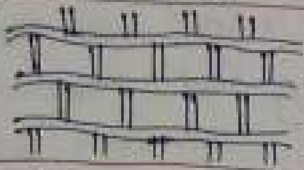
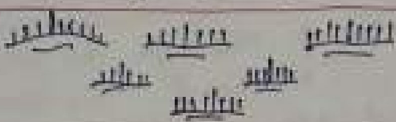
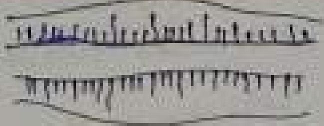
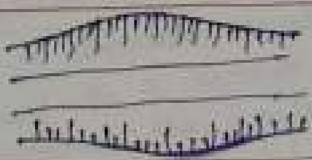
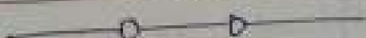
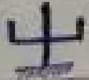
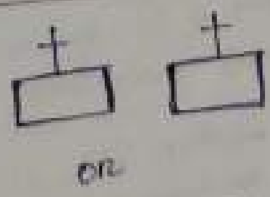
→ A topography survey is an accurate depiction of a site (property, area of land, defined boundary) which is scaled and detailed according to the spatial considerations and is the summary of the one-site data capture processes.

* STUDY OF SIGNS AND SYMBOLS :—

In a map of drawings or plans or in land surveying, the objects or an area is depicted by symbols not by names. So the surveyor should know the following standard conventional symbo-ls for some common objects.

Sl No.	Object	Symbol	Colour
1.	North Line		Black
2.	Main stations or triangulation stations.		Red or crimson lake.
3.	Traverse stations or substations.		Red or crimson lake
4.	chain line		Red or crimson lake.
5.	River.		Prussian blue.
6.	Canal		Prussian blue.
7.	Lake or pond		Prussian blue.
8.	Open well		Prussian lake.
9.	Tube well		Black.
10.	Footpath		Black.
11.	Metalled road		Burnt sienn-a
12.	Unmetalled road		Burnt sienna

Sl No.	Object	Symbol	Colour
13.	Railway line (single)		Black
14.	Railway line (double)		Black.
15.	Road bridge or culvert		Black
16	Railway bridge or culvert.		Black
17	Level-crossing		Black and Burnt sienna.
18	Wall with gate.		Black.
19	Boundary line		Black.
20	Hedge		Green.
21	Wire fencing.		Black.
22	Pipe fencing		Prussian blue.
23	Wood fencing		Yellow.
24	Building (Pukka)		Crimson lake.
25	Building (Kutcha)		Umber (brown)
26	Huts		Yellow
27	Temple.		Crimson lake.
28	Church.		Crimson lake.
29.	Mosque		Crimson lake.

Sl No.	Object	Symbol	Colour
30.	Benchmark		Black.
31.	Tree		Green
32.	Jungle		Green
33.	Orchard		Green
34	Cultivated land		Black and Green
35.	Barren Land		Black
36.	Rough pasture		Black.
37	Marsh or swamp		Black.
38.	Embankment		Black.
39.	Cutting.		Black.
40.	(a) Telegraph line		Black
	(b) Telegraph post		Black
41.	(a) Electric line		Black.
	(b) Electric post.		Black.
42.	Burial ground or cemetery	 OR 	Crimson lake

THEODOLITE SURVEYING AND TRAVERSING CHAPTER 6

The introduction is an intricate instrument used mainly for accurate measurement of horizontal and vertical angles up to 10" or 20", depending upon the least count of the instrument. Because of its various uses, the theodolite is sometimes known as a universal instrument.

→ The following are the different purposes for which the theodolite can be used :—

* Measuring horizontal angles.
* Measuring vertical angles.
* Measuring deflection angles.
* Measuring magnetic bearings.
* Measuring the horizontal distance between two points.
* Finding the vertical height of an object.
* Finding the difference of elevation between various points.
* Ranging a line.

* TYPES OF THEODOLITE :—

Theodolites may be of two types — (i) transit theodolite, and (ii) non-transit theodolite.

→ In transit theodolite, the telescope can be revolved through a complete revolution about its horizontal axis in a vertical plane.

→ In non-transit theodolite, the telescope cannot be revolved through a complete revolution in vertical plane. But it can be revolved to a certain extent in the vertical plane in order to measure the angle of elevation or depression.

* Theodolites may also be classified as :- (i) vernier theodolites — when fitted with a vernier scale, and (ii) micrometer theodolites — when fitted with a micrometer.

Note → The size of the theodolite is defined according to the diameter of the main horizontal graduated circle.

* DEFINITIONS OF THEODOLITE SURVEYING :—

(1) Centring :— The setting of a theodolite exactly over a station mark by means of a plumb-bob is known as centring. The plumb-bob is suspended from a hook fixed below the vertical axis.

(2) Transiting :— The method of turning the telescope about its horizontal axis in a vertical plane through 180° is termed as transiting. Transiting results in a change in face.

(3) Face left :— 'Face left' means that the vertical circle of the theodolite is on the left of the observer at the time of taking readings. The observation taken in the face left position

is called faced left observation.

(4) Face Right :— The situation when the vertical circle of the instrument is on the right of the observer when the reading is taken. The observation taken in the face right position.

(5) Telescope Normal :— The face left position is known as telescope normal, or telescope direct. It is also referred to as 'bubble up'.

(6) Telescope Inverted :— The face right position is called telescope inverted or telescope reversed. It is also referred to as "bubble down".

(7) Changing face :— The operation of bringing the vertical circle from one side of the observer to the other is known as changing face.

(8) Swinging the telescope :— This indicates turning of the telescope in a horizontal plane. It is called 'right swing' when the telescope is turned clockwise and 'left swing' when the telescope is turned anticlockwise.

(9) Line of Collimation :— It is an imaginary line passing through the intersection of the cross hairs at the diaphragm and the optical centre of the object glass and its continuation.

(10) Axis of the telescope :— This axis is an imaginary line passing through the optical centre of the object glass and the optical centre of the eye-piece.

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

(12) Vertical axis :— It is the axis of rotation of the telescope in the horizontal plane.

(13) Horizontal axis :— It is the axis of rotation of the telescope in the vertical plane. It is also known as the trunnion axis.

(14) Temporary adjustment :— The setting of the theodolite over a station at the time of taking any observation is called temporary adjustment.

(15) Permanent adjustment :— When the desired relationship between the fundamental lines of a theodolite is disturbed, then some procedures are adopted to establish this relationship.

(16) Least count of the vernier :— This is the difference between the values of the smallest division of the main scale and that of the smallest division of the vernier scale. It is the smallest value that can be measured by a theodolite.

* If V = Value of smallest division of vernier scale.
d = value of smallest division of main scale.

Least count = $\frac{d}{n}$

⇒ If d = 20′ and n = 60

Least count = $\frac{20}{60} \times 60 = \underline{20''}$

⇒ If d = 15′ and n = 60

Least count = $\frac{15}{60} \times 60 = \underline{15''}$.

(14) Magnification or magnifying power of telescope :— The magnifying power of a telescope is the ratio of the focal length of the objective to that of the eye-piece.

(15) The diaphragm :— The diaphragm is a brass ring consisting of cross-hairs, or one containing a glass disc with fine lines engraved on it. It is fixed in front of the eye-piece. The cross-hairs may be made of spider web or fine platinum wire. They may also be in the form of a fine scratch mark engraved on glass.

(19) Sensitiveness of bubble tube :— The ability of a bubble tube show a very small deviation of the bubble from its horizontal position is termed as the sensitiveness of the bubble tube.

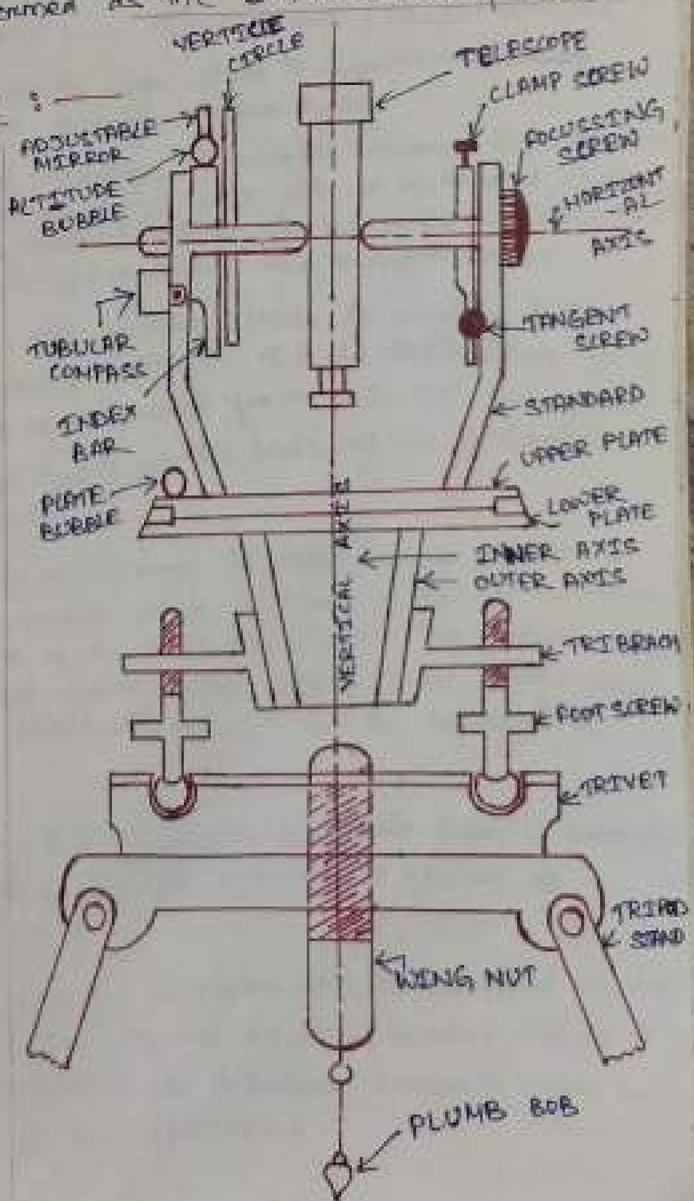
* TRANSIT THEODOLITE :—

The following are the essential parts of a theodolite :—

(1) Trivet :— It is a circular plate having a central, threaded hole for fixing the theodolite on the tripod stand by a wing nut. It is also called the base plate. Three foot screws are secured to this plate by means of a ball-and-socket arrangement.

(2) Foot screws :— These are meant for levelling this instrument. The lower part of the foot screws are secured in the trivet by means of a ball-and-socket arrangement and the upper threaded part passes through the threaded hole in the tribach plate.

(3) Tribach :— It is a triangular plate carrying three foot screws at its ends.



(4) Levelling Head :— The trivet, foot screws and the tribrach constitute a body which is known as the levelling head.

(5) Spindles :— The theodolite consists of two spindles of axes – one inner and the other outer. The inner axis is solid and conical, and the outer is hollow. The two spindles are coaxial.

(6) Lower plate :— The lower plate is attached to the outer axis, and is also known as the scale plate. It is bevelled and the scale is graduated from 0° to 360° in a clockwise direction. Each degree is again subdivided into two, three or four divisions; thus the value of one small division may be 30′, 20′ or 15′ respectively.

→ The lower plate is provided with a clamp screw and a tangent screw which control its movement.

(7) Upper plate :— The upper plate contains the vernier scales A and B. It is attached to the inner axis. Its motion is controlled by the upper clamp screw and the upper tangent screw. When the clamp screw is tightened the vernier scales are fixed with the inner axis, and for the fine adjustment of the scales the tangent screw is rotated.

(8) Plate bubble :— Two plate bubbles are mounted at right angles to each other on the upper surface of the vernier plate. One bubbles is kept parallel to the horizontal axis of the theodolite.

(9) Standard or 'A' frame :— Two frames (shaped like the letter 'A') are provided on the upper plate to support the ~~telescop~~ telescope, the vertical circle and the vernier scales.

(10) The telescope :— The telescope is pivoted between the standards at right angles to the horizontal axis. It can be rotated about its horizontal axis in a vertical plane. The telescope is provided with a focussing screw, clamping screw and tangent screw.

(11) Vertical Circle :— The vertical circle is rigidly fixed with the telescope and moves with it. It is divided into four quadrants. Each quadrant is graduated from 0° to 90° in opposite directions, with the 'zero' mark at the ends of the horizontal diameter of the vertical circle.

(12) Index bar or T-frame :— The index bar is provided on the standard in front of the vertical circle. It carries two verniers (C and D) at the two ends of the horizontal arm. The vertical leg of the index bar is provided with a clip screw at the lower end by means of which the altitude bubble can be brought to the centre.

(13) Altitude bubble :— A long sensitive bubble tube is provided on the top of index bar. The bubble it contains is known as altitude bubble.

(14) Compass :— Sometimes a circular box is mounted on the vernier scale between the standards. In modern theodolites an adjustable trough compass or tubular compass can be fitted with a screw to the standard.

* FUNDAMENTAL AXES OF A THEODOLITE :—

The fundamental axes of theodolite — (1) Line of collimation, (2) Vertical Axis, (3) Axis of telescope, (4) Axis of bubble tube or bubble axis, (5) Horizontal axis.

* CONCEPT OF VERNIER :—

Vernier theodolite is also known as a transit. Vernier is used to determine the least count.

* READING THE VERNIER THEODOLITE :—

The least count of the vernier is to be determined first. Let it be 20". The main division of the main scale is of one degree. Suppose it is divided into three parts. Then each part accounts for 20' (i.e., d = 20')

The vernier scale has 20 big and 60 small divisions.

So, least count $= \frac{d}{n} = \frac{20}{60} \times 60 = 20''$.

Here, least count of one small division $= 20''$.

∴ Least count for one big division $= (20'' \times 3) = 60'' = 1'$.

* TEMPORARY ADJUSTMENT OF THE THEODOLITE :—

Temporary adjustment of the theodolite :—

(1) Setting the theodolite over the station :—

The tripod stand is placed over the required station. The theodolite is fixed on the stand by means of a wing nut.

(2) Approximate levelling by tripod stand :—

The legs of the tripod stand are placed well apart and firmly fixed on the ground.

(3) Centring :—

Centring is the process of setting the instrument exactly over a station. The plumb bob suspended from the hook under the vertical axis lies approximately over the station peg.

(4) Levelling :—

Before starting the levelling operation, all the footscrews are brought to the centre of their run.

→ The plate bubble is placed parallel to any pair of foot screws. By turning both screws equally inwards or outwards, the bubble is brought to the centre.

→ The plate bubble is turned through 90° so that it is perpendicular to the line joining the first and second foot screws. Then by turning the third foot screw either clockwise or anticlockwise the bubble is brought to the centre.

→ The process is repeated several times, so that the bubble remains in the central position of the plate bubble, both directions perpendicular to each other.

→ The instrument is rotated through 360° about its vertical axis. If the bubble still remains in the central position, the adjustment of the bubble is perfect and the vertical axis is truly vertical.

→ **Focussing the eye-piece :—**

The eye-piece is focussed so that the cross-hairs can be seen clearly. To do this, the telescope is directed towards the sky or a piece of white paper is held in front of the object glass, and the eye-piece is moved in or out by turning it clockwise or anticlockwise until the cross-hairs appear distinct and sharp.

→ **Focussing the object glass :—**

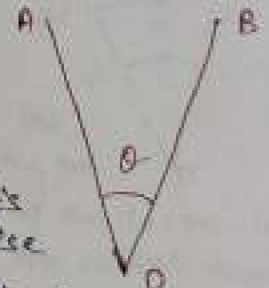
This is done to bring a sharp image of the object or target in the plane of cross-hairs and to eliminate parallax. The focussing screw is turned clockwise or anticlockwise until the image appears clear and sharp there is no relative movement between the image and cross-hairs.

→ **Setting the Vernier :—**

The vernier A is set to 0° and vernier B to 180°. To do this the upper clamp is loosened and the the lower clamp is fixed. The upper plate turned until the arrow of vernier A approximately coincides with zero (i.e., the 360° mark) and that of vernier B approximately coincides with the 180° mark. Then the upper clamp is tightened, and by turning the upper tangent screw the arrows are brought to a position of exact coincidence.

*** DIRECT METHOD OF MEASURING HORIZONTAL ANGLE :—**

Suppose an angle ∠AOB is to be measured. The following procedure is adopted.



→ The instrument is set up over 'O', then it is centred, levelled. The instrument was initially in the face left position.

→ The lower clamp is fixed and the upper clamp is loosened, and by turning the telescope clockwise vernier A is set at 0° and vernier B is set at 180° approximately.

→ The upper clamp is then tightened. Now by turning the upper tangent screw, verniers A and B are set exactly 0° and 180°.

→ The upper clamp is tightly fixed. The lower one is loosened and the telescope is directed to the left-hand object A. The ranging rod at A is bisected approximately by properly focussing the telescope and eliminating parallax. The lower clamp is tightened, and by turning the lower tangent screw the ranging rod at A is accurately bisected.

→ The lower clamp is kept fixed. The upper clamp is loosened and the telescope is turned clockwise to approximately bisect the ranging rod at B by properly focussing the telescope. The upper clamp is tightened, and the ranging rod at B bisected accurately by turning the upper tangent screw.

→ The readings on verniers A and B are noted. Vernier A gives the angle directly. But in the case of vernier B, the angle is obtained by subtracting the initial reading from the final readings. The readings are noted in tabular form.

→ The face of the instrument is changed and the previous procedure is followed. The readings of the verniers are noted in the table.

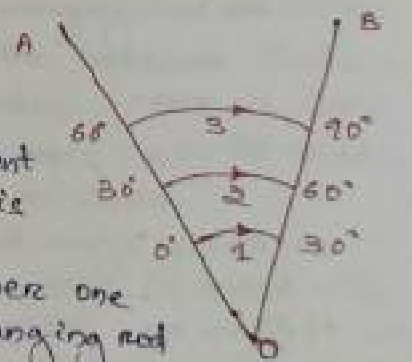
→ The mean of the observations (i.e., face left and face right) is the actual ∠AOB. The two observations are taken to eliminate any possible error due to imperfect adjustment of the instrument.

* There are two methods of measuring horizontal angles :—
  (1) Repetition and (2) Reiteration Method.

(1) Repetition Method :—

In this method, the angle is added a number of times. The total is divided by the number of readings to get the angle. The angle should be measured clockwise in the face left and face right positions, with three repetition at each face. The final reading of the first observation will be the initial reading of the second observation, and so on.

* Procedure :—



→ Suppose the angle ∠AOB is to be measured by the repetition process. The theodolite is set up at 'O'. The instrument is centred and levelled properly. Vernier 'A' is set to 0° and vernier 'B' to 180°.

→ The upper clamp is fixed, and the lower one loosened. By turning the telescope, the ranging rod at A is perfectly bisected with the help of the lower clamp screw and the lower tangent screw. Here the initial reading of vernier A is 0°.

→ The upper clamp is loosened and the telescope is turned clockwise to perfectly bisect the ranging rod at B. The upper clamp is clamped. Suppose the reading on vernier A is 30°.

→ The lower clamp is loosened and the telescope turned anticlockwise to exactly bisect the ranging rod at A. Here, the initial reading is 30° for the second observation.

→ The lower clamp is tightened. The upper one is loosened and the telescope is turned clockwise to exactly bisect the ranging rod at B. Let the reading on vernier A be 60°.

→ The initial reading for the third observation is set to 60°. ∠AOB is again measured. Let the final reading on the vernier A be 90°, which is the accumulated angle.

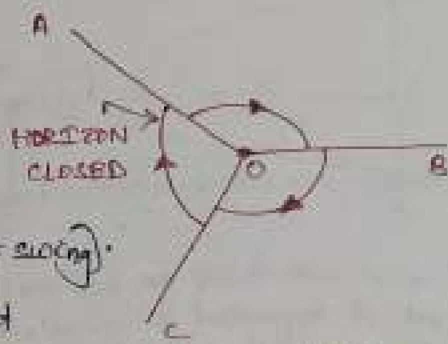
$$\angle AOB = \frac{\text{accumulated angle}}{\text{no. of reading}} = \frac{90°}{3} = 30°.$$

- The face of the instrument is changed and the previous procedure is followed.
- The mean of the two observations gives the actual angle ∠AOB.

(2) Reitration Method :—

This method is suitable when several angles are measured from a single station. In this method all the angles are measured successively and finally the horizon is closed. So, the final reading of the leading vernier should be the same as its initial reading. Suppose it is required to measure ∠AOB and ∠BOC from station 'O'. The procedure is as follows:-

* First Set :—



- The theodolite is perfectly centred over 'O' and levelled properly in the usual manner. Suppose, the observation is taken in the face left position and the telescope is turned clockwise (right swing).
- Vernier A is set to 0° (i.e 360°) and vernier B to 180°.
- The upper clamp is fixed and the lower one loosened. The ranging rod at A is bisected perfectly. Now, the lower clamp is tightened.
- The upper clamp is loosened, and the ranging rod or object at B is bisected properly by turning the telescope clockwise. The readings on both the verniers are taken. ∠AOB is noted.
- Similarly, the object 'C' is bisected properly, and the readings on the verniers are noted. ∠BOC is recorded.
- Now the horizon is closed, i.e., the last angle ∠COA is measured. The position of the leading vernier is noted. The leading vernier should show the initial reading on which it was set.

* Second Set :—

- The face of the instrument is changed. Again the verniers are set at their initial positions. This time the angles are measured anticlockwise (Left swing).
- The upper clamp is fixed, and the lower one loosened. Then the object 'A' is perfectly bisected.
- The lower clamp is tightened. The telescope is turned anticlockwise, and the object 'C' bisected by loosening the upper clamp screw. The readings on both the verniers are taken. ∠COA is noted.
- Then the object B is bisected by turning the telescope anticlockwise, and the readings on the verniers are taken. ∠BOC is recorded.
- Finally, the horizon is closed i.e., the object A is

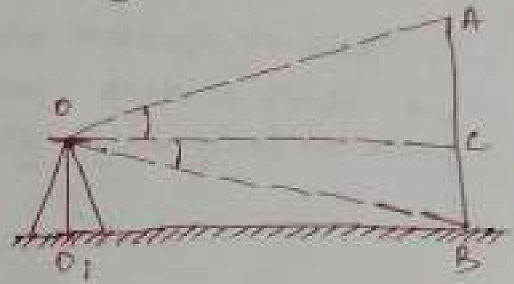
bisected. Here the leading vernier A should show a reading of 0°. The ~~initially~~ last angle ∠AOB is noted.

The mean angles of the two sets give the actual values of the angles.

* MEASURING VERTICAL ANGLE :—

The vertical angle is the one between the horizontal line (line of collimation) and the inclined line of sight. When it is above the horizontal line, it is known as the angle of elevation. When this angle is below the horizontal line, it is called the angle of depression.

* Consider, the angle of elevation ∠AOC and that of depression ∠BOC are to be measured. The following procedure is adopted.



→ The theodolite is set up at O. It is centred and levelled properly. The zeros of the verniers (generally C and D) are set at the 0°–0° mark of the vertical circle (which is fixed to the telescope). The telescope is then clamped.

→ The plate bubble is brought to the centre ~~on~~ with the help of footscrews. Then the altitude bubble is brought to the centre by means of a clip screw. At this position the line of collimation is exactly horizontal.

→ To measure the angle of elevation, the telescope is raised slowly to bisect the point A accurately. The readings on both the verniers are noted, and the angle of elevation recorded.

→ The face of the instrument is changed and the point A is again bisected. The readings on the verniers are noted. The mean of the angles of the observed is assumed to be the correct angle of elevation.

→ To measure the angle of depression, the telescope is lowered slowly and the point B is bisected. The readings on the verniers are noted for the two observations (face left and face right). The mean angle of the observation is taken to be the correct angle of depression.

* MEASUREMENT OF DEFLECTION ANGLE :—

The deflection angle is the angle by which a line is deflected from its original direction. It is the angle which a survey line makes with the extension of the preceding line.

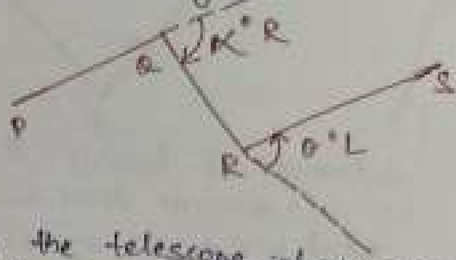
The deflection may be towards the right or the left, depending upon whether the angle is measured in the clockwise or anticlockwise direction from the extension of the preceeding line. Deflection angles are measured for designing horizontal curves in railways, highways, etc.

To measure the deflection angles at Q :—

→ Set the instrument at 'Q' and level it.

→ With both plates clamped at 0°, take back sight on 'P'.

→ Plunge the telescope. Thus the line of sight is in the direction PQ produced when the reading on vernier A is 0°.



→ Unclamp the upper clamp and turn the telescope clockwise to take the foresight on R. Read both the verniers.

→ Unclamp the lower clamp and turn the telescope to sight P again. The verniers still read the same reading as above. Plunge the telescope.

→ Unclamp the upper clamp and turn the telescope to sight 'R'. Read both verniers. Since the deflection angle is double by taking both face readings, one-half of the final reading gives the deflection angle at 'Q'.

* MEASUREMENT OF MAGNETIC BEARING :—

In order to measure the magnetic bearing of a line, the theodolite should be provided with either a tubular compass or trough compass.

→ Set the instrument at P and level it accurately.

→ Set ~~accuracy~~ accurately the vernier A to zero.

→ Loose the lower clamp. Release the needle of the compass. Rotate the instrument about its outer axis till the magnetic needle roughly points to north. Clamp the lower clamp. Using the lower tangent screw, bring the needle exactly against the mark so that it is in magnetic meridian. The line of sight will also be in the magnetic meridian.

→ Loose the upper clamp and point the telescope towards "Q". Bisect Q accurately using the upper tangent ~~screw~~ screw.

→ Read verniers "A" and 'B'.

→ Change the face and repeat the above steps. The average of the two will give the correct bearing of the line "PQ".

* TO MEASURE DIRECT ANGLES :—

"Direct angles" are the angles measured clockwise from the preceding (previous) line to the following (next) line. They are also known as angles to the right or azimuths from the back line and may vary from 0° to 360°. To measure the angle PQR.

→ Set the theodolite at Q and level it accurately. With face left, set the reading on vernier A to zero.

→ Unclamp the lower clamp and direct the telescope to 'P'. Bisect it accurately using the lower tangent screw.

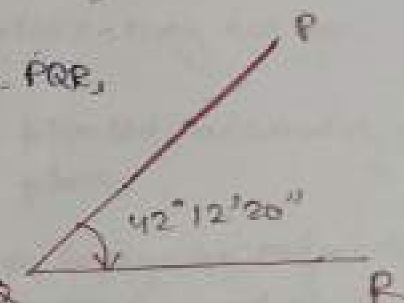
→ Unclamped the upper clamp and swing telescope clockwise and sight R. Bisect R accurately using the upper tangent screw. Read both verniers.

→ Plunge the telescope, unclamp the lower clamp and take backsight on P. Reading on the vernier will be the same as above step.

→ Unclamp the upper clamp and bisect R again. Read the verniers. The reading will be equal to twice the angle. ∠PQR will then be obtained by dividing the final reading by two.

* SETTING OUT ANGLES :—

Let it be required to lay off the angle PQR, say 42°12'20".



→ Set the instrument at 'Q' and level it.

→ Using upper clamp and upper tangent screw, set the reading on vernier A to 0°.

→ Loose the lower clamp and sight "P". Using lower tangent screw, bisect P accurately.

→ Loose upper clamp and turn the telescope till the reading is approximately equal to the angle PQR. Using upper tangent screw, set the reading exactly equal to 42°12'20".

→ Depress the telescope and establish R in the line of sight.

* PROLONGING A STRAIGHT LINE WITH THEODOLITE :-

The prolonging a straight line such as 'AB' to a point 'P' which is not already defined upon the ground and is invisible from A and B.

→ Set the instrument at A and sight B accurately. Establish a point 'C' in the line of sight. Shift the instrument at 'B', sight 'C' and establish point 'D'. The process is continued until 'P' is established.

* ERRORS IN THEODOLITE :—

The sources of errors in theodolite are :—

(1) Instrumental Errors, (2) Personal Errors,
(3) Natural Errors.

(1) Instrumental Errors :—

- → Non-adjustment of plate bubble.
- → Line of collimation not being perpendicular to horizontal axis.
- → Horizontal axis not being perpendicular to vertical axis.
- → Line of collimation not being parallel to axis of telescope.
- → Eccentricity of inner and outer axes.
- → Graduations not being uniform.
- → Verniers being eccentric.

(2) Personal Errors :—

- → The centring, levelling may not be done perfectly.
- → The clamp screws are not properly fixed, the instrument may slip.
- → The proper target screw may not be ~~oriented~~ operated by mistake.
- → The focussing in order to avoid parallax may not be perfectly done.
- → The object or ranging rod may not be bisected accurately.
- → The verniers may not be set in proper place.

(3) Natural Errors :—

- → High temperature causes error due to irregular refraction.
- → High winds cause vibration in the instrument, and this may lead to wrong readings on the vernier.

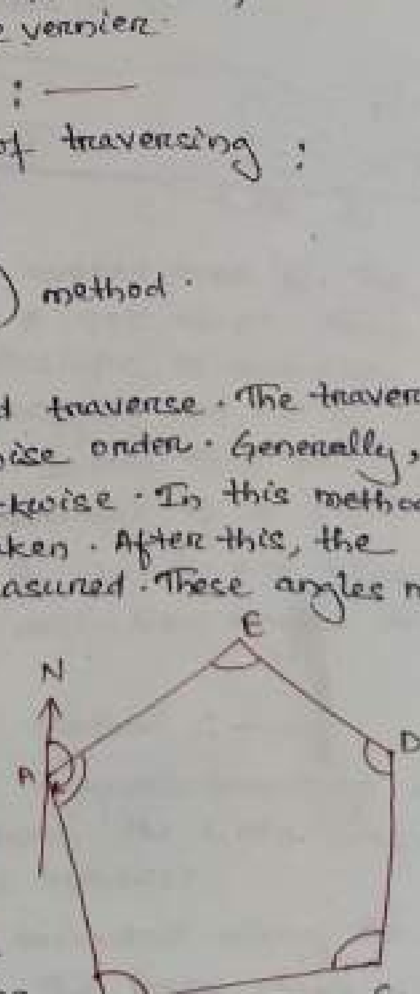
* METHODS OF THEODOLITE TRAVERSE :—

The following are the different methods of traversing :

1) Included angle method.
2) Deflection angle method.
3) Fast angle (or magnetic bearing) method.

1) Included Angle Method :—

This method is most suitable for closed traverse. The traverse may be taken in clockwise or anticlockwise order. Generally, a closed traverse is taken in the anticlockwise. In this method the bearing of the initial line is taken. After this, the included angles of the traverse are measured. These angles may be interior or exterior.



⟹ Procedure :—

→ The theodolite is setup and centred over A. The plate bubble is levelled. Vernier A is set at 0° and Vernier 'B' at 180°. The upper clamp is fixed.

→ The telescope is oriented along the north line with the help of the tubular compass fitted to the instrument.

Then the magnetic bearing of AB is measured.

→ Again vernier A is set at 0° and the upper clamp is kept fixed.

→ The lower clamp is loosened and the ranging rod at E is bisected. Now, this clamp is tightened and the upper one opened. By turning the telescope clockwise, the ranging rod at 'B' is bisected. The readings on the verniers are noted. ∠A is obtained in this fashion.

The face of the instrument is changed and ∠A is measured once more. The mean of the two observations gives the correct value of ∠A.

→ Similarly, the other angles are measured by centring the theodolite at B, C, D and E.

The arithmetical check is applied as follows:

$$(2n-4)\times 90^\circ = \text{Sum of interior angles.}$$

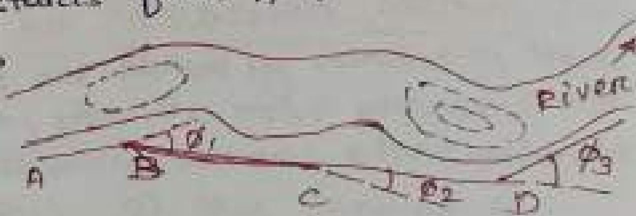
→ For plotting the traverse, latitudes and departures of the traverse legs are calculated. The interior details are marked by applying the ~~planeta~~ plane-table.

2) Deflection Angle Method :—

This method is suitable for open traverse and is mostly employed in the survey of rivers, coast lines, roads, railways etc.

Suppose an open traverse starts from A.

→ The theodolite is set up at A, and then centred and levelled. After this, bearing of the line AB is measured in the usual manner.



→ The theodolite is now shifted and centred over 'B'. The plate bubble is levelled and vernier A set at 0°. Then a backsight is taken on 'A'. The telescope is transited and by turning it clockwise the ranging rod at 'C' is bisected. The vernier readings are taken. Then the deflection angle ϕ_1 is determined (average of A & B).

→ Similarly, the other deflection angles ϕ_2 and ϕ_3 are measured.

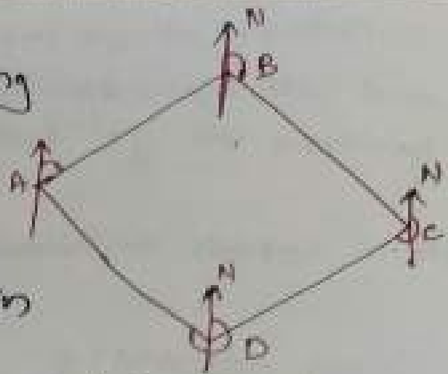
→ A field book is prepared in which the deflection angles and offsets are clearly noted.

3) Fast Needle (Magnetic Bearing) Method :—

This method is used to measure the magnetic bearings and lengths of traverse legs. The angles between the lines are not measured. Suppose ABCDA is a closed traverse.

→ The theodolite is set up at A and oriented along the north line with the help of compass. The lower clamp is fixed.

→ The upper clamp is loosened and the ranging rod at B is bisected. The reading on vernier A gives the fore bearing of AB : say it is 30°. The back-bearing of the DA is also measured from 'A'. Now the upper clamp is also fixed. The traverse is considered in clockwise direction.



→ The instrument is shifted and set up at B with vernier A fixed at the reading of 30°. The lower clamp is loosened and the ranging rod at A is bisected. The telescope is now transited. The upper clamp is then released and the ranging rod at 'C' bisected. Now the reading on vernier 'A' gives the bearing of 'BC' : say it is 100°.

→ Again the instrument is shifted and set up at C with vernier A fixed at 100°.

→ The same process is repeated to get the fore bearing of 'CD'.

→ Similarly, the fore bearings of the remaining sides are measured.

→ At the end of the traverse the FB and BB of DA should differ by 180°.

* PLOTTING THE TRAVERSE BY COORDINATE METHOD :—

In this method, survey stations are plotted by calculating their co-ordinates. This method is by far the most practical and accurate one for plotting traverse or any other extensive system of horizontal control. In this method of plotting the closing error can be eliminated by balancing, prior to plotting.

→ consecutive coordinates — Latitude and Departure.

* CHECKS FOR OPEN AND CLOSED TRAVERSE :—

→ check in closed traverse :—

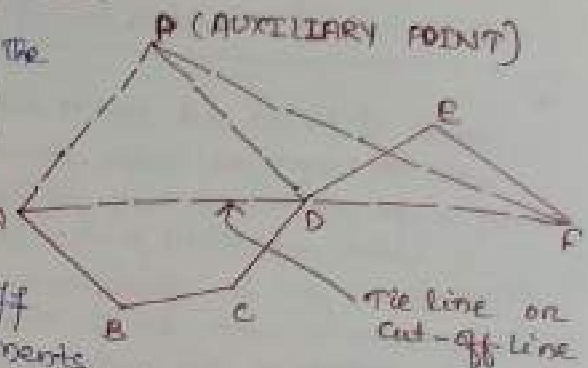
* The sum of the measured interior angles should be equal to $(2N-4) \times 90°$.
* The sum of the measured exterior angles should be equal to $(2N+4) \times 90°$, where 'N' is the number of sides
* The algebraic sum of the deflection angles should be equal to 360°, considering right-hand deflection to be positive and left-hand deflection negative.
* The fore bearing and back bearing of the finishing line should differ by 180°.
* The chaining of each line should be done twice, along opposite direction.

* Check after computation : The sum of the northings should be equal to that of the southings, and the sum of the ~~eastings~~ eastings should equal that of the westings.

2) Check in Open Traverse :—

In open traverse, measurements cannot be checked in the field.

(i) Tie line or cut-off line :—



Suppose ABCDEF is an open traverse. The cut-off lines AD and DF are suitably taken. The FB and BB of lines AD and DF are measured, and so are distances AD and DF. If after plotting the traverse, the distances, FB and BB of the cut-off line tally with the field measurements then the traverse is said to be correct.

(ii) Auxiliary point :—

An auxiliary point 'P' is suitably selected on one side of the traverse. Then magnetic bearings of this point are taken from A, D and F. If the traverse and plotting have been done accurately, then all these bearings must meet at 'P' when plotted from the stations.

The traverse can also be checked by calculating the coordinates of the point 'P', considering ADP and DFP as closed figures.

If the co-ordinates of 'P', calculated from both sides, are equal, then the traverse may be assumed to be correct.

* COMPUTATION OF LATITUDE AND DEPARTURE :—

The theodolite traverse is not plotted according to interior angles or bearings. It is plotted by computing the latitudes and departures of the points (consecutive coordinates) and then finding the independent coordinates of the points.

The latitude of a line is the distance measured parallel to the North-South line and the departure of a line is measured parallel to the East-West line.

→ Latitude = $l\cos\theta$, Departure = $l\sin\theta$.

→ Northing = latitude towards north = +L

Southing = latitude towards south = −L

Easting = departure towards east = +D

Westing = departure towards west = −D.

* Check for closed traverse

(1) Sum of northings = sum of southings.

(2) Sum of eastings = Sum of westings.

Line	Length	Reduced Bearing (θ)	Latitude ($L\cos\theta$)	Departure ($L\sin\theta$)
AB	L	NθE	$+L\cos\theta$	$+L\sin\theta$
BC	L	SθE	$-L\cos\theta$	$+L\sin\theta$
CD	L	SθW	$-L\cos\theta$	$-L\sin\theta$
DA	L	NθW	$+L\cos\theta$	$-L\sin\theta$

Check for closed traverse

→ The algebraic sum of latitudes must be equal to zero

→ The algebraic sum of departures must be equal to zero.

1) Consecutive Coordinates:— The latitude and departure of a point calculated with reference to the preceding point for what.

2) Independent Coordinates:— The coordinates of any point with respect to a common origin are said to be the independent coordinates of that point. The origin may be a station of the survey or a point entirely outside the traverse.

* GALE'S TRAVERSE TABLE :—

Traverse computations are usually done in a tabular form, a more common form being Gales Traverse Table. For complete traverse computations, the following steps are usually necessary:

→ Adjust the interior angles to satisfy the geometrical conditions i.e., sum of interior angles to be equal to $(2N-4)\times 90^\circ$ and exterior angles $(2N+4)\times 90^\circ$.

→ In the case of a local attraction the bearings are adjusted, if any.

→ Starting with observed bearings of one line, calculate the bearings of all other lines. Reduce all bearings to quadrantal system.

→ Calculate the consecutive co-ordinates (i.e., latitudes and departures).

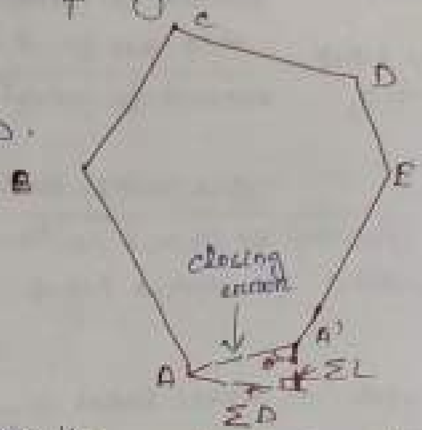
→ Calculate ΣL and ΣD

→ Apply necessary corrections to the latitudes and departures of the lines so that $\Sigma L = 0$ and $\Sigma D = 0$. The corrections may be applied either by transit rule or by compass rule depending upon the type of traverse.

→ Using the corrected consecutive coordinates, calculate the independent co-ordinates to the points so that they are all positive, the whole of the traverse thus lying in the North East quadrant.

* CLOSING ERROR :—

If a closed traverse is plotted according to the field measurements, the end point of the traverse will not coincide exactly with the starting point, owing to the error in the field measurements of angles and distances. Such error is known as closing error.



→ The error of closure for such traverse may be ascertained by finding ΣL and ΣD.

→ closing error (e) $= AA' = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$

The direction of closing error is

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

→ The sign of ΣD and ΣL will thus define the quadrant in which the closing error lies.

$$= \frac{\text{Error of closure}}{\text{Perimeter of traverse}} = \frac{e}{P} = \frac{1}{P/e}$$

→ **Adjustment of the Angular Error :—**

Before calculating latitudes and departures, the traverse angles should be adjusted to satisfy geometric conditions. In a closed traverse, the sum of interior angles should be equal to $(2N-4)\times 90°$. If the angles are measured with the precision, the error in the sum of angles may be distributed equally to each angle of the traverse. If the angular error is small, it may be arbitrarily distributed among two or three angles.

→ **Adjustment of Bearings :—**

In a closed traverse in which bearings are observed the closing error in bearing may be determined by comparing the two bearings of the last line as observed at the first and last stations of traverse. Let 'e' be the closing error in bearing of last line of a closed traverse having 'N' sides.

Correction for first line $= \frac{e}{N}$

Correction for second line $= \frac{2e}{N}$

Correction for third line $= \frac{3e}{N}$

Correction for last line $= \frac{Ne}{N} = e$

* BALANCING OF TRAVERSE :—

In case of closed traverse, the algebraic sum of latitudes must be equal to zero and that of departures must also be equal to zero in the ideal condition.

→ The total errors ~~are to~~ in latitude and departure are determined. These errors are then distributed among the traverse stations ~~proportionately~~ proportionately, according to Bowditch's rule, Transit rule etc.

(1) Bowditch Rule :— The total error (in latitude or departure) is distributed in proportion to the lengths of the traverse legs.

(a) Correction to latitude of any side = $\frac{\text{length of that side}}{\text{perimeter of traverse}} \times$ total error in latitude.

(b) Correction to departure of any side = $\frac{\text{length of that side}}{\text{perimeter of traverse}} \times$ total error in departure

(2) Transit Rule :—

(a) Correction to latitude of any side = $\frac{\text{latitude of that side}}{\text{arithmetical sum of all latitude}} \times$ total error in latitude.

(b) Correction to departure of any side
= $\frac{\text{departure of that side}}{\text{arithmetical sum of all departure}} \times$ total error in departure.

(3) Graphical Method :—

A graphical method of applying the Bowditch's rule is employed to avoid calculating latitudes and departures.

→ Draw a line equal to the perimeter of the traverse to a suitable scale. Mark the stations A, B, C, D, etc. as per their length and the scale chosen.

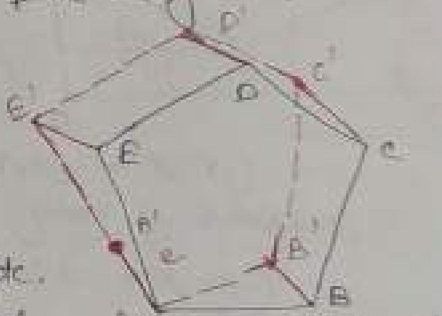
→ Draw at the end of the line the closing error in magnitude and direction. For this, draw a line parallel to the closing error and mark on it the magnitude of the closing error to the same scale as the traverse scale. Join the starting point to the end of the closing error.

→ Draw lines parallel to the closing error through points B, C, D, etc.

→ Draw parallel lines through the traverse points to the closing error.

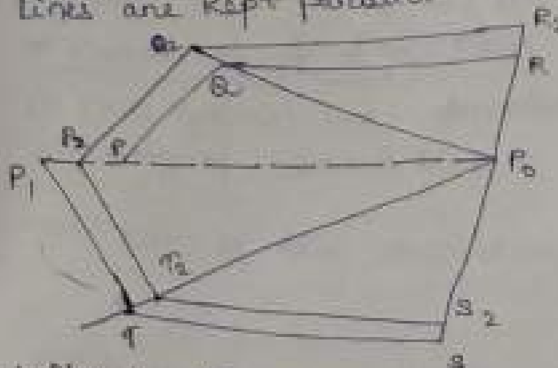
→ Mark on these lines corrections BB', CC', etc.

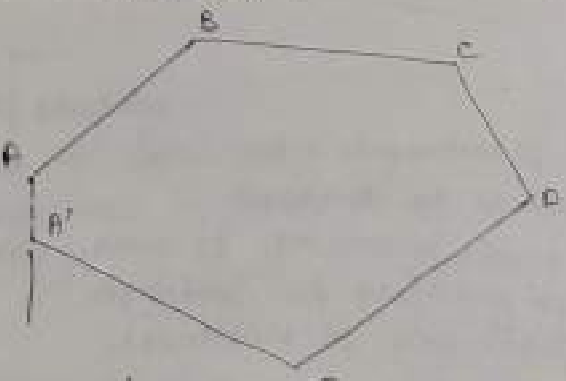
→ Join A', B', C', D', E', A' to get the corrected traverse.



(4) AXIS METHOD :—

In this method, it is assumed that the angles are measured very precisely. In the adjusted traverse, the lengths are altered but the lines are kept parallel to their initial positions.





→ Plot the traverse to a suitable scale and find the closing error. The plot obtained is PQRSTP₁. The closing error is PP₁.

→ Extend the closing error into the traverse; the line may cut the traverse approximately into two equal parts.

→ In case the closing error line is extended to cut a traverse line at a point P_1. This line is known as the axis for correction. This line should divide the traverse into two nearly equal parts.

→ Divide the closing error PP_1 into two equal parts with the point P_2. Thus, $PP_1 = P_1P_2$.

→ Carry out the corrections taking P_2 as the correct location of P. Connect the axis point P to points Q, R, S, and T of the traverse and extend if necessary.

→ Draw lines parallel to the existing lines of the traverse. Thus, draw a line parallel to PQ through P_2 to get the point Q_2 on the line PQ or its extension.

→ Repeat this with all the stations to get the connected traverse $P_2-Q_2-R_2-S_2-T_2-P_2$. This traverse closes keeping the directions of the lines the same as before.

* CALCULATION OF AREA OF CLOSED AREA :—

The area of a closed traverse may be calculated from :—

(1) The coordinates (x and y)

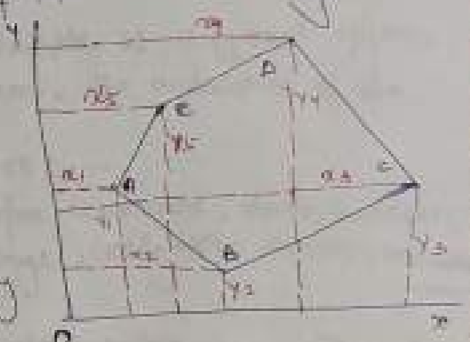
(2) The latitude and double meridian distance.

(3) The departure and total latitudes.

(1) Calculation of area from coordinates :—

The given consecutive coordinates of a traverse are converted into independent coordinates with reference to the coordinates of the most westerly station.

Then the coordinates are arranged in determinant form as follows:



$$\begin{matrix} y_1 & & y_2 & & y_3 & & y_4 & & y_5 & & y_1 \\ & \times & & \times & & \times & & \times & & \times & \\ x_1 & & x_2 & & x_3 & & x_4 & & x_5 & & x_1 \end{matrix}$$

The sum of the products of coordinates joined by solid lines.

$$\Sigma P = (y_1 x_2 + y_2 x_3 + y_3 x_4 + y_4 x_5 + y_5 x_1)$$

The sum of the products of coordinates joined by dotted lines,

$$\Sigma Q = (x_1 y_2 + x_2 y_3 + x_3 y_4 + x_4 y_5 + x_5 y_1)$$

∴ Double area $= \Sigma P - \Sigma Q$

So, required area $= \frac{1}{2} \times (\Sigma P - \Sigma Q)$

(2) Calculation of Area from Latitude and Double Meridian Distance (DMD) :—

It is the distance equal to the sum of the meridian distances of the two ends of the lines.

* Methods of finding DMD :—

(i) DMD of first line = departure of first line.

(ii) DMD of second line = DMD of first line + departure of first line + departure of second line

(iii) DMD of any succeeding line = DMD of preceeding line + departure of preceding line + departure of line itself.

(iv) DMD of last line = departure of last line with opposite sign.

Procedure calculating area :—

→ Each DMD is multiplied by the latitude of that line.
→ The algebraic sum of these products is worked out.
→ The sum is equal to twice the area.
→ Half of this sum gives the required area of the traverse.

(3) Calculating of Area from Departure and Total Latitude :—

→ The total latitude (the latitude with respect to the reference point) of each station of the traverse is found out.
→ The algebraic sum of departures of the two lines meeting at a station is determined.
→ The total latitude is multiplied by the algebraic sum of departures for each individual point.
→ The algebraic sum of this product gives twice the area.
→ Half of this sum gives the required area.

—— x ——

LEVELLING AND CONTOURING

CH-7

→ Definition and Purpose of levelling :—

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

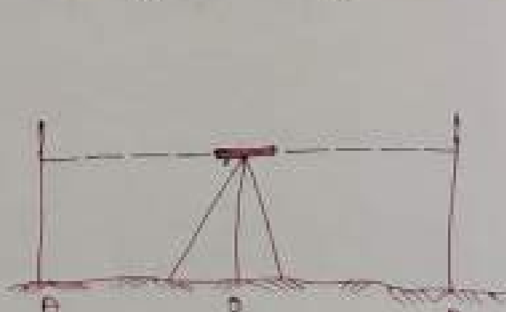
→ Levelling is done for the following purposes :—

① To prepare a contour map for fixing sites for reservoirs, dams, barrages etc, and to fix the alignment of roads, railways, irrigation canals, and so on.
② To determine the altitudes of different important points on a hill or to know the reduced the levels of different points on or below the surface of the earth.
③ To prepare a longitudinal section and cross-sections of a project (roads, railways, irrigation canals, etc) in order to determine the volume of earthwork.
④ To prepare a layout map for watersupply, sanitary or drainage schemes.

⇒ Types of Levelling :—

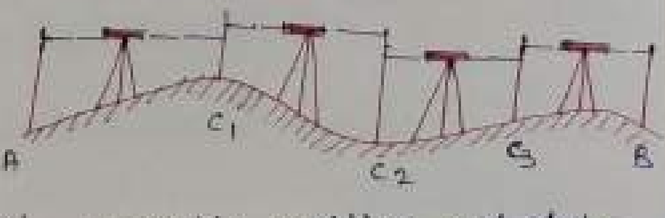
1) Simple levelling :—

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



2) Differential levelling :—

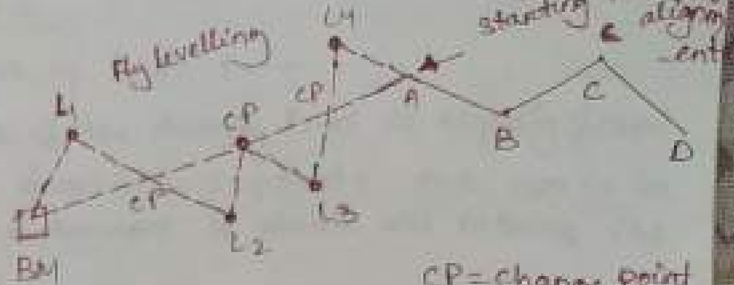
It is adopted when :— the points are a great distance apart, the difference of elevation between the points is large, there are obstacles between the points.



→ This method is also known as compound levelling or continuous levelling. In this method, the level is set up at several suitable positions and staff readings are taken at all of these.

(3) Fly levelling :—

When differential levelling is done in order to connect a benchmark to the starting point of the alignment of any project, is called fly levelling.

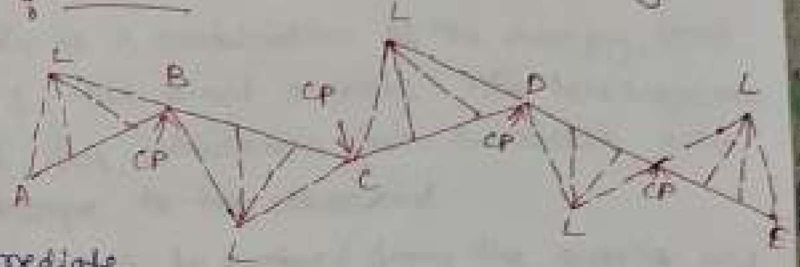


CP = change point
L1 L2 ... = position of level.

→ It is done to connect the BM to any intermediate point of the alignment for checking the accuracy of the work.

(4) Longitudinal or profile levelling :—

The operation of taking levels along the centre line of any alignment (road, railway, etc.) at regular intervals is known as longitudinal or profile levelling.



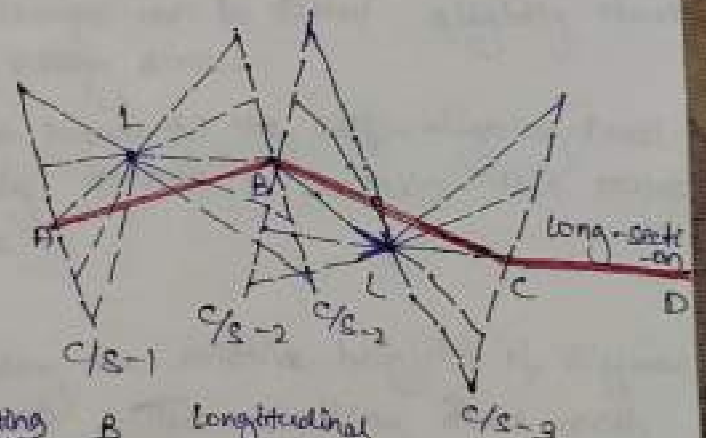
→ In this operation, the backsight, intermediate sight and foresight reading are taken at regular intervals, at every set up of the instrument.

L = position of level.
C.P = change position.

(5) Cross-Sectional Levelling :—

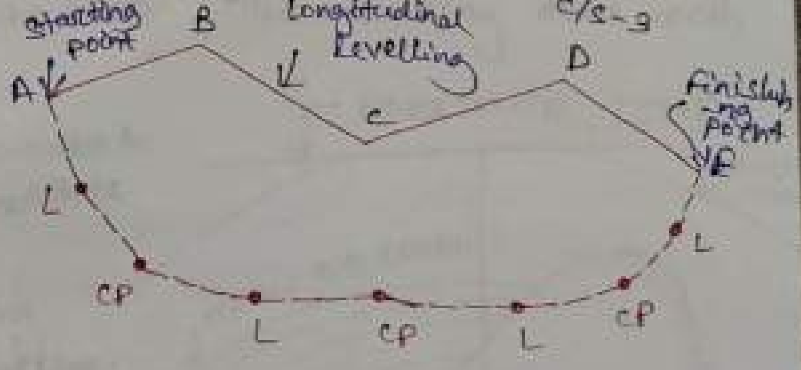
The operation of taking levels transverse to the direction of longitudinal levelling, is known as cross-sectional levelling.

The cross-sections are taken at regular intervals (such as 20m, 40m, 60m, etc.) along the alignment.



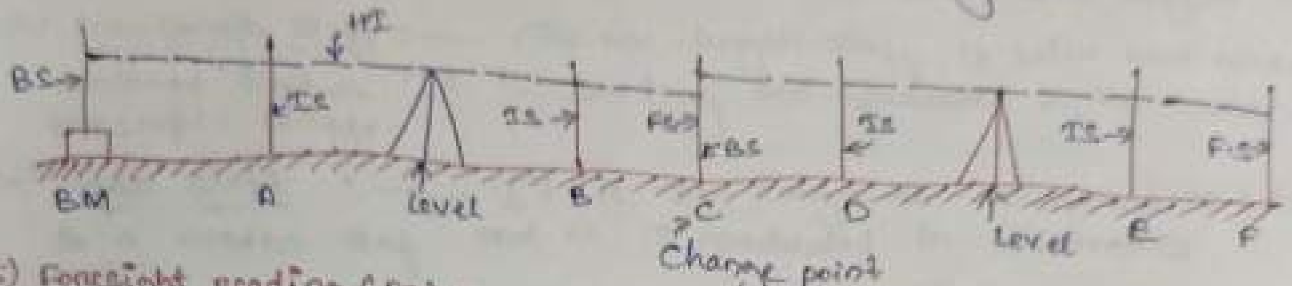
(6) Check Levelling :—

The fly levelling done at the end of day's work to connect the finishing point with the starting point on that particular day is known as check levelling.



13) Temporary Bench-marks :— When the bench-marks are established temporarily at the end of a day's work, they are said to be temporary bench-marks.

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



15) Foresight reading (FS) :— It is the last staff reading in any setup of the instrument, and indicates the shifting of the latter.

16) Intermediat sight reading (IS) :— It is any other staff reading between the BS and FS in the same set up of the instrument.

17) Change point (CP) :— This point indicates the shifting of the instrument. At this point, an FS is taken from one setting and a BS from the next setting.

18) Height of Instrument (HI) :— When the levelling instrument is properly levelled, the RL of the line of collimation is HI. This is done by ~~[illegible]~~ adding the BS reading to the RL of the BM or CP on which the staff reading was taken.

19) Focussing :— The operation of setting the eye-piece and the object glass a proper distance apart for clear vision of the object.

20) Parallax :— The apparent movement of the image relative to the cross-hair is known as parallax. This occurs due to imperfect focussing, when the image doesnot fall in the plane of the diaphragm.

* LEVELLING STAFF :— The levelling staff is a graduated wooden rod used for measuring the vertical distances between the points on the ground and the line of collimation. They are classified as :— The target staff, The self-reading staff.

(i) Target staff :— It consists of a movable target. The target is provided with a vernier which is adjusted by the staffman, according to directions from the Levelman, so that the target coincides with the collimation hair.

* DIFFERENT TYPES OF LEVELLING :—

The following are the different types of level :—

1) **The dumpy level** :— The telescope of the dumpy level is rigidly fixed to its support. It cannot be removed from its supports nor can it be rotated adjust its longitudinal axis. The instrument is stable and retains its permanent adjustment for a long time.

2) **The wye level (Y-level)** :— The telescope is held in two 'Y' supports. It can be removed from the supports and reversed from one end of the telescope to the other end.

3) **Cooke's reversible level** :— This is a combination of the dumpy level and the Y level. It is supported by two rigid sockets. The telescope can be rotated about its longitudinal axis, withdrawn from the socket and replaced from one end of the telescope to the other end.

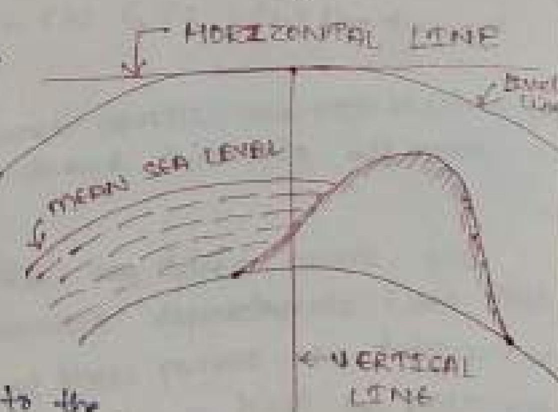
4) **Cushing's level** :— The telescope cannot be removed from the sockets and rotated about its longitudinal axis. The eye-piece and object glass are removable and can be interchanged from one end of the telescope to the other end.

5) **The modern tilting level** :— The telescope can be tilted slightly about its horizontal axis with the help of a tilting screw.

6) **The automatic level** :— This is also known as the self-aligning level. This instrument is levelled automatically within a certain tilt range by means of a compensating device.

* DEFINITIONS :—

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



2) **Level Surface** :— Any surface parallel to the mean spheroidal surface of the earth is said to be a level surface.

3) **Level line** :— Any line lying on a level surface is called a level line. This line is normal to the plumbline at all points.

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

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

6) **Vertical line** :— The direction indicated by a plumbline is known as the vertical line. This line is perpendicular to the horizontal line.

7) Vertical Plane :— Any plane passing through the vertical line.

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

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

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

11) Axis of the telescope :— The axis is an imaginary line passing through the optical centre of the object glass and the optical centre of the eye-piece.

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

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

→ Benchmarks may be of four types :- (a) GTS, (b) Permanent, (c) Temporary, and (d) Arbitrary.

(a) GTS Bench-marks :— These bench-marks are established by the survey of India Department at a large intervals all over the country.

(b) Permanent Bench-marks :— These are fixed points or marks established by different Government departments like PWD, Railways, Irrigation, etc. The RLs of these points are determined with reference to the GTS bench-mark and are kept on permanent points like the plinth of a building, a parapet of a bridge or culvert and so on.

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

(ii) Self-reading staff :— The following are the different types of self-reading staff.

(a) Sop-with Telescope staff :— It is arranged in three lengths placed one into the other. It can be extended to the full length by pulling. The total length of staff is 4m.

(b) Folding Metric Staff :— This is made of well-seasoned timber, and is of width 75mm, thickness 18mm and length 4m.

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

(d) Invar Staff :— It is 3m long. An invar band is fitted to a wooden staf and it is graduated in millimetres.

* DUMPY-LEVEL PARTS :—

(1) Tripod stand :— The tripod stand consists of three legs which may be solid or framed. The lower end of the leg is fitted with steel shoes.

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

(3) Foot screws :— Three foot screws are provided between the trivet and tribatch.

(4) Telescope :— The telescope consists of two metal tubes, one moving within other. It also consists of an object glass and an eye-piece on opposite ends. A diaphragm is fixed with the telescope just in front of the eye-piece. The diaphragm carries cross-hairs.

(5) Bubble Tubes :— Two bubble tubes, one called the longitudinal bubble tube and other the cross-tube, are placed at right angles to each other. These tubes contain spirit bubble.

(6) Compass :— A compass is provided just below the telescope for taking the magnetic bearing of a line when required.

* TEMPORARY ADJUSTMENT OF LEVEL :—

The different steps to be followed in temporary adjustment.

1) Selection of suitable position :— A suitable position is selected for setting the level. The ground should be fairly level.

2) Fixing level with tripod stand :— The tripod stand is placed at the required position with its legs well apart and pressed firmly into the ground.

3) Approximately levelling by legs of tripod stand :— The footscrews are brought to the centre of their run. Two legs of the tripod stand are firmly fixed into the ground. Then the third leg is moved to the left or right in or out until the bubble is approximately at the centre of its run.

4) Perfect levelling by foot screw :— The longitudinal bubble is placed parallel to any pair of footscrews and the bubble is brought to the centre by turning the footscrews equally either both inwards or both outwards. The telescope is then turned through 90° and brought over the third footscrew, and the bubble is brought to the centre by turning this foot screw clockwise or anticlockwise. This process is repeated several times until the bubble remains in the central position in the first as well as second position.

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

6) Focussing the object glass :— The telescope is directed towards the levelling staff. Looking through the eye-piece the focussing screw is turned clockwise or anticlockwise until the graduation of the staff is distinctly visible and the parallax is eliminated.

7) Taking the staff reading :— Finally, the levelling of the instrument is verified by turning the telescope in any direction. When the bubble remains in the central position for any direction of the telescope, the staff readings are taken.

* METHODS OF CALCULATION OF REDUCED LEVEL :-

The following are the ~~ay~~ two systems of calculating reduced level :-

(1) The collimation system or height of instrument system (HI).

(2) The rise and fall system.

(1) The Collimation System or height of instrument system (HI) :-

The reduced level of the line of collimation is said to be the height of the instrument. In this system, the height of the line of collimation is found out by adding the backsight reading ~~of~~ to the RL of the BM on which the BS is taken.

$$HI = RL + BS$$

→ Then the RL of the intermediate points and the change point are obtained by subtracting the respective staff readings from the height of the instrument (HI).

RL of intermediate sight (IS) = HI − IS

RL of ~~inter~~ foresight (FS) = HI − FS

→ **Arithmetical check** :—

$$\Sigma BS - \Sigma FS = \text{Last RL} - \text{1st RL}$$

(2) The Rise-and-fall System :—

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

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

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

→ **Arithmetical Check** :—

$$\Sigma BS - \Sigma FS = \Sigma \text{Rise} - \Sigma \text{Fall} = \text{Last RL} - \text{1st RL}.$$

* Comparison of collimation system and Rise-and-fall system :—

Collimation System	Rise-and-fall System
(1) It is rapid as it involves few calculation.	(1) It is laborious, involving several calculation.
(2) There is no check on the RL of intermediate points.	(2) There is a check on the RL of intermediate points.
(3) Errors in intermediate RLs cannot be detected.	(3) Errors in intermediate RLs can be detected as all the points are correlated.
(4) There are two checks on the accuracy of RL calculation.	(4) There are three checks on the accuracy of RL calculation.
(5) This system is suitable for longitudinal levelling where there are a number of intermediate sights.	(5) This system is suitable for fly levelling where there are no intermediate sights.

* CORRECTIONS TO BE APPLIED :—

1) Curvature Correction :— (Cc)

For long sights, the curvature of the earth affects staff readings. The line of sights is horizontal, but the level line is curved and parallel to the mean spheroidal surface of the earth.

Curvature Correction = $C_c = \frac{D^2}{2R}$

$$C_c = \frac{D^2 \times 1000}{12,742} = 0.0785 D^2 \text{ m}$$ (negative)

Hence, True staff reading = observed staff reading − Curvature correction.

2) Refraction Correction :— C(r)

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

The refraction correction is taken as one-seventh of the curvature correction.

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

Refraction Correction, $C_r = \frac{1}{7} \times 0.0785 D^2 = 0.0112 D^2$ m (positive)

3) **Combined Correction :—**

The combined correction is the effect of curvature and refraction

Combined Correction = Curvature correction + refraction correction

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

$$= \boxed{-0.0673D^2} \text{ m.}$$

* The combined correction is always subtractive (i.e., negative)

True staff reading = observed staff reading − combined correction

→ It may be expressed as

$$\frac{D^2}{2R} - \frac{1}{7} \times \frac{D^2}{2R}$$

$$= \boxed{\frac{6D^2}{14R}} \quad \text{(negative)} \quad \text{or} \quad \boxed{-0.00673D^2}$$

4) **Visible horizon distance :—**

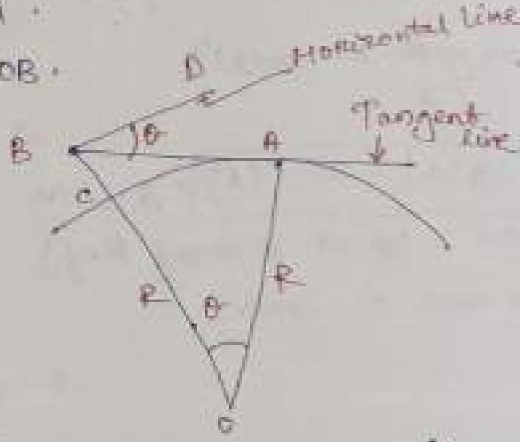
AB = D = visible horizon distance in kilometres.

h = height of the point above mean sealevel in metres.

$$h = 0.0673D^2$$

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

5) **Dip of horizon :—**



AB = D = tangent to the earth at A.

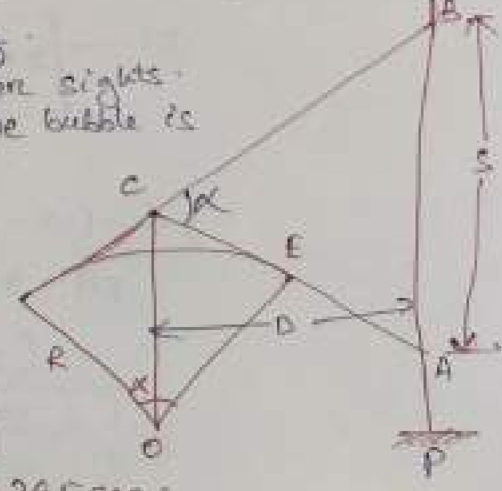
BD = horizontal line perpendicular to OB.

θ = dip of horizon.

$$\boxed{\theta = \frac{D}{R} \text{ in radians}}$$

$$\boxed{\theta = \frac{D}{R} \times \frac{180 \times 60}{\pi}} \text{ minutes.}$$

6) **Sensitiveness of the bubble :—**



D = distance between the level and staff

S = intercept between the upper and lower sights.

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

R = radius of curvature of the tube.

α = angle subtended by arc EE1

d = length of one division of the graduation.

$$\alpha' = \frac{S}{Dn} \text{ radian}$$

$$\boxed{R = \frac{nd \times D}{S}}, \quad \boxed{\alpha' = \frac{S}{Dn} \times 206{,}265 \text{ sec}}$$

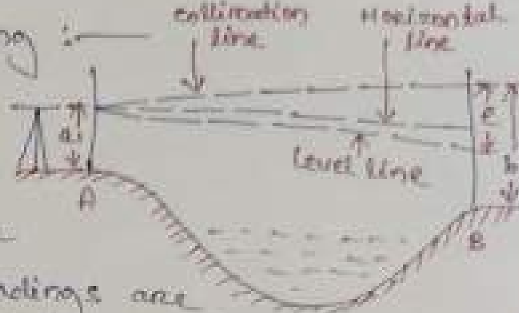
1 radian = 206,265 sec.

RECIPROCAL LEVELLING :—

In case of river or valley, the level is set between two points on opposite banks. In such case, reciprocal levelling is adopted which involves reciprocal ~~levelling~~ observations from both banks of the river or valley.

The level is set up on both banks of the river or valley and two sets of staff readings are taken by holding the staff on both banks.

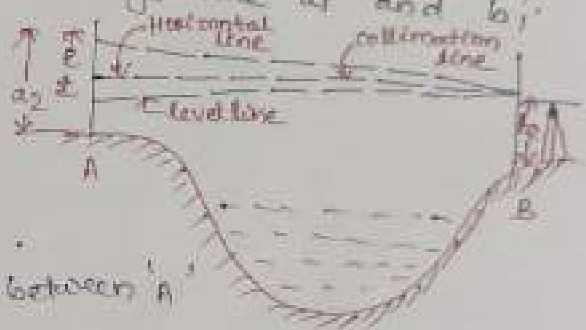
→ The principle of reciprocal levelling :—



Procedure :—

① → Suppose A and B are two points on the opposite banks of a river. The level is set up very near A and after proper temporary adjustment, staff readings are taken at A and B. Suppose the readings are 'a_1' and 'b_1'.

② → The level is shifted and setup very near 'B' and after proper adjustment, staff readings are taken at A and B. Suppose the readings are a_2 and b_2.



Let 'h' = true difference of level between 'A' and 'B'.

e = combined error due to curvature, refraction and collimation.

⇒ In the first case, (Level is very near A)

Correct staff reading at A = a_1

Correct staff reading at B = $b_1 - e$

True difference of level between A and B – $(h) = a_1 - (b_1 - e)$ —(1)

(fall from B to A)

→ In second case, (level is near B)

correct staff reading at B = b_2

Correct staff reading at A = $a_2 - e$

So, true difference of level,

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

From (1) and (2)

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

$$2h = a_1 - b_1 + e + a_2 - e - b_2$$

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

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

* DETERMINATION OF AREAS :—

The term 'area' in the context of surveying refers to the area of a tract of land projected upon the horizontal plane, and not to the actual area of the land surface. Area may be expressed in the following units :— Square metres, Hectare, Square-feet, Acres.

→ The representation of the various methods of computation of area.

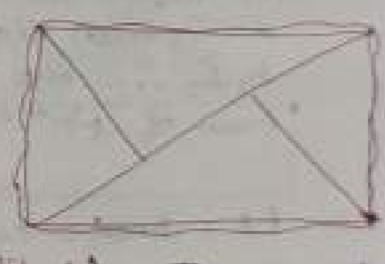
- Area
  - Graphical Method
    - From field notes
    - From plotted plan
      - Entire area
      - Boundary area
        - Mid-ordinate rule
        - Average ordinate rule
        - Trapezoidal rule
        - Simpson's rule
  - Instrumental Method.

* COMPUTATION OF AREA FROM PLOTTED PLAN :—

The area may be calculated in the following ways.

* Case 1 – Considering the entire area :— The entire area is divided into regions of a convenient shape, and calculated as follows :—

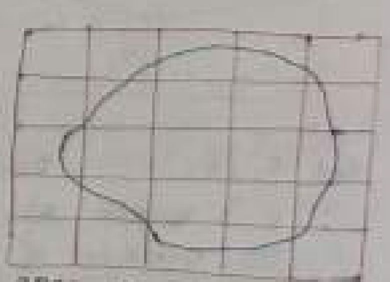
(a) By dividing the area into triangles :—



The triangles are drawn as to equalise the irregular boundary line.

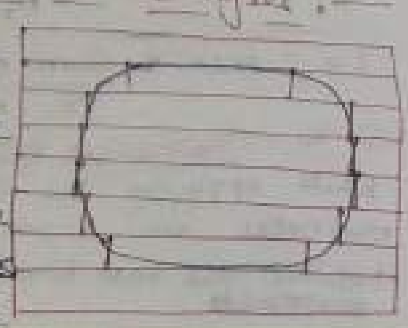
Then the bases and altitudes of the triangles are determined according to the scale to which the plan was drawn. After this, the areas of these triangles are calculated (area = $\frac{1}{2}$ × base × altitude). The areas are then added to obtain the total area.

(b) By dividing the area into squares :—



In this method, squares of equal size are ruled out on a piece of tracing paper. Each square represents a unit area, which could be $1cm^2$ or $1m^2$. The tracing paper is placed over the plan and the number of full squares are counted. The total area is then calculated by multiplying the number of squares by the unit area of each square.

(c) By drawing parallel lines and converting them to rectangles :—



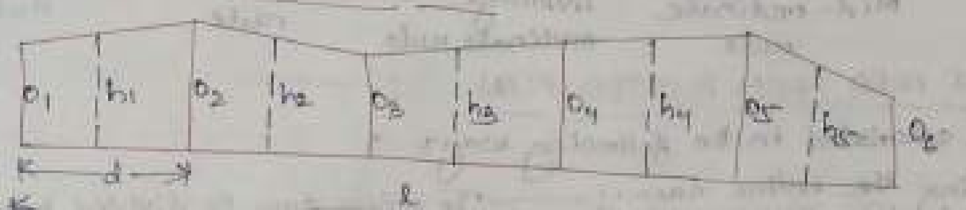
In this method, a series of equidistant parallel lines are drawn on a tracing paper. The constant distance represents a metre or centimetre. The tracing paper is placed over the plan in such a way that the area in enclosed between the two line area is divided into a number of strips. The curved ends of the strips are replaced by perpendicular lines and a number of rectangles are formed. The sum of the lengths are the rectangles is then calculated. Then,

Required area = Σ length of rectangles × constant distance.

* **Case II:—** In this method, a large square or rectangle is formed within the area in the plan. Then ordinates are drawn at regular intervals from the side of the square to the curved boundary. The middle area is calculated in the usual way. The boundary area is calculated according to one of the following rules.

1) The mid-ordinate rule.
2) The average ordinate rule.
3) The trapezoidal rule.
4) Simpson's rule.

1) **The mid-ordinate rule:—**



Let $O_1, O_2, O_3, \ldots O_n$ = ordinates at equal intervals.
L = length of base line,
d = common distance between ordinates.
$h_1, h_2, \ldots h_n$ = mid-ordinates.

$$\text{Area of plot} = h_1 \times d + h_2 \times d + \cdots + h_n \times d = d(h_1 + h_2 + \cdots + h_n)$$

i.e., Area = common distance × sum of mid-ordinates.

(2) **The Average-ordinate rule:—**

Let $O_1, O_2, \ldots, O_n$ = ordinates or offsets at regular intervals.
L = length of base line,
n = number of division, $n+1$ = number of ordinates

$$\text{Area} = \frac{O_1 + O_2 + \cdots + O_n}{O_{n+1}} \times L$$

i.e., Area = $\frac{\text{Sum of ordinates}}{\text{no. of ordinates}}$ × length of base line.

(3) **The Trapezoidal Rule:—**

While applying the trapezoidal rule, boundaries between the ends of ordinates are assumed to be straight. Thus the areas enclosed between the base line and the irregular boundary line are considered as trapezoids

Let $O_1, O_2, \ldots, O_n$ = ordinates at equal intervals

d = common distance.

1st area = $\frac{O_1 + O_2}{2} \times d$

2nd area = $\frac{O_2 + O_3}{2} \times d$

3rd area = $\frac{O_3 + O_4}{2} \times d$

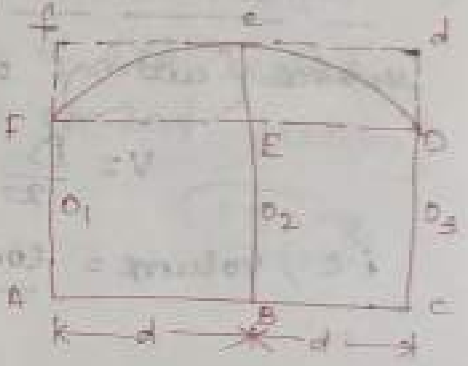
....

Last area = $\frac{O_{n-1} + O_n}{2} \times d$

Total area = $\frac{d}{2} \{ O_1 + 2O_1 + 2O_2 + \ldots\ldots + 2O_{n-1} + O_n \}$

= $\frac{\text{common distance}}{2} \{ (\text{1st ordinate} + \text{last ordinate} + 2(\text{sum of other ordinate}) \}$

(4) Simpson's rule :—



The boundaries between the ends of ordinates are assumed to form an arc of a parabola. Hence Simpson's rule is sometimes called the parabolic rule.

Let, O_1, O_2, O_3 = three consecutive ordinates

d = common distance between the ordinates.

Area AFeDC = area of trapezium AFDC + area of segment FeDEF.

Here,

Area of trapezium = $\frac{O_1 + O_3}{2} \times 2d$

Area of segment = $\frac{2}{3} \times$ area of parallelogram FfdD

$= \frac{2}{3} \times Ee \times 2d = \frac{2}{3} \times \left\{ O_2 - \frac{O_1 + O_3}{2} \right\} \times 2d$

So, the area between the first two divisions,

$\Delta_1 = \frac{O_1 + O_3}{2} \times 2d + \frac{2}{3} \left\{ O_2 - \frac{O_1 + O_3}{2} \right\} \times 2d$

$= \frac{d}{3} (O_1 + 4O_2 + O_3)$

Similarly, the area between next two divisions,

$\Delta_2 = \frac{d}{3} (O_3 + 4O_4 + O_5)$ and so on.

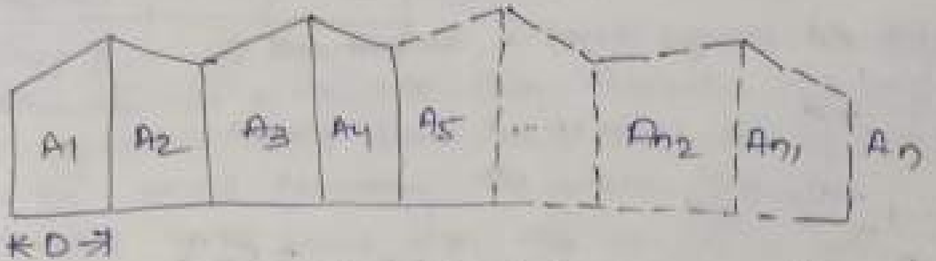
$\therefore$ Total area $= \frac{d}{3} (O_1 + 4O_2 + 2O_3 + 4O_4 + \ldots + O_n)$

$= \frac{d}{3} \{ (O_1 + O_n + 4(O_2 + O_4 + \ldots) + 2(O_3 + O_5 + \ldots) \}$

$= \frac{d}{3} (O_1 + O_n$

= $\frac{\text{common distance}}{2} \{ \text{1st ordinate} + \text{last ordinate}) + 4(\text{sum of even ordinates}) + 2(\text{sum of odd ordinates}) \}$

* FORMULA FOR CALCULATION OF VOLUME :—



D = common distance between sections.

(1) Trapezoidal Rule (Average End Area Rule) :—

volume (cutting or filling),

$$V = \frac{D}{2}\left\{A_1 + A_n + 2(A_2 + A_3 + \dots + A_{n-1})\right\}$$

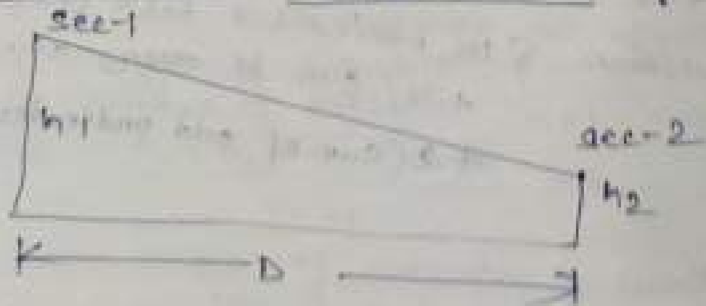
i.e., volume = $\frac{\text{common distance}}{2}$ {area of 1st section + area of last section + 2 (sum of area of other sections)}

(2) Prismoidal formula :—

Volume (cutting or filling), $V = \frac{D}{3}\left\{A_1 + A_n + 4(A_2 + A_4 + A_{n-1}) + 2(A_3 + A_5 \dots + A_{n-2})\right\}$

i.e., V = $\frac{\text{common distance}}{3}$ {Area of 1st section + area of last section + 4 (sum of areas of even sections) + 2 (sum of areas of odd sections)}

* PRISMOIDAL CORRECTIONS :—



1) Prismoidal correction for level sections:

$$C_P = \frac{D \times S}{6}(h_1 - h_2)^2$$

(considering, transverse slope = 1 in n, side slope = s:1)

2.) Prismoidal correction for two-level section :-

$$C_P = \frac{D \times S}{6}\left(\frac{n^2}{n^2 - s^2}\right) \times (h_1 - h_2)^2$$

3.) Prismoidal correction for side hill two-level section :-

(a) C_P (for cutting) $= \dfrac{D}{12(n - s_1)} \times n^2 (h_1 - h_2)^2$ (side slope $= s_1 : 1$)

(b) C_P (for filling) $= \dfrac{D}{12(n + s_2)} \times n^2 (h_1 - h_2)^2$ (side slope $= s_2 : 1$)

4.) Prismoidal correction for three-level section :-

$C_P = \frac{D}{12}(h_1 - h_2)$ (whole width of 1st section − whole width of 2nd section)

* CURVATURE CORRECTION FOR VOLUME :- (Pg - 408)

1)* Level section :- the ground is level along transverse direction

Area $= (b + sh)h$

2) Two level section :- Ground has a transverse slope.

$\frac{b + b + 2sh}{2} \times h$

$$\text{Area} = \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)(b_1 + b_2) - \left(\frac{b^2}{2s}\right)\right\}$$

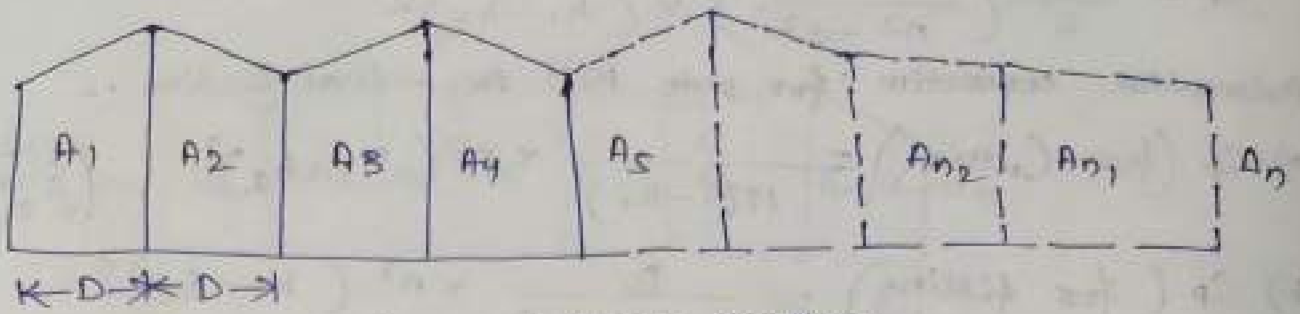
3) Three level section :- slope is not uniform

$$A = \left\{\frac{h}{2}(b_1 + b_2) + \frac{b}{4}(h_1 + h_2)\right\}$$

4) Side-hill two level section :- transverse slope (cutting & filling).

5) Multi-level section.

* COMPUTATION OF VOLUME :—



D= common distance between sections.

1) Trapezoidal Rule (Average End Area Rule) :—

volume (cutting or filling), $V = \frac{D}{2}\{A_1 + A_n + 2(A_2 + A_3 + \ldots + A_{n-1})\}$

i.e., volume = $\frac{\text{common distance}}{2}$ {area of 1st section + area of last section + 2 (sum of area of other section)}

2) Prismoidal Formula :—

volume (cutting or filling), $V = \frac{D}{3}\{A_1 + A_n + 4(A_2 + A_4 + A_{n-1}) + 2(A_3 + A_5 + \ldots + A_{n-2})\}$

i.e., $V = \frac{\text{Common distance}}{3}$ {Area of 1st section + area of last section + 4 (sum of areas of even section) + 2 (sum of areas of odd section)}

* FORMULA FOR CALCULATION OF CROSS-SECTIONAL AREA :—

The cross-sectional area may be of different types :—

i) Level section.
ii) Two-level section.
iii) Three-level section.
iv) Side-hill two level section.
v) multi-level section.

i) Level-Section :—

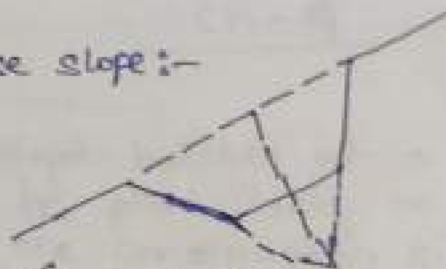
When the ground is level along the transverse direction :—

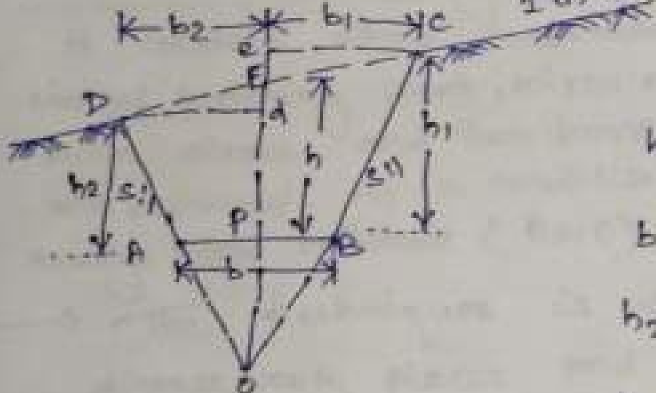
$Area = \frac{b + b + 2sh}{2} \times h$

$\boxed{Area = (b + Sh)h}$

(ii) Two-Level Section :—

When the ground surface has a transverse slope :-





$$h_1 = \frac{n}{(n-s)} \times \left(h + \frac{b}{2n}\right)$$

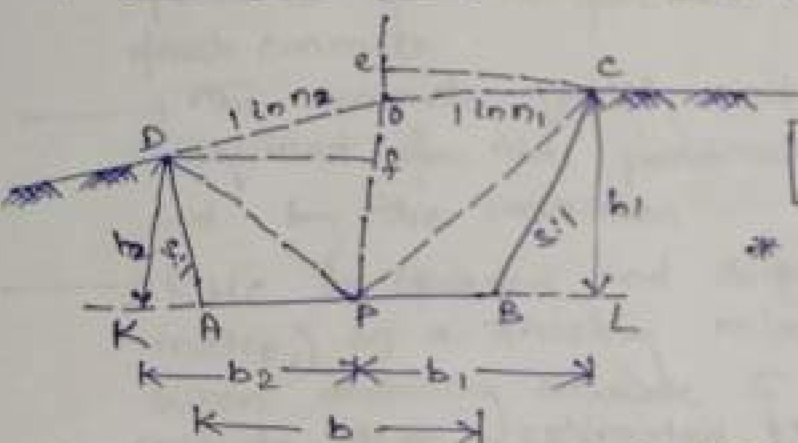
$$b_1 = \frac{b}{2} + \frac{n \cdot s}{n-s}\left(h + \frac{b}{2n}\right)$$

$$h_2 = \frac{n}{(n+s)}\left(h - \frac{b}{2n}\right)$$

$$b_2 = \frac{b}{2} + \frac{n \cdot s}{n+s} \times \left(h \mp \frac{b}{2n}\right)$$

$$\text{Area} = \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)(b_1 + b_2) - \frac{b^2}{2s}\right\}$$

(iii) Three-Level Section :— When the transverse slope is not uniform.

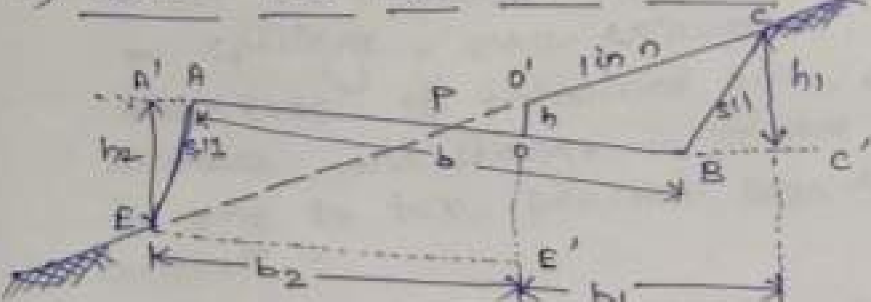


$$\text{Area} = \left\{\frac{h}{2}(b_1 + b_2) + \frac{b}{4}(h_1 + h_2)\right\}$$

$$* \; b_1 = \frac{n_1 s}{n_1 - s}\left(h + \frac{b}{2s}\right)$$

$$b_2 = \frac{n_2 s}{n_2 + s}\left(h + \frac{b}{2s}\right)$$

(iv) Side-Hill Two-Level Section :—



$$h_1 = \frac{n}{n-s} \times \left(\frac{b}{2n} + h\right)$$

$$b_1 = \frac{b}{2} + \frac{n \cdot s}{(n-s)} \times \left(\frac{b}{2n} + h\right)$$

$$h_2 = \frac{n}{(n-s)} \times \left(\frac{b}{2n} - h\right)$$

$$b_2 = \frac{b}{2} + \frac{n \cdot s}{(n-s)} \times \left(\frac{b}{2n} - h\right)$$

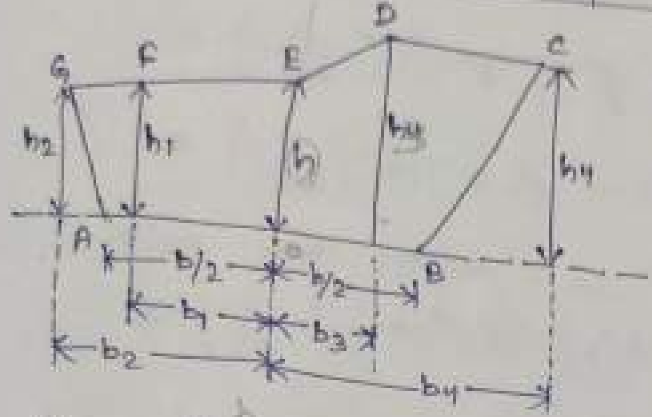
$$* \; A_1 = \frac{1}{2}\left[\frac{\{(b/2) + nh\}^2}{n-s}\right]$$

$$* \; A_2 = \frac{1}{2}\left[\frac{\{(b/2) - nh\}^2}{n-s}\right]$$

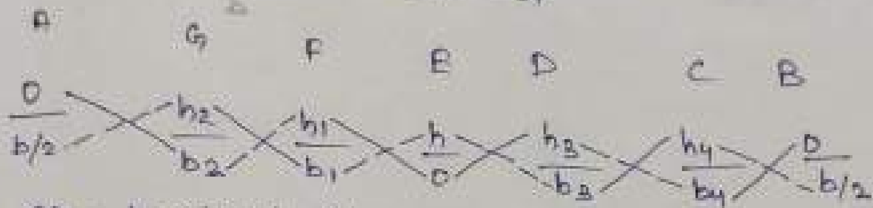
(v) Multi-level section :—

The cross-sectional data pertaining to an irregular section are noted in the following form :—

Left		Centre	Right	
$\frac{\pm h_2}{b_2}$	$\frac{\pm h_1}{b_1}$	$\frac{\pm h}{0}$	$\frac{\pm h_3}{b_3}$	$\frac{\pm h_4}{b_4}$



A positive sign in the numerator denotes a cut, and a negative sign indicates a fill.

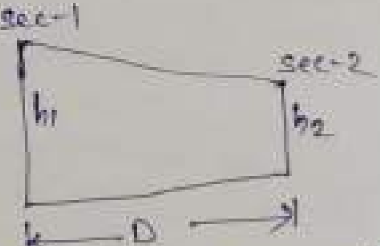


$\Sigma P = h_3 \times 0 + h_4 \times b_3 + 0 \times b_4 + h_1 \times 0 + h_2 \times b_1 + 0 \times b_2$

$\Sigma Q = h \times b_3 + h_3 \times b_4 + h_4 \times \frac{b}{2} + h \times b_1 + h_1 \times b_1 + h_2 \times \frac{b}{2}$

$$\text{Area} = \frac{1}{2}(\Sigma P - \Sigma Q)$$

* PRISMOIDAL CORRECTION :-



1) Prismoidal correction for level section

$$C_p = \frac{D \times S}{6}(h_1 - h_2)^2$$

2.) Prismoidal correction for two-level section :-

$$C_p = \frac{D \times S}{6} \times \left(\frac{n^2}{n^2 - S^2}\right)(h_1 - h_2)^2$$

(considering transverse slope = 1 in n side slope = S : 1)

3.) Prismoidal correction for side hill two-level sections :-

(a) C_p (for cutting) $= \frac{D}{12(n - S_1)} \times n^2 (h_1 - h_2)^2$ * side slope $= S_1 : 1$

(b) C_p (for filling) $= \frac{D}{12(n - S_2)} \times n^2 (h_1 - h_2)^2$ * side slope $S_2 : 1$

4.) Prismoidal correction for three-level section :-

$$C_p = \frac{D}{12}(h_1 - h_2)(\text{whole width of 1 section} - \text{whole width of 2nd section})$$

* **CURVATURE CORRECTION :—**

a) For two-level section :—

$$\pm \frac{D}{6R}\left(b_1^2 + b_2^2\right)\left(h + \frac{b}{2s}\right)$$

b) Side-hill two-level section :—

$$r\left(\frac{A}{3R}\right)\left(b_2 + \frac{b}{2} - rh\right)$$ (for ~~filling~~ cutting)

$$\left(\frac{A}{3R}\right)\left(b_1 + \frac{b}{2} + rh\right)$$ (for filling)

c) Three-level section :—

$$\pm\left(\frac{D}{6R}\right)\left(b_1^2 - b_2^2\right)\left(h + \frac{b}{2s}\right)$$