JAWAHARLAL NEHRU TECHNOLOGICAL UNIVERSITY

II B.TECH. (CIVIL ENGINEERING)

II Year B.Tech. – I Sem.

SURVEYING

UNIT – I

Introduction: definition-Uses of surveying- overview of plane surveying (chain, compass and plane table), Objectives, Principles and classifications – Errors in survey measurements

UNIT – II

Distances And Direction: Electronic distance measurements (EDM)- principles of electro optical EDM-Errors and corrections to linear measurements- Compass survey Meridians, Azimuths and Bearings, declination, computation of angle. Traversing-Purpose-types of traverse-traverse computation-traverse adjustments-Introduction omitted measurements

UNIT – III

Levelling And Contouring: Concept and Terminology, Levelling Instruments and their Temporary and permanent adjustments- method of levelling. Characteristics and Uses of contours- methods of conducting contour surveys.

UNIT – IV

Theodolite: Description, principles-uses and adjustments – temporary and permanent, measurement of horizontal and vertical angles. Principles of Electronic Theodolite – Introduction to Trigonometrical levelling, Tachometric Surveying: Stadia and tangential methods of Tacheometry. Distance and Elevation formulae for Staff vertical position.

UNIT – V

Curves: Types of curves, design and setting out – simple and compound curves. Introduction to geodetic surveying, Total Station and Global positioning system

UNIT – VI

Computation Of Areas And Volumes: Area from field notes, computation of areas along irregular boundaries and area consisting of regular boundaries. Embankments and cutting for a level section and two level sections with and without transverse slopes, determination of the capacity of reservoir, volume of barrow pits.

Text Books:

1. Surveying, Vol No.1, 2 &3, B. C. Punmia, Ashok Kumar Jain and Arun Kumar Jain – Laxmi Publications Ltd, New Delhi.

2. Advance Surveying, Satish Gopi, R. Sathi Kumar and N. Madhu, Pearson Publications.

3. Text book of Surveying, C. Venkataramaiah, University press, India Limited.

4. Surveying and levelling, R. Subramanian, Oxford University press.

References:

1. Text book of Surveying, S.K. Duggal (Vol No. 1&2), Tata McGraw Hill Publishing Co. Ltd. New Delhi.

2. Text book of Surveying, Arora (Vol No. 1&2), Standard Book House, Delhi.

3. Higher Surveying, A.M. Chandra, New Age International Pvt ltd.

4. Fundamentals of surveying, S.K. Roy – PHI learning (P) ltd.

5. Plane Surveying, Alak de, S. Chand & Company, New Delhi.

INDEX

UNIT – I

INTRODUCTION

1.1 INTRODUCTION:

Surveying is the art of determining the relative positions of points on, above or beneath the surface of the earth by means of direct or indirect measurements of distance, direction and elevation. It also includes the art of establishing points by predetermined angular and linear measurements.

Surveying also includes the art of setting out or locating the points on the ground from the plan or map. The points and lines have to be located on the ground before starting the construction of engineering work, such as buildings, roads, bridges, and dams.

The first stage in all the big projects is generally to survey the area and to prepare plans. These plans are used in the preparation of the detailed drawing, design and estimate of the project. After finalizing the drawings, setting out is done by establishing the various points and lines on the ground from the drawing.

The objects of surveying can be summarized as under:

- To take measurements to determine the relative positions of the existing features on or near the ground.
- To layout or to mark the positions of the proposed structure on the ground.
- To determine areas, volumes and other related quantities.

1.2 PRIMARY DIVISION OF SURVEYING:

Primary division of surveying is made on the basic whether the curvature of the earth is considered or whether the earth is assumed to be a flat plane. The actual shape of the earth is an oblate spheroid. It is ellipsoid of revolution, flattened at the poles and bulging at the equator.

Because of the curvature of the earth's surface, the measured distances on earth's surface, the measured distances on earth's surface are actually curved. However, when the distances are small, compared with the radius of the earth, there is no significant difference between the curved distances and the corresponding straight line distances, and the curvature of the earth can be neglected.

Surveying is thus primarily divided into two types:

- Plane surveying
- Geodetic surveying

www.FirstRanker.com

1.2.1 Plane Surveying

It is the type of surveying in which the curvature of the earth is neglected and it is assumed to a flat surface. All distance and horizontal angles are assumed to be projected onto a horizontal plane. A horizontal plane at a point is the plane which is perpendicular to the vertical line at that point. The vertical line is indicated by a freely suspended plumb bob. A single horizontal plane of reference is selected for the entire survey of the small area. Thus the plumb bob lines at all points of the area are assumed to be parallel.

Plane surveying can safely be used when one is concerned with a small portions of the earth‟s surface and the areas involved are less than 250 sq. km or so. It is worth noting that the difference between an arc distance of 18.5 km on the surface of the earth and the corresponding chord distance is less than 10 mm.

1.2.2 Geodetic Surveying

It is the type of surveying in which the curvature of the earth is taken into consideration, and a very high standard of accuracy is maintained. The main object of geodetic surveying is to determine the precise location of a system of widely spaced points on the surface of the earth. The points so located are used as control station of the primary surveys. The secondary surveys of less precision are connected to these control stations.

In Geodetic surveying, the earth's major and minor axes are computed accurately and a spheroid of reference is visualized. The spheroid is a mathematical surface obtained by revolving an ellipse about the earth's polar axis. The earth's mean sea-level surface which is perpendicular to the direction of gravity at every point is represented by a geoid.

The above discussions may be summarized as under

(a). Plane Surveying

- It is used for relatively small areas.
- A curved line on the surface of the earth is considered as mathematically straight.
- The directions of the plumb lines at various points are assumed to be parallel to one another.
- The spherical angles are considered as plane angles.
- The standard of accuracy is lower than that in geodetic surveying.

(b). Geodetic Surveying

- It is used for large areas.
- It is used for establishing precise points of reference or control points.
- The surface of the earth is considered as curved.
- The directions of plumb lines at various points are different. The earth's mean sea level is perpendicular to the direction of gravity indicated by plumb bobs.
- The standard of accuracy is very high. Very precise instruments are used.

1.3 CLASSIFICATION:

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

- Classification based upon the nature of the field survey.
- Classification based on the object of survey.
- Classification based on instruments used

1.3.1 Classification based upon the nature of the field survey

- Land Surveying
- Marine or Hydro graphic Survey
- Astronomical Survey

1.3.1.1 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:

These 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.

1.3.1.2 Marine or Hydrographic Survey

It deals with bodies of water for purpose of navigation, water supply, harbor 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.

1.3.1.3 Astronomical Survey

The astronomical survey offers the surveyor 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.

1.3.2 Classification Based On The Object Of Survey

(i). 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.

(ii). Military Survey

This is used for determining points of strategic importance.

(iii). Mine Survey

This is used for the exploring mineral wealth.

(iv). Geological Survey

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

(v). Archaeological Survey

This is used for unearthing relics of antiquity.

1.3.3 Classification Based On Instruments Used

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

- Chain Survey
- Theodolite Survey
- Traverse Survey
- Triangulation Survey
- Tacheometric Survey
- Plane table Survey
- Photogrammetric Survey and
- Aerial survey

1.3.3.1 Chain Survey

Chain surveying is that type of surveying in which only linear measurements are made in the field. This type of surveying is suitable for surveys of small extent on open ground to secure data for exact description of the boundaries of a piece of land or to take simple details.

1.3.3.2 Theodolite Survey

The theodolite is the most precise instrument designed for the measurement of horizontal and vertical angles and has wide applicability in surveying such as laying off horizontal angles, locating points on line, prolonging survey lines, establishing grades, determining difference in elevation, setting out curves etc.,

1.3.3.3 Traverse Survey

Traversing is that type of survey in which a number of connected survey lines form the framework and the direction and lengths of survey lines are measured with the help of an angle (or direction) measuring instrument and a tape (or chain).

- When the lines form a circuit which ends at the starting point, it is known as a closed traverse.
- If the circuit ends elsewhere, it is said to be an open traverse.

1.3.3.4 Triangulation Survey

The horizontal control in geodetic survey is established either by triangulation or by precise traverse. In triangulation, the system consists of a number of inter-connected triangles in which the length of only one line called the base line, and the angles of the triangles are measured very precisely. Knowing the length of one side and the three angles, the lengths of the other two sides of each triangle can be computed. The apexes of the triangles are known as the triangulation survey.

1.3.3.5 Tachometric survey

It is a branch of angular surveying in which the horizontal and vertical distances of points are obtained by optical means as opposed to the ordinary slower process of measurements by tape or chain. The method is very rapid and convenient.

1.3.3.6 Plane Table Survey

It 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 system by triangulation or traverse and by lines of levels.

1.3.3.7 Photogrammetric Survey

It is the science and art of obtaining accurate measurements by use of photographs, for various purposes such as the construction of plainimetric and topographic maps, Classification of soils, interpretation of geology, acquisition of military intelligence and the preparation of composite pictures of the ground.

1.3.3.8 Aerial Survey

It is a branch of photogrammetry wherein the photographs are taken by a camera mounted in an aircraft flying over the area. Mapping from aerial photographs is the best mapping procedure yet developed for large projects, and are invaluable for military intelligence. The major users of aerial mapping methods are the civilian and military mapping agencies of the government.

1.4 OVERVIEW OF PLANE SURVEYING

Survey in which the mean surface of earth is regarded as plane surface and not curved as it really is, is known as plane surveying.

Plane surveying is carried out by

- Chain Survey
- Compass Survey and
- Plane Table Survey

1.4.1Chain Survey:

It is the branch of surveying in which the distances are measured with a chain and tape and the operation is called chaining. All the distances measured should be horizontal. It is recommended for planes involving development of buildings, roads, water supply and seweage schemes.

1.4.2 Compass Survey:

The direction of survey lines is measured with the help of an instrument known as compass.

The direction of survey lines may be defined in two ways:

- Relative to each other
- Relative to some reference direction.

In the first case, the directions are expressed in terms of angles between two consecutive lines, measured with a Theodolite. In the second case, there are expressed in terms of bearings, measured with a compass.

Compass being light and portable, is most suited for reconnaissance and exploratory survey. It is particularly advantageous, when the survey lines have to be short due to obstructions or irregularities of details. The applications and uses of compass survey are:

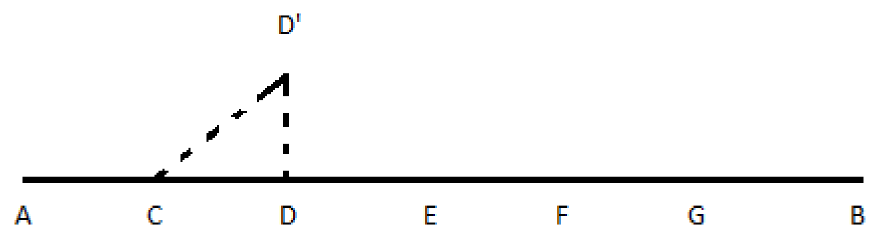
- To find out the magnetic bearing of a line.
- To fill in details.
- To find the direction during night marching.
- Tracing streams.
- Plotting irregular shore lines.
- Recconnaissance survey.
- Clearings in roads.

1.4.3 Plane Table Surveying:

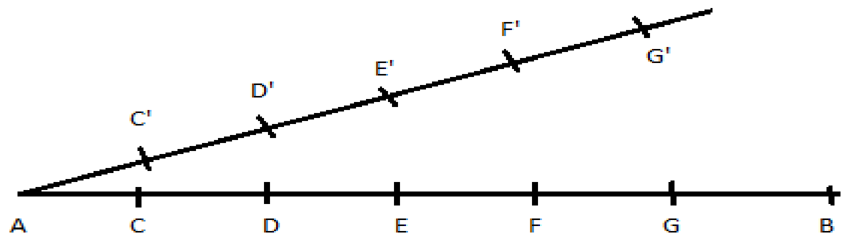
The plane table is an instrument used for surveying by a graphical method in which the field work and plotting are done simultaneously. The main advantage of plane tabling introduced in establishing any intermediate point will be carried in establishing the other points.

For Example:

If the point D has been established out of line AB, as D and E,F.....etc., have been established correctly. The actual distance DC and DE will be in error (D’C and D’E) but all other distances AC, EF, FG, etc., will be connected. Therefore the error in their procedure is localized at point D and it is not magnified.



On the other hand if we work from part to whole, say if intermediate points are established with respect to AC as shown in fig. As shown in fig. As C is already established out of line AB, there will be error in the actual length AC.



Therefore the remaining points C’, D’, E’..... etc., will also be established out of line AB with increasing magnitude of error and finally the survey will become uncontrollable. Therefore working from part to whole is never recommended.

1.5 PRINCIPLES OF SURVEYING

The fundamental principles upon which the various methods of plane surveying are based are of very simple nature and can be stated under the following two aspects;

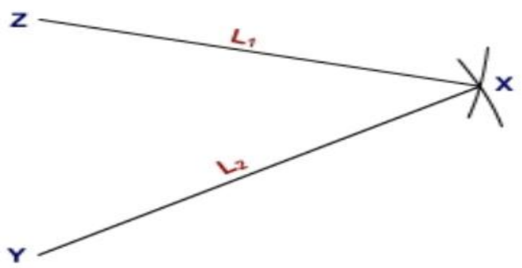
- After deciding the position of any point, its reference must be kept from at least two permanent objects or stations whose position have already been well defined.
- Working from whole to part.

The purpose of working from whole to part is

- to localize the errors and
- to control the accumulation of errors.

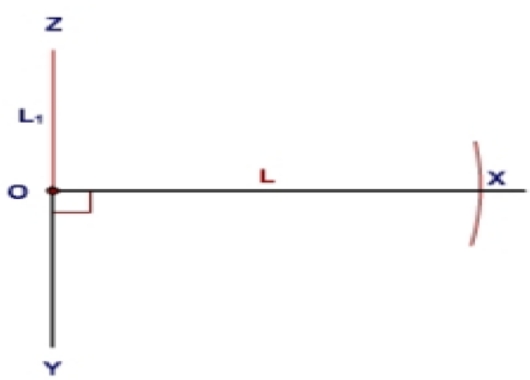
The reference of any point, say X, has to kept with respect to, at least, two permanent objects or well defined points, say Y and Z. Generally, this has been achieved by taking measurement of two parameters. The location of a point, say X can be done as shown in the figure below.

(a) Distances YX and



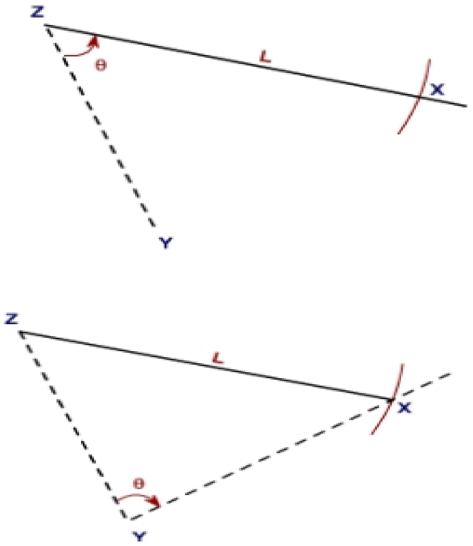
Reference of a point using two well defined distance

(b) Perpendicular distance OX and distance OY or OZ



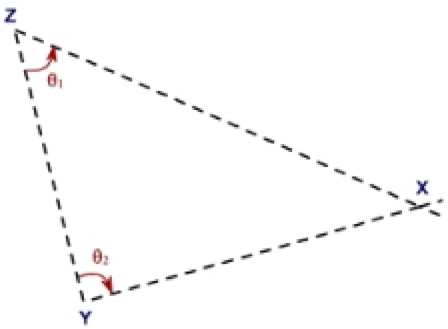
Reference of a point using perpendicular distance

(c) Distance YX or ZX and angle YZX or ZYX



Reference of a point using a distance and an angle

(d).Angles YZX and ZYX



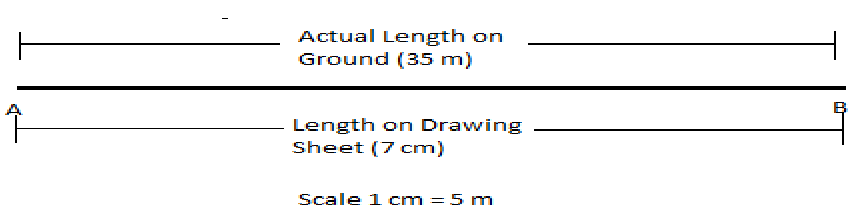
Reference of a point using two angles

The point of intersection of the two measured parameters defines the position of the point.

1.6 ERRORS IN SURVEY MEASURMENTS:

1.6.1 Shrinkage of Scale:

We usually represent the scale of a map at the bottom of drawing sheet as shown below, which is known as graphical scale.



When a map shrinks or expands, the scale line also shrinks or expands with it and thus the measurements made from the map are not affected.

Shrunk Scale = Shrinkage factor x original Scale.

1.6.2 Shrinkage Ratio (S.R) or Shrinkage Factor (S.F):

The ratio of the shrunk length to the actual length is known as shrinkage ratio (S.R) or Shrinkage factor (S.F).

$$\textbf{S.F} = \frac{\textbf{Shrunk Length}}{\textbf{Original Length}} = \frac{\textbf{Shrunk Scale}}{\textbf{Original Scale}} = \frac{\textbf{Shrunk R.F}}{\textbf{Original R.F}}$$

$$\textbf{Thus Correct distance} = \frac{\textbf{Measured Distance}}{\textbf{S.F}}$$

$$\textbf{And Correct Area} = \frac{\textbf{Measured Area}}{(\textbf{S.F})^2}$$

1.6.3 Wrong Scale:

If a wrong measuring scale is used to measure the length of a line already drawn on a plan or map, the measured length will be erroneous. Then

$$\textbf{Correct Length} = \frac{\textbf{R.F of Wrong Scale}}{\textbf{R.F of Correct Scale}} \textbf{ x Measured Length}$$

$$\textbf{And Correct Area} = \left[\frac{\textbf{R. F of Wrong Scale}}{\textbf{R.F of Correct Scale}}\right]^2 \textbf{ x Measured Area}$$

UNIT – II

DISTANCES AND DIRECTION

2.1 ELECTROMAGNETIC DISTANCE MEASUREMENT (EDM)

There are three methods of measuring distance between any two given points.

- ✓ Direct distance measurement (DDM), such as the one by chaining or taping.
- ✓ Optical distance measurement (ODM), such as the one by tacheometry, Horizontal subtense method or telemetric method using optical wedge attachments.
- ✓ Electromagnetic distance measurement (EDM) such as the one by Geodimeter, tellurometer or distomat etc.,

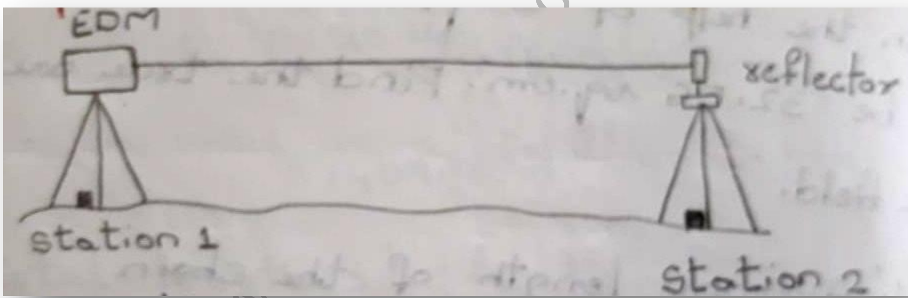
  (i). But in ODM method also, the range is limited to 150 m and the occuracy obtained is 1 in 1000 to 1 in 10,000.

  (ii). EDM enables the occuracies upto 1 in 10^5 over ranges upto 100 km.

2.1.1 PRINCIPLES OF EDM

The basic principle of EDM instrument is the determination of time required for electro magnetic waves to travel between two stations. Here the velocity of electro-manetic wave is the basis for computations of the distance.

Wave Length $\lambda = V/f$



1. Transmitter of modulated electromagnetic radiation.
2. Reflector.

2.2 LINEAR MEASUREMENT

For measuring horizontal distance, various methods such as rope stretching, bamboo, pacing, chaining, optical (tacheometry) and electromagnetic distance measurement have existed since our ancestors till date.

- ➢ Rope stretching and bamboo measurement are very crude methods and are absolute.
- ➢ Pacing can be recommended if an error of 5% is permissible and if the ground is flat.

- In the optical methods, the distances are not actually measured in field but are computed indirectly by using the principle of optics. The instrument used is tacheometer (Tacheometry). Tacheometry may be employed when the ground is rough, undulating and not suitable for chaining.
- The electromagnetic distance measurements can be made by using light waves or radio waves. Geodimeter and Tacheometer make use of light waves, where as densitometer uses radio waves.
- Electronic methods and aerial photo grammetry yield results with high precisions, but are expensive.
- Among the above methods, the most common method of measuring the horizontal distance is by the use of chain and tape and the operation is called chaining.

2.2 CHAIN SURVEYING

It is the branch of surveying in which the distances are measured with a chain and tape and the operation is called chaining. All the distances should be horizontal. However, if measured on slopes, the measurements are to be subsequently reduced to horizontal equivalents.

2.2.1 MAIN EQUIPMENTS USED FOR CHAIN SURVEYING:

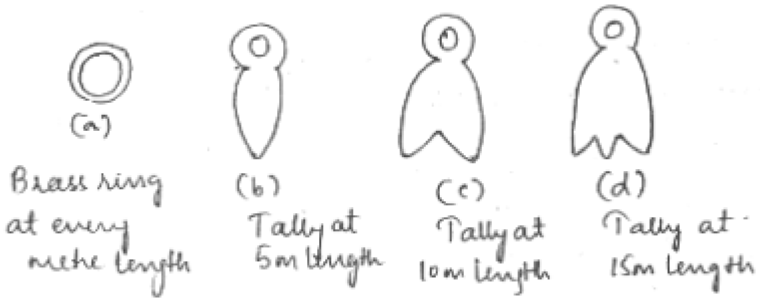
2.2.1.1 Chain:

Gunter, Revenue, engineer and metric chains are the various types of chains which are normally used for surveying.
The chains are mostly divided into 100 links.

- Gunter's chain is 66 ft. long (100 links).
- Revenue chain is 33 ft. long (16 links).
- Engineers chain is 100 ft. long (100 links).
- Metric chain are either 30 m (150 links) or 20 m (100 links) in length.

2.2.1.2 Metric Survey Chain (20 or 30 m)

A metric chain is divided into 100 links and is made of galvanized mild steel wire 4mm in diameter. The length of the link is the distance between the centers of the two consecutive middle rings. The ends of the chain are provided with brass handles. The outside of the handle is the zero point or the end point of the chain. The length of chain is measured from outer end of the handles. Metallic tags are used 5, 10, 15, 20, 25m intervals for quick reading. The metallic tags are called tallies. Small brass rings are provided at every meter length.



2.2.1.3 Suitability of chain

- It is suitable for rough usage
- It can be easily repaired in field
- It can be read easily

2.2.1.4 Unsuitability of chain

- Being heavier, it sags considerably when suspended in air.
- Its length alters by shortening/lengthening of links. therefore it is suitable for ordinary work only.

2.2.1.5 Unfolding the chain(Undoing the chain):

The leather strap is removed and with both the handles of the chain in the left hand the chain is thrown well forward with the right hand. The leader then takes one of the handles of the chain and moves forward until the chain is intended to full length. The chain is checked and kinks of bent links are removed.

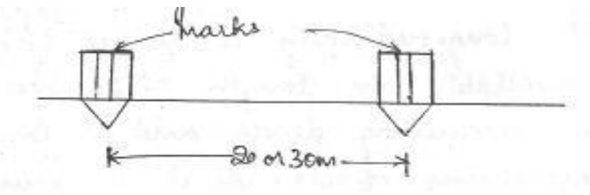
2.2.1.6 Folding the chain (Doing the chain):

The chain is pulled from the middle and the two halves of the chain are so placed as to lie along side each other. Commencing from the middle, two pairs of links are taken at a time with the right hand and are placed across the other in the left hand. The chain is then folded into a bundle and fastened with a leather strap.

2.3 TESTING OF CHAIN:

During its use, the links of a chain get bent and the length is shortened. On the other hand, the length of a chain may increase by stretching of links and usage, and rough handling through hedges, fences etc., therefore, it becomes necessary to check the length of the chain before commencing the survey work.

Procedure:



- Two pegs are at required distance of 20 m or 30 m are inserted on a flat ground. The overall length of the chain is computed with the marks and the difference is noted.
- If the chain is found to be too long. It may be adjusted by closing the opened joints of rings: reshaping the elongated links, remaining one or more circular rings: and replacing the worm out rings.
- If chain is found too short, it may be adjusted by straightening the bent links, flattening the circular rings: replacing circular rings by bigger rings, and inserting additional rings.

2.4 TAPE:

They are available in a variety of material lengths and weights.

2.4.1 Cloth (or) Linen Tape:

These are closely woven linen or synthetic material and are varnished to resist the moisture. These are available in lengths eg: 10 – 30 m and width of 12 -15 mm.

The disadvantages of this type are

- It is affected by moisture and gets shrunk.
- Its length gets altered by stretching.
- It is likely to twist and does not remain straight in strong winds.

2.4.2 Metallic Tape

It is a linear tape with brass or copper wires woven into it, longitudinally to reduce stretching. These are available in length of 20 – 30 m. it si an accurate measuring device and is commonly used for measuring offsets. As it is reinforced with wires, all the defects of linen tape are overcome.

2.4.3 Steel Tape

- These are 1 – 50 m in length and 6-10 mm wide.
- A brass ring is attached at the end of the tape, whose outer end is zero point of tape.
- It cannot be used in ground with vegetation and weeds.

2.4.4 Invar Tape

- This is made of an alloy of nickel and steel
- It has very low co-efficient of thermal expansion
- They are available in length of 30, 50 and 100m and in a width of 6mm.

Advantages of Invar Tape

- Highly precise
- It is less affected by temperature changes when compared to other tapes.

Disadvantages of Invar Tape

- It is raft and so deforms easily
- It requires much attention in handling

2.5 ACCESSORIES FOR CHAINING:

2.5.1 Pegs:

- These are used to mark definite points on the ground either temporarily or semi-permanently.
- Generally, hard creosoted wood 2.5 – 7.5 cm^2 and 15-19 cm long, flat at one end and pointed at the other end is used as a peg.
- Iron or tubular pegs of 1-2 cm diameter are also used.
- For permanent marking of stations, a small concrete pillar is used as a peg. The size varies from 15 to 30 cm^2 and 7.5 to 60 cm in height and is built in situ.

2.5.2 Arrows

- These are also known as chaining pins.
- They are used to mark the end of each chain.
- They are made of hardened and tempered steel wire 4 mm diameter and of length 400 mm.
- These are pointed at one end where as a circular ring is formed at its other end.

2.5.3 Ranging Rods

- Also known as flag poles or lining rods
- These are made of well- seasoned timber of teak, deodar etc., or steel tubular rods.
- They are used for marking a point so that the point can be clearly and exactly seen from some distance away.
- These are 30 mm in diameter and 2 or 3 m long.
- They are painted either alternate bands of either red and white or black and white of 250 mm length so that rod can be used for rough measurement of short length.
- A cross – shoe of 15 mm length is provided at the lower end.

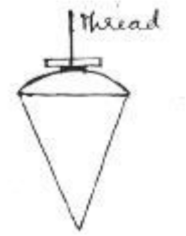
- A flag painted red and white is provided at top.
- Also used to locate intermediate points between the two ends, when the distance between these ends are more than chain length.

2.5.4 Offset Rods

- These are similar to ranging rods except at the top where a stout open ring recessed hook is provided. With this hook, the chain can be pulled or pushed through hedges or other obstructions.
- It is provided with two short narrow vertical slots at right angles to each other, passing through centre of section, at about eye levels which are used to align the offset line.
- It is mainly used to align offset line and measuring the short offsets.

2.5.6 Plumb Bob:

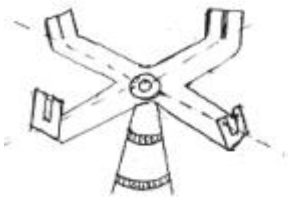
- It is made of steel in a conical shape.
- It is used while measuring distance on slopes and in all the instruments that require centering



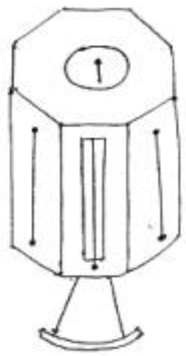
2.5.8 Cross – Staff

- It is an instrument used for setting out right angles.
- In its simplest form it is known as open Cross – staff. Which consists of two pairs of vertical slits providing two lines of sight mutually at right angles.
- Another modified form of cross-staff is French cross-staff which consists of an octagoanal brase tube with slits on all eight sides. This has a distant advantage over the open cross-staff as with it even lines at 45^0 can be set out from the chain line.

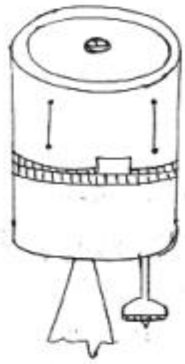
The latest modified cross-staff is the adjustable cross-staff, which consists of two cylinders of equal diameter placed one above the other. The upper cylinder can be rotated over the lower one graduated in degrees and its sub divisions. The upper cylinder carries the vernier and the slits to provide a line of sight. Thus, it may be used to take offsets and to set out any desired angles from the chain line.



Open Cross-staff



French Cross Staff



Adjustable Cross-Staff

2.6 ERRORS IN CHAINING

Errors and mistakes in chaining may arise from any one or more of the following sources such as erroneous length of chain, bad ranging, poor straightening, careless holding and marking, variation of temperature, variation of pull, displacement of arrows, miscounting chain lengths, misreading and erroneous booking. Errors in chaining are classified as follows:

- Compensating errors
- Cumulative errors

2.6.1 Compensating Errors

These are the errors which are liable to occur in both the directions and tend to compensate. Compensating errors are proportional to the square root of the length of the line. They do not affect the result much.

2.6.2 Cumulative Errors

These are the errors which are liable to occur in the same direction and tend to accumulate. The errors thus considerably increase or decrease the actual measurements. The cumulative errors are proportional to the length of the line and may be positive or megative.

2.6.3 Errors:

1. Erroneous length of chain or tape (Cumulative + or -)
2. Bad Ranging (Cumulative +)

3. Careless Marking and holding (Compensating +or -)
4. Bad straightening (Cumulative +)
5. Non – Horizontality (Cumulative +)
6. Sag in Chain (Cumulative +)
7. Variation in temperature (Cumulative + or -)
8. Variation in pull (Cumulative + or -)

2.7 ERROR DUE TO INCORRECT CHAIN:

In the length of the chain used in measuring length of the line is not equal to the tru length or the designated length, the measured length of the line will not be correct and suitable correction will have to be applied.

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

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

Let L = True (or) designated length of the chain (or) tape.

L' = Incorrect length of the chain or tape used.

2.7.1CORRECTIONS

2.7.1.1 CORRECTION TO MEASURED LENGTH:

Let l' = measured length of the line

l = actual or true length of the line

Then true length of line

= Measured length of line x (L'/L)

(or) $l = l' x (L'/L)$.

2.7.1.2 CORRECTION TO AREA:

Let A' = measured area of the ground

A = Actual or true area of the ground.

Then, true area

$$= \text{Measured Area x } (L'/L)^2$$

$$\text{(or) } A = A' \text{ x } (L'/L)^2$$

2.7.1.3 CORRECTION TO VOLUME:

Let V' = Measured or computed volume.

V = Actual or true Volume.

Then, true area

$$= \text{Measured Volume x } (L'/L)^3$$

$$\text{(or) } V = V' \text{ x } (L'/L)^3$$

2.8 COMPASS SURVEY

Compass surveying is the branch of surveying in which the position of an object is located using angular measurements determined by a compass and linear measurements using a chain or tape. Compass surveying is used in following circumstances:

- If the surveying area is large, chain surveying is not adopted for surveying rather compass surveying is employed.
- If the plot for surveying has numerous obstacles and undulations which prevents chaining.
- If there is a time limit for surveying, compass surveying is usually adopted.

Compass surveying is not used in places which contain iron core, power lines etc which usually attracts magnets due to their natural properties and electromagnetic properties respectively. Compass surveying is done by using traversing. A traverse is formed by connecting the points in the plot by means of a series of straight lines.

Compass is used to find out the magnetic bearing of survey lines. The bearings may either measured in Whole Circle Bearing (W.C.B) system or in Quadrantal Bearing (Q.B) system based on the type of compass used. The basic principle of compass is if a strip of steel or iron is magnetized and pivoted exactly at centre so that it can swing freely, then it will establish itself in the magnetic meridian at the place of arrangement.

Major types of magnetic Compass are:

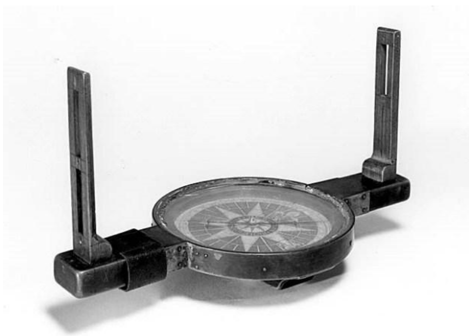
1. Prismatic Compass
2. Surveyor's Compass

2.8.1 Prismatic Compass:

Prismatic compass is a portable magnetic compass which can be either used as a hand instrument or can be fitted on a tripod. It contains a prism which is used for accurate measurement of readings. The greatest advantage of this compass is both sighting and reading can be done simultaneously without changing the position.

Surveyor's compass

Surveyor's compass consists of a circular brass box containing a magnetic needle which swings freely over a brass circle which is divided into 360 degrees. The horizontal angle is measured using a pair of sights located on north – south axis of the compass. They are usually mounted over a tripod and leveled using a ball and socket mechanism.



2.8.2 Adjustments of prismatic compass

Two types of adjustments:

1. Temporary adjustment
2. Permanent adjustment

2.8.2.1Temporary adjustments

- **Centering:** it is the process of fixing the compass exactly over the station. Centering is usually done by adjusting the tripod legs. Also a plumb-bob is used to judge the accurate centering of instruments over the station.
- **Leveling:** the instrument has to be leveled if it is used as in hand or mounted over a tripod. If it is used as in hand, the graduated disc should swing freely and appears to be

completely level in reference to the top edge of the case. If the tripod is used, they usually have a ball and socket arrangement for leveling purpose.

- **Focusing the prism:** Prism can be slide up or down for focusing to make the readings clear and readable.

2.8.2.2 Permanent adjustments:

They are done only in the circumstances where the internal parts of the prism is disturbed or damaged. They are:

- Adjustments in levels
- Adjustment of pivot point
- Adjustment of sight vanes
- Adjustment of needle

2.8.3 Advantages & Disadvantages of Compass surveying

Advantages

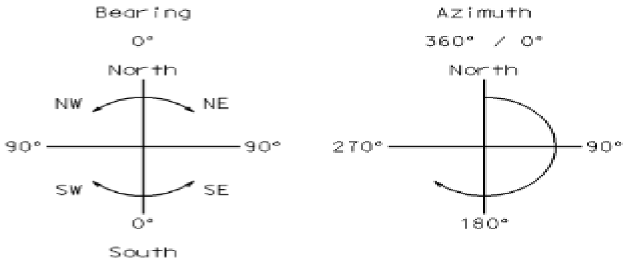
- They are portable and light weight.
- They have fewer settings to fix it on a station
- The error in direction produced in a single survey line does not affect other lines.
- It is suitable to retrace old surveys.

Disadvantages

- It is less precise compared to other advanced methods of surveying.
- It is easily subjected to various errors such as errors adjoining to magnetic meridian, local attraction etc.
- Imperfect sighting of the ranging rods and inaccurate leveling also causes error.

2.8.4. DEFINITIONS

- ✓ **Meridian**: A line on the mean surface of the earth joining the north and south poles.
- ✓ **Geographic North:** True North Sometimes called Geodetic True North-fixed.
- ✓ **Magnetic North:** Taken from a magnetic compass-change with time.
- ✓ **Grid North (Meridian):** Lines parallel to a grid reference meridian (central meridian)-SPC.
- ✓ **Bearing:** - The direction of a line as given by the acute angle between the line and a reference meridian.
- ✓ **Azimuth:** The direction of a line as given by an angle measurement clockwise (usually) from the north end of a reference meridian.

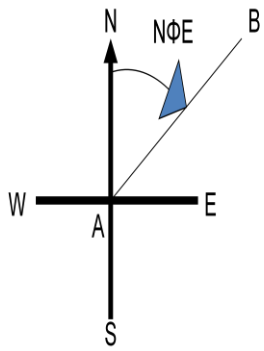


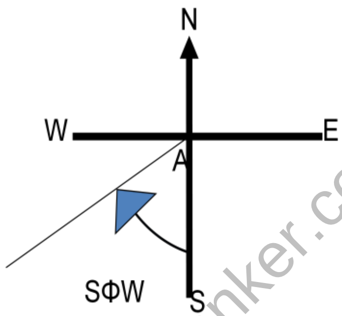
- **Declination:** The horizontal angle between the magnetic meridian and true meridian is known as declination.

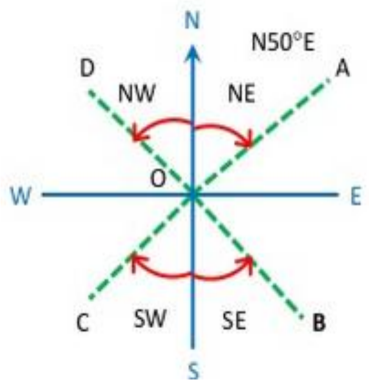
2.9 COMPUTATION OF ANGLES:

2.9.1 Reduced bearing:

- Either from north or south either clockwise or anticlockwise as per convenience
- Value doesn't exceed 90^0
- Denoted as N ΦE or S Φ W
- The system of measuring this bearing is known as Reduced Bearing System (RB System)

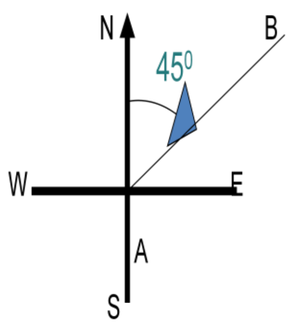


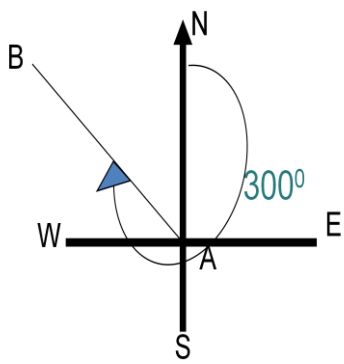


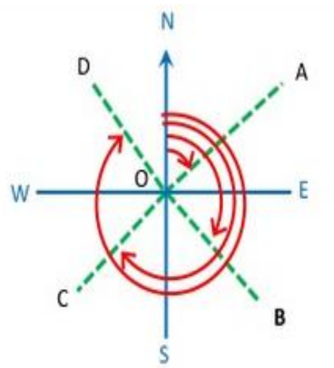


2.9.2 Whole Circle Bearing:

- Always clockwise either from north or south end
- Mostly from north end
- Value varies from $0^0 – 360^0$
- The system of measuring this bearing is known as Whole Circle Bearing System (WCB System)





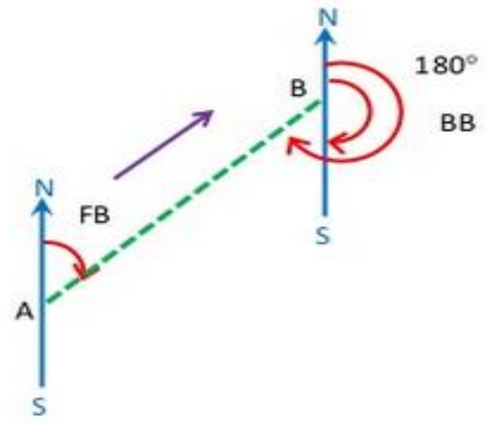


2.9.3 Fore bearing:

The bearing of a line in the direction of the progress of survey, is called as fore bearing or forward bearing.

2.9.4 Back bearing:

The bearing of a line in the direction opposite to the progress of survey, is called as back Bearing or backward bearing.



2.9.5 Conversion of Whole Circle Bearing into Reduced bearing:

Line	W.C.B. Between	Rule For R.B	Quadrant
AB	0^o ------ 90^o	R.B = W.C.B	N --- E
AC	90^o ------ 180^o	R.B = 180^o – W.C.B	S --- E
AD	180^o ------ 270^o	R.B = W.C.B - 180^o	S --- W
AE	270^o ------ 360^o	R.B = 360^o – W.C.B	N ---W

2.9.6 Conversion of Reduced Bearing into Whole circle bearing:

Line	R.B	Rule For W.C.B	W.C.B. Between
AB	N α E	W.C.B = R.B	0^o ------ 90^o
AC	S β E	W.C.B = 180^o – R.B	90^o ------ 180^o
AD	S θ W	W.C.B = 180^o + R.B	180^o ------ 270^o
AE	N φ W	W.C.B = 360^o - R.B	270^o ------ 360^o

2.10 TRAVERSING:

Almost all surveying requires some calculations to reduce measurements into a more useful form for determining distance, earthwork volumes, land areas, etc.

A traverse is developed by measuring the distance and angles between points that found the boundary of a site.

We will learn several different techniques to compute the area inside a traverse.

It is a method of surveying in which the lengths and directions of consecutive lines can be measured with a steel tape, chain and transit Theodolite.

They are 2 types of traversing

1. Closed traversing
2. Open traversing

2.10.1Closed Traversing:

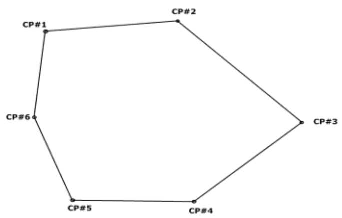
It is said to be closed one if it returns to the starting point, thereby forming a closed polygon. It is used for locating the boundaries of lakes and wood across which tie lines cannot be measured for area determination, control for mapping, and for surveying moderately large area.

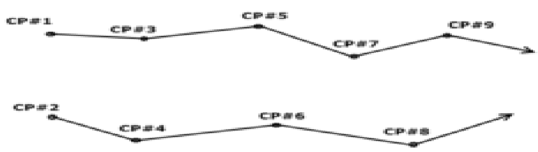
2.10.2 Open Traversing:

An open traverse is one that does not return to the starting point. It consists of a series of lines expanding in the same direction.

An open traverse cannot be checked and adjusted accurately. It is employed for surveying long narrow strips of country.

Example: Path of a highway, railway, canal, pipeline, Coastline, transmission line etc.,





2.10.3 Purpose of traverse:

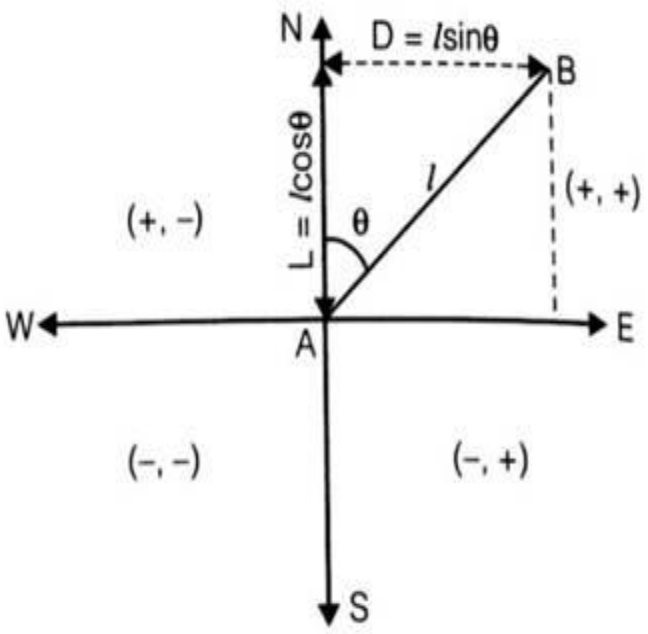
1. To establish the control networks.
2. It involves placing survey stations along a line or path of travel.
3. The previously surveyed points as a base for observing the next point.

2.10.4 Traverse Computation:

Latitude and Departure:

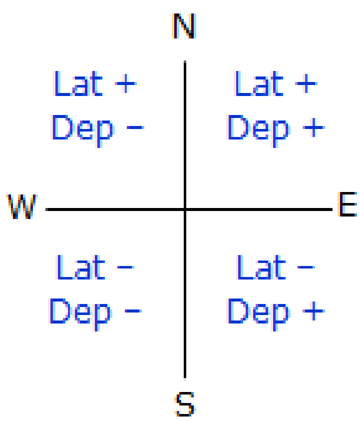
1. The latitude of a survey line may be defined as its co-ordinate length measured parallel to an assumed meridian direction.
2. The departure of survey line may be defined as its co-ordinate length measured at right angles to the meridian direction.
3. The latitude and departure of the line AB of length L and reduced bearing θ are given by

   Latitude (L) = + l Cos θ and Departure (D) = + l Sin θ



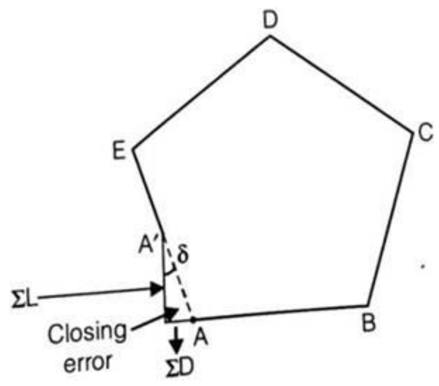
4. The sign of latitudes and departures will depend upon the reduced bearing of a line. The following table gives signs of latitudes and Departures.

W.C.B.	R.B and Quadrants	Sign of	
		Latitude	Departure
0^o to 90^o	N θ E ; I	+	+
90^o to 180^o	S θ E ; II	-	+
180^o to 270^o	S θ W ; III	-	-
270^o to 360^o	N θ W ; IV	+	-



2.10.5 Traverse Adjustment:

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 errors in the field measurements of angles and distance. Such error is known as closing error.



Closing Error $e = AA' = \sqrt{(\sum L)^2 + (\sum D)^2}$

The direction of closing error is given by = $\tan \delta = (\sum D)/ (\sum L)$

2.11 OMITTED MEASUREMENTS:

In conducting traverse survey to lengths and bearings of all lines should be measured in the field itself, but owing the field conditions (obstacles) or due to forgetness of the observer. Some measurements might be omitted in the field. Such omitted quantities can be calculated using the properties of the closed traverse.

The following cases may be arised:

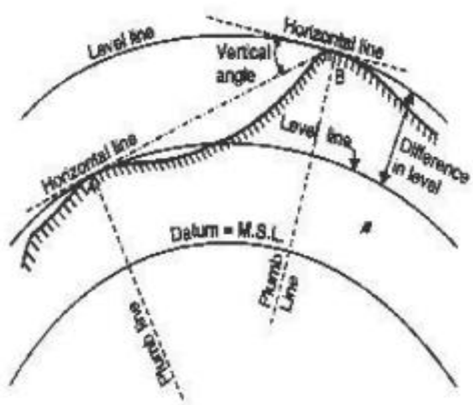
1. Length (or) Bearing (or) both of a line omitted
2. Length of one line, bearing of other line omitted
3. Length of two lines omitted
4. Bearing of two lines omitted

UNIT – III

LEVELLING AND CONTOURING

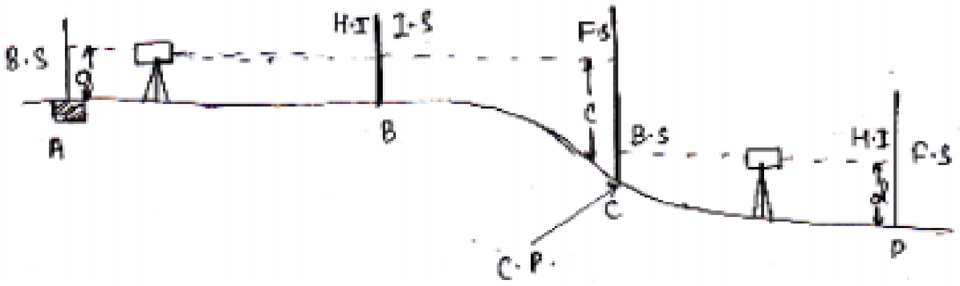
3.1 CONCEPT AND TERMINOLOGY

- **Levelling**: The operation of determine the difference of elevation of points with respect to each other on the surface of the earth is called leveling.
- **Level Surface:** A surface parallel to the mean spherical surface of the earth is called level surface.
- **Vertical line:** It is a line from any point on the earth"s surface to the centre of the earth and is commonly considered to be the line defined by a plumb line.
- **Level Line:** It is a line laying on a level surface and normal to plumb line at all the points.
- **Horizontal Plane:** It is a plane tangential to the level surface at the point under consideration and perpendicular to plumb line.
- **Horizontal Line:** It is a line lying in the horizontal plane. It is a straight line tangential to level line



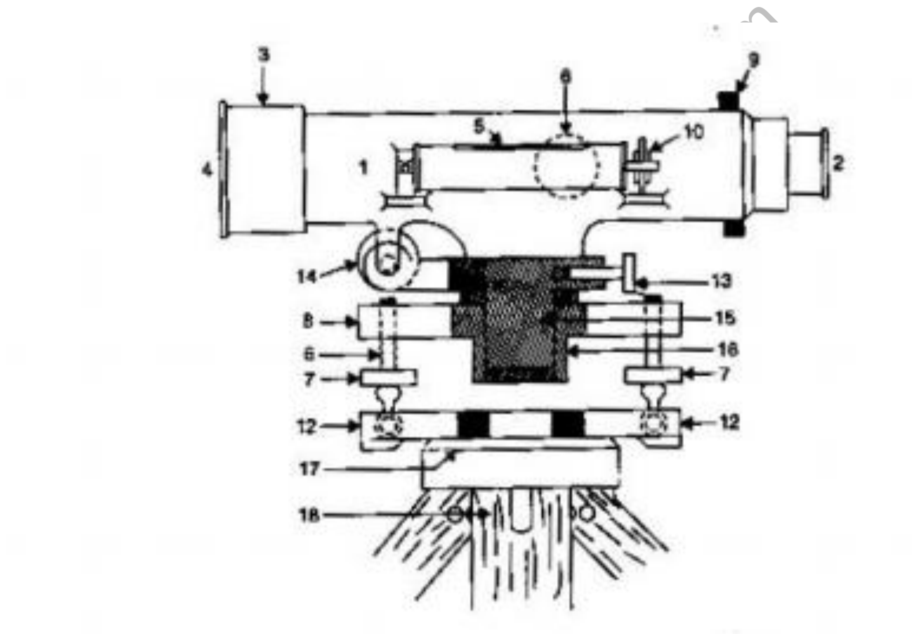
- **Elevation:** It is a vertical distance above or below the datum. It is also known as reduced level (R.L).
- **Axis Of Telescope:** It is a line joining the optical centre of the objective to the centre of the eyepiece
- **Line Of Sight (or) Line Of Collimation**: It is a line joining the intersection of the cross-hirs to the centre of the objective and its continuation.
- **Axis Of Level Tube or Bubble Tube:** It is the elevation of the plane of collimation when the instrument is leveled.
- **Back Sight (B.S):** It is a staff reading taken on a point of known elevation i.e., a right on a bench mark or on a change point i.e., station C in fig.

- **Fore Sight (F.S):** It is a staff reading taken on a point whose elevation is to be determined. Eg. A sight on a change point i.e., station C & D in above fig.
- **Intermediate Sight (I.S):** It is a staff reading taken on a point of unknown elevation between back sight and foresight, Eg. A sight on station B in above fig.



- Change Point (C.P) or Turning Point (T.P): It is a point denoting the shifting of the level. Both F.S and B.S are taken on this point. Eg. Station C in above fig.
- Station: A point whose elevation is to be determined is called station. The shaft is kept at this point, Eg. A,B, and C in above fig.
- Parallax: It is the apparent movement of the image relative to the cross- hairs when the image formed by the objective does not fall in the plane of the diaphragm.
- Bench Mark (B.M): It is a fixed reference point of known elevation

3.2 LEVELLING INSTRUMENT:



1 TELESCOPE
2 EYE-PIECE
3 RAY SHADE
4 OBJECTIVE END
5 LONGITUDINAL BUBBLE
6 FOCUSING SCREW
7 FOOT SCREWS
8 UPPER PARALLEL PLATE (TRIBRACH)
9 DIAPHRAGM ADJUSTING SCREWS
10 BUBBLE TUBE ADJUSTING SCREWS
12 FOOT PLATE (TRIVET STAGE)
13 CLAMP SCREW
14 SLOW MOTION SCREW
15 INNER CONE
16 OUTER CONE
17 TRIPOD HEAD
18 TRIPOD

Level:

The instrument which is used to determine the vertical distance of points is known as level. A level consists of a telescope to provide the line of sight, a level tube to make the line of sight horizontal, a leveling head to bring the bubble of level tube at the centre, and a tripod to support the level.

Telescope:

- A telescope consists of a diaphragm ring and two convex lenses. The lens near the eye is called eyepiece and that towards object is called objective.
- The diaphragm ring consists of cross-hairs near the eye piece.
- Focussing screw is provided to give lateral movement of telescope.
- Adjustment Screw is provided to give small movement of telescope about vertical axis.

External Focussing Telescope:

- Focussing screw is provided to give lateral movement.
- Adjustment screw is provided to give small movement of telescope about vertical axis.

Internal Focussing Telescope:

- Focussing is done internally with a negative lens by moving the negative lens with a focussing screw.
- The objective and the eyepiece are kept at a fixed distance.

Levelling Head

- It is generally a conical socket attached with a triangular base called tribrach. Over which level tube is provided. It is having three or four leveling screws.
- The bubble in the level tube is brought to its centre by adjusting the level screws.

Level Tube

- It is a glass tube filled with a sensitive liquid such as alcohol or either leaving enough space to form a bubble.
- The bubble is made to come at the centre of level by leveling screws in all directions so that the instrument is horizontal.

Sensitiveness Of Level Tube:-

Measurement of sensitivity:

- Fix two points at a known distance apart, say 100m.
- Set up and level the instrument at „O‟.

- Take the reading on staff held vertical at C.
- By turning the foot screw, Move the bubble to n divisions.
- Read the staff again.
- Find the difference in two staff readings.

  $\alpha = S/D = n \times (l/R)$

  the term l/r is called sensitivity of bubble tube and equal to α^1.

  From $\alpha^1 = l/R = S/nD$ (in radians)

  $\alpha^1 = l/R = S/nD \times 20265$ (in seconds)

Where α = angle between the line of sight in radians

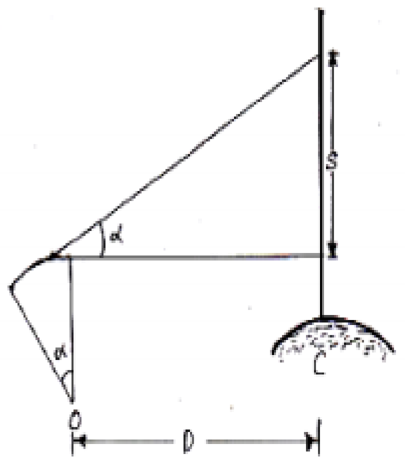
D = distance of the instrument from staff

n = number of divisions through which the bubble is moved.

R = Radius of curvature of tube

S = Staff intercept

l = length of one division of bubble tube (Usually 2 mm).



3.3 TEMPORARY ADJUSTMENTS:

The temporary adjustment consists of setting up leveling and elimination of parallax.

3.3.1 Setting up:

While locating the level, the ground point should be so chosen that

- The instrument is not too low or to high to facilitate reading on a bench mark.
- The length of back sight should preferably be not more than 98.0 m and
- The back sight distance and fore sight distance should be equal and the foresight should be so located that it advances the line of levels.
- Setting up includes fixing the instrument and approximate leveling by leg adjustment.

Fixing the instrument over tripod:

The clamp screw of the instrument is released. The level is held in the right hand. It is fixed on the tripod by turning round the lower part with the left hand and is finally screwed over the tripod.

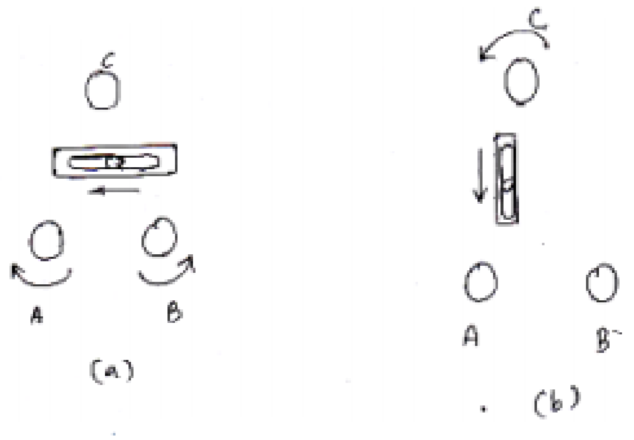
Leg adjustment:

The instrument is placed at a convenient height with the tripod legs. Spread well apart and so adjusted that the tripod head is nearly horizontal as can be judged by eye. Fix any two legs of the tripod firmly into the ground and move the third leg right or left in a circumferential direction until the main bubble is approximately at the centre. The third leg is then pushed in to the ground.

3.3.2 Levelling Up

Levelling with a three – screw head:

1. The clamp is loosened and the upper plate is turned until the longitudinal axis of the plate is parallel a line joining any two leveling screws, say A and B.
2. The foot screws are turned uniformly toward each other or away from each other until the plate bubble is central.
3. The telescope is rotated through 90^0 to its original position and the above procedure is repeated till the bubble remains central in both the positions.
4. The third screw is turned until the plate bubble is central.
5. The telescope is rotated through 90^0 to its original position and the above procedure is repeated till the bubble remains central in both the positions.
6. The telescope is now rotated through 180^0. The bubble should remain central if the instrument is in proper adjustment.



3.3.3 Elimination of Parallax

It consist of focusing the eye piece and objective of the level.

Focussing the eye – piece

The operation is done to make the cross-hairs appear distinct and clear visible. The following steps are involved.

- The telescope is directed skywards or a sheet of white paper is held infront of the objective.
- The eye piece is moved in or out till the cross-hairs appear distinct.

Focussing the objective

This operation is done to bring the image of the object in the plane of cross-hairs. The following steps are involved.

- The telescope is directed towards the staff.
- The focusing screw is turned until the image appears clear and sharp.

3.4 PERMANENT ADJUSTMENTS

These are the adjustments that are done to set the essential parts of the instrument in their time positions relative to each other.

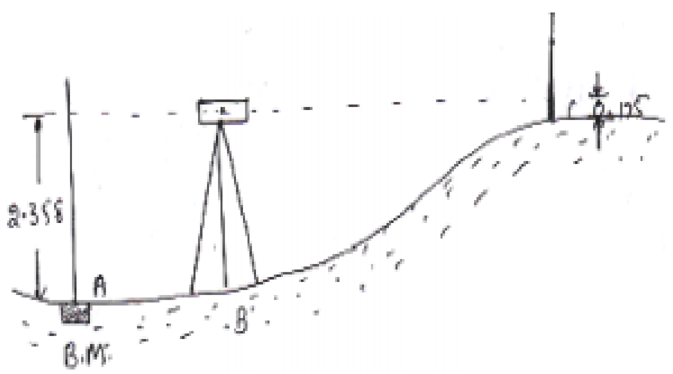
For a level, if care is taken to equalize back sight and fore sight distances, any error due to imperfect permanent adjustment is eliminated.

- Adjustment of level tube
- Adjustment of cross-Hair ring
- Adjustment of line of collimation

3.5 PRINCIPLES OF LEVELLING

3.5.1 Simple Levelling:-

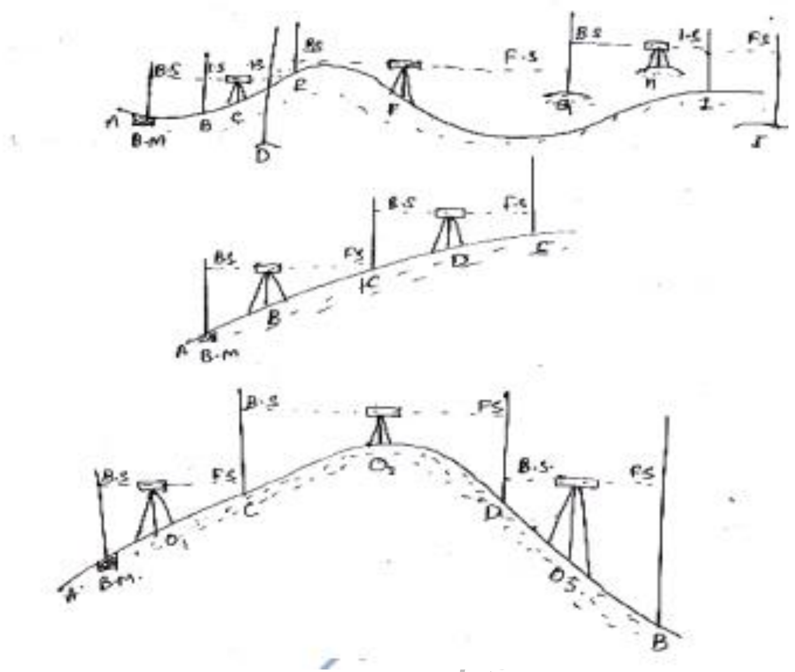
It is the simplest operation when it is required to fine the difference in elevation between two points, both of which are visible from a single position of the level.



3.5.2 Differential Levelling:-

Determining the difference in elevation between two or more points without any regard to the alignment of the points is called differential leveling. It is used when

- Two points at a large distance apart.
- The difference in elevation between the two points is large and
- Some obstacle instruments between the points.



3.6 BOOKING AND READING THE LEVEL

The observations are recorded in a level book. These are two methods of booking and reading the levels of the points from the observed staff readings.

3**.6.1 Collimation Method or Height of Instrument Method:-**

The elevation of plane of collimation or height of instrument for the first set up of the level is determined by adding back sight to the reduced level of a B.M.

The reduced levels of intermediate points and the first change point are obtained by subtracting the staff readings. The rise or fall is obtained by subtracting the staff readings taken on these points and a new plane of collimation is set by taking a B.S on the change point. The height of instrument is obtained by adding this B.S to its R.L. which was already calculated and the process continues.

Check: Σ B.S – Σ F.S = Last R.L – First R.L

3.6.2 Rise and Fall Method:

- It consists of determining the difference of levels between the consecutive points by comparing their staff readings. The rise or fall is obtained by calculating the difference between the consecutive staff readings.
- A rise is indicated if the back right is more than the fore sight and a fall if the back sight is less than the foresight.
- The reduced level of each point is obtained by adding the rise to or by subtracting the fall from the reduced level of the preceding point.

Check : ΣB.S . ΣF.S = Σ Rise – Σ fall = Last RL – First RL.

3.7 METHODS OF LEVELLLING

3.7.1 Reciprocal leveling:

- It is the operation of leveling in which the difference in elevation between two points by two sets of observations.
- This method is very useful when the instrument cannot be setup between the two points due to an obstruction such as a valley, river, etc., and if the sights are much longer than those which are ordinarily permissible.
- For such long sights the errors of reading the staff the curvature of earth, and the imperfect adjustments of the instrument become prominent. Special methods like reciprocal leveling should be used to minimize these errors.
- In this method, the instrument is setup near one point say A on one side on the valley, and a reading is taken on the staff held at A near the instrument and on the staff at B on the other side of valley. Let these readings b would have an error due to curvature, refraction and collimation.
- Let these readings be c and d. The near reading c is without error, where as reading d would contain an error c due to the reasons discussed above. Let n be the true difference of elevation between A and B.

  In the first case (fig (a))

  n = (b-e)-a

  In second case (fig (b))

  h = c-(d-e)

  adding the above two equations we get

  2h = b-e-a+c-d+e

  2h = b-a-c-d

2h = (b-a)+(c-d)

h = ½[(b-a)+(c-d)]

As „e" is eliminated like this, therefore reciprocal leveling eliminates the effect of atmospheric refraction, and earth curvature, as well as the effect of not adjusting the line of collimation.

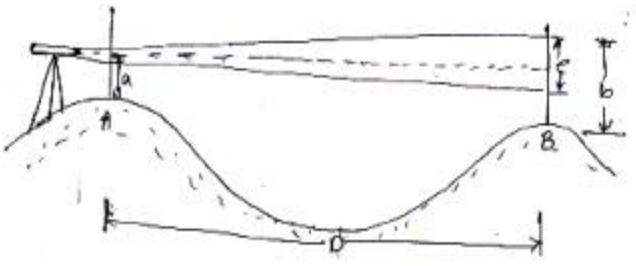
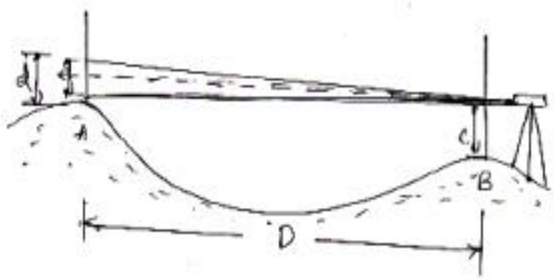


Fig (a)



3.7.2Precise Levelling:

- This is the operation of leveling in which precise instruments are used.
- In ordinary leveling, the distances between check points are relatively short.
- In precise leveling, the level loop may be of substantial length and efforts are made to control all the sources of errors.
- The most important error control in precise leveling is the balancing of foresight and back sight distances. This eliminates the collimation error and errors due to curvature, and minimizes errors due to refraction.
- Temperatures are read at interval to correct the graduations along the length of the staff.
- Precise leveling is used for establishing bench marks with high precision.

3.7.3Fly Levelling:

- It is an operation of leveling in which a line of levels is run to determine the approximate elevations.
- It is carried out for reconnaissance of the area.

3.7.4Check Levelling:

- It is the operation of running levels to check the accuracy of the benchmarks previously fixed. At the end of each days work, a line of levels is run, returning to the BM with a view check the work done on that day.

3.7.5Trigonometric Levelling:

- This is an indirect method of leveling in which the difference in elevation of the points is determined from the observed vertical angles and measured distances.
- The vertical angles are measured with a transit and the distances are measured directly (plane survey) or computed trigonometrically (geodetic survey).

This is commonly used in topographical work to find out the elevation of the top of buildings, chimneys, church spices, and so on.

3.7.6Barometric Levelling:

- The principle used in barometric leveling is that the elevation of a point is inversely proportional to the weight of the air column above the observer.
- The instrument used for measuring pressure is called barometer. The modified form of a barometer used to find relative elevation of points on the surface of earth is called Altimeter.
- The method used to measure as single base method, two altimeters are required. One altimeter is placed at a point of known elevation and the other altimeter is placed at the desired point whose elevation is desired and the readings of these two barometers are noted.
- The difference in elevation between the two points may be obtained by the following formula.

$$H= 18336.6\ (\log_{10}h_1 - \log_{10}h_2)\ [1+((T_1+T_2)/500)]$$

Where H = the difference in elevation between two points.

h_1,h_2 = the barometric readings (in cm) at the lower and higher points respectively,

T_1,T_2 = temperature of air (in 0^o c) at the lower and higher points respectively.

3.7.7Hypometry:

- The altitude of various points may be obtained by using an instrument known as hypsometer.
- It works on the principle that a liquid boils when its vapour pressure is equal to the atmospheric pressure.

- It may be noted that the boiling point of water is lowered as the pressure decreases i.e., as a higher altitude is attained.
- This method consists in determining the boiling point temperature at various station. The corresponding atmospheric pressures may be obtained from the tables. In the absence of tables, the following approximate formula may be used.

h = 76.00 ± 2.679t

where h = pressure in an

t = difference of boiling point from $100^{o}c$

The difference in elevation between two points is obtained by

$H = 18336.6 (\log_{10}h_1 - \log_{10}h_2) [1+((T_1+T_2)/500)]$

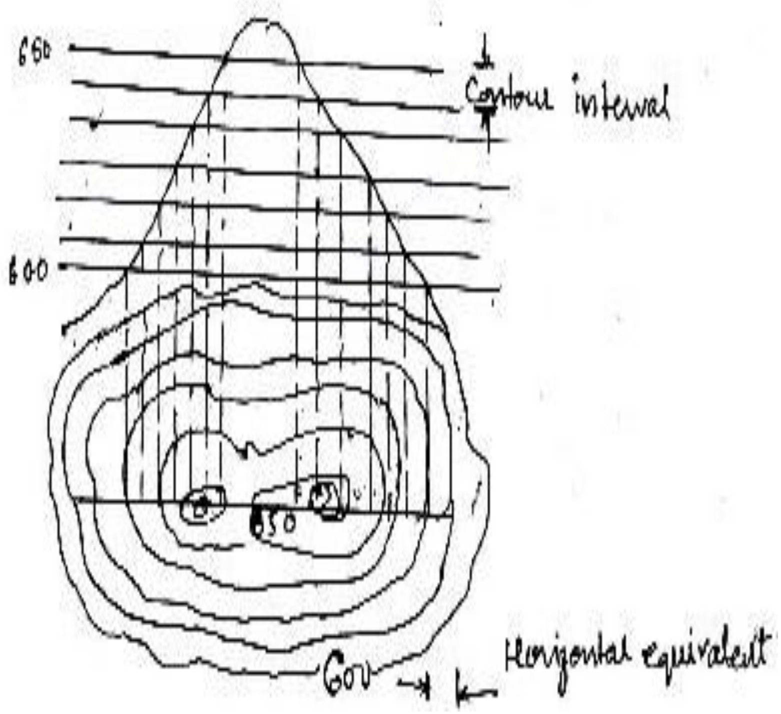
3.8 CONTOURING:-

A contour may be defined as an imaginary line passing through points of equal elevation.

(or) It is defined as the intersection of a level with the surface of earth.

Contour Interval

- The vertical distance between consecutive contours is termed as control interval.
- It is desirable to have a constant interval throughout the map.
- The contour interval depends on the following factors.



3.8.1 Scale of the map

Contour interval is inversely proportional to the scale of the amp. For a topographic map, the interval may range as shown below.

Ground Surface	Large Scale (1 Cm = 1 – 10 m)	Intermediate Scale (1 cm = 10 – 100 m)	Small Scale (1 cm = 100 m onwards)
Flat	0.2 – 0.5 m	0.5, 1 or 1.5	1, 2 or 3
Rolling	0.5 – 1 m	0.5 – 1.5 m	2 – 5 m
Hilly	1, 1.5 or 2 m	2, 2.5 or 3 m	5 – 10 m

3.8.2 Purpose of the map

Contour interval is kept large up to 2.0 m for projects such as highways and railways, whereas it is kept as small as 0.5 m for measurement of earth works, building sites, dams etc., for a city survey, a contour interval of 0.5 m may be adopted, and for more extended surveys such as a geological survey usually 6 – 15 m are adopted.

3.8.3 Nature of the country

Contour interval varies with the topography of the area. It is large for steep grounds and small for flat grounds.

3.8.4 Time

Contour interval is kept large when time is less.

3.8.5 Funds

Contour interval is kept large when funds are short and limited.

3.8.6 Horizontal Equivalent

- The horizontal distance between consecutive contours is termed as horizontal equivalent.
- Steeper the ground, lesser the horizontal equivalent.

3.8.7 Contour Gradient

A line lying on the ground surface throughout and maintaining a constant inclination is called contour gradient.

3.8.8 Grade Contours

- The lines having equal gradient along a slope are called grade contours.
- The difference in elevation of two points of grade contours divided by the distance between than is always a constant gradient.

The gradient of PQ = (12 - 11)/PQ

The gradient of PQ^1 = (12 - 11)/PQ

A PQ = PQ^1

(12 - 11)/PQ = (12 - 11)/PQ^1

Therefore PQ and PQ^1 are grade contours as their gradients are equal.

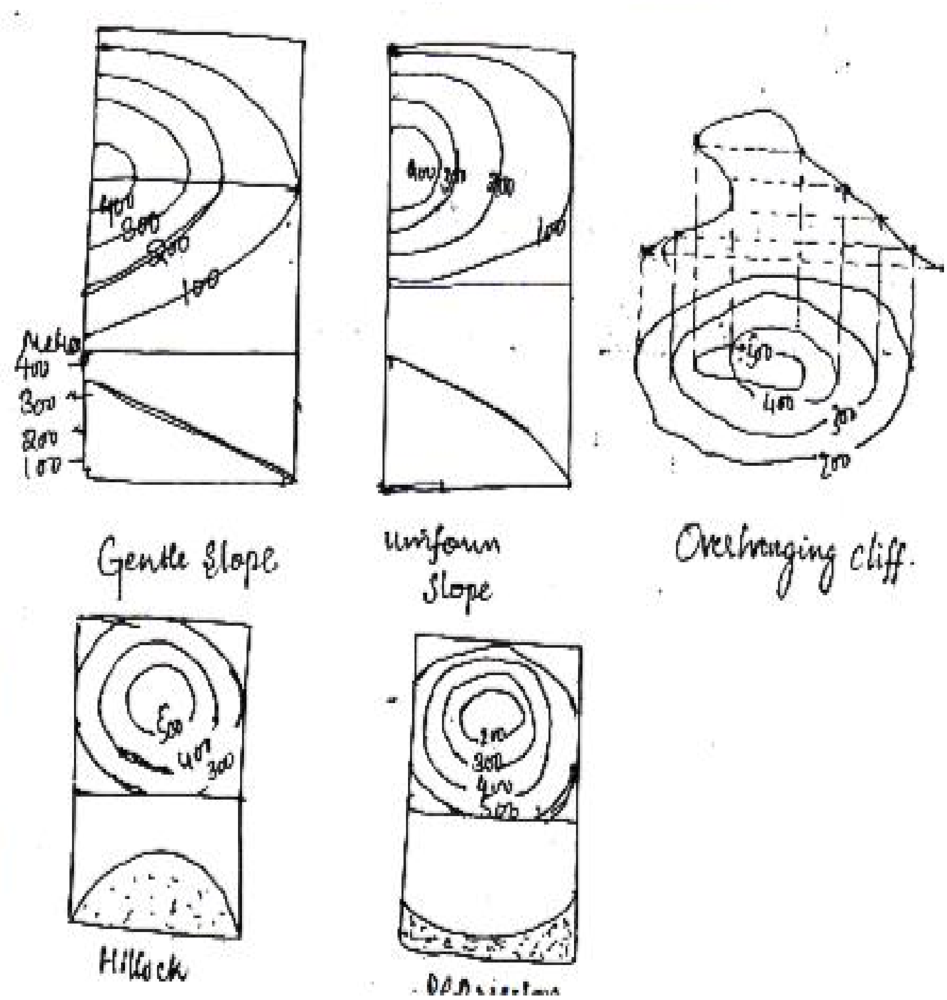
3.9 USES OF CONTOURS

1. With the help of contour map proper and precise location of engineering works such as roads, canals etc., can be decided.
2. In location of water supply, water distribution and to solve the problems of stream pollution etc.,
3. Planning and designing of dams, reservoirs, aqueducts, transmission lines.
4. To select sites for new industrial plants.
5. To ascertain the indivisibility of station.
6. To ascertain the profile of the country along any direction
7. To estimate quantity of cutting, filling and the capacity of reservoirs

3.10 CHARACTERISTICS OF CONTOUR LINES:

1. All the points on a contour line have the same elevation. The elevations of the contour are indicated either by inserting the figure in a break in the respective contour or printed close to the contour. When no value is present, it indicates a flat terrain. A zero meter contour line represents the coast line.

2. Two contour lines do not intersect each other except in the cases of an overhanging cliff or a cave penetrating a hillside.
3. A contour line must close on to itself not necessarily with in the limit of a map.
4. Equally spaced contour represent a uniform slope and contours that are well apart indicates a gentle slope.
5. A set of close contours with higher fig inside and lower fig outside indicate a hillock, whereas in the case of depressions, lakes, etc., the higher fig are outside and the lower and the lower fig are inside.



6. A watershed or rigid line (line joining the highest points of a series of hills) and the that weg or valley line(line joining the lowest points of a velley) cross the contours at right angles.
7. Irregular contours represent uneven ground.
8. The direction of steepest slope is along the shortest distance between the contours. The direction of the steepest slope at a point on a contour is, therefore, at right angles to the contour.

3.11 METHODS OF CONTOURING

3.11.1 Direct Methods:

The field work in contouring consists of horizontal and vertical control. The horizontal control for a small area can be exercised by a chain or tape and by compass, theodolite or plane table for a large area. For vertical control either a level and staff or a hand level may be used.

(1) By Level and Staff:

The method consists of locating a series of points on the ground having the same elevation. To do this an instrument ground station is selected so that it commands a view of most of the area to be surveyed. The height of the instrument is fixed from the nearest benchmark. For a particular contour value, the staff reading is worked out. The staff man is then directed to move right or left along the excepted contour until the required reading is observed. A series of points having the same staff readings, and thus the same elevations are plotted and joined by a smooth curve.

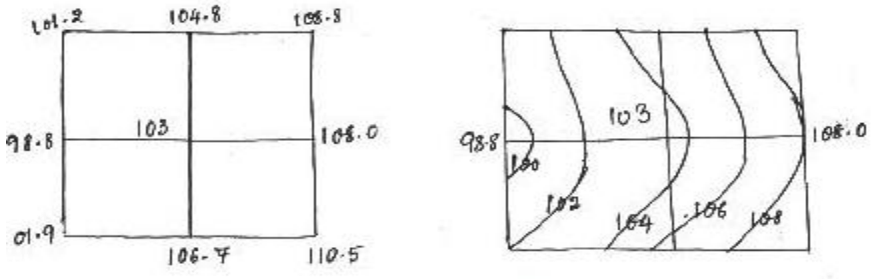
(2) By hand Level:

The principle used is the same as that used in the method using a level and staff. However, this method is very rapid and is preferred for certain works. The instruments used are a hand level, giving an indication of the horizontal line from the eye of the observer and level staff or a pole having a zero mark at the height of the observer‟s eye and graduated up and down from this point. When an observation is made on the staff or pole, the reading on it is the difference in elevation between the foot of the observer and that of the pole. In this method the instrument man stands over the benchmark and the staff man is moved near to a point on the contour which has to be plotted. As soon as the instrument man observers the required staff reading for a particular contour, the instructs the staff man to stop and locate the position of the point.

3.11.2 Indirect Methods:

(1). Method of Squares:

This is also called coordinate method of locating contours. The entire area is divided into squares or rectangles forming a grid. The elevations of the corners are then determined by spirit levelling. The levels are then interpolated. This method is very suitable for a small open area where contours are required at a close vertical interval.



(2).Methods of Cross – Sections:

In this method a transit traverse is run. Then suitably spaced sections are projected from traverse lines. The observations are made in the usual manner with a level, clinometers, or theodolite at points on these transverse lines. The contours are then interpolated. This method is suitable for road, railway and canal survey.

(3). Plane table method:

A plane table is placed on the traverse station and an alidade is sighted on a rod with two targets at a fixed distance apart (1-2m). The direction of the line is drawn along the ruling edge of the alidade. With a tangent clinometers the vertical angles are read corresponding to the two targets. The distance and elevation of the staff point is reduced by trigonometric relations. The contours are then interpolated. The observer scales the computed distance along the plotted line to locate the point and writes the computed elevation in such a way that the plotted position of the point coincides with the decimal point of the elevation value. Meanwhile the staff man selects and moves to the next point and continues the work till sufficient observations for interpolating contours are made.

(4).Tacheometric Method:

This method is particularly suitable for hilly areas and at places where plane tabling is impractical. First of all, reconnaissance of the area is done and a network of traverse is arranged in such a way that the entire area can be covered. The traverse stations are so chosen that large vertical angles. Particularly for long sights are avoided. From these traverse stations a number of radial lines are drawn at some angular interval depending upon the nature of the country. A tacheometer, fitted with an anollartic lens is placed on the traverse stations. The observations corresponding to cross – wire, stadia wires and vertical angles are carried out on all the control stations and on the points of detail. The elevations and distances are then calculated and from the observed data. Contours are interpolated.

3.12 INTERPOLATION OF CONTOURS:

(1). By Estimation:

This is a very crude method and is usually adopted where the ground forms are quite regular, the scale of the map is small, and high accuracy is not required. The positions of the contour points between the ground points are estimated and contours are drawn through the slope between the ground points in uniform.

(2).By Arithmetic Calculations:

This method is used when high accuracy is required and the scale of the map is of intermediate or large. In this method the distance between two points of known elevations are accurately measured. Then with the help of arithmetic calculations the positions of the required elevation points are computed.

Let A and B be two points with RL 52.60m and 55.80m respectively and at a distance of 16.00m apart. Let the contour interval be 1.00m and let it be required to locate a contour between the two points with values 53.00m. The contour can be located as follows. Difference of level between A and B = 55.50 – 52.60 = 3.20m

Difference of level between A and 53.00m contour = 53.00 – 52.60 = 0.40m
Distance of 53.00m contour form point A = (0.40/3.20) X 16 = 2.00m

(3).By Graphical Method:

When high accuracy is required and many interpolations are to be made this method of plotting contours proves to be the most rapid and convenient. On tracing paper parallel lines are drawn at some fixed interval, say 0.5m. Every tenth line is made thicken. Let A and B be two points of elevation at 50.50m and 64.50m respectively. The tracing paper is placed with point A on the line 50.50m and is turned till the point B is on line 64.50m. The intersectionsof the line AB and the lines of the required elevation point will give the position of the point on the respective contour.

3.13 PROBLEMS

The following consecutive readings were taken with a dumpy level: 6.21,4.92,6.12,8.42,9.1,6.63,7.91,8.26,9.71,10.21. the level was shifted after 4^{th}, 6^{th} and 9^{th} readings. The R.L of first point was 125.00. Rule out a page of level field work and fill all the columns. Calculate the reduced levels and apply usual check by both height of instrument method and rise and fall method.

Sol:

Height of instrument Method:

Station	B.S	I.S	F.S	H.I	R.L	Remarks
1	6.21			131.210	125.00	B.M
2		4.92			126.290	
3		6.12			125.090	
4	9.1		8.42	131.890	122.790	C.P
5	7.91		6.63	133.170	125.260	C.P
6		8.26			124.910	
7	10.21		9.71	133.670	123.460	C.P

Σ B.S = 23.22; Σ F.S = 24.76

Check: Σ B.S – Σ F.S = Last R.L – First R.L = -1.54 m.

Rise and fall method:

Station	B.S	I.S	F.S	Rise	Fall	R.L	Remark
1	6.21					125.00	
2		4.92		1.290		126.290	
3		6.12			1.200	125.090	C.P
4	9.1		8.42		2.300	122.790	C.P
5	7.91		6.63	2.470		125.260	
6		8.26			0.35	124.910	C.P
7	10.21		9.71		1.45	123.460	

Check: Σ B.S – Σ F.S = Last R.L – First R.L = ΣRise – Σfall = -1.54 m.

UNIT – IV

THEODOLITE

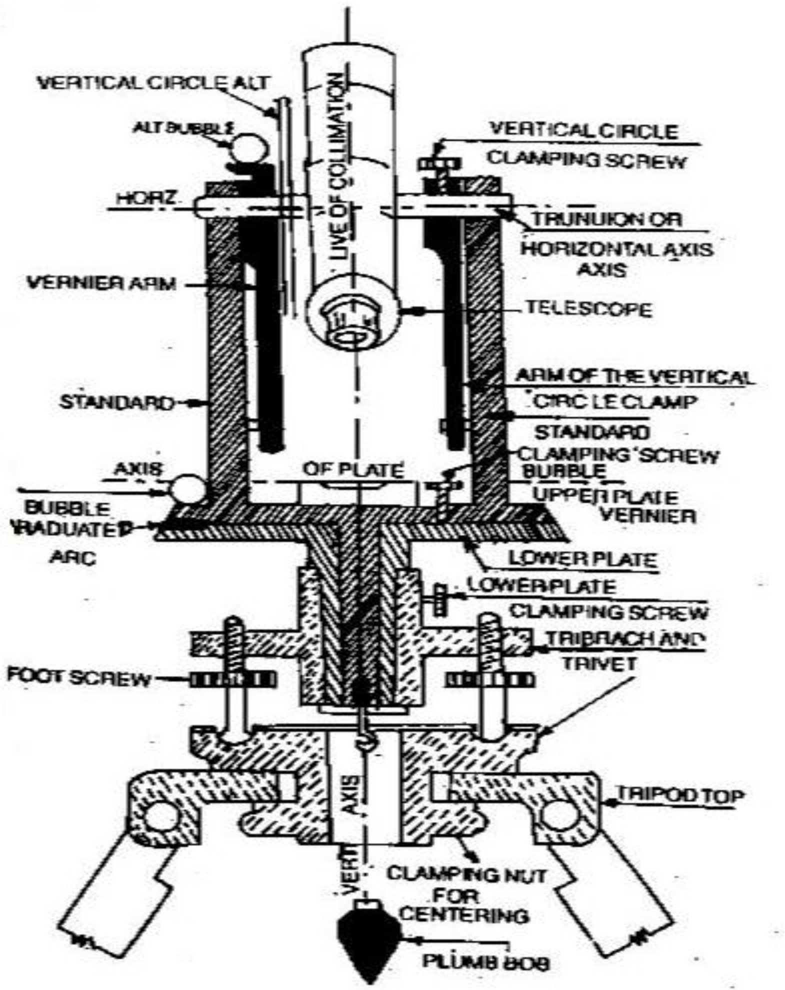
4.1 INTRODUCTION

The theodolite is the most precise instrument designed for the measurement of horizontal and vertical angles and has wide applicability in surveying such as laying off horizontal angles, locating points on line, prolonging survey lines, establishing grades, determining difference in elevation, setting out curves etc.,

Theodolite may be classified as

- Transit theodolite
- Non – Transit theodolite

A transit theodolite (or simply transit) is one is which the line of sight can be reversed by revolving the telescope through 180^0 in the vertical plane. The non- transit theodolites are either plain theodolites or Y- theodolites in which the telescope cannot be transited. The transit is mainly used and non- transit theodolites have now becomes obsolete.



4.2 DEFINITIONS AND TERMS

1. The Vertical axis

The vertical axis is the axis about which the instrument can be rotated in a horizontal plane. This is the axis about which the lower and upper plates rotate.

2. The Horizontal Axis

The horizontal or trunnion axis is the axis about which the telescope and the vertical circle rotate in vertical plane.

3. The line of sight or line of collimation

It is the line passing through the intersection of the horizontal and vertical cross – hairs and the optical centre of the object glass and its continuation.

4. The axis of level tube

The axis of the level tube or the bubble line is a straight line tangential to the longitudinal curve of the level tube at its centre. The axis of the level – tube is horizontal when the bubble is central.

5. Centring

The process of setting the theodolite exactly over the station mark is known as centring.

6. Transiting

It is the process of turning the telescope in vertical plane through 180^0 about the trunnion axis. Since the line of sight is reversed in this operation, it is also known as plunging or reversing.

7. Swinging the telescope

It is the process of turning the telescope in horizontal plane. If the telescope is rotated in clock-wise direction, it is known as right swing. If telescope is rotated in the anti – clockwise direction, it is known as the left swing.

8. Face left observations

If the face of the vertical circle is to the left of the observer, the observation of the angle (horizontal or vertical) is known as face right observation.

9. Face right observation

If the face of the vertical circle is to the right of the observer, the observation is known as face right observation.

10. Telescope normal

A telescope is said to be normal or direct when the face of the vertical circle is to the left and the “bubble (of the telescope) up”.

11. Telescope inveted

A telescope is said to be inverted or reversed when of the vertical circle is to the right and the "bubble down".

12. Changing face

It is an operation of bringing the face of the telescope from left to right and vice versa.

4.3 ADJUSTMENTS OF A THEODOLITE

The adjustments of a theodolite are of two kinds:

- Permanent Adjustments
- Temporary Adjustments

4.3.1 Permanent Adjustments

The permanent adjustments are made to establish the fixed relationships between the fundamental lines of the instrument, and once made, they last for a long time. They are essential for the accuracy of observations. The permanent adjustments in case of transit theodolite are:

- Adjustment of the horizontal plate levels.
- Collimation adjustment.
- Horizontal axis adjustment.
- Adjustment of the telescope level or the altitude level.
- Vertical circle index adjustment.

4.3.2 Temporary Adjustments

The temporary adjustment are made at each set up of the instrument before starting taking observations with the instrument. There are three temporary adjustments of a theodolite.

- Centering
- Levelling
- Focussing

4.3.2 Centering

It means bring the vertical axis of the theodolite immediately over a station mark. The station mark should be represented by a well- defined point such as end of a nail driven on the top of a peg or the intersection points of a cross marked at the surface below the instrument etc., to do this,

A. Attach the string of the plumb – bob to the hook under the vertical axis of the instrument.

B. Place the instrument over the station by spreading the legs well apart so that the telescope is at a convenient height, the plumb – bob is approximately over the station mark, and the levelling head is approximately levelled.

C. Lift the instrument bodily without disturbing the relative positions of the legs and move it until the plumb – bob hangs within 1 cm horizontally of station mark.

D. Unclamp the centre-shifting arrangement and move the instrument until the plumb – bob is exactly over the station-mark. The pointer end of the plumb-bob should hang within 3 mm vertically above the station mark. Then clamp the shifting head.

4.3.2 Levelling

Having centered and approximately leveled the instrument, it is accurately leveled with reference to the plate levels by means of foot- screws so that the vertical axis is made truly vertical. To level the instrument,

A. Loosen all clamps and turn the instrument about either of its axis until the longer plate level is parallel to any pair of foot-screws, the other plate level will then be parallel to the line joining the third foot-screw and the mid-point of the line joining the first pair.

B. Bring the long bubble to the centre of its run by turning both screws equally, either either inwards or both outwards.

C. Similarly bring the other bubble to the centre of its run by turning only the third foot-screw.

D. Repeat this until both the bubbles are exactly centered.

Now rotate the instrument about the vertical axis through a completes revolution. Each bubble will now remain central provided the plate levels are in correct adjustments. The vertical axis is thus made truly vertical.

If the vertical angles are to be measured, the instrument should be leveled with reference to the altitude level fixed on the index arm. To do this,

A. First level the instrument by plate levels. Then turn the telescope so that the altitude level is parallel to the line joining a pair of foot-screws and bring the bubble to the centre of its run by means of these screws.

B. Turn the telescope through 90^0 in the horizontal plane and make the bubble central by the third foot-screw.

C. Repeat this until the bubble remains central in these two positions.

D. Bring the altitude level over the third foot-screw and swing the telescope through 180^0. If now the bubble does not remain constant, correct half its deviation by clip screw and the other half by the third foot-screw.

4.3.2 Focussing

This is done in two steps

- Focusing of the eye-piece for distinct vision of the cross-hairs at diaphragm, and
- Focussing the object – glass for bringing the image of the object into the plane of the diaphragm.

A. Focussing the eye-piece: point the telescope to the sky or hold a piece of white paper in front of the telescope. Move the eye-piece in and out until a distinct and sharp black image of the cross-hairs is seen.

B. Focussing the object-glass: Direct the telescope towards the object and turn the focusing screw until a clear and sharp image of the object is obtained. It may be noted that parallax is completely eliminated if there is no movement of the image of the object when the eye is moved up and down.

4.4 MEASUREMENT OF HORIZONTAL ANGLES:

There are three method of measuring horizontal angles

- Ordinary Method
- Repetition method
- Reiteration Method

4.4.1 Ordinary Method:

To measure horizontal angle AOB:

(i). Set up the theodolite at station point O and level it accurately.

(ii). Set the vernier A to the zero or 360^0 of the horizontal circle so do this, loosen the upper clamp and turn the upper plate until the zero of vernier A nearly coincides with the zero of the horizontal circle. Tighten the upper clamp and turn its tangent screw to bring the two zeros into exact coincidence.

(iii). Loosen the lower clamp. Turn the instrument and direct the telescope approximately to the left hand object (A) by sighting over the top of the telescope. Tighten the lower clamp and bisect A exactly by turning the lower tangent screw. Bring the point A into exact coincidence with the point of intersection of the cross-hairs at diaphragm by using the vertical circle clamp and tangent screws. Alternatively bring the vertical cross-hair exactly on the lowest visible portion of the arrow or the ranging rod representing the point A in order to minimize the error due to non-vertically of the object.

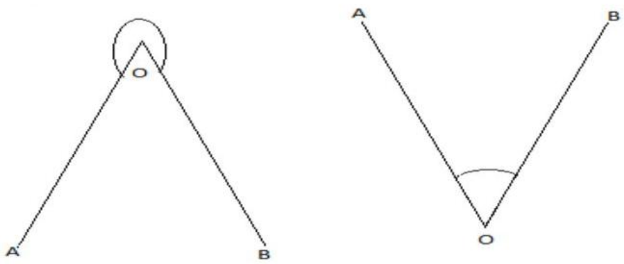
(iv). Having sighted the object A, see whether the vernier A still reads Zero. This is done to detect the error caused by turning the wrong tangent screw. Read the vernier B and record both vernier readings.

(v). Loosen the upper clamp and turn the telescope clockwise until the line of sight is set nearly on the right hand object (B). then tighten the upper clamp and by turning its tangent screw, bisect B exactly. In this operation, the lower clamp and its tangent screws should not be touched.

(vi). Read both verniers. The reading of the vernier A which was initially set at zero gives the value of the angle AOB directly and that of the other vernier B by deducting 180^0. The mean of the two vernier readings (after deducting 180^0 from the reading on vernier B gives the value of the required angle AOB.)

(vii). Change the face of the instrument and repeat the whole process. The mean of the two vernier readings given the second value of the angle ABC which should be approximately or exactly equal to the previous value.

(viii). The mean of the two values of the angle AOB, one with the face left and other with the face right, gives the required angle free from all instrumental errors.



4.4.2. Repetition Method:

This method is used for very accurate work. In this method, the same angle is added several times mechanically and the correct value of the angle is obtained by dividing the accumulated reading by the number of repetitions. The number of repetitions made usually is six, three with the face left and three with the face right. In this way, angles can be measured to a finer degree of accuracy than that obtainable with the least count of the vernier. However, it cannot be said that any desired degree of accuracy can be obtained by increasing the number of repetitions considerably because the errors due to frequent clamping etc., are introduced. There is therefore, no advantage in increasing the number of observations beyond a certain limit. Three repetitions with face left and three repetitions with face right are quite sufficient except in cases of very precise work.

To measure the horizontal angle AOB by repetition,

(1). Set up the theodolite at station-point O and level it accurately.

(2). Set the vernier A to zero or 360^0 by using the upper clamp and its tangent screw. Then loosen the lower clamp, direct the telescope to the left hand object A, and bisect A exactly by using the lower clamp and its tangent screw.

(3). Check the radius of the vernier A and see whether it still reads zero, and then read the other vernier.

(4). Loosen the upper clamp, turn the telescope clock-wise and bisect the right hand object (B) exactly by using the upper clamp and its tangent screw.

(5). Read both verniers. The object of reading the verniers is to obtain the approximate value of the angle.

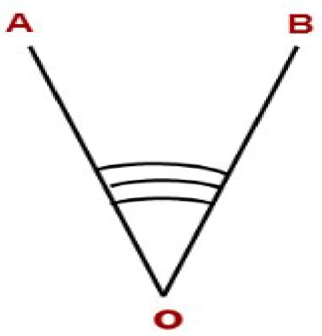
(6). Loosen the lower clamp and turn the telescope clock-wise until the object (A) is sighted again. Bisect A accurately using the lower tangent screw. Check the vernier readings which must be the same as before.

(7). Loosen the upper clamp. Turn the telescope clock-wise and again sight towards B. Bisect B accurately by using the upper tangent screw. The verniers will now read twice the value of the angle.

(8). Repeat the process until the angle is repeated the required number of times. Read both verniers. The final readings after n repetition should be approximately n x (50^0 4‘‘). Divide the sum by the number of repetitions and the result thus obtained gives the correct value of the angle AOB.

(9). Change the face of the instrument. Repeat exactly in the same manner and find another value of the angle (AOB).

(10). The average of the two values of the angle thus obtained gives the required precise value of the angle (AOB).



4.4.2.1Errors eliminated by changing face of theodolite

(i). Errors eliminated by changing face of theodolite:

- Error due to the line of collimation not being perpendicular to the horizontal axis of the telescope.
- Error due to the horizontal axis of the telescope not being perpendicular to the vertical axis.
- Error due to the line of collimation not coinciding with the axis of the telescope.

(ii). Errors eliminated by reading both verniers and averaging the readings:

- Error due to the axis of the vernier – plate not coinciding with the axis of the main scale plate.
- Error due to the unequal graduations.

(iii). Error eliminated by measuring the angle on different parts of the circle:

- Error due to unequal graduations.
- The errors in the pointings tend to compensate each other and the remaining error is minimized by the division.

- The error due to dislevelment of the bubble can be minimized by taking precautions in leveling.

4.4.3 Reiteration method

Reiteration is another precise and comparatively less tedious method of measuring the horizontal angles. It is generally preferred when several angles are to be measured at a particular station. This method consists in measuring the several angles successively, and finally closing the horizon at the starting point. The final reading of the vernier A should be the same as its initial reading. If not, the discrepancy is equally distributed among all the measured angles.

Suppose it is required to measure the angles AOB, BOC and COD. Then to measure these angles by reiteration method:

(1). Set up the instrument over station point O and level it accurately.

(2). Set the vernier A to 0 or 360^0 by using the upper clamp and its tangent screw.

(3). Direct the telescope to some well defined object (P) or say, the station point A, which is known as the “Reference object”. Bisect it accurately by using the lower clamp and its tangent screw. Check the reading at vernier A which should still be 0 or 360^0 and note the reading at vernier B.

(4). Loosen the upper clamp and turn the telescope clockwise until the point B is exactly sighted by using the upper tangent screw. Read both verniers. The mean of the two vernier readings (after deducting 180^0 from the reading at vernier B) will give the value of the angle AOB.

(5). Similary bisect C and D successively, read both verniers at each bisection, find the values of the angles BOC and COD.

(6). Finally, close the horizontal by sighting towards the reference object (P) or the station-point A.

(7). The vernier A should now read 360^0, if not note down the error. This error occurs due to slip etc.,

(8). If the error is small, it is equally distributed among the several observed angles. If large, the readings should be discarded and a new set of readings be taken.

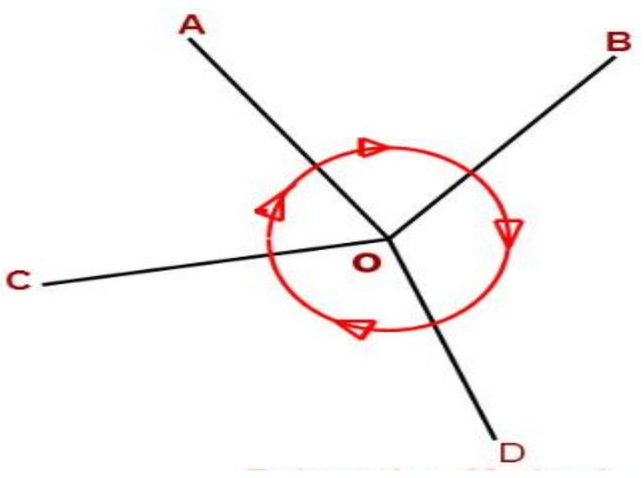
(9). Change the face of the instrument.

(10). Set the vernier A to a reading other than 0^0 say 60^0 or 90^0 this is done to avoid errors of graduation.

(11). Again measure the angles in the same manner by turning the telescope this time in the counter – clockwise direction to compensate or slip and error due to twisting of the instrument.

(12). Close the horizon and apply the necessary correction to all the angles as before.

(13). The mean of the two results for each angle is taken as its true value.



4.5 MEASUREMENT OF VERTICAL ANGLES

A vertical angle is an angle between the inclined line of sight and the horizontal. It may be an angle of elevation or depression according as the object is above or below the horizontal plane.

4.5.1 To measure the vertical angle of an object A at a station O

1. Set up the theodolite at station point O and level it accurately with reference to the altitude bubble.
2. Set the zero of vertical vernier exactly to the zero of the vertical circle by using the vertical circle clamp and tangent screw.
3. Bring the bubble of the altitude level in the central position by using the clip screw. The line of sight is thus made horizontal, while the vernier reads zero.
4. Loosen the vernier circle clamp screw and direct the telescope towards the object A and sight it exactly by using the vertical circle tangent screw.
5. Read both verniers on the vertical circle. The mean of tne two vernier readings gives the value of the required angle.
6. Change the face of the instrument and repeat the process. The mean of two vernier readings gives the second value of the required angle.
7. The average of the two values of the angle thus obtained, is the required value of the angle free from instrumental errors.

4.5.2 To measure the vertical angle between two points A and B

1. Sight A as before and take the mean of the two vernier readings at the vertical circle. Let it be α.
2. Similarly sight B and take the mean of the two vernier readings at the vertical circle, Let it be β.
3. The sum or difference of these readings will give the value of the vertical angle between A and B according as one of the points is above and the other below the horizontal plane or both points are on the same side of the horizontal plane.

4.6 PRINCIPLES OF ELECTRONIC THEODOLITE

- Circles can be instantaneously zeroed, or initialized to any value
- Angles can be measured with increasing values either left or right
- Angles measured by repetition can be added to provide a total larger than 360°
- Mistakes in reading angles are greatly reduced
- Speed of operation is increased
- Cost of instruments is lower

4.7 TRIGONOMETRICAL LEVELLING

It is the process of determining the differences of elevations of stations from observed vertical angles and known distances, which are assumed to be either horizontal or geodetic lengths at mean sea level. The vertical angles may be measured or computed. We shall discuss the trigonometrical levelling under two heads.

- Observations for heights and distances, and
- Geodetical Levelling

In the first case, the principle of plane surveying will be used. It is assumed that the distances between the points observed are not large so that either the effect of curvature and refraction may be neglected or proper corrections may be applied linearly to the calculated difference in elevation. Under this head fall the various methods of angular levelling for determining the elevations of particular points such as top of chimney, or church spire etc.,

In the geodetical observations of trigonometrical levelling, the distance between the points measured is geodetic and is large. The ordinary principles of plane surveying are not applicable. The corrections for curvature and refraction are applied in angular measure directly to the observed angles. The geodetically observations of trigonometrical levelling have been dealt with in the second volume.

In order to get the differences in elevation between the instrument station and the object under observation, we shall consider the following cases:

- Base of the object accessible.
- Base of the object inaccessible: instrument stations in the same vertical plane as the elevated object.
- Base of the object inaccessible: Instrument stations not in the same vertical plane as the elevated object.

4.8 METHODS OF TRAVERSING

The chief methods by which the relative directions of lines may be determined are:

- By measurement of angles between two successive lines
- By the direct observation of bearings of the survey lines.

The first method is generally used for long traverses and where the high degree of accuracy is required, while the second is employed for topographical surveys or for short traverses where high precision is not essential.

4.8.1 Theodolite traversing by direct observation of angles.

In this method, the angles between the successive lines are measured, and the bearing of the starting line is observed. The bearings of the remaining lines are then found from the observed bearing and the measured angles. This method includes:

- Traversing by the method of included angles.

- Traversing by the method of direct angles.
- Traversing by the method of deflection angles.

4.8.1.1 Traversing by the method of included angles

In a closed traverse, the angles measured are either interior or exterior according as the traverse is run in a counter clock wise direction or in a clockwise direction as shown in the figure shown but in both the cases the result is the same. The common practice is to run a closed traverse in the counter-clockwise direction. The angles can be measured by repetition that is why, any desired degree of accuracy can be obtained in this method. Procedure:

In running the traverse ABCDEFG, set up the theodolite at first station A and observer the bearing of the line AB. Then measure the angle GAB. Shift the instrument to each of the successive stations B, C etc., and measure and angles ABC, BCD etc. Measure the lines AB, BC, etc and the take the necessary offsets to locate the required details.

4.8.1.2 Traversing by the method of direct angles:

This method is commonly used for open traverses. In running the traverse as shown in figure, set up the theodolite at the starting station A and observe the bearing of line AB. Shift the theodolite to B. Set the vernier A to zero, take a backsight on the preceding station A.

Unclamp the upper plate, turn the telescope clockwise, take a fore sight on the following station C, and read both verniers. The mean of the two vernier readings is the required direct angle ABC. Take other angles in the similar manner. Chain the lines AB, BC etc., and take the necessary offsets in the usual way.

4.8.1.3 Traversing by the method of deflection angles:

This method is used for open traverses. This is much suitable when the survey lines make small deflection angles with each other as in the case of surveys for roads, railways, pipe lines etc., In running a traverse as in figure set up the theodolite at the starting station A and observe the bearing line AB. Shift the instrument to station B, set the vernier A to zero and take a back sight on A. Then transit the telescope, loosen the upper clamp, turn the telescope clockwise and take a foresight on C. Read both verniers, the mean of these readings is the required deflection angle of BC form AB. Also note down its direction. In this case, it is α_1 R (i.e., α_1 Right). Then set up the theodolite at each of the successive stations C, D, E, etc., and observer the deflection angles, and record them in the field-book. Chaining and offsetting is done in the usual manner.

4.8.2 Theodolite traversing by direct observation of Bearings:

This is also call fast needle method. There are three methods of observing bearings directly in the field.

- Direct method in which the telescope is transited.
- Direct method in which the telescope is not transited
- Back bearing method

4.8.2.1Direct method in which the Telescope is Transited:

1. Set up the transit at A and level it. Set the vernier A to zero. Point the telescope to the magnetic north as indicated by the magnetic needle attached to the transit by using the lower clamp and tangent screw.
2. Loosen the upper clamp and bisect B exactly by using the upper clamp and its tangent screw. Read the vernier A which gives the bearing of AB say 50^0. If the traverse is closed one, then observe also the back bearing of the last line (EA). This will serve as a check.
3. Shift the instrument and set it up at B. check whether the vernier A still reads the bearing of AB i.e., 50^0. If due to slip the readings differs, correct it with the upper tangent screw.
4. Using the lower clamp and tangent screws, backsight on A.
5. Transit the telescope. The line of sight is thus directed towards AB produced and the vernier A still reads the bearing of AB. Loosen the upper clamp, turn the telescope, and bisect C exactly by using the upper clamp and tangent screw.
6. Read vernier A, which now gives the bearing of line BC, say 115^0.
7. With the vernier A clamped at 115^0, transfer the instrument to C and repeat the process.
8. As a check upon the accuracy of work in a closed traverse, the back bearing of the last line (EA) observed in step (ii) at the first station A and its fore bearing taken at the last station (E) must differ exactly by 180^0. It should be noted that in this method, the telescope is transited for alternate back and fore readings.

4.8.2.2 Direct method in which the telescope is not transited.

This method is preferable in the case of non-transit instrument or in the case of transited instruments having poor adjustment.

The procedure is similar to that followed in the first method except that the telescope at B is not transited after the backs sight is taken on A, but rotated in a horizontal plane to sight C. the orientation thus becomes out by 180^o and so a correction of 180^0 has to be applied to the reading of the vernier A taken at B. Substract 180^0 if the reading if the reading is more than 180^0, and add 180^0, if the reading is less than 180^0 . At C, the orientation becomes out by 360^0, and is therefore correct, i.e., there is no need of applying the correction of 180^0. Therefore, this correction is necessary only at even instrument stationsi.e., 2^{nd} 4^{th},6^{th} and so on.

4.8.2.3 Back Bearing Method

1. Set up the instrument at A and observe the fore bearing of line AB.
2. Shift the instrument and set it up at B.
3. Set the vernier A to back bearing of AB.
4. With the vernier A kept clamped at the same reading, back sight on A by using the lower clamp and its tangent screw. As the vernier A is set to the back bearing of AB and the line of sight is directed towards BA, the instrument is in correct orientation.
5. Unclamp the upper plate and turn the telescope until C is sighted. Bisect C exactly by using the upper clamp and its tangent screw.
6. Read the vernier A which gives the bearing of BC.
7. Repeat the process at each of the subsequent stations.

Out of the three methods discussed above, the second is the best.

4.9 CHECKS ON CLOSED TRAVERSE

- Check on angular measurements
- Check on Linear Measurements

4.9.1Check on Angular Measurements:

a). Traverse by included angles:

i) Sum of the measured interior angles should equal (2N – 4) right angles.

ii) The sum of the measured exterior angles should equal (2N + 4) right angles Where N is number of sides of the traverse.

b). Traverse by deflection angles:

i) The algebraic sum of the deflection angles should equal 360^o.

ii) Consider the right hand deflection angles as positive, and left hand ones as negative.

c) Traverse by direct observation of bearings:

The back bearing on the last line observed at the first station, and the fore bearing of this line observed at the last station should different exactly by 180^o

4.9.2 Checks on Linear Measurements:

In a closed traverse, the sum of the northings (distances measured towards north) should be equal to the sum of the southings (distances measured towards south). Similarly the sum of the eastings (distances measured towards east) should be equal to the sum of the westings (distances measured towards west).

4.10: TACHOMETRIC SURVEYING:

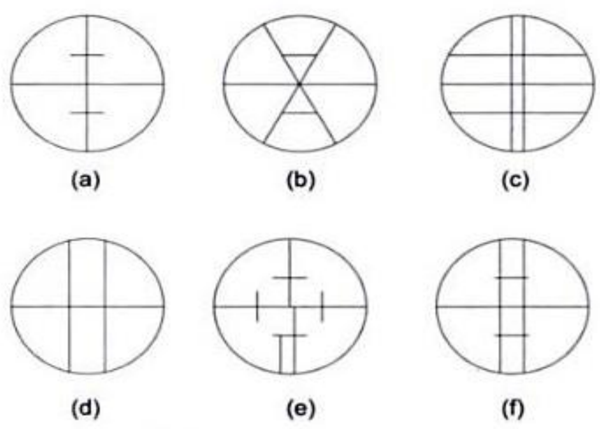
- It is a branch of angular surverying in which both horizontal and vertical distances between stations are determined from instrumental observations.
- In this method, measurements by a chain or tape are completely omitted.
- This method is very rapid and convenient.
- Though accuracy of tacheometric distances is low as compared to direct chaining on flat ground but the accuracy achieved by tacheometry is better as compared to chaining in broken ground or across large water bodies.
- The instrument employed for tachometric purpose is generally known as tacheometer which is similar to theodolite having diaphragm fitted with two additional horizontal wires called stadia-hairs.
- The accuracy of tachometric distance is such that under favorable condition, the error will not exceed is 1 in 1000.

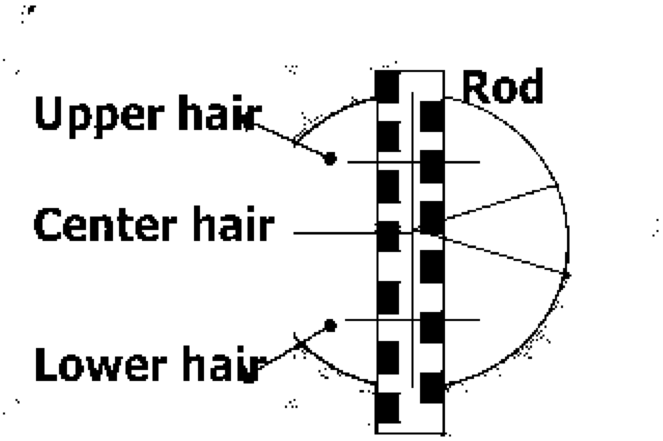
4.10.1 Purpose Of Tacheometric Surveying:

The primary object of tacheometry is to prepare contour plans. Also during surveys of higher accuracy it provides a check on distances measured with the tape or a chain.

4.11 INSTRUMENTS USED IN TACHEOMETRY:

A tacheometer is nothing but a transit theodolite fitted with a stadia diaphragm and an analytical lens. Different forms of stadia diaphragms commonly used are as given below:





Generally ordinary leveling staves are used for short distances marked with least count of 0.005m or 5mm and 4m long. For greater distances a specially designed stadia rod of 4m long may be used. The graduations are comparatively bold and clear and the least count is .0001m.

4.12 CHARACTERISTICS OF A TACHEOMETER:

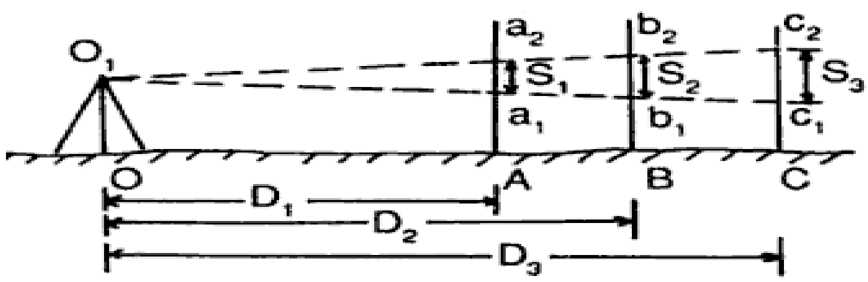
1. The value of multiplying constant F/I Or K should be 100.
2. The telescope should have a magnification of atleast 20-30%.
3. The aperture of the objective should be 35mm – 45mm so that a bright image can be obtained.
4. The eyepiece should be of greater magnifying power than usual, so that it is possible to obtain a clear staff reading form a long distance.
5. The telescope should be fitted with an anallatic lens to make the additive constant (f + d) or c is exactly equal to zero.

4.13 PRINCIPLE OF TACHEOMETRY

The principle of tacheometry is based on the property of isosceles triangle, where the ratio of the distance of the base from the apex and the length of the base is always constant.

From the below figure

$\Delta O_1 a_1a_2$, $\Delta O_1 a_1a_2$, & $\Delta O_1 a_1a_2$, are isosceles triangles where D_1, D_2, D_3 are the distances of the true bases one from the apex O_1 and S_1, S_2 and S_3 are the length of the bases.



As per the principles stated above we have a relation.

$$\frac{D1}{S1} = \frac{D2}{S2} = \frac{D3}{S3} = \frac{f}{i} \text{ (or) K (Constant)}$$

The f/i (or) K is known as the multiplying Constant,

Where f = focal length of objective and

i = Stadia intercept

$$\frac{D1}{S1} = K$$

$$\Rightarrow \quad D_1 = S_1 \times K$$

4.13.1Problem:

The readings of upper and lower stadia hairs are 3.255 and 1.365 respectively. Then find out the distance.

Solution

$S_1 = 3.255$

$S_2 = 1.365$

Distance = S x K

= (3.255 – 1.365) x 100

= 1.89 x 100

= 189 m.

4.14 DIFFERENT SYSTEMS OF TACHEOMETRIC MEASUREMENTS

The various systems of tacheometric survey may be classified as follows:-

- The Stadia System
- The Tangential System

4.14.1 Stadia Tacheometry

In this method, the diaphgram of tacheometer is provided with two stadia hairs one upper and another lower. Looking through the telescope the stadia hair readings are taken and noted.

The difference between upper and lower stadia hair reading is known as stadia intercepts. To determine the distance between the instrument station and the staff, the staff intercept 's' is multiplied by the stadia constant (k or f/I) generally 100.

The stadia method is further divided into two types.

1. Fixed hair method
2. Movable hair method

Fixed hair method:

This is the most commonly used method of tacheometry. In this method the stadia hair are kept at fixed interval and the value of staff intercept(S) varies with the distance from instrument station.

Movable hair method or subtense method:

In this method the staff intercept on the leveling staff is kept constant by fixing two targets or vanes at a known distance apart. During the observation the distance between stadia hair adjusted that the upper hair coincides upper target and the lower hair coincides with the lower target, by adjusting the micrometer screws fitted on the diaphgram.

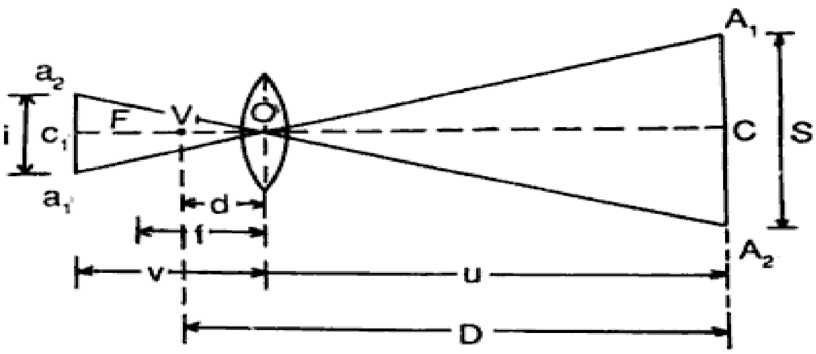
The variable stadia intercept is measured and the required distance is then calculated. Generally this method is not adopted.

4.14.2 Tangential Tacheometry:

In this method, the stadia hairs are not used. Readings on a staff are taken against the horizontal cross hair. To measure the staff intercept two pointings of the telescope is necessary. This method is generally not adopted as two vertical angles are required to be measured for one single observation.

4.15 THEORY OF STADIA TACHEOMETRY:

Let us assume that line of sight is horizontal and telescope is used to external focusing type. Above figure shows the outlines of the telescope with its axis horizontal. The staff is held vertically at point. Let us first consider the two extreme rays $A_1$0 and $A_2$0 coming from the points a_1 and a_2 respectively and passing through the optical centre.



$a_{1,}$ $a_{2,}$ c_1 = bottom, top and central hairs of diaphragm,

A_1, A_2, C = Reading on staff cut by three hairs.

a_1a_2 = i = length of image

A_1 A_2 = s = staff intercept

O = optical centre of object glass.

F = focus

V = vertical axis of instrument

f = Focal length of object glass.

d = Distance between optical centre and vertical axis of instrument.

u = Distance between optical centre and staff

v = Distance between optical centre and image

From Triangle a_1oa_2 and A_1OA_2 are similar triangles.

$$\frac{i}{s} = \frac{v}{u}$$

Or $$v = \frac{iu}{s}$$

From the properties of lenses,

$$\frac{1}{v} + \frac{1}{u} = \frac{1}{f}$$

Putting the value of v in equation

$$\frac{1}{iu/s} + \frac{1}{u} = \frac{1}{f}$$

Or $$\frac{s}{iu} + \frac{1}{u} = \frac{1}{f}$$

Or $$u = [\frac{s}{i} + 1]\, f$$

But $$D = u + d$$

$$D = [\frac{s}{i} + 1]\, f + d$$

$$D = \frac{f}{i} \times s + (f + d)$$

$$D = Ks + C$$

Where Constants

$$K = \frac{f}{i} \text{ and } C = (f + d)$$

K is known as multiplying constant (or) stadia interval factor. Its value is usually100. The constant “C” is known as additive constant. Its value is generally 0.3 to 0.6 m in external focusing telescope. Its value is 0.08 to 0.2 m for interval focusing telescope.

4.15.1. Problems

1. A tacheometer has a diaphragm with three cross-hairs faced at distance 1.15mm. the focal length of object glass is 23 cm and distance from object glass to the horizontal axis is 10 cm, calculate tacheometric constant.

Solution

f = 23 cm

d = 10 cm

i/2 = 1.15

K =?

$$K = \frac{f}{i}$$

i/2 = 1.15 ➔ i = 1.15 x 2 = 0.23 cm

$$K = \frac{23}{0.23} = 100.$$

2. The stadia readings with horizontal sight on a vertical staff held 50 m from a tacheometer were 1.285 m and 1.780 m. the focal length of the object glass was 25 cm. the distance between the object glass and the vertical axis of the tacheometer was 15 cm, calculate the stadia interval.

Solution:

f = 25 cm

d = 15 cm

S = 1.780 – 1.285 = 0.495 m

D = 50 m

$$D = KS + C$$

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

$$50 = \frac{25}{i} \times 0.495 + (25 + 15)$$

$$50 = \frac{25}{i} \times 0.495 + (0.25 + 0.15)$$

$$(50 - 0.4) = \frac{0.25}{i} \times 0.495$$

$$\frac{49.6}{0.495} = \frac{0.25}{i}$$

$i = 2.49 \times 10^{-3}$ m

$= 2.49$ mm

3. Determine constant of tacheometer from following readings taken with it.

Distance of staff From instrument Vertical axis	Reading against Lower Wire	Stadia wire Upper Wire
30 m	1.086 m	1.383 m
60 m	0.924 m	1.521 m

Solution

$D_1 = 30$ m, $D_2 = 60$ m, $S_1 = 0.297$, $S_2 = 0.597$

$D_1 = KS_1 + C$

$D_2 = KS_2 + C$

Therefore $K = \dfrac{D2 - D1}{S2 - S1}$

$= \dfrac{60-30}{(0.597-0.297)}$

$= 100.$

$D_1 = KS_1 + C$

$30 = 100 \times 0.297 + c$

$C = 30 – 29.7 = 0.3$ m

$D_2 = KS_2 + C$

$60 = 100 \times 0.597 + c$

$C = 60 – 0.59 = 0.3$ m

3. The following readings are taken with tacheometer in the line of sight horizontal on a staff held vertical 0.950, 1.285, 1.620. Determine the horizontal distance from instrumental station to staff station if K = 100 and C = 0.15 and also determine the R.L of the staff station. If the R.L of the instrument station is 101.580 m and height of the horizontal axis is 1.460 m.

Solution

S = 0.67

K = 100

C = 0.15

R.L of instrument station = 101.580 m

H.I = 1.460 m

$D = KS + C$

$= 100 \times 0.67 + 0.15 = 67.15$

R.L of the staff station = 101.580 + 1.460 – 1.285

= 101.755 m.

4.15.2 DISTANCE AND ELEVATION FORMULAE FOR DIFFERENT POSITION:-

4.15.2.1Stadia Method (fixed hair):-

Case – I

Distance and elevation for horizontal sights

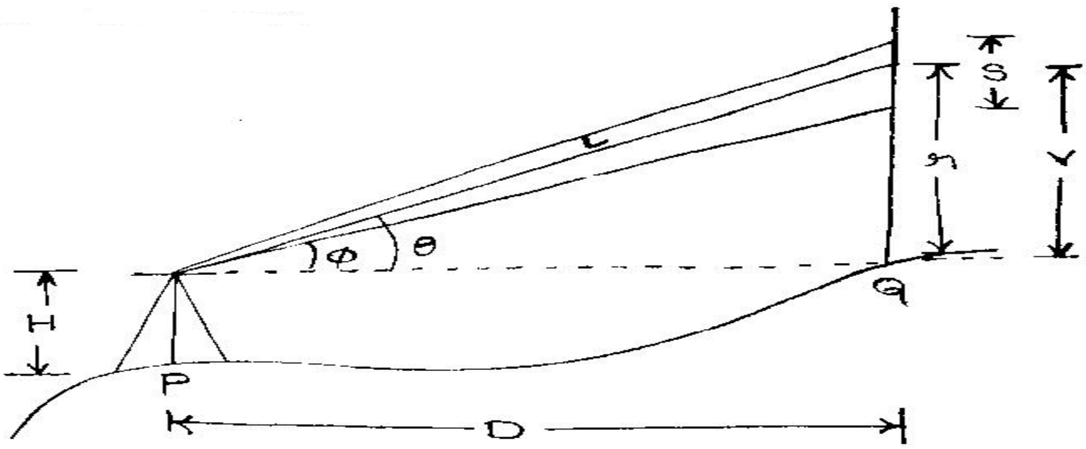
$$D = L = KS + C$$

Elevation of Q = elevation of (P + h) – r.

Case - II

Distance and elevation for inclined sight, when the staff held vertical

Angle Of Elevation



$L = KS \cos\theta + C$

$D = L \cos\theta$

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

$V = L \sin\theta$

$\quad = \dfrac{KS \sin 2\theta}{2} + C.\sin\theta$

Elevation of staff station = Elevation of P + h + V – r.

Angle of Depression

$L = KS \cos\theta + C$

$D = L \cos\theta$

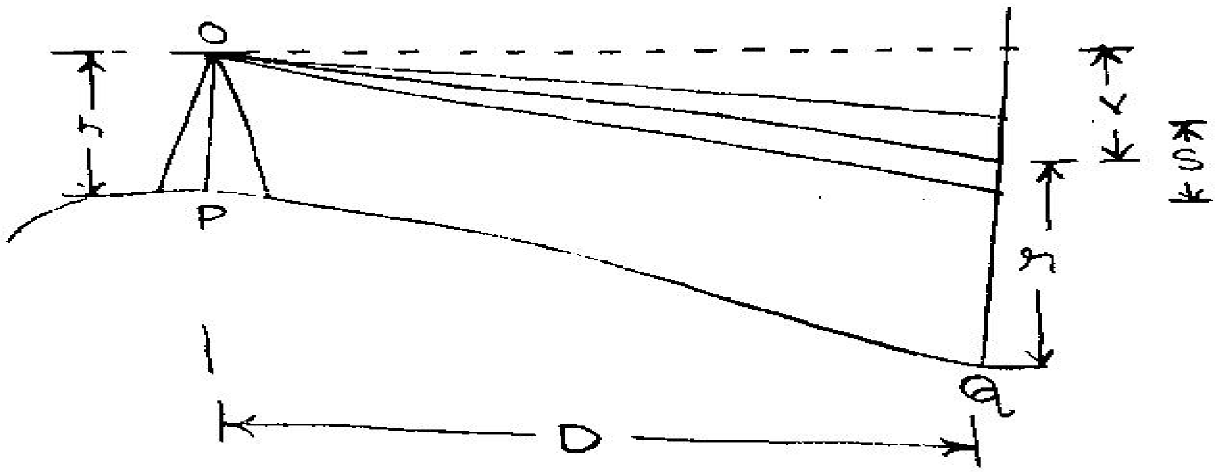
$\quad = KS \cos^2\theta + C \cos\theta$

www.FirstRanker.com

V = L Sin θ

$$= \frac{KS\ Sin\ 2\theta}{2} + C.\ Sin\ \theta$$

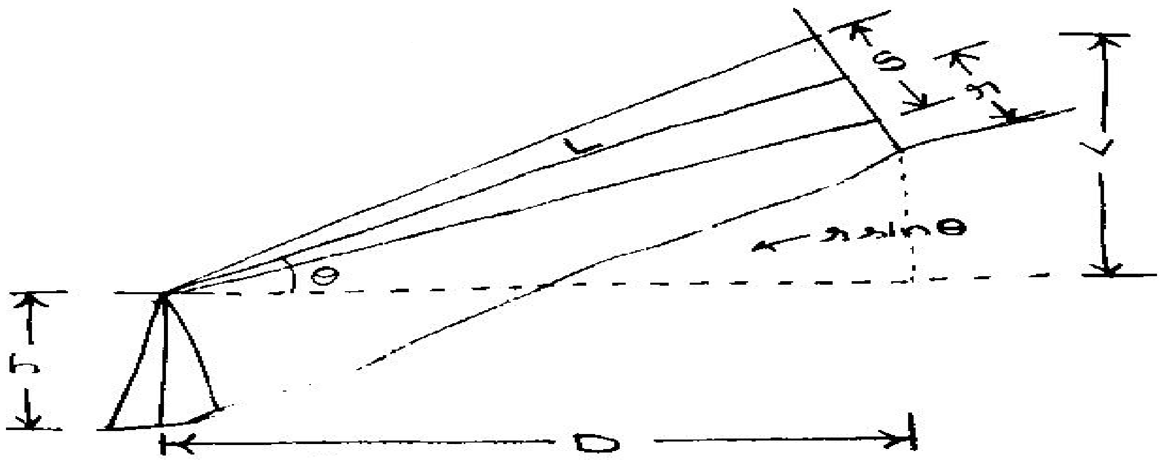
Elevation of staff station = Elevation of P + h - V – r.



Case - III

Distance and elevation for inclined sights for staff held normal.

<u>Angle of Elevation</u>



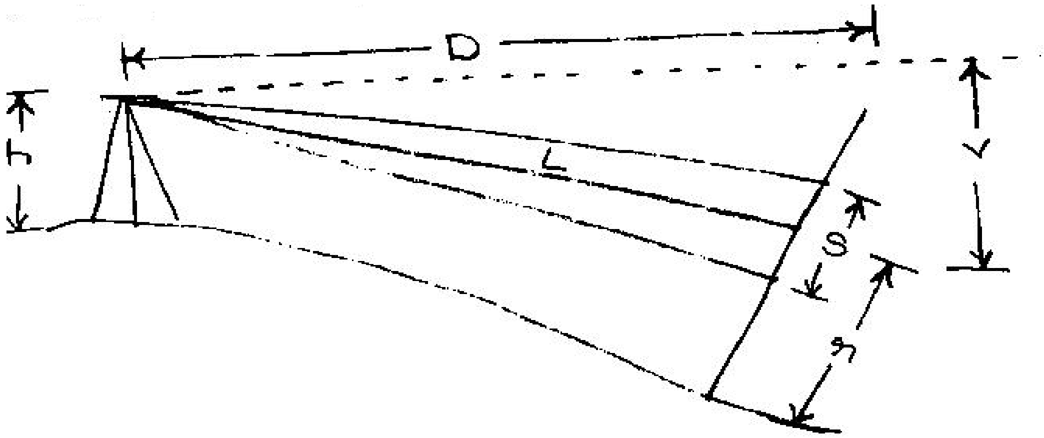
L = KS + C

D = L Cos θ + r Sin θ

V = l Sin θ

Elevation of staff station = Elevation of P + h + V – r Cos θ

Angle of Depression



L = KS + C

D = L Cos θ + r Sin θ

r = l Sin θ

Elevation of staff station = Elevation of P + h - V – r Cos θ

Where

i = interval between stadia hairs

S = Staff intercept

f = focal length of the objective

d = distance of vertical axis of instrument from optical centre ‘ O ‘

D = horizontal distance of the staff from vertical axis of the instrument.

K = (f/i) = Multiplying constant (or) stadia interval factor

L = length measured along the line of sight

C = f + d = additive constant of instrument

h = height of the instrument

r = Distance of central hair from ground level (Central hair reading)

V = Vertical intercept at ‘Q’ between line of sight and horizontal line.

P = instrumental station

Q = Staff station.

4.15. 2.2 Problems

1. Determine the distance between instrument station P and Staff station’Q’ from following data. H.I is 1.49 m vertical angle is + 4^o 30’, staff readings (staff vertical) 0.645, 0.998, 1.351 and also determine R.L of Q if that P is 200.10 m take K = 100, C = 0.

Solution

H = 1.49m, θ = 4^0 30’, S = 0.706, r = 0.998, K = 100, C = 0

$L = KS \cos\theta + C$

$= 100 \times 0.706 \cos 4^0\ 30^' + 0$

$= 70.38$

$D = L \cos\theta$

$= 70.38 \cos 4^0\ 30^'$

$= 70.16$ m

$V = l \sin\theta$

$= 70.38 \sin 4^0\ 30^'$

$= 5.52$ m

R.L of Q = R.L of P + B.S + V – r

= 200.41 + 1.49 + 5.52 – 0.998

= 206.42 m.

2. Determine the distance between station ' P ' and staff station ' Q ' from following data. R.L of instrument axis = 200.15 m Vertical angle = - 3^0 45', staff readings = 1.450 , 0.9 , 0.350 and also determine the R.L of ' Q ' take K = 100, C = 0.

Solution

$\theta = -\ 3^0\ 45'$

S = 1.450 – 0.350 = 1.1

K = 100, C = 0

$L = KS \cos\theta + C$

$= 100 \times 1.1 \cos -\ 3^0\ 45' + 0$

$= 109.76$

$D = L \cos\theta$

$= 109.76 \cos -\ 3^0\ 45'$

$= 109.52$

$V = l \sin\theta$

$= 109.76 \sin -\ 3^0\ 45'$

$= -\ 7.17$

R.L Of Q = 200.150 – 0.9 – 7.14 = 192.08 m

3. The following observations are taken, if the tachometer at station 'P' to staff at 'Q' held normal to the line of sight reading = 1.450 , 1.915 , 2.380. Angle of depression = 15^0 30'. R.L of

P = 201.45 m. Height of the horizontal axis at ' P ' is 1.315 m. Determine the horizontal distance between P and Q and R.L of Q Take K = 100, C = 0.

Solution

S = 0.93, $\theta = 15^0$ 30' R.L of P = 201.45 m, H = 1.315, K = 100 , C = 0

L = KS + C

= 100 x 0.93 + 0 = 93

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

$= 93 \cos 15^0\ 30' - 1.915 \sin 15^0\ 30'$

= 89.10 m

$V = L \sin\theta$

$= 93 \sin 15^0\ 30'$

= 24.85 m.

R.L of Q = $201.45 + 1.315 - 24.85 - r \cos 15^0\ 30'$

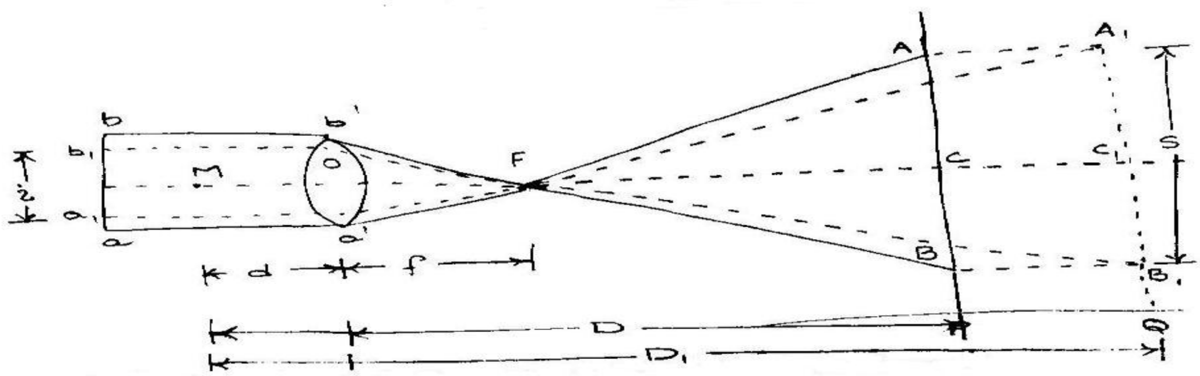
= 177.915 – 1.845

= 176.07 m

4.15.3MOVABLE HAIR METHOD (Or) SUBSTENSE METHOD

- For Horizontal Sight
- For Inclined Sight
  - (a). For Staff held vertically
  - (b). For staff normal to the line of sight

4.15.3.1 For Horizontal Sight (Vertical base Subtense Method)



Let S = AB = A_1B_1 be the staff intercept

i = ab, the stadia interval

F = exterior principal focus of the objective and

M = Centre of the instrument

From similar triangles ABF and a'b'F'

$$\frac{FC}{S} + \frac{FO}{a'b'} = \frac{f}{i}$$

$$FC = \frac{f}{i} S$$

Now, the distance D = FC + MF

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

This is the distance equation for the subtense method for a horizontal line of sight, whereas the stadia interval is variable, so $\frac{f}{i}$ Varies with the staff position. The stadia interval is measured with the help of micrometer screw.

Let m = total number of revolution of micrometer screw,
P = Pitch of micrometer screw , and
e = Index error.
Thus, i = mp
Substituting this value in the distance equation

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

$$= \frac{K}{m} S + C$$

Where K = $\frac{f}{P}$ = multiplying constant and varies from 600 to 1000.

C = (f + d) = additive Constant

If the index error e is there, the distance equation can be written as

$$D = \frac{KS}{m-e} + C$$

For Inclined Sight

(a). For staff held vertical

$$D = \frac{KS}{m-e} \cos^2\theta + C \cos\theta$$

$$V = \frac{KS}{m-e} \times \frac{\sin 2\theta}{2} + C \sin\theta$$

(b). For Staff Held Normal To The Line Of Sight

$$D = \{[\frac{KS}{m-e}] + C\}\cos\theta + C \sin\theta$$

$$V = \{[\frac{KS}{m-e}] + C\} \sin\theta$$

4.15.3.1.1 Problems

A substense theodolite was used to determine the horizontal distance of the point from instrument station. The micrometer reading of the drum respectively 3.425 and 3.930, when staff intercept was 3 m. the micrometer screw has 100 threads to 1 cm. the focal length of object glass was 225 mm, the distance of instrument axis from centre of object glass measured as 200 mm.

Solution

S = 3 m, f = 225 mm = 0.225, d = 200 mm, P = 1/100 = 0.01 cm = 0.01 x 10^{-2} m

$$K = \frac{f}{P} = \frac{0.225}{0.01 \, x \, 10-^{2}}$$

$$= \frac{22.5}{0.01}$$

$$= 2250$$

m = 3.425 + 3.930

= 7.355

C = f +d = 0.225 + 0.2 = 0.425 m

$$D = \frac{KS}{m} + C$$

$$= \frac{2250 \, x \, 3}{7.355} + 0.425$$

= 918.16 m

2. A substense theodolite whose K was 600 was used to measured the horizontal distance, when line of sight was inclined at 45^0 to the horizontal. The staff intercept on a vertical staff was 2 m, when the sum of micrometer reading on the drum was 4.85 m. if C = 0.5. what is horizontal distance.

Solution

K = 100, $\theta = 45^0$

$$D = \frac{KS}{m} \text{ Cos }^2 \theta + C \text{ Cos } \theta$$

$$= \frac{600 \, x \, 2}{4.85} \text{ Cos }^2 45^0 + 0.5 \text{ Cos } 45^0$$

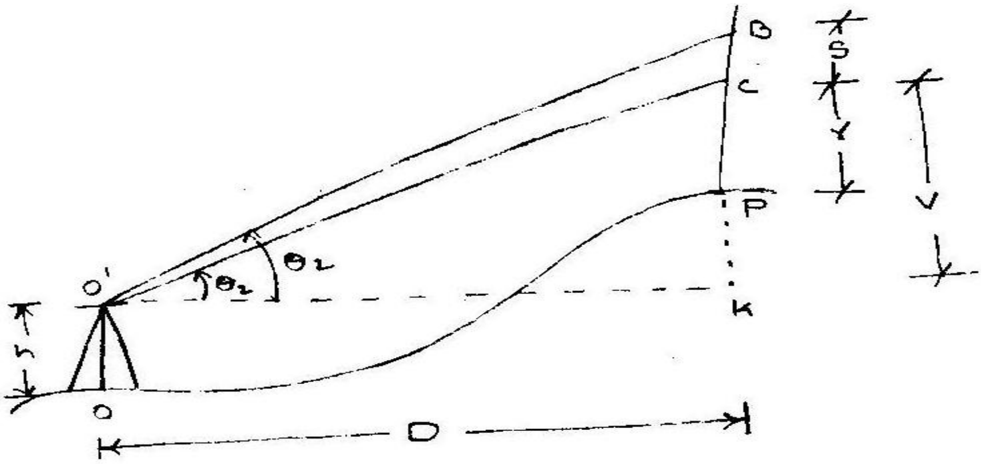
= 124.06 m.

4.16 TANGENTIAL METHOD:-

The vertical and horizontal distances are measured by means of observed vertical angles to the vanes fixed at a constant distance apart on the staff. These method is used when the theodolite is not fitted with stadia wires.

Case – I

Both angles are angle of elevation



From Triangle O’KB, V+S = D tan θ_1

From Triangle O’KC, V = D tan θ_2

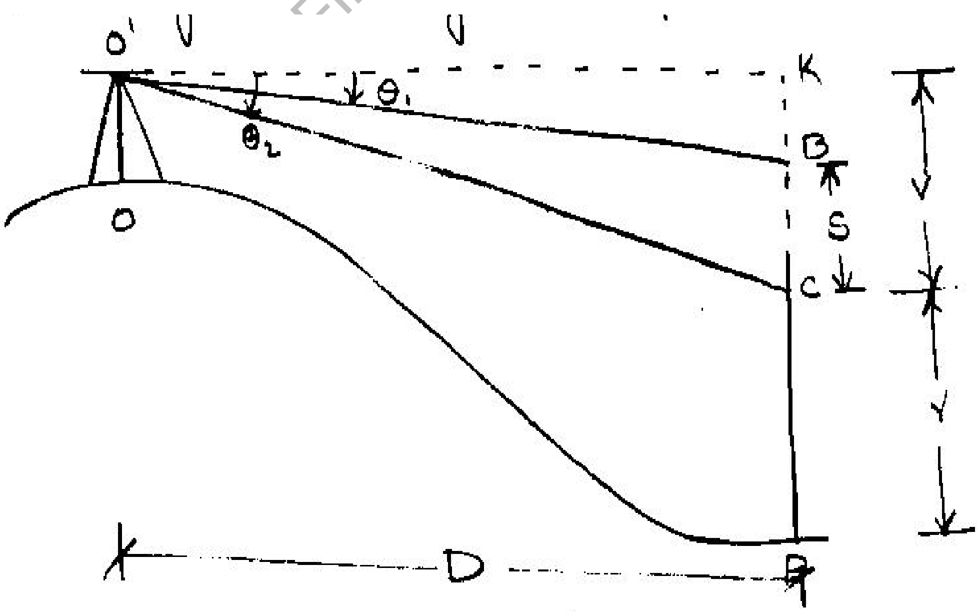
From above equations

S = D (tan θ_1 - tan θ_2)

(or) D = $\frac{S}{(\tan\ \theta 1 - \tan\ \theta 2)}$

Elevation of station P = Elevation of instrument axis + V – r.

Case – II



Both the angles are angle of depression

From Triangle O'KB, V - S = D tan θ_1

From Triangle O'KC, V = D tan θ_2

From above equations

D tan θ_1 + S = D tan θ_2

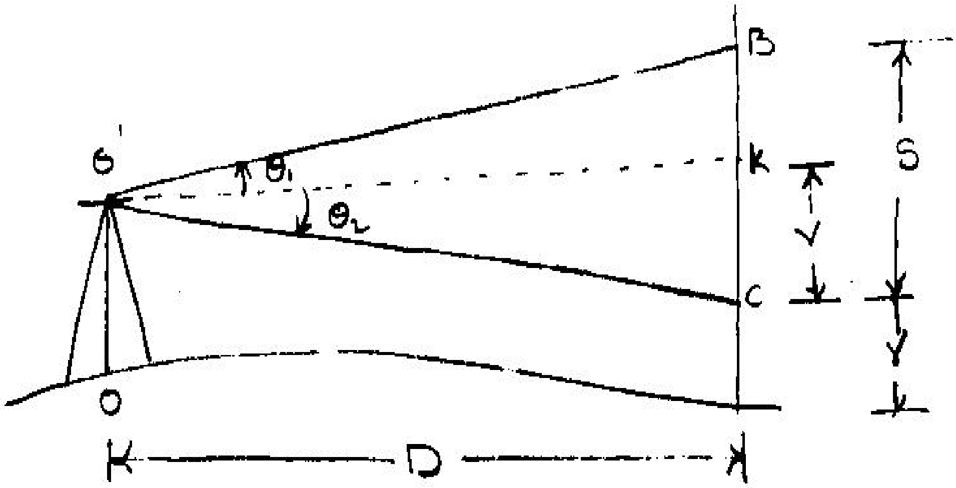
S = D (tan θ_2 - tan θ_1)

(or) D = $\dfrac{S}{(\tan\ \theta 2 - \tan\ \theta 1)}$

Elevation of station P = Elevation of station Q + h – V - r.

Case – III

One angle is angle of elevation and the other angle is angle of depression



From Triangle O'KB, S - V = D tan θ_1

From Triangle O'KC, V = D tan θ_2

From above equations

D tan θ_1 + S = D tan θ_2

S = D (tan θ_1 + tan θ_2)

(or) D = $\dfrac{S}{(\tan\ \theta 2 + \tan\ \theta 1)}$

Elevation of station P = Elevation of station O + h – V - r.

4.15.1 Problems

In the tangential method of tacheometry, two vanes were fixed at an interval of 1m on a 3m staff with the bottom vane at 1.0 m. the staff was held vertical at station A and the vertical angles measured for the two vanes were 5^0 30' and 3^0 15' respectively. Find the reduced level and horizontal distance of A, if the R.L of a B.M was 400 m (Instrumental axis).

Solution

S = 1m

$\theta_1 = 5^0\ 30'$

$\theta_2 = 3^0\ 15'$

B.M + R.L = 400m

$$D = \frac{1}{(\tan\ 5.30 + \tan\ 3.15\ )}$$

$$= 25.313\ m$$

$V = D \tan \theta_2$

$= 25.313 \tan \theta_2$

$= 1.4370$ m

R.L of A = 400 + 1.437 – 1.0 = 400.437 m

UNIT – V

CURVES

5.1 INTRODUCTION

A regular curved path followed by a railway (or) highway alignment is called a curve. A curve may be circular, parabolic or spiral and is always tangential to the two straight directions at its ends.

The curves may be further classified as:

- Simple Curves
- Compound Curves
- Reverse Curves
- Transition Curves

5.1.1 Simple Curve

A simple circular curve is a single circular arc with a constant radius connecting two straight (or) tangents are known as simple circular curves.

5.1.2 Compound Curve

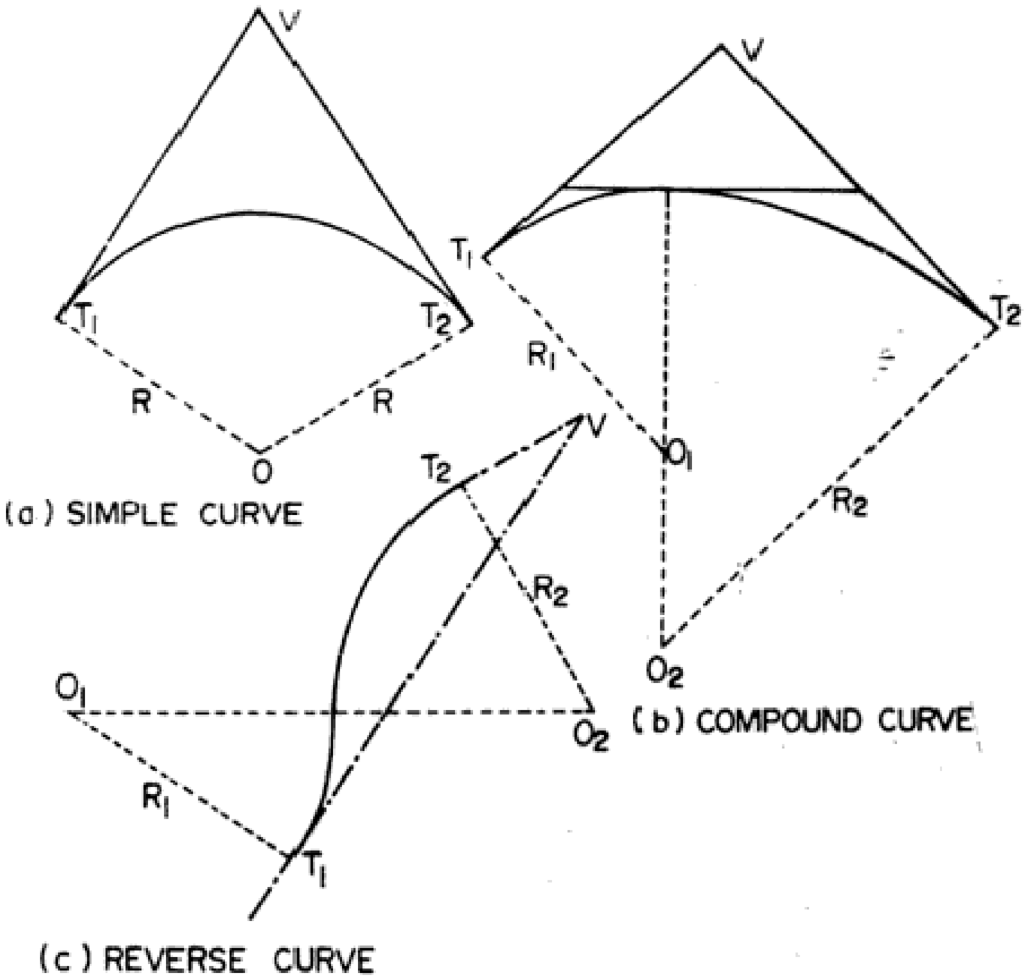
A compound curve consists of two circular arcs of different radii curving in the same direction. It can be visualized as connecting three straights, the middle one being the common tangent for both the arcs.

5.1.3 Reverse Curve

A curve which consists of two arcs of different circles of same or different radii is known as reverse curves. In such curve, centre's of the arcs are on opposite sides of curves. The two arcs are on opposite sides of curves. The two arcs turn in opposite direction with a common tangent at the junction of two arcs.

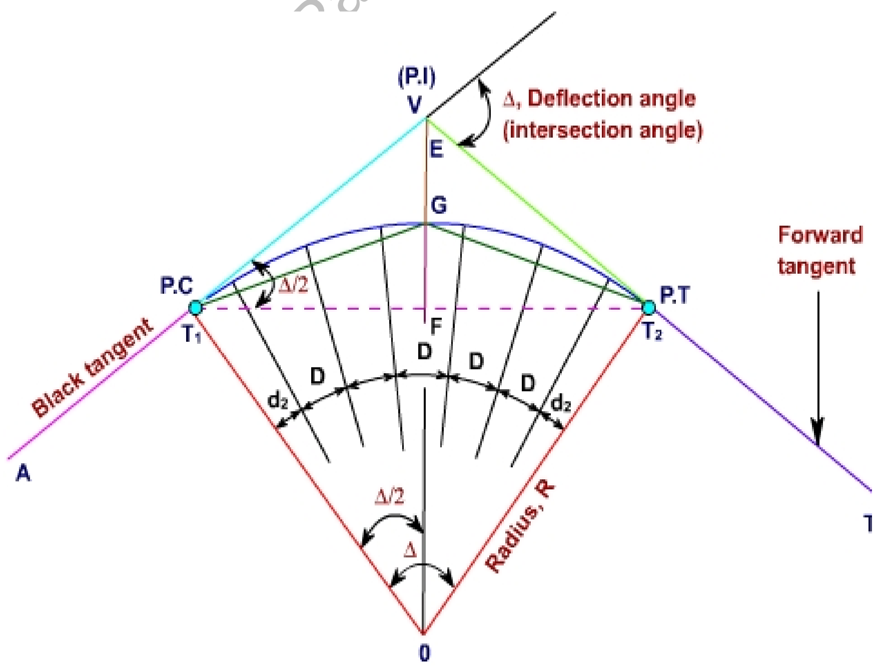
5.1.4 Transition Curve

A curve of varying radius introducing between a straight and circular curve is called a transition curve.



Different types of curves

5.2 ELEMENTS OF SIMPLE CURVE



5.2.1 Back Tangent:

The curve is set out between two straight lines which are tangent to the curve, the tangent line before beginning of the curve is called Back – tangent (or) rare tangent. The line AT_1 is the back tangent.

5.2.2 Forward Tangent:

The tangent line after the end of the curve is forward tangent. The line T_2B is the forward tangent.

5.2.3 Vertex (or) Point of intersection:

The back tangent and forward tangent when extended intersect a point known as vertex (v) or Point of intersection (P.I).

5.2.4 Intersection Angle:

The angle BB'C between the line AB produced beyond the vertex δ and line BC is known as Angle of Intersection(I).

It is the deflection angle between the back tangent and forward tangent. Sometimes it is also called the angle of deflection or angle of deviation. It is also denoted by ϕ or Δ.

5.2.5 Point of Curvature (PC):

It is the point on the back tangent at the beginning of the curve. At this point, the alignment of the route changes from a straight line to a curve. The point is also known as tangent curve(T_c) point. The point of curvature also called point of curve. Here in the diagram represented by T_1.

5.2.6 Point of Tangency(PT):

It is the forward tangent point at the end of the curve at this point alignment of the route changes from a curve to a straight line. The poing is also known as curve tangent.

5.2.7 Tangent Distance(T):

It is the distance between the point of curvature to the point of intersection. It is also equal to the distance between the point of tangency to point of intersection.

5.2.8 External distance(E):

It is the distance between the point of intersection and middle point 'c' of the curve.

5.2.9 Long chord:

It is the chord of circular curve. It joins point of curvature and point of tangency. The length of the long chord represented bu 'L'.

5.2.10 Mid ordinate:

It is the distance between middle point 'c' of the curve and middle point 'D' of long chord.

5.2.11 Length of the Curve:

It is the length of the curve between point of curvature and point of tangency. It is represented by 'l'.

5.2.12 Right hand Curve:

It is the curve which delfects to the right of progress of the route. The curve shown in the fig is a right hand curve as route is progressing from A→B

5.2.13 Left hand Curve:

It is the curve which deflects to the left off direction of progressive of route. The direction of progress of route had been from B → A. The curve should have been a left hand curve.

5.2.14 Degree of the Curve:

It may be defined either with respect to a fixed length of an arc of the curve or with respect to a fixed length of a normal chord of the curve.

5.2.15 Fixed length of an arc:

The degree of curve may be defined as the central angle of curve i.e., subtended by arc or 30m or 100 feet. This definition is generally adopted for railway curves.

5.2.16 Fixed Length of a Chord:

It may be defined as the fixed central angle of the curve i.e., subtended by a chord of 30m or 100 feet. This definition is generally adopted for road curves.

5.3 RELATION BETWEEN THE RADIUS (R) AND DEGREE OF THE CURVE:

Let D be the angle subtended by an arc of 30m length of a circle whose radius is 'R'. Total circumference of the circle $= 2\pi R$.

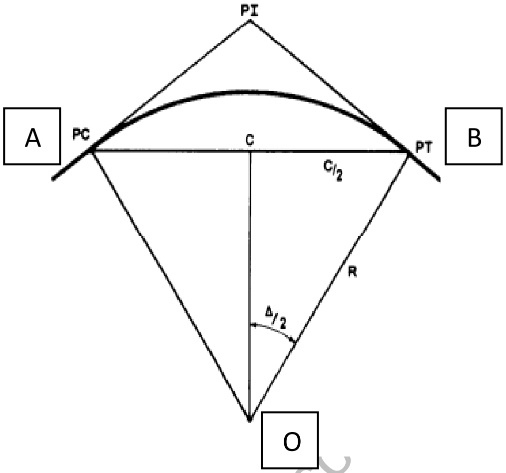
$$D^o = \frac{360}{2\pi R} \text{ x } 30 = \frac{10800}{2\pi R} \text{ degrees}$$

OR

$$D = \frac{1718.9}{R}$$

Where R = Radius of curve in m. D = Degree of Curve in degrees.

5.3.1Based on fixed length of Chord:



Chord Distance

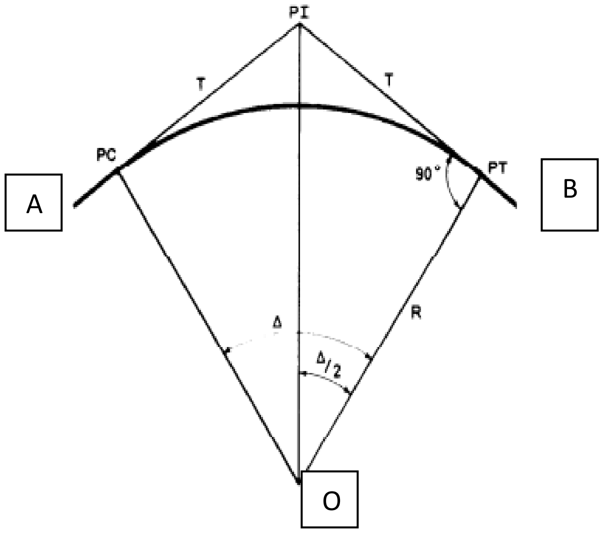
From Triangle AOB

$$\text{Sin } \frac{D}{2} = \frac{Cd/2}{R} \quad \text{or}$$

$$R = \frac{Cd/2}{Sin\ (\frac{D}{2})}$$

$$R = \frac{15}{\left(\frac{D}{2}\right) X \frac{\pi}{180}} = \frac{15\ X\ 360}{\pi D} = \frac{1718.9}{D}$$

5.3.2 Based on fixed length of an Arc:



From Triangle AOB

$$\frac{as}{2\pi R} \quad \frac{D}{360} = R = \frac{360}{D} \quad \frac{2\pi R}{as}$$

a_s = Standard arc length

$$R = \frac{360}{D} \quad \frac{2\pi R}{as} = \frac{1718.9}{D} \text{ m.}$$

5.3.3Problem

Compute the radius of a simple circular curve of 6^o used in the alignment of a highway taking unit chord/arc length as 30 m.

Solution

Length of unit chord/ arc = 30 m

Degree of curve D = 6^o

$$R = \frac{1718.9}{D}$$

$$= \frac{1718.9}{6}$$

= 286.48 m.

5.3.4 Calculation of Various Elements of a Curve

- Tangent Length (T)

  $$T = R \tan \phi/2$$

- Length of curve (l)

  $$L = T_1T_2 = R\phi \text{ (}\phi\text{ in Radians)}$$

  $$L = \frac{\pi R\phi}{180}, \ \phi \text{ in degrees}$$

- Length of Long Chord

  $$l = T_1T_2 = 2R\sin \phi/2$$

- Apex distance (C_v)

  $$Cv = OV - CO = R \sec \phi/2 - R$$

  $$= R (\sec \phi/2 - 1)$$

- Mid Ordinate

  $$CD = OC - OD = R - R \cos \phi/2$$

  $$= R (1 - \cos \phi/2)$$

- Chainage T_1 = chainage of V – J

  $$T_2 = \text{Chainage of } T_1 + \frac{\pi R\phi}{180}$$

5.3.4.1 Problems

Two straight lines intersect at chainage 1150.5 and angle of intersection is 60^o. If radius of curve is 500m. Determine (a) Tangent Distance (b) Length of Curve (c) Chainage of PC and tangency point CT_2 (d) Length of long chord (e) Degree of Curve (f) Apex distance (g) Mid Ordinate.

Solution

(a). $T = R \tan \phi/2$

$= 500 \tan 60/2$

$= 288.67$

(b). $L = \frac{\pi R\phi}{180}$

$= \frac{\pi x \ 500 \ x \ 60}{180}$

= 523.60 m

(c). Chainage of T_1 = 1150.5 – 288.67

= 861.83

Chainage of T_2 = T_1 + $\frac{\pi R \emptyset}{180}$

= 861.83 + 523.60

= 1385.43 m

(d). Length of Long Chord = 2R Sin ϕ/2

= 2 x 500 Sin 30

= 500

(e). Apex distance = 500 (Sec 30 – 1)

= 77.35 m

(f). Degree of Curve D = $\frac{1718.9}{D}$

= $\frac{1718.9}{500}$

= 3^o 26' 16"

(g). Mid Ordinate = R(1- Cos ϕ/2)

= 66.99 m.

5.4 METHODS OF SETTING SIMPLE CURVES

1. Methods using chain and tape only (linear methods)
2. Instrumental methods using angle measuring instruments (Angular methods)

5.4.1 Linear Methods

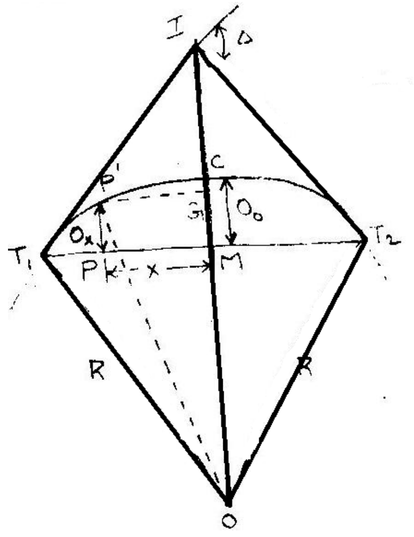
- Offsets from the long chord.
- Perpendicular offsets from the tangents.
- Radial offsets from the tangent.
- Successive bisection of arcs.
- Offsets from the chord produced.

5.4.2 Angular Methods

- Rankine's method of deflection angle [one – Theodolite Method].
- Two – theodolite method
- Tacheometric method

5.4.1.1 Offsets from long Chord:-

Let it be required to lay a curve T_1CT_2 between the two intersecting straights T_1I and T_2I. R is the radius of the curve. O_o the mid – ordinate and O_x the offset at a point P at a distance x from the mid point (M) of the long chord.



From triangle OMT_1

$$OM = \sqrt{(OT_1)^2 - (MT_1)^2}$$

$$= \sqrt{((R^2 - (L/2)^2)}$$

Now, CM = OC – OM

(or) $O_o = R - OM$

(or) $O_o = R - \sqrt{((R^2 - (L/2)^2)}$

In triangle OP'G

$$OG = \sqrt{R^2 - X^2}$$

and $OM = R - O_o$

The required offset

$$PP^1 = OG - OM$$

Hence, $PP^1 = \sqrt{R^2 - X^2} - (R - O_o)$

$$O_x = \sqrt{R^2 - X^2} - (R - O_o)$$

$= R\,(1 - x^2/R^2)^{1/2} - R + O_0$

$= R(1 - x^2/(2R^2) + \text{-------------}) - R + O_0$

$= O_o - [x^2/2R].$

By assigning different values to x, the corresponding values of offsets O_x can be calculated. The calculated offsets can be laid from the long chord and points can be established in the field which when joined produce the required curve.

5.4.1.1.1 Problem

A simple circular curve has a radius of 300m and a long chord of length 120 m. Calculate offsets to the curve from the long chord at 10 m intervals.

Solution

R = 300 m

L = 120 m

Interval of Offsets = 10 m

L = 2R Sin ϕ/2

120 = 2 x 300 Sin ϕ/2

Sin ϕ/2 = 120/600 = 0.2

$\Phi = 2\,(\sin^{-1} 0.2)$

$= 23^o\,04'\,26.11''$

$= 23^o\,04'\,26''$

(1). Curve Length = $\dfrac{\pi R \emptyset}{180} = \dfrac{\pi\ x\ 300\ x\ 23.04'\,26''}{180}$

= 120.815m

(2). The long chord is divided into two equal halves each half = L/2

= 120/2 = 60 m.

(3). Mid – Ordinate

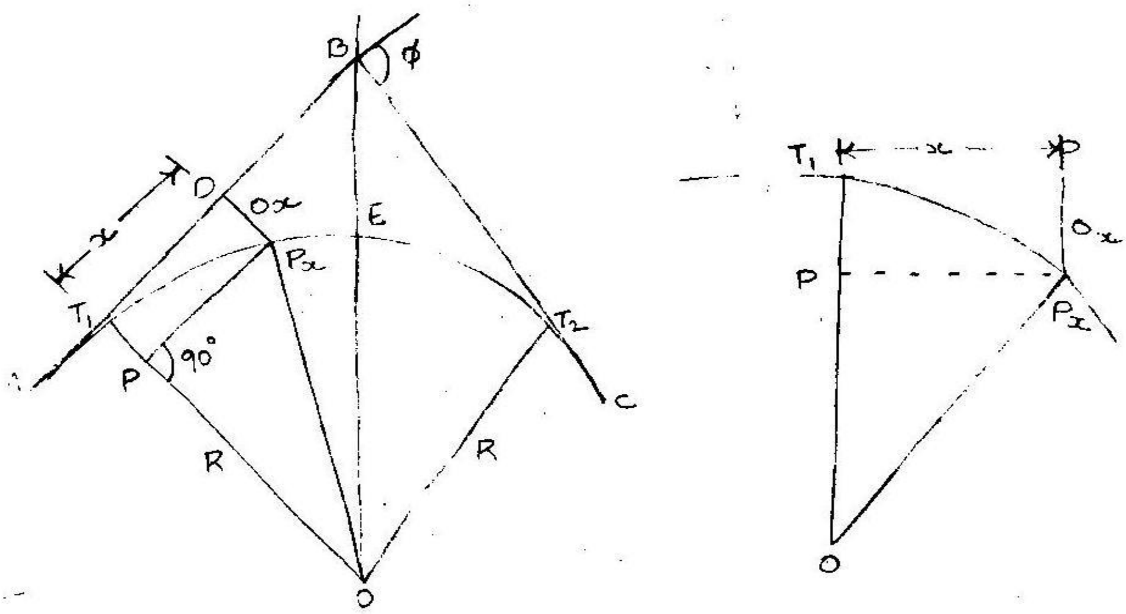
$$O_o = R - \sqrt{(R^2 - (L/2)^2)}$$
$$= 300 - \sqrt{(300^2 - (120/2)^2}$$
$$= 6.061 \text{ m}$$

(4). The ordinates are calculated at 10 m intervals starting from the centre towards T_1 for the left half.

$$O_{10} = \sqrt{R^2 - X^2} - (R - O_o)$$
$$= \sqrt{300^2 - 10^2} - (300 - 6.061)$$
$$= 5.894 \text{ m}$$
$$O_{20} = \sqrt{300^2 - 20^2} - (300 - 6.061) = 5.394 \text{ m}$$
$$O_{30} = \sqrt{300^2 - 30^2} - (300 - 6.061) = 4.557 \text{ m}$$
$$O_{40} = \sqrt{300^2 - 40^2} - (300 - 6.061) = 3.382 \text{ m}$$
$$O_{50} = \sqrt{300^2 - 50^2} - (300 - 6.061) = 1.865 \text{ m}$$
$$O6_0 = \sqrt{300^2 - 60^2} - (300 - 6.061) = 0.00 \text{ m}$$

The ordinates for the right half are similar to those for the left half.

5.4.1.2 Perpendicular offsets from Tangents;



Let AB and BC are two tangents meeting at a point B. the tangent Length is calculated and the tangent point T_1 and T_2 are marked on the ground.

A point D is taken on rear tangent AB at a distance x from T_1. Let O_x is the perpendicular offset at D. the line PP_x is drawn parallel to T_1D.

Now from Triangle OPP_x

$$OP_x^2 = OP^2 + P_xP^2$$

$OP_x = R$, $PP_x = T_1D = x$

And $OP = OT_1 – T_1P = R - O_x$

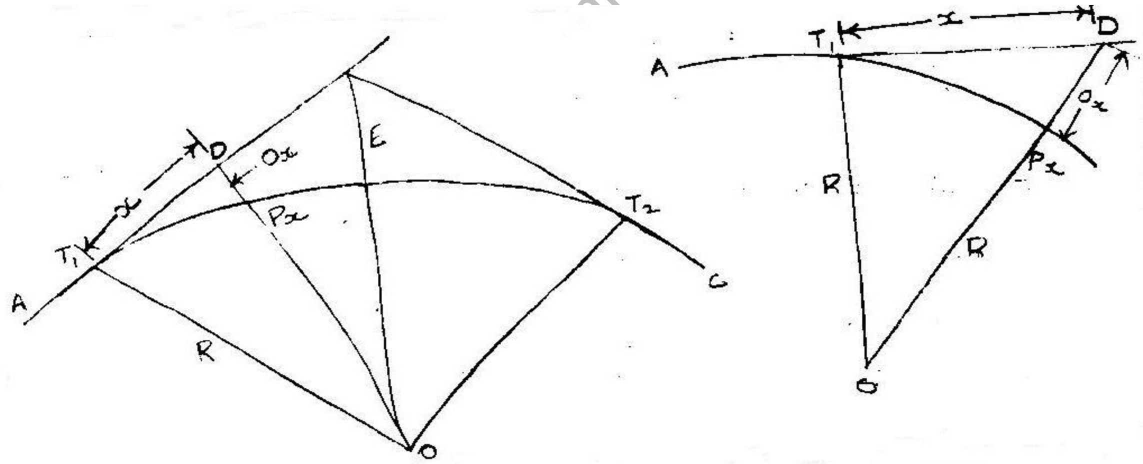
Therefore $R^2 = (R - O_x)^2 + x^2$

$$(R - O_x) = \sqrt{(R^2 – x^2)}$$

$$O_x = R - \sqrt{(R^2 – x^2)}$$

Assigning different values to x, the corresponding values of offsets Ox can be calculated. These calculated offsets can be laid from the tangent at known distance x and the points can be established in the field which when joined produce the required curve.

5.4.1.3 Radial Offsets from tangent:-



Ox is the radial offset P_xD at any distance x along the tangent from T_1.

From Triangle OT_1D

$$OD^2 = OT_1^2 + T_1D^2$$

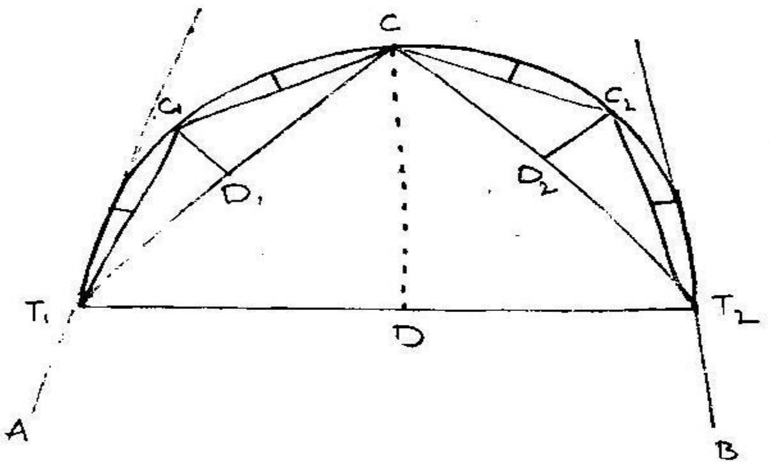
$T_1D = x$, $OT_1 = R$ and $OD = R + O_x$

$(R + O_x)^2 = R^2 + x^2$

$R + O_x = \sqrt{R^2 + x^2}$

$O_x = \sqrt{R^2 + x^2} - R$

5.4.1.4 Successive bisection of arc:-



1. Let T_1 and T_2 be the tangent points
2. The long chord T_1T_2 is bisected at D.
3. Mid- Ordinate is equal to R (1 - Cos∆/2).
4. Thus point C is established.
5. T_1C and T_2C are joined
6. T_1C and T_2C are bisected at D_1 and D_2.
7. Perpendicular offsets D_1C and D_2C each will be equal to R (1 - Cos∆/4).
8. These offsets are set out giving points C_1 and C_2 on the curve.
9. By the successive bisection of the chords T_1C and C_1 C, CC_2 and C_2T_2, more points may be obtained which when joined produce the required curve.

5.4.1.5 Offsets from the chords Produced:-

1. Let AB be the rear tangent and T_1 be the first tangent point of the curve of radius R.
2. Let b_1 be the length of the initial sub-chord and b_n be the final sub chord. Let the intermediate full chords be b_2, b_3, ----- $b_{n-1.}$
3. Let P_1, P_2, P_3 -------- be the points to be set out on the curve.
4. Let the tangent at P_1 meet the rear tangent at D.

$T_1P_1{}^1 = T_1P_1 = b_1$

Since b_1 is small, the offset O_1 at $P_1 = P_1P_1{}^1 = O_1 = b_1{}^2/2R$

5. Produce the first chord T_1P_1 to $P_2{}^1$ such that

   $P_1P_2{}^1 = P_1P_2{}^1 = b_2$

Let tangent DP_1 be produced to E such that $P_1E = b_2$. Consider the approximate triangles $T_1P_1P_2{}^1$ which are both isosceles and the angle at P_1 are equal.they can be seen as similar.

Therefore $P_2{}^1E = P_1P_2{}^1 \text{ x } = \frac{P1P2}{T1P1} = = \frac{b1^2}{2R} \text{ x } \frac{b2}{b1} = \frac{b1b2}{2R}$

$$EP_2 = \frac{b2^2}{2R} \text{ [offset from tangent]}$$

Therefore $O_2 = P_2P_2{}^1 = P_2E + EP_2$

$$= \frac{b1b2}{2R} + \frac{b2^2}{2R} = \frac{b2}{2R} (b_1 + b_2).$$

Next produce chord b_2 to $P_3{}^1$, such that $P_2P_3' = b_3$ and tangent length $P_2f = b_3$. Similarly.

$$O_3 = P_3F + FP_3$$

$$= \frac{b2b3}{2R} + \frac{b3^2}{2R}$$

$$= \frac{b3}{2R} (b_2 + b_3).$$

$$= \frac{b3^2}{2R} \text{ [Since } b_2 = b_3]$$

For the last chord, the offset on will be given by

$$O_n = \frac{(bn - 1 + bn)bn}{2R}$$

5.4.1.6 Problem:

It is required to set out a curve of radius 100m with pegs at approximately 10m centres. The deflection angle is 60^0. Draw up the data necessary for pegging out the curve by each of the following methods.

(a). Offsets from long chord. (b). Chord bisection (c). Offsets from tangent.

Solution

Offsets from long chord:

$$O_o = R - \sqrt{(R^2 - (L/2)^2)}$$

$$= 100 - \sqrt{(100^2 - (L/2)^2}$$

$$= 13.40 \text{ m}$$

Length of long chord, L = 2R sin ∆/2

$$= 2 \times 100 \times \sin 60/2$$

$$= 100 \text{ m.}$$

Let peg interval: x = 10m, so that

$$O_x = \sqrt{R^2 - X^2} - (R - O_o)$$

$$O_{10} = \sqrt{100^2 - 20^2} - (100 - 13.40) = 12.9 \text{ m}$$

$$O_{20} = \sqrt{100^2 - 30^2} - (100 - 13.40) = 11.40 \text{ m}$$

$$O_{30} = \sqrt{100^2 - 40^2} - (100 - 13.40) = 8.90 \text{ m}$$

$$O_{40} = \sqrt{100^2 - 50^2} - (100 - 13.40) = 5.40 \text{ m}$$

$$O_{50} = \sqrt{100^2 - 60^2} - (100 - 13.40) = 0.00 \text{ m}$$

Chord bisection

$$\text{Length of curve} = R\Delta \frac{\pi}{180}$$

$$= 100 \times 60 \times \frac{\pi}{180}$$

$$= 104.72 \text{ m}$$

Thus 11 pegs are required at 10.47 m centres.

Tangent length = R tan ∆/2

$$= 100 \tan 60/2$$

= 57.73m

Therefore x (Mid ordinate) = R (1- cos∆/2)

= 100 (1- cos (60/2))

= 13.40m

Y = R (1- cos ∆/4)

= 100 (1 – Cos 60/4)

= 3.41m.

Z = R(1- cos ∆/8)

= 100(1 – cos 7.30')

= 0.86m

Offsets from Tangent

Tangent length = R Sin ∆/2 (perpendicular offset from tangen)

= 100 Sin 30

= 50.0m

Set pegs at 10m interval

Offset from tangent = R - $\sqrt{(R^2 - x^2)}$

X = 10 m , 20 m , 30 m , 40 m , 50 m.

$O_{10} = \sqrt{100^2 - 10^2} - 100 = 0.5$ m

$O_{20} = \sqrt{100^2 - 20^2} - 100 = 2.02$ m

$O_{30} = \sqrt{100^2 - 30^2} - 100 = 4.60$ m

$O_{40} = \sqrt{100^2 - 40^2} - 100 = 8.35$ m

$O_{50} = \sqrt{100^2 - 50^2} - 100 = 13.40$ m

5.4.2.1 Rankine's Method of deflection angle (One theodolite method) :-

Procedure:

1. Set the theodolite at the point of curve (T_1). With both plates clamped to zero, direct the theodolite to bisect the point of intersection (V). the line of sight is thus in the direction of the rear tangent.
2. Release the vernier plate and set angle Δ_1 on the vernier. The line of sight is thus directed along chord T_1A.

$$\Delta_1 = \delta_1 = \frac{cD}{40}$$

3. With the zero end of the tape pointed at T_1 and an arrow held at a distance T_1A = c along it, swing the tape around T_1 till the arrow is bisected by the cross-hairs. Thus, the first point A is fixed.
4. Set the second deflection angle Δ_2 on the vernier so that the line of sight is directed along T_1B.

$$\Delta_2 = \delta_2 = \frac{cD}{40} + \frac{1}{2}\text{ D}$$

5. With the zero end of the tape pinned at A, and an arrow held at distance AB = C along it, swing the tape around A till the arrow is bisected by the cross-hairs,thus fixing the point B.
6. Repeat steps (4) and (5) till the last point T_2 is reached.

5.4.2.2 Two Theodolite Method

Procedure:

1. Set up one transit at P.C (T_1) and the other at P.T (T_2).
2. Clamp both the plates of each transit to zero reading.
3. With the zero reading, direct the line of sight of the transit at T_1 towards V. Similarly, direct the line of sight of the order transit at T_2 towards T_1 when the reading is zero. Both the transit are thus correctly oriented.
4. Set the reading of each of the transit to the deflection angle for the first point A. the line of sight of both the theodolites are thus directed towards A along T_1A and respectively.
5. Move a ranging rod or an arrow in such a way that it is bisected simultaneously by cross-hairs of both the instruments. Thus, point A is fixed.
6. To fix the second point B, set reading Δ_2 on both the instruments and bisect the ranging rod.
7. Repeat steps (4) and (5) for location of all the points.

5.4.2.3 Tacheometric Method

Procedure:

1. Set the tacheometer at T_1 and sight the point of intersection (V) when the reading is zero. The line of sight is thus oriented along the rear tangent.
2. Set the angle Δ_1 on the vernier, thus directing the line of sight along T_1A.

3. Direct a staffman to move in the direction T_1A till the calculated staff intercept s_1 is obtained. The staff is generally held vertical. Thus, the first point A is fixed.
4. Set the angle Δ_2 now, thus directing the line of sight along T_1B. Move the staff backward or forward along T_1B until the staff intercept s_2 is obtained, thus fixing the point B.
5. Fix other points similarly.

5.5 COMPOUND CURVE

A compound curve is a combination of two or more simple circular curves with different radii. The two centred compound curve has two circular arcs of different radii that deviate in the same direction and join at a common tangent point also known as point of compound curvature.

5.5.1Elements of compound curves:-

AI and BI are two straights intersecting at I(P.I). T_1DT_2 is the compound curve consisting of two arcs of radii R_1 and R_2 and D is the point of compound curvature. MN is the common tangent making deflection angles Δ_1 and Δ_2 at M and N.

So that $\Delta = \Delta_1 + \Delta_2$

Froem

The triangle IMN

$$\frac{IM}{Sin\ \Delta1} = \frac{IN}{Sin\ \Delta1} = \frac{MN}{Sin\,(180-(\Delta1+\Delta2))}$$

Hence IM = $\dfrac{MN \sin \Delta2}{Sin\,(\Delta1+\Delta2))}$

(or) IM = $\dfrac{MN \sin \Delta2}{Sin\,(\Delta)}$,

Also, IN = $\dfrac{MN \sin \Delta1}{Sin\,(\Delta1+\Delta2))}$

IN = $\dfrac{MN \sin \Delta1}{Sin\,(\Delta)}$

Common tangent MN

$MD = R_1 \tan \Delta_1/2$

$DN = R_2 \tan \Delta_2/2$

Now,

$$MN = MD + DN$$

$$\text{(or) } MN = R_1 \tan \Delta_1/2 + R_2 \tan \Delta_2/2$$

Length of main tangents IT_1 and IT_2

$$IT_1 = T_1M + MI$$

$$= R_1 \text{ Tan } \Delta_1/2 + \frac{MN \sin \Delta 2}{Sin\,(\Delta)}$$

$$IT_2 = T_2N + NI$$

$$= R_2 \tan \Delta_2/2 + \frac{MN \sin \Delta 1}{Sin\,(\Delta)}$$

5.5.2 Problems

A compound Curve, consisting of two simple circular curves of radii 350 m and 500 m, is to be laid out between two straights. The angles of intersection between the tangents and the two straights are 25° and 55°. Calculate the various elements of the compound curve.

Solution

Length of the common tangent

$$PQ = PD + DQ$$

$$= R_1 \tan \Delta_1/2 + R_2 \tan \Delta_2/2$$

$$= 350 \tan 55/2 + 500 \tan 25/2$$

$$= 293.04 \text{ m}$$

$$\Delta = 55 + 25 = 80^o$$

Triangle IPQ by the Sine rule

$$\frac{IP}{Sin\ 25} = \frac{IQ}{Sin\ 55} = \frac{IQ}{Sin\ 80}$$

$$IP = \frac{293.04 \sin 25}{Sin\ 80} = 125.754 \text{ m}$$

$$\text{And } IQ = \frac{293.04 \sin 55}{Sin\ 80} = 243.74 \text{ m}$$

Length of tangent $IT_1 = IP + PT_1$

= 125.75 + 350 tan 55/2

= 307.95 m

Length of tangent $IT_2 = IQ + QT_2$

= 243.74 + 500 tan 25/2 = 354.59 m.

2. The centre – line of a new railway is to be set out along a valley. The frist straight AI bears 75°, while the connecting straight IB bears 120°. Due to site conditions it has been decided to join the straights with a compound curve.

Solution

$\Delta = 120 - 75 = 45^o$

$\Delta_1 = 25^o$

$\Delta_2 = 20^o$

Tangent length = $MD = T_1M = R_1 \tan \Delta_1/2$

$= 500 \tan 12^o\ 30'$

$= 110.8$ m

Triangle MIN $<NIM = 180^o - \Delta = 180^o - 45^o = 135^o$

Length $T_1I - T_1M = 300 - 110.8 = 189.2$ m

Now, $MN = \dfrac{IM\ Sin\ NIM}{Sin\ (\Delta 2)} = \dfrac{189.2\ Sin\ 135}{Sin\ 20} = 391.2$ m

$IN = \dfrac{IM\ Sin\ \Delta 1}{Sin\ (\Delta 2)} = \dfrac{189.2\ Sin\ 25}{Sin\ 20} = 233.8$ m

$T_2N = ND = MN - MD = MN - T_1M = 391.2 - 110.8 = 280.4$ m

$T_2N = R_2 \tan \Delta_2/2$

$280.4 = R \tan 10$

$R_2 = 1590$ m

$IT_2 = IN + NT_2$

$= 233.8 + 280.4 = 514.2$ m

5.6 INTRODUCTION OF GEODETIC SURVEYING :

The art of Surveying the earth surface considering its shape and size is called Geodetic Surveying . Geodetic Surveying is suitable for finding out the area of any region on the earth surface, the length and directions of the border lines, contour lines and location of basic points.
It is assumed that the shape of earth is spheroid, measurement of distances are taken along curved surfaces and not along straight lines. Therefore for Geodetic Surveying, earth's both the diameters are considered. The latitudes and longitudes are determined considering the spheroidcal shape of the earth. The points which are used to find out the shape, size and coordinates of the earth surface is called Geodetic Datum in Geodetic Surveying. Hundreds of such points are marked for carrying out Geodetic Survey .

Geodetic Surveying: Finding exact location of an object

- **Triangulation:** As the name indicates, a triangle is incorporated to find out the location of the point in respect of latitude and longitude. The measurements of the sides of the triangle and the angles in the triangle which is drawn with respect to the particular pointis found out. With the help of these measurements, longitude and latitude of the triangulation point is calculated.
- **Bench mark:** In Geodetic Surveying , benchmarks are also used for determining the height or elevation of a point. The surveyor gives a permanent mark in the area which shows the benchmark for ages.
- **GPS based control station**: The GPS or Global Positioning System based control station capture the radio signal given by the satellite. This signal is then processed and analyzed to find out the latitudes and logitudes of the given point.

5.6.1Main instrument for Geodetic Surveying :

Theodolite: It is the basic surveying unit used for Geodetic Surveying. Theodolite consists of a telescope which is placed on a swivel and it can be rotated both horizontally and vertically.
Triangulation point are determined by the theodolite in Geodetic Surveying. Two circles-one vertical and another horizontal, are used to read out the readings. But in the modern theodolite the reading is done electronically. Geodetic Surveying can be done by geographers, engineers and surveyors specialised in related disciplines.

5.6.2Use of Geodetic Surveying :

- Engineering purposes: The engineers uses Geodetic Surveying for finding out the exact location of the concerned point or area. Latitudes and longitudes are needed for any engineering constructions.

- Construction purposes: The builders used Geodetic Surveying for finding out the direction of the buildings or their exact location for vaastu shastra.
- Land Surveying and assessment: The vertical elevation and the horzontal attributes , the latitude and longitudes of the area surveyed are found out through Geodetic Surveying .

Geodetic Surveying is thus considered as an important method of Surveying.

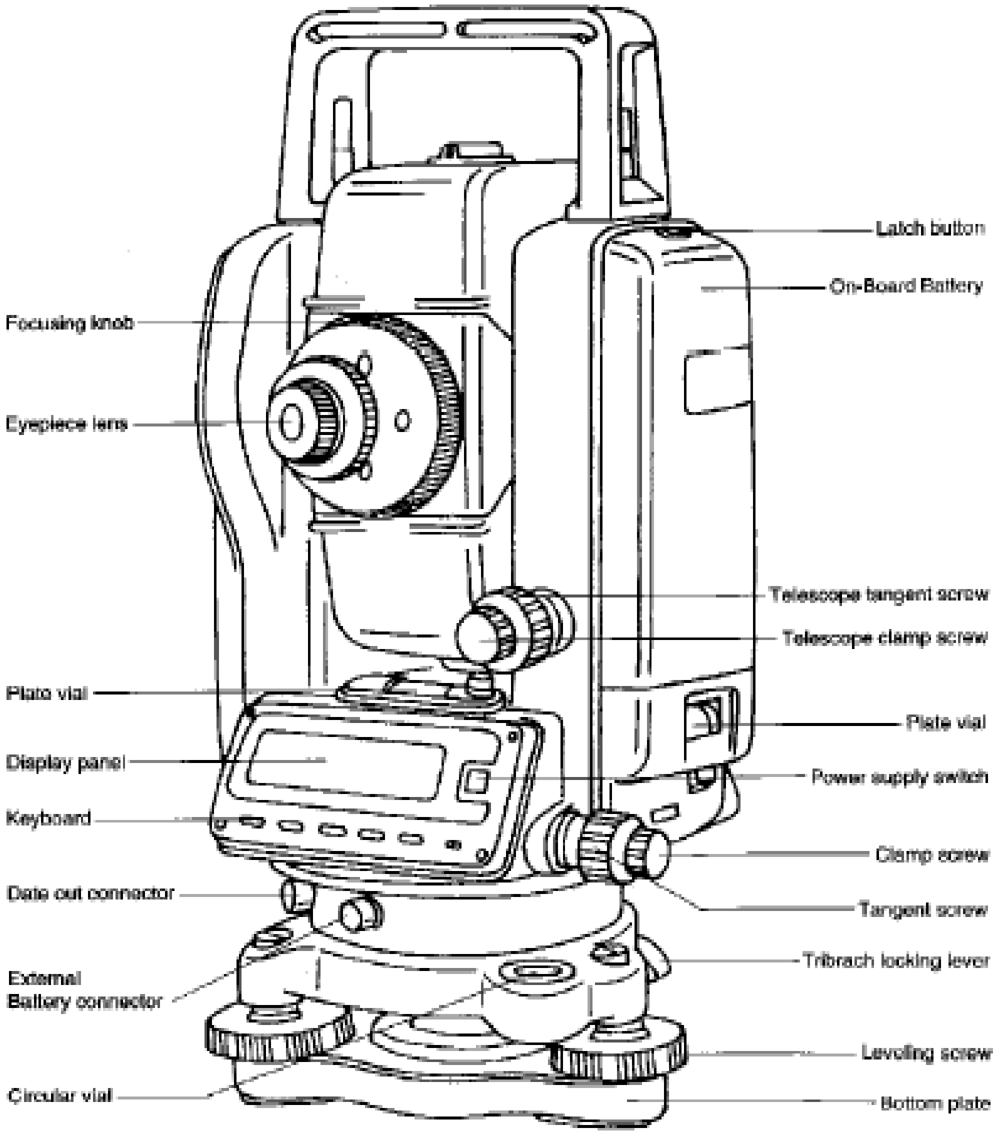
5.6.2.1 Temporal change

In geodesy, temporal change can be studied by a variety of techniques. Points on the Earth's surface change their location due to a variety of mechanisms:

- Continental plate motion, plate tectonics
- Episodic motion of tectonic origin, esp. close to fault lines
- Periodic effects due to Earth tides
- Postglacial land uplift due to isostatic adjustment
- Various anthropogenic movements due to, for instance, petroleum or water extraction or reservoir construction

5.7 TOTAL STATION:

A total station is an electronic/optical instrument used in modern surveying. The total station is an electronic theodolite (transit) integrated with an electronic distancemeter (EDM) to read slope distances from the instrument to a particular point.



5.7.1 Applications of Total station

A total station is used to record the absolute location of the tunnel walls (stopes), ceilings (backs), and floors as the drifts of an underground mine are driven. The recorded data is then downloaded into a CAD program, and compared to the designed layout of the tunnel.

5.7.1.1. Coordinate Measurement

Coordinates of an unknown point relative to a known coordinate can be determined using the total station as long as a direct line of sight can be established between the two points. Angles and distances are measured from the total station to points under survey, and the coordinates(X, Y, and Z or northing, easting and elevation) of surveyed points relative to the total station position are calculated using trigonometry and triangulation. To determine an absolute location a

Total Station requires line of sight observations and must be set up over a known point or with line of sight to 2 or more points with known location.

5.7.1.2. Angle measurement

Most modern total station instruments measure angles by means of electro-optical scanning of extremely precise digital bar-codes etched on rotating glass cylinders or discs within the instrument. The best quality total stations are capable of measuring angles to 0.5 arc-second. Inexpensive "construction grade" total stations can generally measure angles to 5 or 10 arc seconds.

5.7.1.3. Distance Measurement

Measurement of distance is accomplished with a modulated microwave or infrared carrier signal, generated by a small solid-state emitter within the instrument's optical path, and reflected by a prism reflector or the object under survey. The modulation pattern in the returning signal is read and interpreted by the computer in the total station. The distance is determined by emitting and receiving multiple frequencies, and determining the integer number of wavelengths to the target for each frequency. Most total stations use purpose-built glass corner cube prism reflectors for the EDM signal. A typical total station can measure distances with an accuracy of about 1.5 millimetres (0.0049 ft) + 2 parts per million over a distance of up to 1,500 metres (4,900 ft).[2]

5.7.1.4. Data processing

Some models include internal electronic data storage to record distance, horizontal angle, and vertical angle measured, while other models are equipped to write these measurements to an external data collector, such as a hand-held computer. When data is downloaded from a total station onto a computer, application software can be used to compute results and generate a map of the surveyed area.

5.8 GLOBAL POSITIONING SYSTEM

The **Global Positioning System** (**GPS**) is a space-based global navigation satellite system (GNSS) that provides location and time information in all weather, anywhere on or near the Earth, where there is an unobstructed line of sight to four or more GPS satellites.

5.8.1Basic concept of GPS

A GPS receiver calculates its position by precisely timing the signals sent by GPS satellites high above the Earth. Each satellite continually transmits messages that include

- the time the message was transmitted
- precise orbital information (the ephemeris)
- the general system health and rough orbits of all GPS satellites (the almanac).

A satellite's position and pseudorange define a sphere, centered on the satellite, with radius equal to the pseudorange. The position of the receiver is somewhere on the surface of this sphere. Thus with four satellites, the indicated position of the GPS receiver is at or near the intersection of the surfaces of four spheres. The methods are trilateration, Bancroft's method, Multidimensional Newton-Raphson calculations

5.8.2 GPS Applications

While originally a military project, GPS is considered a *dual-use* technology, meaning it has significant military and civilian applications.

5.8.2.1Military applications:

- Navigation: GPS allows soldiers to find objectives, even in the dark or in unfamiliar territory, and to coordinate troop and supply movement. In the United States armed forces, commanders use the *Commanders Digital Assistant* and lower ranks use the *Soldier DigitalAssistant*.
- Target tracking: Various military weapons systems use GPS to track potential ground and air targets before flagging them as hostile. These weapon systems pass target coordinates to precision-guided munitions to allow them to engage targets accurately.
- Missile and projectile guidance:
- Search and Rescue: Downed pilots can be located faster if their position is known.
- Reconnaissance: Patrol movement can be managed more closely.

5.8.2.2Civilian Applications:

- Disaster relief/emergency services: Depend upon GPS for location and timing capabilities.
- Geofencing: Vehicle tracking systems, person tracking systems, and pet tracking systems use GPS to locate a vehicle, person, or pet. These devices are attached to the vehicle, person, or the pet collar. The application provides continuous tracking and mobile or Internet updates should the target leave a designated area.
- Geotagging: Applying location coordinates to digital objects such as photographs and other documents for purposes such as creating map overlays.
- GPS Aircraft Tracking
- GPS tours: Location determines what content to display; for instance, information about an approaching point of interest.
- Map-making: Both civilian and military cartographers use GPS extensively.
- Navigation: Navigators value digitally precise velocity and orientation measurements.
- Phasor measurements: GPS enables highly accurate time stamping of power system

measurements, making it possible to compute phasors.

- Robotics: Self-navigating, autonomous robots using a GPS sensors, which calculate latitude, longitude, time, speed, and heading.
- Recreation: For example, geocaching, geodashing, GPS drawing and way marking.
- Surveying: Surveyors use absolute locations to make maps and determine property boundaries.
- Tectonics: GPS enables direct fault motion measurement in earthquakes.
- Telematics: GPS technology integrated with computers and mobile communications technology in automotive navigation systems
- Fleet Tracking: The use of GPS technology to identify, locate and maintain contact reports with one or more fleet vehicles in real-time.

5.9 GEOGRAPHICAL INFORMATION SYSTEM:

A geographic information system, geographical information science, or geospatial information studies is a system designed to capture, store, manipulate, analyze, manage, and present all types of geographically referenced data. In the simplest terms, GIS is the merging of cartography, statistical analysis, and database technology.

GIS consists of five key components: -

1. Hardware
2. Software
3. Data
4. People
5. Methods

Computer Hardware Module: It is generally a computer or central processing unit. It is linked to a disk drive storage unit, which provides space for storing data and programs. A digitizer, scanner and other device is used to convert data from maps and documents into digital form and send them to computer.

Computer Software Module: The GIS software includes the programs and the user interface for driving the hardware. GIS software generates, stores, analyzes, manipulates and presents geographic information or data. It is essential for driving the hardware.

Data: Geographic data and related tabular data can be collected in house, compiled to custom specifications and requirements, or purchased from a commercial data provider.

People: People are the core to GIS. Both technical specialists who develop and manage the system as well as end users who employ the technology are critical to the success of GIS.

Methods: A neatly conceived implementation plan and business rules are the models and operating practices, are unique to each organization

5.9.1Spatial analysis with GIS

1. Slope and aspect
2. Data analysis
3. Topological Modeling
4. Networks
5. Hydrological Modeling
6. Cartographic Modeling
7. Map overlay
8. Automated cartography
9. Geostatistics
10. Address geocoding
11. Reverse geocoding

5.9.2 Data representation

GIS data represents real objects (such as roads, land use, elevation, trees, waterways, etc.) with digital data determining the mix. Real objects can be divided into two abstractions: discrete objects (e.g., a house) and continuous fields (such as rainfall amount, or elevations). Traditionally, there are two broad methods used to store data in a GIS for both kinds of abstractions mapping references: raster images and vector. Points, lines, and polygons are the stuff of mapped location attribute references

5.9.3 Raster

A raster data type is, in essence, any type of digital image represented by reducible and enlargeable grids. Anyone who is familiar with digital photography will recognize the Raster graphics pixel as the smallest individual grid unit building block of an image, usually not readily identified as an artifact shape until an image is produced on a very large scale.

5.9.4 Vector

In a GIS, geographical features are often expressed as vectors, by considering those features as geometrical shapes. Different geographical features are expressed by different types of geometry:

- Points: Zero-dimensional points are used for geographical features that can best be expressed by a single point reference

- Lines or polylines: One-dimensional lines or polylines are used for linear features such as rivers, roads, railroads, trails, and topographic lines.
- Polygons: Two-dimensional polygons are used for geographical features that cover a particular area of the earth's surface. Such features may include lakes, park boundaries, buildings, city boundaries, or land uses.

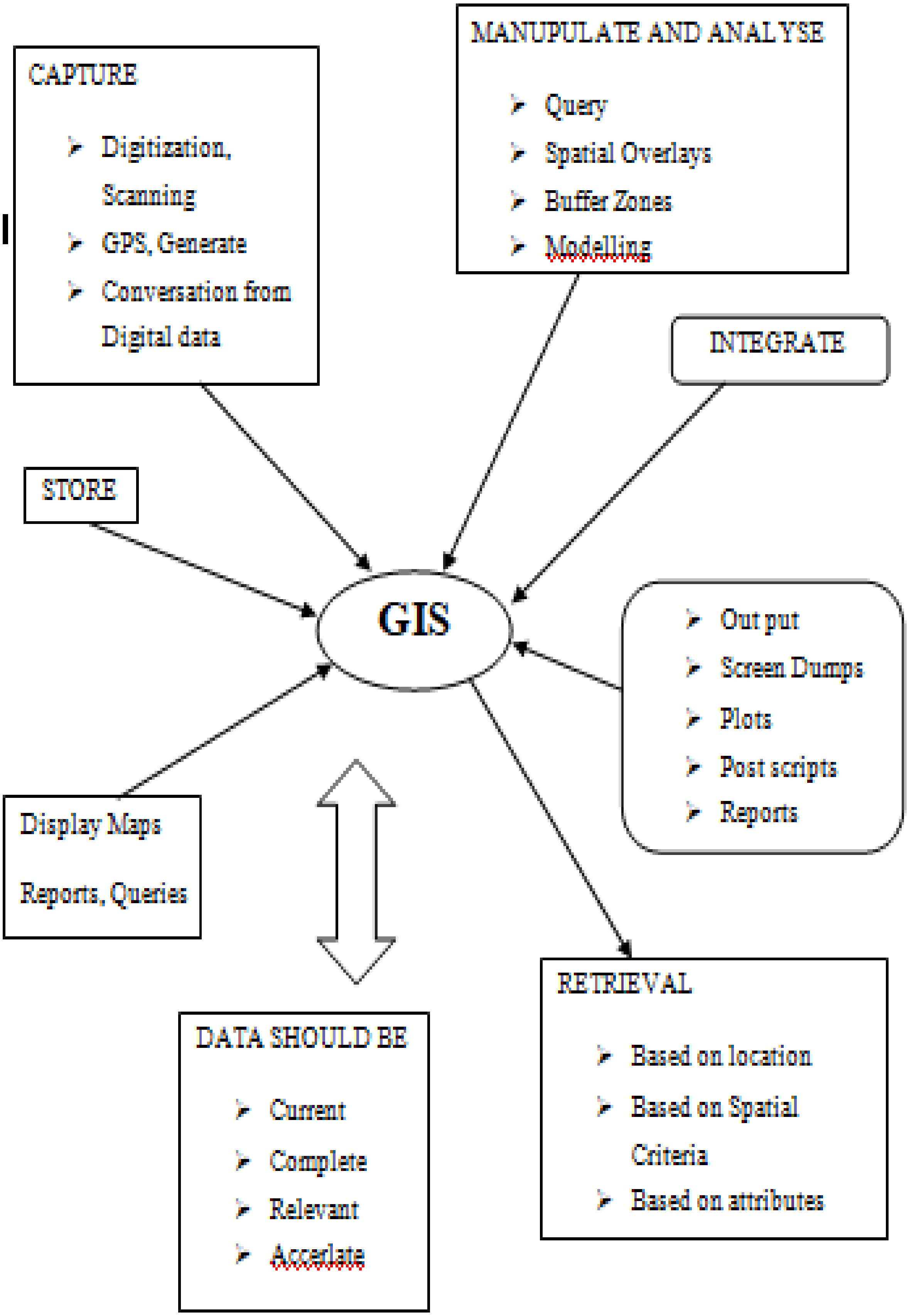
5.9.5 Advantages and disadvantages

There are some important advantages and disadvantages to using a raster or vector data model to represent reality:

- Raster datasets record a value for all points in the area covered which may require more storage space than representing data in a vector format that can store data only where needed.
- Raster data is computationally less expensive to render than vector graphics
- There are transparency and aliasing problems when overlaying multiple stacked pieces of raster images
- Vector data allows for visually smooth and easy implementation of overlay operations, especially in terms of graphics and shape-driven information like maps, routes and custom fonts, which are more difficult with raster data.
- Vector data can be displayed as vector graphics used on traditional maps, whereas raster data will appear as an image that may have a blocky appearance for object boundaries. (depending on the resolution of the raster file)
- Vector data can be easier to register, scale, and re-project, which can simplify combining vector layers from different sources.

Vector data is more compatible with relational database environments, where they can be part of a relation

ESSENTIALS OF GIS



UNIT – VI

COMPUTATION OF AREAS AND VOLUMES

6.1 AREA FROM FIELD NOTES

Area of a plot may be determined by the direct use of the field noted as in a cross – staff survey. In this method of surveying a chain line is run through the centre of the area so that the offsets to the boundaries on either side of it are fairly equal. These offsets aare taken in order of their changes. A cross staff is used to set out the perpendicular directions of the offsets which are usually more than 15 m in length. For accurate work, an optical square or a prism square is pereferable used. The plot is divide into right angled triangles and trapezoids, and the area is calculated.

6.2 COMPUTATION OF AREAS ALONG IRREGULAR BOUNDARIES AND AREA CONSISTING OF REGULAR BOUNDARIES

One of the primary objects of land surveying of land surveying is to determine the area of the tract surveyed and to determine the quantites of earthwork. The area of land in plane surveying means the area as projected on a horizontal plane. The units of measurements of area in English units are sq. ft or acres, while in metric units, the units are sq. metres or hectares.

- British units of square measure with metrics equivalents.
- Metric units of square measure with british equivalents.

General Methods of Determining areas:

The following are the general methods of calculating areas:

1) By computations based directly on field measurements:

   These include :

   i) By dividing the area into a number of triangles.

   ii) By offsets to base line

   iii) By latitudes and departures:

      a) By double meridian distance (DMD method)

      b) By double paralles distance (DPM method)

   iv) By co-ordinates.

2) By computation based on measurements scaled from a map.

3) By mechanical method : Usually by means of a planimeter.

6.2.1Areas computed by subdivision into triangles:

In this method, the area is divided into a number of triangles, and the area of each triangle is calculated. The total area of the tract will then be equal to the sum of areas of individual triangles. The figure shows an area divided into several triangles. For field work, a transit may be set up at O, and the lengths and directions of each of the lines OA, OB, ... etc, may be measured. The area of each triangle can then be computed. In addition, the sides AB, BC,…… etc. can also be measured and a check may be applied by calculating the area from the three known sides of a triangle. Thus, if two sides and one included angle of a triangle is measured, the area of the triangle is given by

$$\text{Area} = \tfrac{1}{2}\, ab \sin C$$

When the lengths of the three sides of a triangle are measured, its area is computed by the equation.

$$\text{Area} = \sqrt{s(s-a)(s-b)(s-c)}$$

Where s = half perimeter = ½ (a+b+c).

The method is suitable only for work of small nature where the determination of the closing error of the figure is not important, and hence the computation of latitudes and departure is unnecessary. The accuracy of the field work, in such cases, may be determined by measuring the diagonal in the field and comparing its length to the computed length.

6.2.2Areas from offsets to a baseline : Offsets at regular intervals:

This method is suitable for long narrow strips of land. The offsets are measured from the boundary to the base line or a survey line at regular intervals. The method can also be applied to a plotted plan from which the offsets to a line can be scaled off. The area may be calculated by the following rules:

i) Mid – ordinate rule
ii) Average ordinate rule
iii) Trapezodial rule
iv) Simpson's one- third rule

6.2.2.1Mid Ordinate Rule:

The method is used with the assumption that the boundaries between the extremities of the ordinates (or offsets) are straight lines. The base line is divided into a number of divisions and the ordinates are measured at the mid – points of each division as illustrated in the figure. The area is calculated by the formula

$$\text{Area} = \Delta = \text{Average ordinate x Length of base}$$

$$= L = (O_1 + O_2 + O_3 + \text{------} + O_n)\, d = d\,\Sigma O$$

Where O_1,O_2 ---- = the ordinate at the mid points of each division.

ΣO = sum of the mid-ordinates

n = number of divisions

L = Length of base line = nd

d = distance of each division

6.2.2.2.Average ordinate Rule:

This rule also assumes that the boundaries between the extremities of the ordinates are straight lines. The offsets are measured to each of the points of the divisions of the base line.

The area is given by ∆ = Average ordinate x length of the base

$$= \frac{Oo + O1 + \_\_ + On}{n+1} L = \frac{L}{n+1} \Sigma O$$

Where Oo = ordinate at one end of the base

O_n = ordinate at the other end of the base divided into n equal divisions.

O_1, O_2 = ordinate at the end of each division.

6.2.2.3 Trapezoidal Rule

This rule is based on the assumption that the figures are trapezoids. The rule is more accurate than the previous two rules which are approximate versions of the trapezoidal rule.

The area of the first trapezoid is given by

$$\Delta_1 = \frac{Oo + O1}{2} d$$

Similarly, the area of the second trapezoid is given by $\Delta_2 = \frac{O1 + O2}{2} d$

Area of the last trapezoid is given by

$$\Delta_3 = \frac{On - 1 + On}{2} d$$

Hence the total area is given by

$$\Delta = \Delta_1 + \Delta_2 + \text{------ } \Delta n = \frac{Oo + O1}{2} d + \frac{O1 + O2}{2} d + \frac{On - 1 + On}{2} d$$

Or

$$\Delta = [\frac{Oo + On}{2} + O_1 + O_2 + \text{-------} + O_{n-1}] d$$

6.2.2.4 Simpsons One-third Rule

This rule assumes that the short lengths of boundary between the ordinates are parabolic arcs. This method is more useful when the boundary line departs considerably from the straight line.

The area between the line AB and the curve DFC may be considered to be equal to the area of the trapezoid ABCD plus the area of the segment between the parabolic arc DFC and the corresponding Chord DC.

Let O_0, O_1, O_2 = any three consecutive ordinates taken at regular interval of d.

Area of trapezoid ABCD = $\frac{Oo + O2}{2}$ 2d.

The total area

Δ = d/3 [O_0 + 4 O_1 + 2O_2 + 4O_3 + -------+2O_{n-2} + 4 O_{n-1} + O_n].

Δ = d/3 [(O_0 +O_n) + 4(O_1 + O_3 + -------+O_{n-1} + 2 (O_2 + O_4 + O_{n-2})].

6.2.2.5 Offsets at irregular intervals

In this method, the area of each trapezoid is calculated separately and then added together to calculate the total area.

Δ = d_1/2 (O_1 + O_2)+ d_2/2 (O_2 + O_3) + d_3/2 (O_3 + O_4).

6.3 MEASUREMENT OF VOLUME

These are three methods generally adopted for measuring the volume. They are

- From Cross-sections
- From Spot levels
- From contours

The first two methods are commonly used for the calculation of earth work while the third method is generally adopted for the calculation of reservoir capacities.

Formulae for Areas of Cross - sections:

The following are the various cross sections usually met with whose areas are to be computed:

- Level Section
- Two Level section
- Side hill tow level section
- Three level section
- Multi level section

Level Section:

$$A = \tfrac{1}{2} [b + (b+2sh)]h$$

$$= (b + sh)\, h$$

Two level section:

$$A = \tfrac{1}{2} [(w_1 + w_2)(h + b/2s) - b^2/2s]$$

$$= \left[\frac{s\left(\frac{h}{2}\right)^2 + r^2bh + r^2sh^2}{(r^2 - s^2)}\right]$$

Slide hill two level section:

$$\text{Area of Cutting, } A_1 = \tfrac{1}{2}\left[\frac{(\frac{b}{2}+rh)^2}{r-s}\right]$$

Area of Filling, $A_2 = ½[\frac{(\frac{b}{2} - rh)^2}{r - s}]$

Three level section:

$A = [½h (w_1 + w_2) + b/4 (h_1 + h_2)]$

Multi – level Section:

A = ½ (ΣF – ΣD) … … … … … ./.. …

Formulae for volume:

To calculate the volumes of the solids between sections, it must be assumed that they have some geometrical form. They must nearly take the form of prismoids and therefore, in calculation work, they are considered to be prismoids.

Let A_1, A_2, A_3, ... A_n = the areas at the 1st, 2nd, 3rd, .. ……. Last cross section.

D = the common distance between the cross section.

V = the volume of cutting or filling

Trapezoidal Formula:

$V = D\ \{\frac{A1+An}{2} + A2 + A3 + \cdots ... + An - 1\ \}$

$= \text{Common distance}\ \{\frac{1st\ area + last\ area}{2} + \ the\ sum\ of\ remaining\ areas\}$

The number of cross sections giving the areas may be odd or even. Since the areas at ends are averaged in this formula, therefore, it is also known as Average end Area formula.

Prismoidal Forumula:

$V = \frac{D}{3}\ \{A_1 + 4A_2 + 2A_3 + 4A_4 + 2A_5 + \ldots\ldots + 2A_{n-2} + 4A_{n-1} + A_n\}$

$= \frac{D}{3}\ \{(A_1 + A_n) + 2(A_3 + A_5 + \ldots\ldots + 2A_{n-2} + 4(A_1 + A_4 + \text{-------} + A_{n-1})\}$

$= \frac{Common\ \ distance}{3}$ {(1st area + last area)}

+2{ (the sum of remaining odd area)

+4 (the sum of even area)}

In order to apply the prismoidal formula, it is necessary to have odd number of sections giving the areas. If there be even areas, the prismoidal formula may be applied to odd number of areas and the volume between the last two sections may be obtained separately by trapezoidal formula and added.

Problems:-

1. The distance AB on the ground as measured on a plan drawn to a scale 1 cm = 50m was found to be 500m. Later it was detected that the surveyor wrongly used a scale of 1cm = 40m in the calculations. Find the true length of the line AB.

Sol:- Let AB be the correct length on plan

The length ab on plan with the scale 1 cm = 40m used for calculating the distance AB on the ground

$$= \frac{500}{40} = 12.5 \text{ cm}$$

True scale is 1 cm = 50m

True distance on ground, AB = 50 x 12.5
= 625 m.

2, The area of the plan of an old survey plotted to a scale of 15m to 1 cm now measures as 80.2 cm² as found by a plainmeter. The plan is found to have shrunk, so that a line original 10 cm long now measures 9.8 cm only. Find the shrunk scale and the true area of the survey.

Sol:-

$$S.F = \frac{\text{shrunk length}}{\text{original length}} = \frac{9.8}{10} = 0.98$$

original scale is 1 cm = 15 m

Shrunk scale = SF x original scale

$= 0.98 \times 15 = 14.70\,m$

Shrunk scale is 1 cm = 14.70 m

$$\text{Correct area} = \frac{\text{measured area}}{(SF)^2}$$

$$= \frac{80.2}{(0.98)^2}$$

$= 83.51\,m^2.$

3. A surveyor measured the distance b/w two points on the plan drawn to a scale of 1cm = 40 mts and the result was 468 mts later, however he discovered that he used a scale of 1cm = 20 mts. Find the true distance b/w the points.

Sol:- Corrected length = $\dfrac{\text{R.F of wrong scale}}{\text{R.F of correct scale}} \times \text{M.D}$

$$= \frac{\frac{1}{20 \times 100}}{\frac{1}{40 \times 100}} \times 468$$

$= 936$ mts

4. The plan of an area as shunk such that a line originally 10 cms long now measures 9.5 cms. If the original scale of the plan was 1cm = 50 m. Determine (1) shrinkage factor (2) shrunk scale (3) correct distance corresponded to a measured distance of 980 mts. (4) correct area corresponding to a measured area of 10,000 m^2.

Sol:- original length = 10 cms

shrunk length = 9.5 cms

original scale = 50 mts

Measured distance = 980 mts

Measured area = 10,000 mts

Shrinkage factor = $\frac{9.5}{10} = 0.95$

$S.S = S.F \times O.S$

$= 0.95 \times \frac{1}{50 \times 100}$

$= \frac{1}{5263.157}$

Correct distance = $\frac{\text{Measured Distance}}{S.F}$

$= \frac{980}{0.95}$

$= 1031.578$ mts

Correct area = $\frac{\text{Measured Area}}{(S.F)^2}$

$= \frac{10,000}{(0.95)^2}$

$= 11080.33\ m^2.$

2. The length of a survey line was measured with a 20m chain and was found to be equal to 1200m. As a check, the length was again measured with a 25m chain and was found to be 1212m. On comparing the 20m chain with the test gauge, it was found to be 1 decimetre too long. Find the actual length of the 25m chain used.

Sol:- With 20m chain; $L' = 20 + 0.10 = 20.10\,m$

$$l = l'\left[\frac{L'}{L}\right] = 1200 \times \frac{20.10}{20}$$

$$= 1206\,m \quad \text{True length}$$

With 25m chain

$$l = l'\left[\frac{L'}{L}\right]$$

$$1206 = \left[\frac{L'}{25}\right] 1212$$

$$L' = \frac{1206 \times 25}{1212} = 24.88\,m.$$

Thus, the 25m chain was 12cm too short

... was found to be 10cm too long after chaining a distance of 1500m. It was found to be 18cm too long at the end of day's work after chaining a total distance of 2900m. Find the true distance if the chain was correct before the commencement of the work.

Sol:- For first 1500m.

$$\text{Average error} = e = \frac{0+10}{2} = 5\,cm = 0.05\,m$$

$$\therefore L' = 20 + 0.05 = 20.05\,m$$

$$\text{Hence } L_1 = \frac{20.05}{20} \times 1500$$

$$= 1503.75\,m.$$

For next 1400 m;

$$\text{Average error} = e = \left(\frac{10+18}{2}\right) = 14\,cm = 0.14\,m$$

$$L' = 20 + 0.14 = 20.14\,m$$

$$L_2 = \frac{20.14}{20} \times 1400$$

$$= 1409.80\,m.$$

$$\text{Total length} = L = l_1 + l_2$$

$$= 1503.75 + 1409.80$$

$$= 2913.55\,m.$$

A surveyor measured the distance between two points on the plan drawn to a scale of 1cm = 40m and the result was 468 m. Later, however, he discovered that he used a scale of 1cm = 20m. Find the true distance between the two points.

Sol:- Distance b/w two points measured with a scale of 1 cm to 20 m

$$= \frac{468}{20} = 23.4 \text{ cm.}$$

Actual scale of the plan is 1 cm = 40m.

True distance b/w the points $= 23.4 \times 40$

$= 936$ m.

5. A 20m chain used for a survey was found to be 20.10m at the beginning and 20.30m at the end of the work. The area of the plan drawn to a scale of 1cm = 8m was measured with the help of a plainmeter and was found to be 32.56 sq.cm. Find the true area of the field.

Sol:- L: Average length of the chain

$$= \frac{20.10 + 20.30}{2}$$

$= 20.20$ m.

Area of plan = 32.56 sq.cm

Area of the ground $= 32.56\,(8)^2$

$= 2083.84$ sq.m

True area $= A = \left[\frac{L'}{L}\right]^2 \times A'$

$= \left[\frac{20.20}{20}\right]^2 \times 2083.84$

$= 2125.73$ sq.m

Alternatively:-

$$A \approx (1+2e)A'$$

$$e = \frac{\Delta L}{L} = \frac{20.20-20}{20} = \frac{0.20}{20}$$

$$= 0.01$$

$$A \approx (1+2\times0.01)\times 2083.84$$

$$= 2125.52\ m^2$$

1. Convert the following whole circle bearing to Quadrantal bearing.

(1) $22° 30'$ (2) $170° 12'$ (3) $211° 54'$ (4) $327° 24'$

Sol:- R.B = W.C.B = $22° 30'$ = N $22° 30'$ E

R.B = 180° − W.C.B = 180° − $170° 12'$ = $9° 48'$

= S $9° 48'$ E

R.B = W.C.B − 180° = $211° 54'$ − 180° = $31° 54'$

= S $31° 54'$ W

R.B = 360° − W.C.B = 360° − $327° 24'$ = $32° 36'$

= N $32° 36'$ W.

2. Convert the following R.B to W.C.B.

(i) N $12° 24'$ E
(ii) S $31° 36'$ E
(iii) S $68° 6'$ W
(iv) N $5° 52'$ W.

Sol:- (i) W.C.B = R.B = N $12° 24'$ E = $12° 24'$

(ii) S $31° 36'$ E = W.C.B = 180 − R.B

= 180° − $31° 36'$ = $148° 24'$

(iii) S $68° 6'$ W = W.C.B = 180 + R.B

= 180° + $68° 6'$ = $248° 6'$

(iv) N $5° 52'$ W = W.C.B = 360° − R.B

= 360° − $5° 52'$ = $354° 8'$

3. The following are the observations of fore bearing of the lines

(1) AB − 12° 24' (2) BC − 119° 48' (3) CD − 266° 30'

(4) DE − 354° 18' (5) PQ − N 18° 0' E (6) QR − S 12° 24'

(7) RS − S 59° 18' W (8) ST − N 86° 12' W

Find their backbearings.

Sol:- AB ⟶ 12° 24' ⟶ 12° 24' + 180° = 192° 24'

BC ⟶ 119° 48' ⟶ 119° 48' + 180° = 299° 48'

CD ⟶ 266° 30' ⟶ 266° 30' − 180° = 86° 30'

DE ⟶ 354° 18' ⟶ 354° ~~48'~~ 18' − 180° = 174° 18'

PQ ⟶ N 180° E ⟶ S 180° W

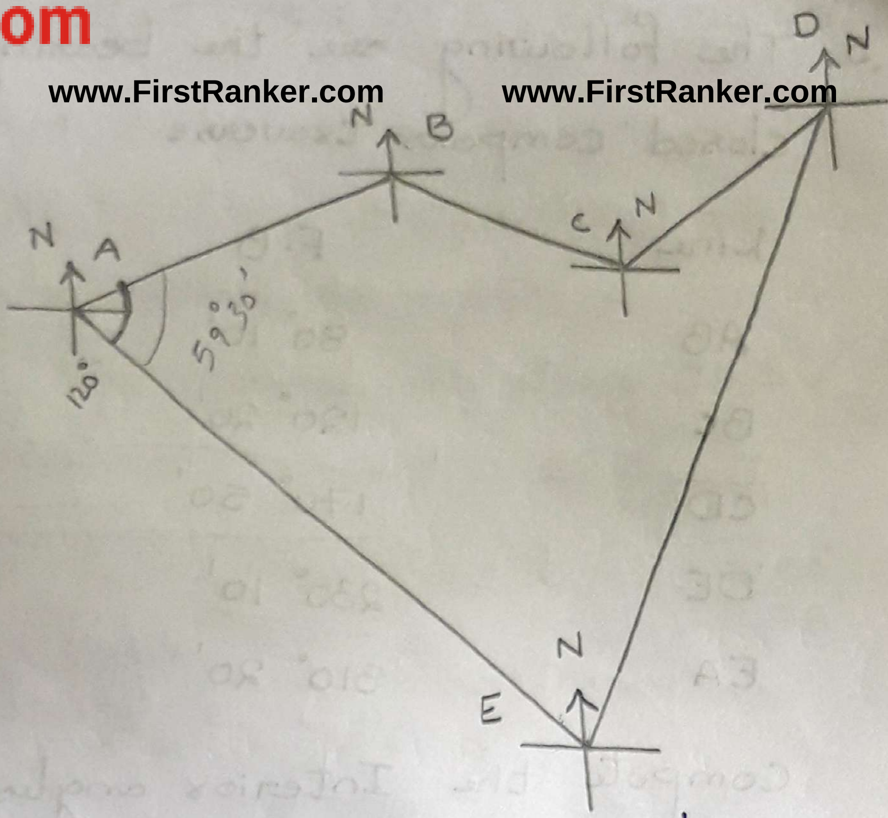
QR ⟶ S 12° 24 E ⟶ N 12° 24' W

RS ⟶ S 59° 18' W ⟶ N 59° 18' E

ST ⟶ N 86° 12' W ⟶ S 86° 12' E

4. The following bearings were observed with a compass calculate the interior angles

Line	F.B		B.B
AB	60° 30'	180°	240° 30'
BC	122°	180°	302° 0'
CD	46°	180°	226° 0'
DE	205° 30'	180°	25° 30'



Included angle = bearing of previous line − bearing of next line

$\angle A = (300^\circ - 180^\circ) - 60^\circ 30' = 59^\circ 30'$

$\angle B = (60^\circ 30' + 180^\circ) - 122^\circ = 118^\circ 30'$

$\angle C = (122^\circ + 180^\circ) - 46^\circ = 256^\circ$

$\angle D = (46^\circ + 180^\circ) - 205^\circ 30' = 20^\circ 30'$

$\angle E = (205^\circ 30' - 180^\circ) - 300^\circ + 360^\circ = 85^\circ 30'$

$\angle A + \angle B + \angle C + \angle D + \angle E = 540^\circ$

$59^\circ 30' + 118^\circ 30' + 256^\circ + 20^\circ 30' + 85^\circ 30' = 540^\circ$

check:-

$(2n - 4)\, 90^\circ$

$= (2 \times 5 - 4)\, 90^\circ$

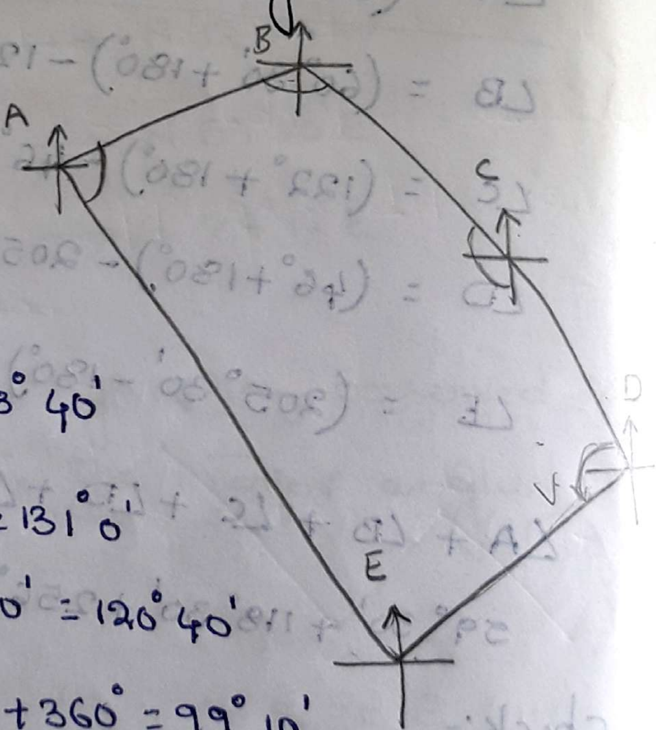
$= 540^\circ$

5) The following are the bearings taken on a closed compass traverse

Line	F.B	B.B
AB	80° 10'	259° 0'
BC	120° 20'	301° 50'
CD	170° 50'	350° 50'
DE	230° 10'	49° 30'
EA	310° 20'	130° 15'

Compute the Interior angles and correct them for observational errors. Assuming the observed bearings of the line CD to be correct adjust the bearings of the remaining sides.

Sol:-



$\angle A = 130°15' - 80°10'$
$= 50° 5'$

$\angle B = 259° 0' - 120° 20'$
$= 138° 40'$

$\angle C = 301° 50' - 170° 50' = 131° 0'$

$\angle D = 350° 50' - 230° 10' = 120° 40'$

$\angle E = 49° 30' - 310° 20' + 360° = 99° 10'$

check

$(2n-4)\, 90°$

$= (2\times5-4)\, 90°$

$= 540°$ $\therefore$ Error $= -\dfrac{0°25}{5} = 5'$

$\angle B = 138^\circ 45'$, $\angle C = 131^\circ 5'$, $\angle D = 120^\circ 45'$,
$\angle E = 99^\circ 15'$

Correction:-

Bearing of D.E = B. of DC − ∠D
$= 350^\circ 50' - 120^\circ 45' = 230^\circ 5'$ ✓

Bearing of ED = $230^\circ 5' - 180^\circ = 50^\circ 5'$

Bearing of EA = B. of ED − ∠E + 360° = $310^\circ 50'$
$50^\circ 5' - 99^\circ 15' + 360^\circ = 310^\circ 50'$

Bearing of AE = $310^\circ 50' - 180^\circ = 130^\circ 50'$

Bearing of AB = B. of AE − ∠A
$= 130^\circ 50' - 50^\circ 10' = 80^\circ 40'$

Bearing of BA = $80^\circ 40' + 180^\circ = 260^\circ 40'$

Bearing of BC = B. of BA − ∠B
$= 260^\circ 40' - 138^\circ 45' = 121^\circ 55'$

Bearing of CB = $121^\circ 55' + 180^\circ = 301^\circ 55'$

Bearing of CD = B. of CB − ∠C
$= 301^\circ 55' - 131^\circ 5' = 170^\circ 50'$

Bearing of DC = $170^\circ 50' + 180^\circ = 350^\circ 50'$

Problems:-

1. In a traverse the latitudes and departures of the sides were calculated and it was observed that Σlatitude $= 1.39$ and Σdeparture $= -2.17$. Calculate the length and bearing of the closing error.

Sol:- closing error, $e = \sqrt{\Sigma L^2 + \Sigma D^2}$

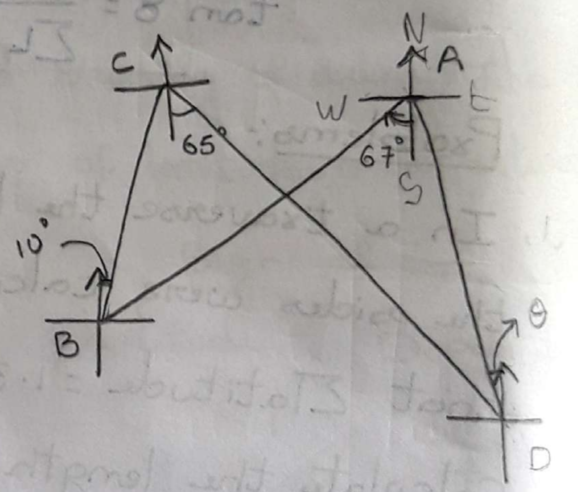
$$= \sqrt{(1.39)^2 + (-2.17)^2}$$

$$= 2.577 \text{ m.}$$

Reduced bearing of closing error $\delta = \tan^{-1} \frac{-2.17}{1.39}$

$$= 57^\circ \, 21'$$

While making a reconnaisance survey through the woods, a surveyor with a ... started from a point A and walked a thousand steps in the direction S 67° W and reached a point B. Then he changed his direction & walked 512 steps in the direction N 10° E and reached a point C. Then again he changed his direction and walked 1504 steps in the direction S 65° E and reached a point D as shown in fig. Now the surveyor wants to return to the starting point A. In which direction should he move and how many steps should he take.



Sol:-

Line	Length	R.B	Quadrant
AB	1000 steps	67°	SW
BC	512 steps	10°	NE
CD	1504 steps	65°	SE
DA	?	?	?

Line AB

Latitude $= -1000 \cos 67° = -390.73$

Departure $= -1000 \sin 67° = -920.50$

Latitude = 512 cos 10° = +504.22

Departure = 512 Sin 10° = +88.90

Line CD

Latitude = −1504 Cos 65° = −635.61

Departure = +1504 Sin 65° = +1363.08

Let the latitude and departure of the line DA be $l\cos\theta$ and $l\sin\theta$.

Then $\Sigma L = 0$

$$0 = -390.73 + 504.22 - 635.61 \boxed{+ l.\cos\theta}$$

$$l\cos\theta = 522.12$$

$$\Sigma D = 0 = -920.50 + 88.90 + 1363.08 + l\sin\theta$$

$$l\sin\theta = -531.48$$

Length of DA, $l = \sqrt{\Sigma L^2 + \Sigma D^2}$

$$= \sqrt{(522.12)^2 + (-531.48)^2}$$

$$= 745.03 \simeq 745 \text{ steps.}$$

Reduced bearing, $\theta = \tan^{-1}\dfrac{531.48}{522.12}$

$$= 45°\,30'\,32''$$

Hence, the required direction is N 45° 30' 32'' W.

[NW quadrant, since latitude is '+'ve and departure is '−'ve].