KNOWLEDGE CRITERIA WEEK - 01

SN	COURSE CONTENT
1.1	Contours, Importance of contour Maps, characteristics of contours.
1.2	Methods of plotting contours, Factors affecting contour interval.
1.3	Method of preparing contour map from the given RLs of grid points with examples

WEEK 01 CLASS 01

GENERAL CONCEPT OF CONTOUR:

Most of the civil engineering projects such as railways, highways & canals require the knowledge of nature of ground surface for locating the suitable alignments and estimating the volume of earthwork. Therefore in a topographic survey both horizontal as well as vertical controls are required to enhance the relative position of the points both horizontally & vertically. Hence a map has been prepared to show the details of an area on earth in horizontal plane. On a plan a relative altitudes of the points can be represented by shading, hachure, and contour lines. Out of these contour lines are most widely used because they indicate the elevations directly.

TERMS USED IN CONTOURING:

- **CONTOUR**: A contour is an imaginary line on the surface of the earth joining the points of equal elevation.
- **CONTOUR LINE:** A contour line is a line on the map representing a contour.
- **CONTOUR INTERVAL:** The vertical distance between any two consecutive contours is called contour interval.
- **HORIZONTAL EQUIVALENT:** The horizontal distance between two points on two consecutive contours is known as horizontal equivalent.

IMPORTANCE OF CONTOUR MAPS:

- ✓ By the inspection of contour map, information regarding the character of the tract of the country is obtained, whether it is flat, undulating or mountainous.
- ✓ The most economical or suitable site for engineering works such as reservoir, canal, sewer, road or railway may be approximately selected.
- ✓ Quantities of earthwork may be computed from contour maps.
- ✓ Contour maps are used to determine the area of the drainage basin and the capacity of the reservoir.
- ✓ The sections may be easily drawn from contour maps in any direction.
- ✓ Contour maps can be used to draw longitudinal and cross section.
- ✓ Intervisibility of two given points can be ascertained from the map.
- ✓ A route of a given grade line can be traced on the map.

CHARACTERISTICS OF CONTOURS:

1) All the points on a contour line have the same elevation. A zero meter contour line represents the coast line.
2) Contour should be labelled to the elevation value.
3) Two contour lines having the same elevation cannot unite and continue as one line. Similarly a single contour cannot split into two lines. This is evident because the single line would, otherwise indicate a knife edge ridge or depression which does not occur in nature. Two different contours of the same elevation may approach very near to each other.
4) Contours do not pass through permanent structures such as buildings. (Fig 1.1)
5) Contour lines cross a water shed or ridge line at right angles. They form curves of U shaped it with the concave side of the curve towards the higher ground. (Fig 1.2)
6) Contour lines cross a valley line at right angles. They form sharp curves of V shaped it with the convex side of the curve towards the higher ground. (Fig 1.2)
7) A contour line must close upon itself, though not necessarily within the limits of the map depending upon the topography. (Fig 1.3)
8) A closed contour line with one or more higher ones inside it represents a hill. (Fig 1.4)
9) A closed contour line with one or more lower ones inside it represents a depression without an outlet. (Fig 1.5)
10) Contour lines of different elevations can unite to form one line only in the case of a vertical cliff. (Fig 1.6)
11) Two contour lines of different elevations cannot cross each other. If they did, the point of intersection would have two different elevations which are absurd. However contour lines of different elevations can intersect only in the case of an overhanging cliff or cave. (Fig 1.7)
12) Contour lines close together indicate steep slope. They indicate a gentle slope if they are far apart. If they are equally spaced, uniform slope is indicated. A series of straight, parallel and equally spaced contours represent a plane surface. (Fig 1.8)
13) A contour passing through any point is perpendicular to the line of steepest slope at that point. This agrees with 12 since the perpendicular distance between contour lines is the shortest distance.

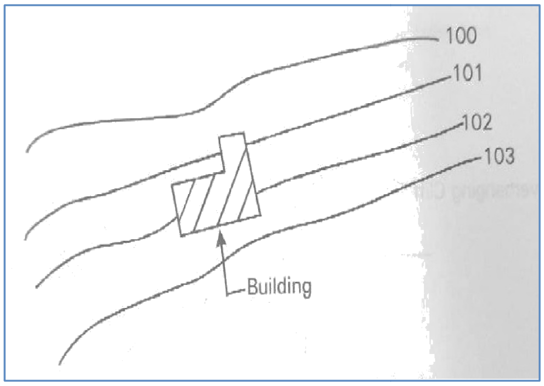


FIG 1.1

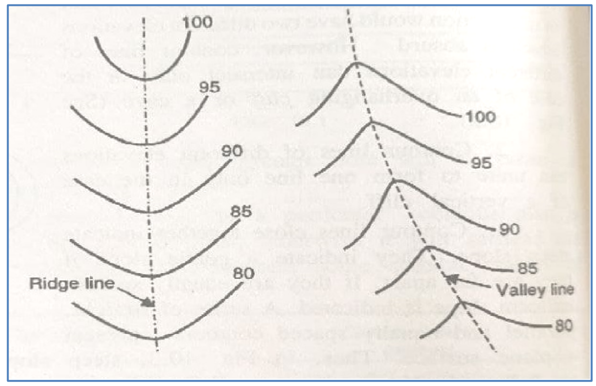


FIG 1.2

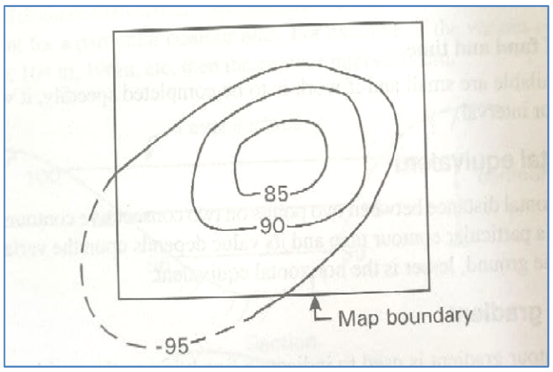


FIG 1.3

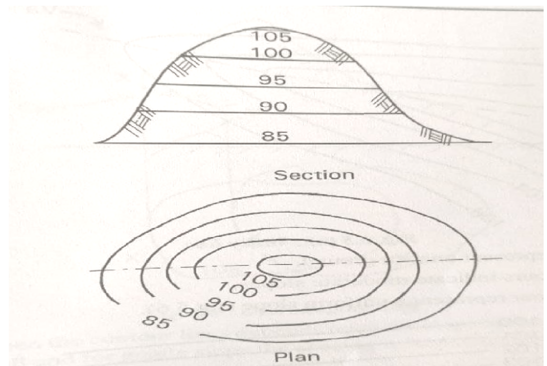


FIG 1.4

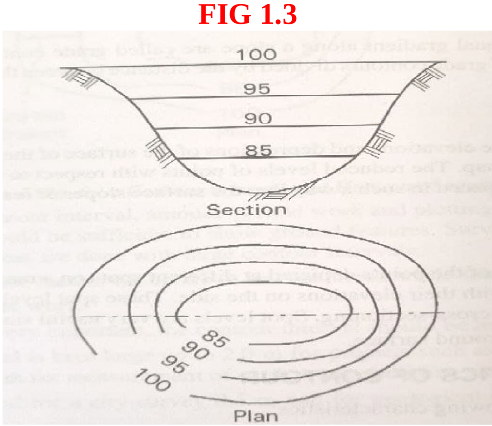


FIG 1.5

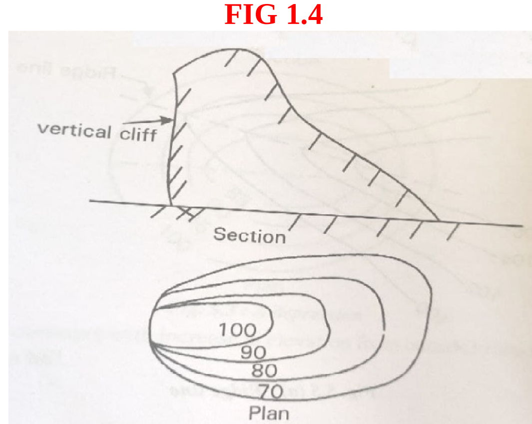


FIG 1.6

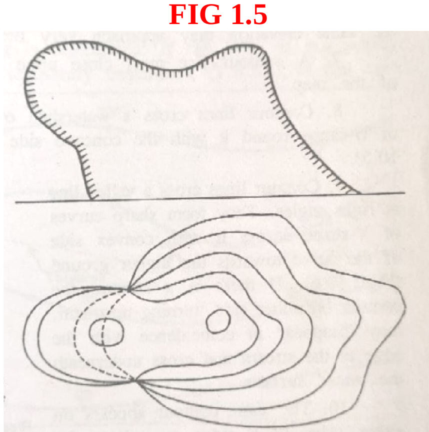


FIG 1.7

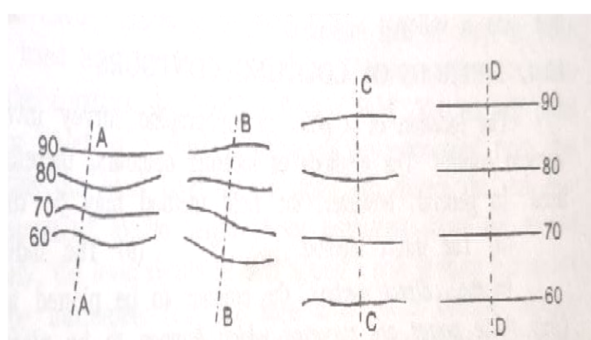
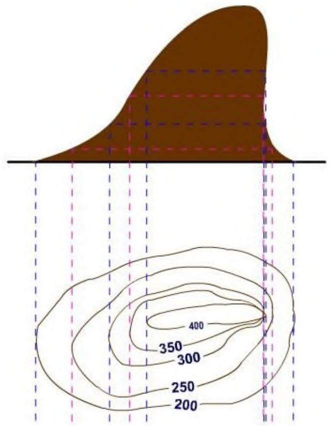
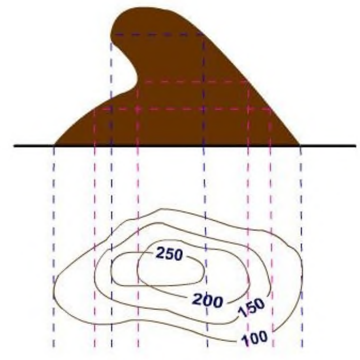


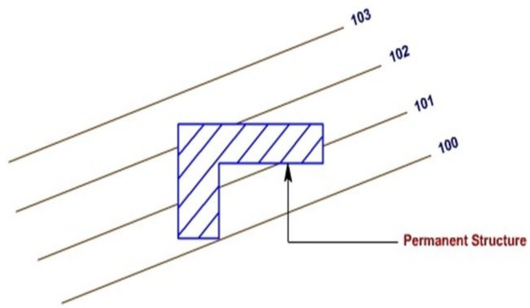
FIG 1.8



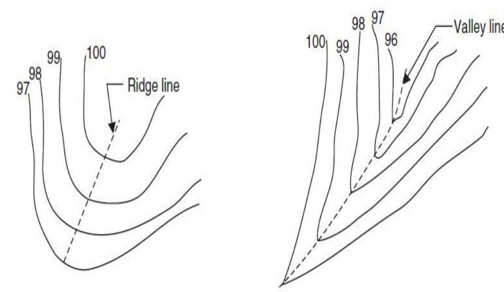
Vertical cliff and its contours

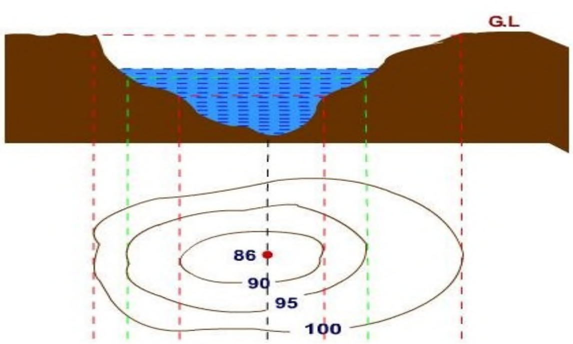


Overhanging cliff and its contour

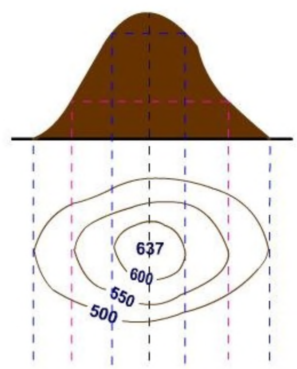


Contours across permanent structure

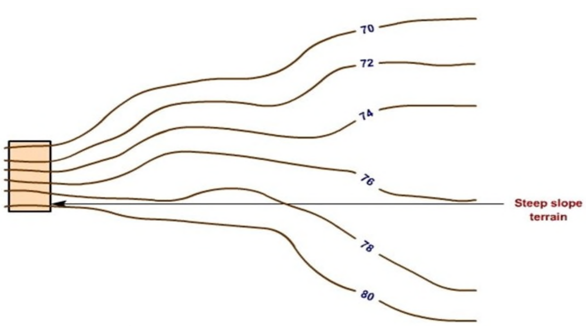




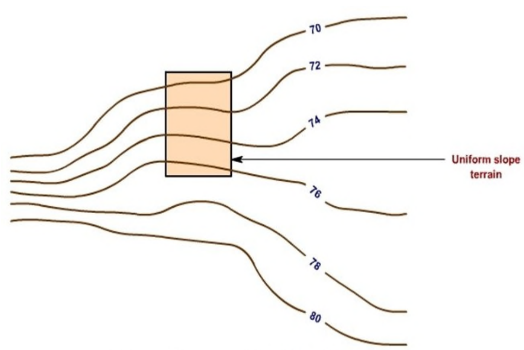
Pond and its contour



Hill and its contour



Contours of Terrain having different types of slope



Contours of Terrain having different types of slope

WEEK 01 CLASS 02

METHODS OF PLOTTING CONTOURS:

METHODS OF LOCATING CONTOURS:

The location of a point in topographic survey involves both horizontal as well as vertical control. The methods of locating contours, therefore depends upon the instruments used. In general the field method may be divided into two classes.

- **THE DIRECT METHOD:** The contour to be plotted is actually traced on the ground. Only these points are surveyed which happen to be plotted. After having surveyed those points, they are plotted and contours are drawn through them. The method is slow and tedious and is used for small areas and where great accuracy is required.
- **THE INDIRECT METHOD:** In this method some suitable guide points are surveyed, the guide points need not to be on the contours. These guide points, having been plotted serve as basis for the interpolation of contours. This method is most commonly used in engineering survey.

DIRECT METHOD:

As stated earlier in the direct method each contour is located by determining the positions of series of points through which the contour passes. The operation is also sometimes called tracing out of contours.

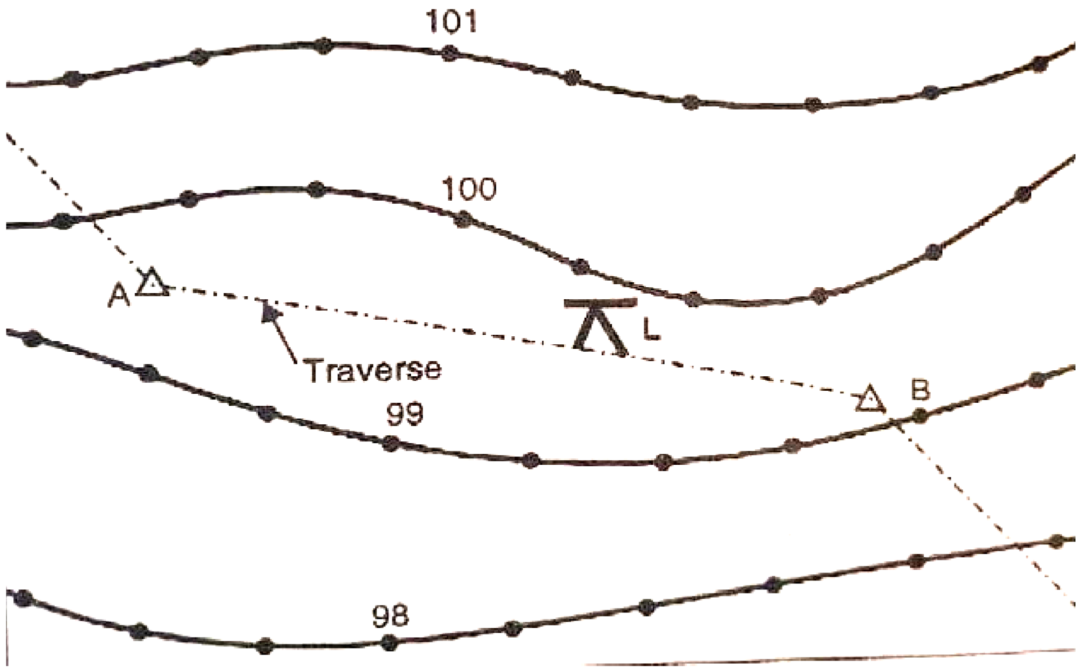
The field work is two-fold:

Vertical control: Location of points on the contour.

Horizontal control: Survey of those points.

- **Vertical Control:** The points on the contours are traced either with the help of a level and staff or with the help of a hand level. In the former case, the level is set at a point to command as much area as is possible and is levelled. The staff is kept on the BM and the height of the instrument is determined, If the BM is not nearby. Fly leveling may be performed to establish a temporary benchmark (TBM) in that area. Having known the height of the instrument, the staff reading is calculated so that the bottom of the staff is at an elevation equal to the value of the contour.

  For example, if the height of the instrument is 101.80 meters, the staff reading to get a point on the contour of 100.00 meters will be 1.80 meters. Taking one con- tour at a time (say 100.0 m contour), the staff man is directed to keep the staff on the points on contour so that reading of 1.80 m is obtained every time. Thus, in Figure the dots represent the points determined by this method explained above.



- **Horizontal Control:** After having located the points on various contours, they are to be surveyed with a suitable control system. The system to be adopted depends mainly on the type and extent of areas. For small area chain surveying may be used and the points may be located by offsets from the survey lines. In a work of larger nature, a traverse may be used. The traverse may be a Theodolite or compass or plane table traversing. In this method two survey parties generally work simultaneously.

  One locating the points on the contours and the other surveying those points. However if the work is of a small nature, the points may be located first and then surveyed by the same party. In figure the points shown by dots have been surveyed with respect to points A & B which may be tied by a traverse shown by chain dotted lines.

INDIRECT METHOD:

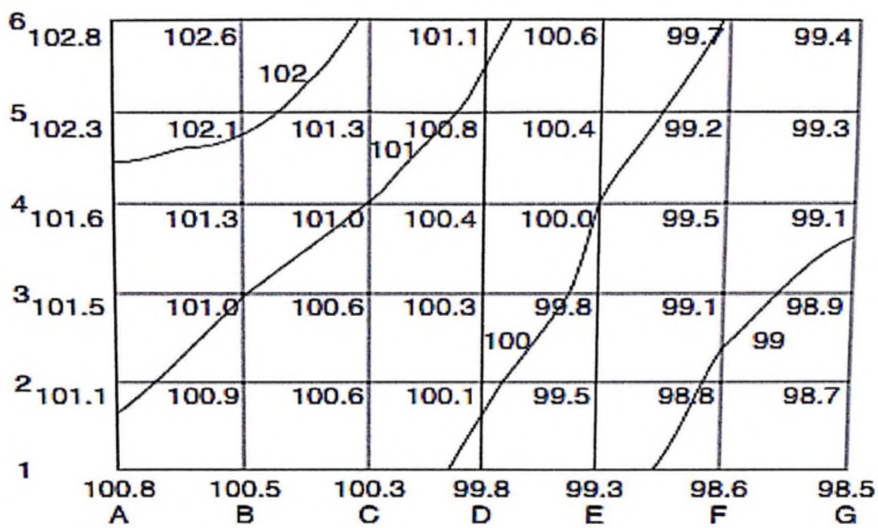
In this method some guide points are selected along a system of straight lines and their elevations are found. The points are then plotted and contours are then drawn by interpolation. These guide points are not, except by coincidence points on the contour to be located. While interpolating it is assumed that the slope between any two adjacent guide points is uniform.

The following are some of the indirect methods of locating the ground points.

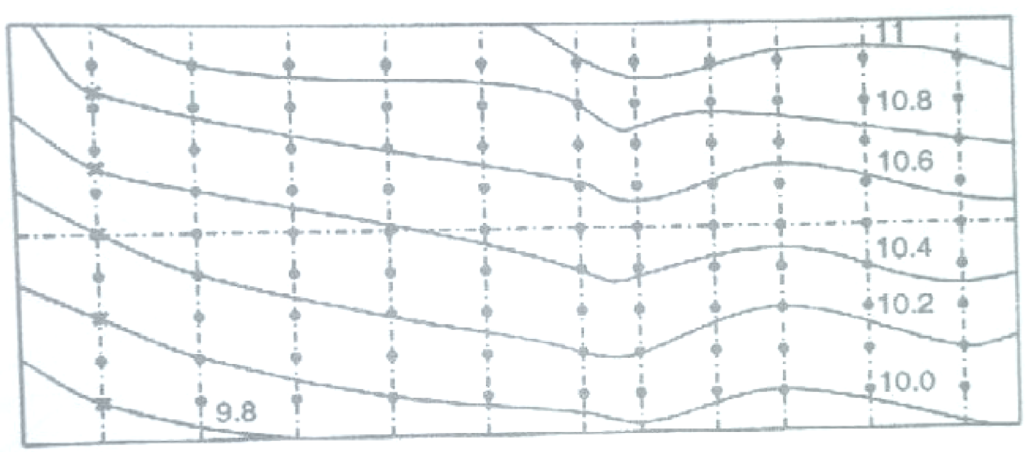
A. Grid Method.
B. Cross Section Method.
C. Radial Method.

✓ **Grid Method:**

The method is used when the area to be surveyed is small and the ground is not very much undulating. The area to be surveyed is divided into a number of squares. The size of the square may vary from 5 to 20 m depending upon the nature of the contour and contour interval. The elevations of the corners of the square are then determined by means of a level and a staff. The contour lines may then be drawn by interpolation. It is not necessary that the squares may be of the same size. Sometimes rectangles are also used in place of squares. When there are appreciable breaks in the surface between corners, guide points in addition to those at corners may also be used. The squares should be as long as practicable, yet small enough to conform to the in equalities of the ground and to the accuracy required. The method is also known spot leveling.



✓ **Cross Section Method:**

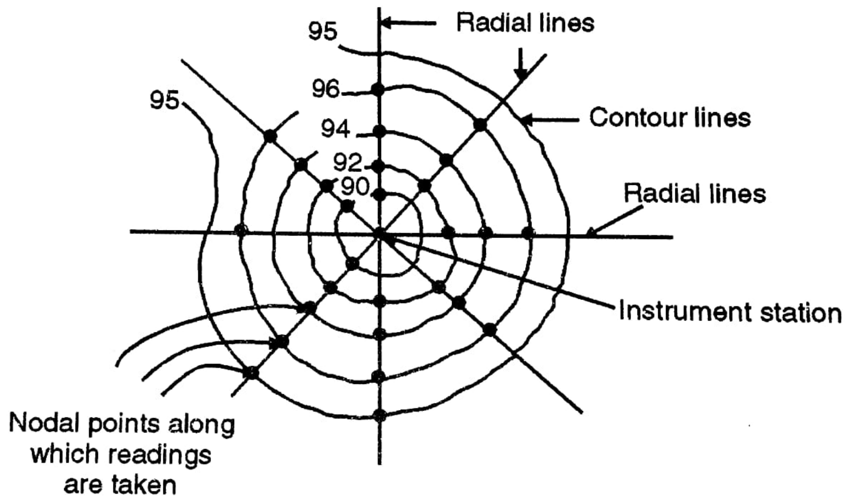


This method is used for determination of contours along a fixed route such as canal, road & railways. Cross sections are located on the ground at right angles to the center line. The spacing of cross section depends on nature of ground, the contour interval and purpose of contouring. The space should be kept small in ravines & spur, where contour changes its direction abruptly.

The elevation of guide points on a given cross section is determined using a leveling instrument. Staff readings should also be taken at points on breaksin ground surface and recorded with respective distances from fixed line or central line.

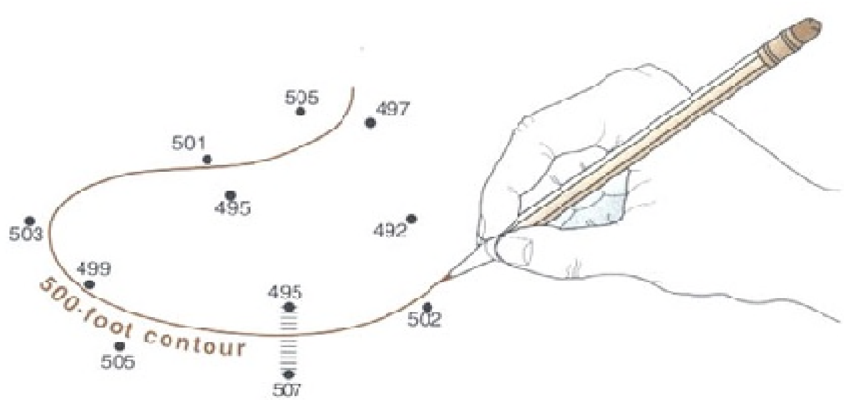
- ✓ **Radial Method:**

For contouring small hilly areas, radial lines are run from the peak to cover the area. The guide points are taken on the radial lines and their elevations are determined. The contour lines are drawn by interpolation as shown in figure.

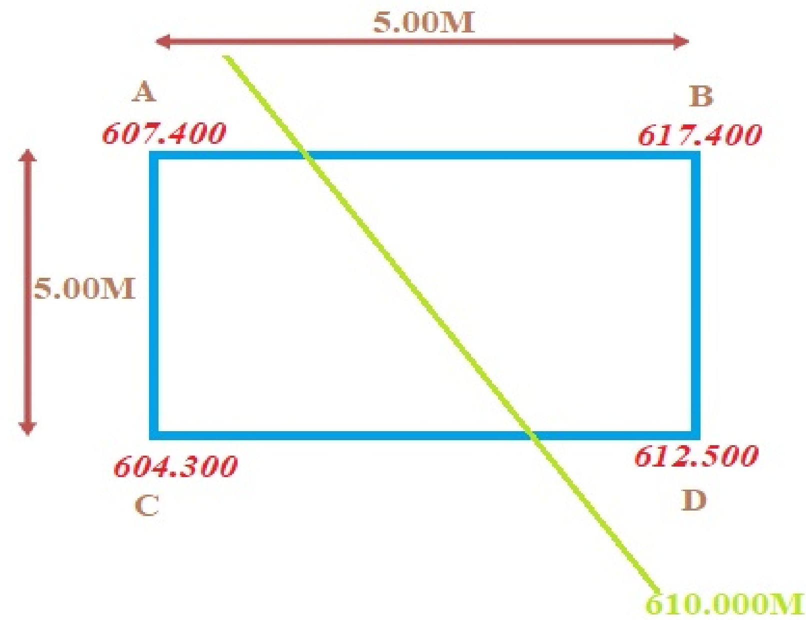


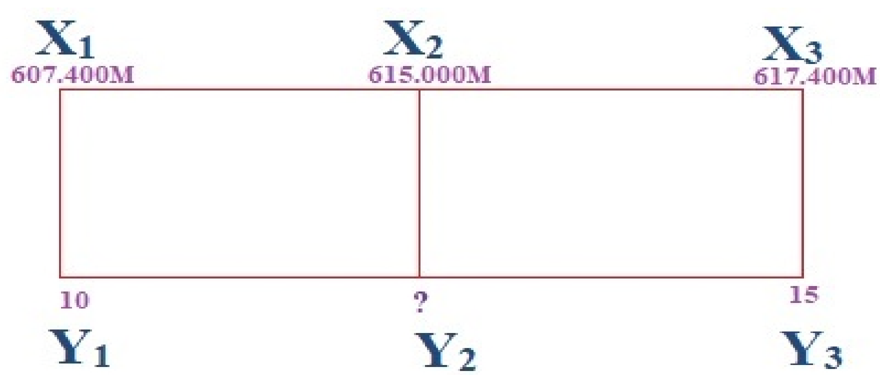
INTERPOLATION OF CONTOURS:

Interpolation of the contours is the process of spacing the contours proportionately between the plotted ground points established by indirect methods. The methods of interpolation are based on the assumption that the slope of ground between the two points is uniform. The chief methods of interpolations are by estimation, By Arithmetic calculations & by graphical Method.



ARITHMETIC CALCULATION: The Arithmetic method of interpolation of contour is accurate and also time consuming. The positions of contour points between the guide points are located by arithmetic calculations. For example A, B, C & D are the guide points having elevation of 607.400, 617.400, 604.300 & 612.500m respectively. Let AB = BC = CD = CA = 5.00M. For Example let us locate contour of RL 615.000m on line AB.





$$\frac{\mathbf{Y_2 - Y_1}}{\mathbf{Y_3 - Y_1}} = \frac{\mathbf{X_2 - X_1}}{\mathbf{X_3 - X_1}}$$

$$\frac{\mathbf{Y_2 - 10}}{\mathbf{15 - 10}} = \frac{\mathbf{615.000 - 607.400}}{\mathbf{617.000 - 607.400}}$$

$$\mathbf{Y_2 - 10} = \left(\frac{\mathbf{7.60 \times 5}}{\mathbf{9.60}}\right)$$

$$\mathbf{Y_2 = 13.96}$$

Similarly contour of RL 615.000m is located on line BD.

FACTORS AFFECTING CONTOUR INTERVAL:

The vertical distance between any two consecutive contours is called contour interval. The contour interval is kept constant for a contour map; otherwise in general appearance of the map will be misleading. The horizontal distance between two points on two consecutive contours is known as horizontal equivalent and depends upon the steepness of the ground. The choice of proper contour interval depends upon the following considerations.

a) THE NATURE OF THE GROUND:

The contour interval depends upon whether the country is flat or highly undulated. A contour interval chosen for a flat ground will be highly unsuitable for undulating ground. For every flat ground, a small interval is necessary. If the ground is more broken, greater contour interval should be adopted; otherwise the contours will come too close to each other.

b) THE SCALE OF THE MAP:

The contour interval should be inversely proportional to the scale. If the scale is small, the contour interval should be large. If the scale is large, the contour interval should be small. The following table suggests some suitable values of contour interval.

SCALE OF MAP	TYPE OF GROUND	CONTOUR INTERVALS (METERS)
LARGE **(1cm = 10m or less)**	FLAT ROLLING HILLY	0.20 TO 0.50 0.50 TO 1.00 1.00, 1.50 TO 2.00
INTERMEDIATE **(1cm = 10m or less)**	FLAT ROLLING HILLY	0.50, 1.00 OR 1.50 1.00, 1.50 OR 2.00 2.00, 2.50 OR 3.00
SMALL **(1cm = 10m or less)**	FLAT ROLLING HILLY MOUNTAINIOUS	1.00, 2.00 OR 3.00 2.00 TO 5.00 5.00 TO 10.00 10.00, 25.00 OR 50.00

c) THE PURPOSE AND EXTENT OF THE SURVEY:

The contour interval largely depends upon the purpose and the extent of the survey. For example, if the survey is intended for detailed design work or for accurate earth work calculations, small contour interval is to be used. The extent of survey in such cases will generally be small. In the case of location surveys for lines of communication and for reservoir and drainage areas, where the extent of survey is large, a large contour interval is to be used.

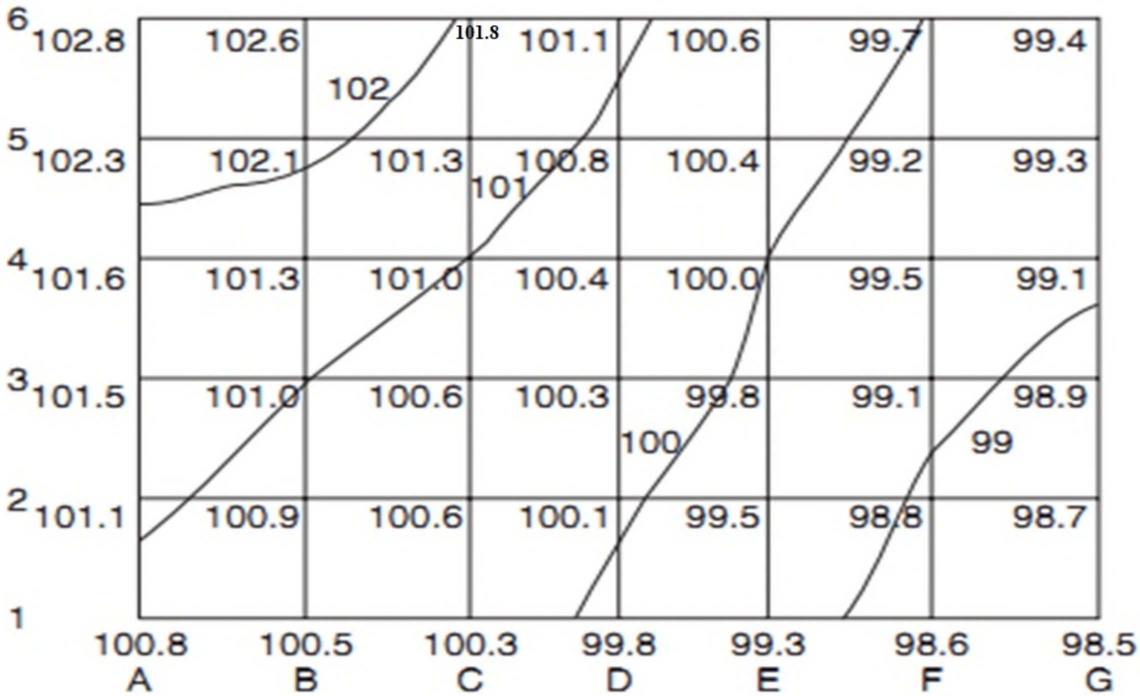
d) TIME AND EXPENSES OF FIELD AND OFFICE WORK:

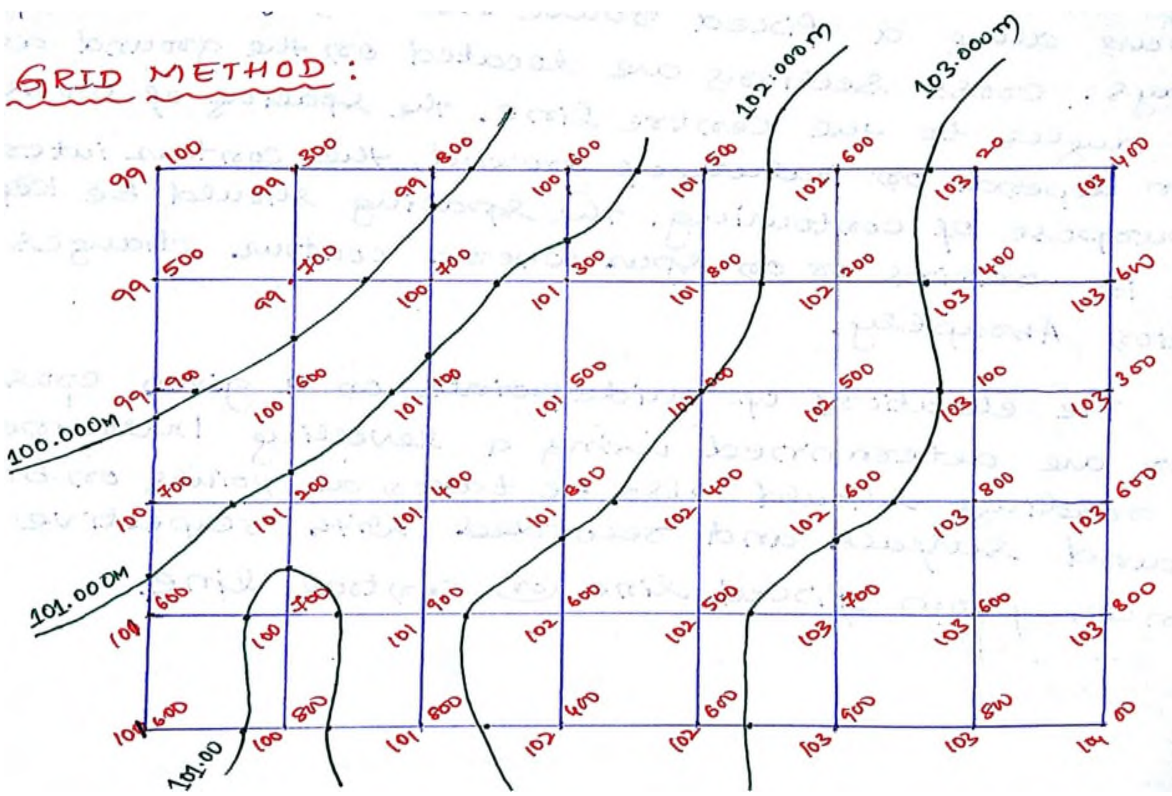
If the time available is less, greater contour interval should be used. If the contour interval is small, greater time will be taken in the field survey in reduction and in plotting of map.

Considering all these aspects the contour interval for a particular contour plan is selected. This contour interval is kept constant in that plan otherwise it will mislead the general appearance of the ground.

WEEK 01 CLASS 03

METHOD OF PREPARING CONTOUR MAP FROM THE GIVEN RLS OF GRID POINTS WITH EXAMPLES





PERFORMANCE CREITERIA WEEK - 01

SN	COURSE CONTENT
1.1	Conduct block contouring for a minimum area of 40 m x 40 m to draw its contour plan at a suitable contour interval.
1.2	Find out the area enclosed by the contours using AutoCAD.

WEEK 01 PRACTICAL 01

AIM: To conduct block contouring for a minimum area of 40 m x 40 m to draw its contour plan at a suitable contour interval.

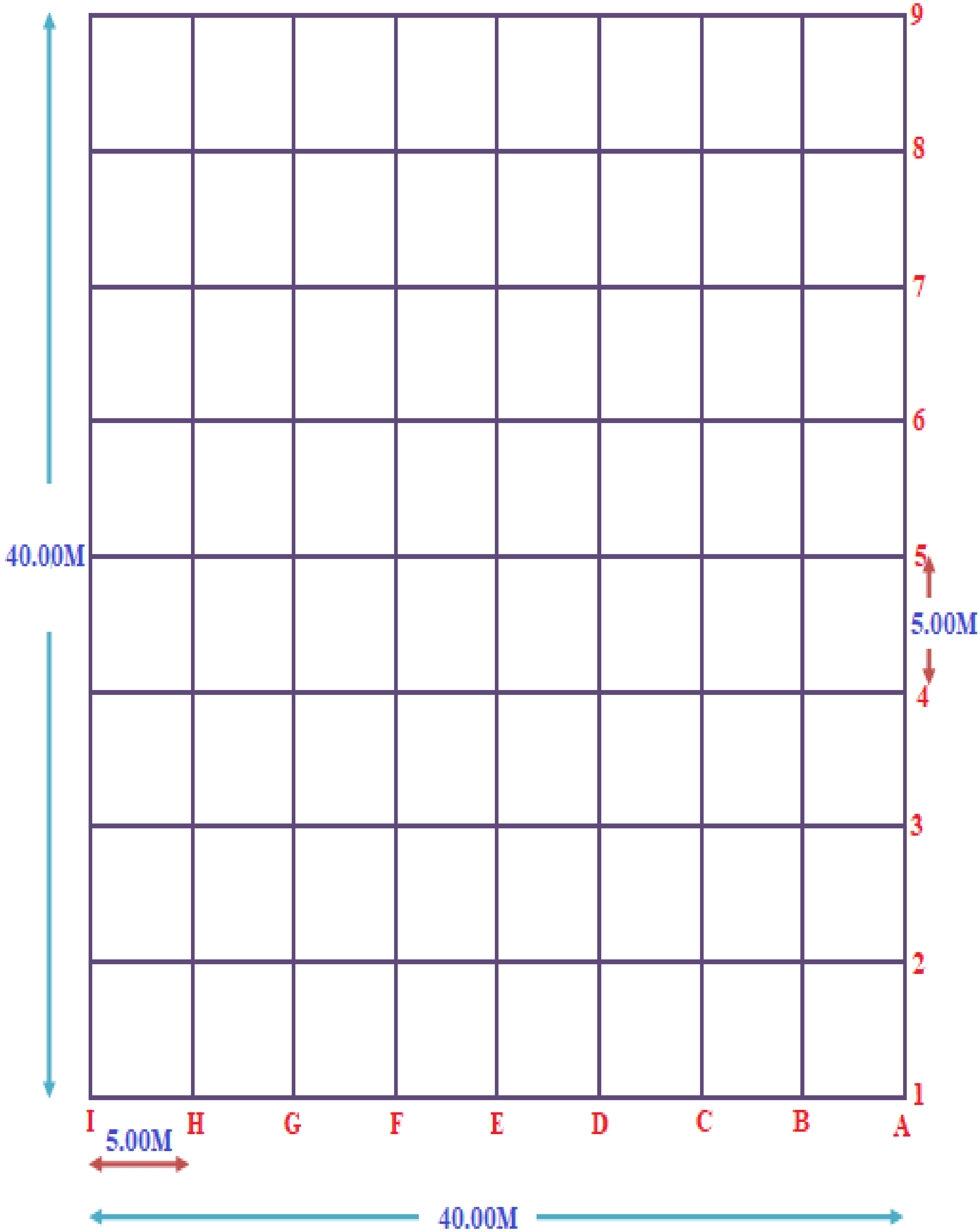
INSTRUMENTS USED

- Dumpy Level
- Ranging rods,
- Pegs or Arrows.
- Open cross staff.
- Levelling staff.

OBSERVATIONS:

LEVEL PAGE:

Back Sight	Intermediate Sight	Fore Sight	Height of Instrument	Reduced Level	Distance	Total Distance	Station	Remarks


9
8
7
6
5
4
3
2
1
40.00M
5.00M
I
H
G
F
E
D
C
B
A
5.00M
40.00M

PROCEDURE:

- Mark an area of 40M in length and 40M in width (1600M^2).
- Divide the marked area into equal grids say 5m by 5m using open cross staff, measuring tape, ranging road & arrows.
- At each corner of the grid points drive an arrow.
- Name the grid points from A to I in horizontal direction and 1 to 9 in vertical direction say A_1, A_2, A_3, A_4, A_5, A_6, A_7, A_8 & A_9.
- Select any suitable place for setting up of the instrument.
- Place the instrument exactly over a station point and do all necessary temporary adjustments (Centring, Levelling up & elimination of parallax).
- Take staff reading on given bench mark of known elevation.
- Take staff readings at every corner of the grid at an interval of 5m and note down in level page.
- Calculate the reduced levels of every corner of grid by rise and fall method or height of instruments method.
- Select suitable scale say 1:200 to draw 40m X 40m grid on drawing sheet.
- Transfer the calculated reduced levels of grid points to drawings from level page.
- Select suitable contour interval say 0.50m to draw contour map.
- By arithmetic method of interpolation calculate the distances and complete the contour map.

RESULT:

- Conducted block contouring for a minimum area of 40 m x 40 m and contour plan at suitable contour intervals is drawn.

WEEK 01 PRACTICAL 02

AIM: To find the area enclosed by contour using AutoCAD.

APPARATUS USED

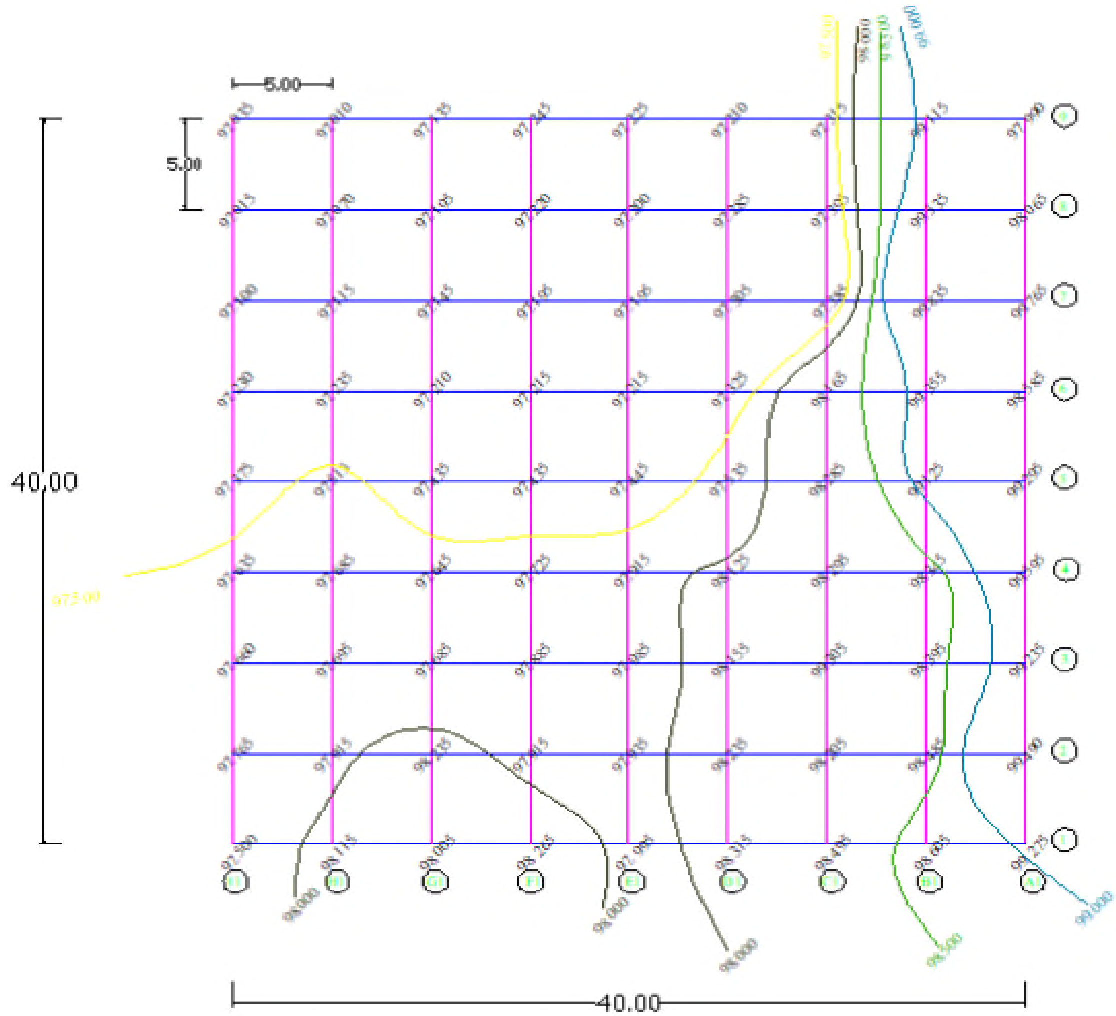
- Computer with accessories.
- AutoCAD.
- Printer.

PROCEDURE

- Open AutoCAD software, set limits and units by using appropriate command.
- By using LINE (L) command draw line of length 40m. Complete the grid of 40M by 40M by using line command with ORTHO (F8) on.
- By using OFFSET (O) command draw offsets at 5m interval in both horizontal and vertical directions.
- The intersection of offsets from both horizontal and vertical direction is referred as grd points.
- By using TEXT (MT) command type the value of reduced level of any point.
- By using ROTATE (RO) command rotate the text to desired direction say 45^0.
- By using COPY (CO) command copy the reduced level to each grid points and edit their values.
- By using SPLINE from DRAW menu draw contour lines.
- By using AREA command determine the area of enclosed contours.

RESULT:

- The area enclosed by a contour of RL_______ is _______M^2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KNOWLEDGE CRITERIA WEEK - 02

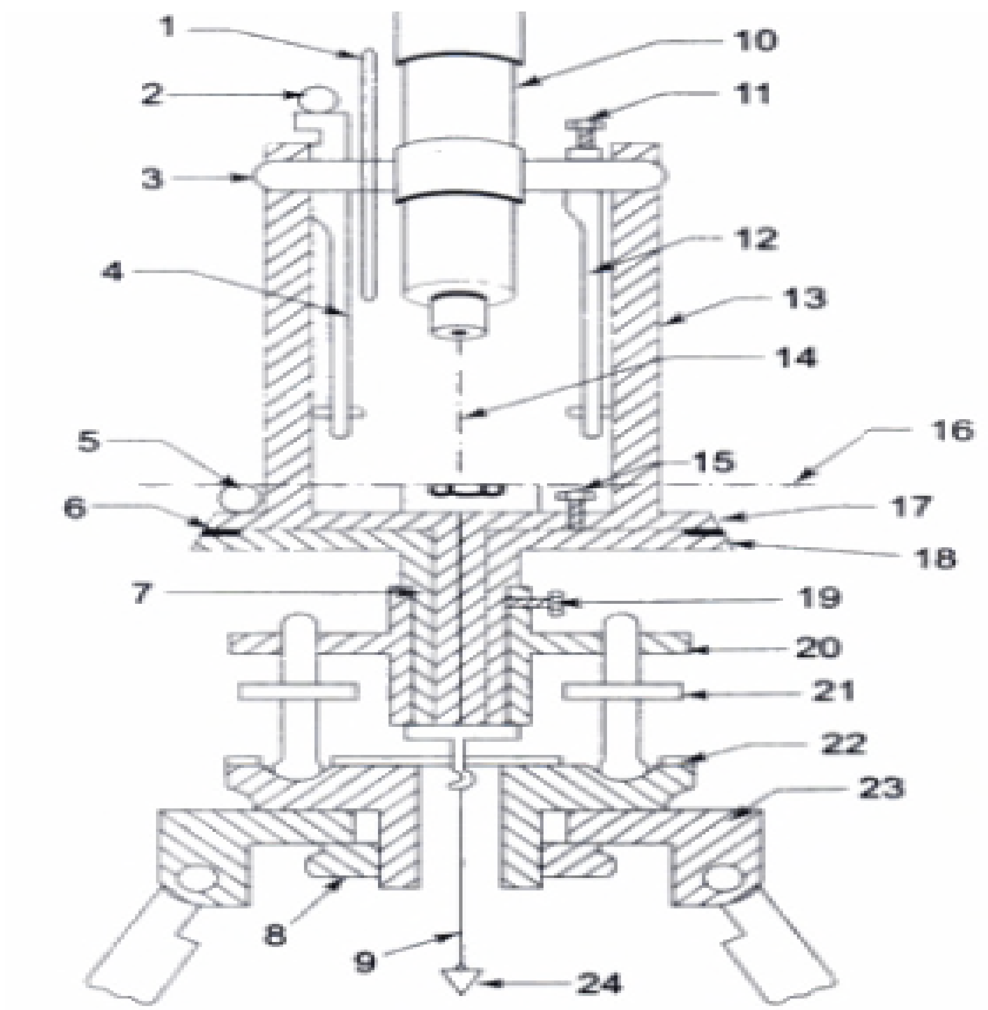
SN	COURSE CONTENT
2.1	Component parts of transit Theodolite and their functions.
2.2	Reading the Vernier, Salient features and relationship between the fundamental axes of transit Theodolite.
2.3	Technical terms used. Temporary adjustments.

WEEK 02 CLASS 01

COMPONENT PARTS OF TRANSIT THEODOLITE AND THEIR FUNCTIONS:

The Theodolite is the most intricate and accurate instrument used for measurement of horizontal and vertical angles. It consists of telescope by means of which distant objects can be sighted. The telescope has two distinct motions on in the horizontal plane and the other in the vertical plane. The former being measured on a graduated Horizontal vertical circle of two verniear.

A Theodolite is called transit Theodolite when its telescope can be resolved through a complete revolution about its horizontal axis. The transit type is largely used. Various parts of transit Theodolite are as shown in figure.



Transit Theodolite and its essential parts:

1. Vertical Circle
2. Altitude bubble
3. Horizontal axes
4. Vernier Arm
5. Plate bubble
6. Graduated Arc
7. Levelling Head
8. Clamping Nut
9. Vertical Axis
10. Telescope
11. Vertical circle clamping screw
12. Arm of the vertical circle clamp
13. Standard
14. Line of sight
15. Upper plate clamping screw
16. Axis of plate bubble
17. Upper plate
18. Lower plate
19. Lower plate clamping screw
20. Tribranch
21. Foot screw
22. Trivet
23. Tripod top
24. Plumb bob

TELESCOPE: It is an integral part and is mounted on the spindle known as horizontal axis or turn on axis. Telescope is either internal or external focusing type. In most of the transits, internal focusing telescope is used.

THE LEVELLING HEAD: The levelling head consists of two parallel triangular planes known as tribranch plates. It may consists of circular plates called as upper and lower Parallel plates. The lower parallel plate has a central aperture through which a plumb bob may be suspended. The upper parallel plate or tribranch is supported by means of four or three levelling screws by which the instrument may be levelled. A levelling head has three distinctive functions.

- To support the main part of the instrument.
- To attach the Theodolite to the tripod.
- To provide a mean for levelling the Theodolite.

THE STANDARDS or A FRAME: Two standards resembling the letter A are mounted on the upper plates. The trunnion axis of the telescope is supported on these. The T frame and the arm of vertical circle clamp are also attached to the A Frame.

LOWER PLATE OR SCREW PLATE: The lower plate is attached to the outer spindle. It carries horizontal circle at its levelled screw. It carries a lower clamp screw and tangent screw with the help of which it can be fixed accurately in any desired position. When the clamp is tightened the lower plate is fixed to the upper tribranch of the levelling head. On turning the tangent screw the lower plate can be rotated slightly. Usually the size of a Theodolite is represented by size of the scale plate. That is 10cm or 12cm Theodolite.

THE UPPER PLATE OR VERNIER PLATE: The upper plate is attached to inner axis and carries two verniers with magnifiers at two extremities diametrically opposite. The upper plate supports the standards. It carries an upper clamp screw and corresponding tangent screw for the purpose of accurately fixing it to the lower plate. On clamping the upper and unclamping the lower clamp, the instrument can rotate on its outer axis without any relative motion between the two plates.

INDEX FRAME: The index frame is a T-shaped frame consisting of a vertical leg known as clipping arm and a horizontal bar known as Vernier arm of index corm. At the two extremities of the index arm are fitted two verniers to read the vertical circle. The index arm is centered on the trunnion axis in front of the vertical circle and remains fixed. When the telescope is moved in the vertical plane, the vertical circle moves relative to the verniers with the help of which the reading can be taken. Glass magnifiers are placed in front of each Vernier to magnify the reading. A long sensitive bubble tube, sometimes known as altitude bubble s placed on the top of the index frame.

VERTICAL CIRCLE: The vertical circle is circular graduated arc attached to the trunnion axis of the telescope. Consequently the graduated arc rotates with the telescope when the latter is turned about the horizontal axis. By means of vertical circle clamp and its corresponding slow motion screw or tangent screw. The telescope can be set accurately to any desired position in vertical plane. The circle is either graduated continuously from 0^0 to 360^0 n clock wise direction or it is usually divided into four quadrants. Clamping the vertical circle restricts the movement of telescope in vertical plane.

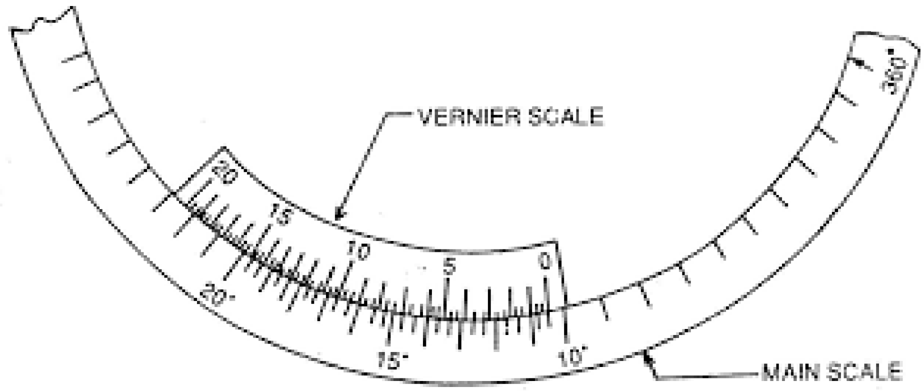
CLAMPS AND TANGENT SCREWS: There are two clamps and associated tangent screws with the plate. These screws facilitate the motion of the instruments in horizontal plane. Lower clamp screw locks or releases the lower plate. When this screw is unlocked both upper and lower plates move together. The associated lower tangent screw allows small motion of the plate in locked position. The upper clamp screw locks or releases the upper Vernier plate. When this clamp is released the lower plate does not move but the upper Vernier plate moves with the instrument. This causes the change in the reading. The upper tangent screw allows the fine adjustment

ALTITUDE LEVEL: A highly sensitive bubble is used for levelling particularly when taking the vertical angle observations

PLUMB BOB: To centre the instrument exactly over a station mark, a plumb bob is suspended from the hook fitted to the bottom of the central vertical axis.

WEEK 02 CLASS 02

READING THE VERNIER:



There are two circular plates considered on Vernier Theodolite.

- **Lower plate:** Outer axis is attached with a lower plate and the lower plate is graduated from 0 to 360 in a clockwise direction. The lower plate is called the main scale .the lower plate is provided with a clamp or slow-motion screw. When the observation is taken then the lower plate is tightened with a lower clamp or tangent screw. In the above picture shown graduation of the lower circle are two types, one is big division and the other is small division. The value of one big division is 1° because number of big division in the lower plate is 360. The one big division or 1° is divided into three small divisions. Then the value of one small division on the main scale 1° or 60 '÷ 3 = 20'

- **Upper plate:** The upper plate is called the vernier plate. It is attached with inner axis and a clamp and A tangent screw provided for fixing the vernier plate to the main scale. There are two vernier plates placed apart 180° on the upper plate with two magnifiers. In the above picture, the vernier plate is divided, 20 big divisions. The value of 1 big division on the vernier plate is 1'. The big division is divided into three small division then the value of 1 small division will be 1'÷3 or 60" ÷3=20".

  Reading the above picture ----> First saw on main scale reading,
  The vernier indicator 0 is crossed between 9° 40' and 10 °. Next, we saw vernier reading, the 14' of big division of the vernier scale is exact coincides with the main scale. Then the vernier scale reading will be 14'0". Here is division of second is not Coincides with the main scale. Total reading of the above picture is 9°40'+14'0"=9°54'0". The same procedure does on vernier B. True reading of vernier Theodolite will be = Reading of vernier A + Reading of vernier B ÷2. Least count = Value of 1 small division of main

scale ÷ total no. of division on the vernier scale --> 20'×60" ÷ 60=20" Then the least count of above picture of the instrument =20"

SALIENT FEATURES & RELATIONSHIP BETWEEN FUNDAMENTAL AXES OF TRANSIT THEODOLITE

The fundamental axes or lines of a transit Theodolite are,

- The vertical axis.
- The horizontal axis.
- The line of collimation.
- Axis of plate level.
- Axis of altitude level.
- Axis of striding level.

The figure shows the relationship between the line of sight, the axes and the circles of the Theodolite. The following relationship should exist.

1. ***The axis of the plate level must lie in a plane perpendicular to the vertical axis.***

   If this condition exists, the vertical axis will be truly vertical when the bubble is in the centre of its run.

2. ***The line of collimation must be perpendicular to the horizontal axis at its intersection with the vertical axis. Also if the telescope is external focusing type, the optical axis, the axis of the objective slide and the line of collimation must coincide.***

   If this condition exists, the line of sight will generate a vertical pane when the telescope is rotated about the horizontal axis.

3. ***The horizontal axis must be perpendicular to the vertical axis.***

   If this condition exists, the line of sight will generate a vertical pane when the telescope is plunged.

4. ***The axis of the altitude level must be parallel to the line of collimation.***

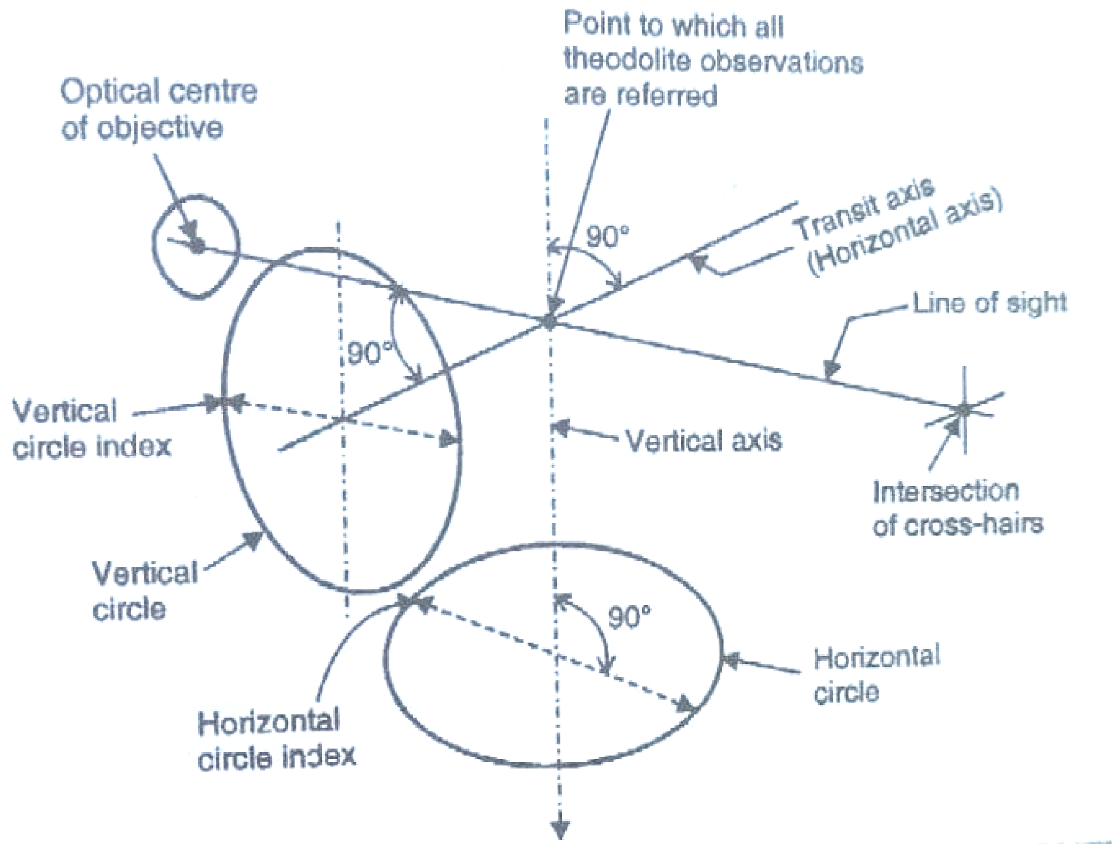
   If this condition exists, the vertical angles will be free from index error due to lack of parallelism.

5. ***The vertical circle must read zero when the line of collimation is horizontal.***

   If this condition exists, the vertical angles will be free from index error due to displacement of the Vernier.

6. ***The axis of the striding level must be parallel to the horizontal axis.***

   If this condition exists, the line of sight will generate a vertical plane when the telescope is plunged, the bubble of striding level being in the centre of its run.


Point to which all theodolite observations are referred
Optical centre of objective
90°
Transit axis (Horizontal axis)
Line of sight
90°
Vertical circle index
Vertical axis
Intersection of cross-hairs
Vertical circle
90°
Horizontal circle
Horizontal circle index

WEEK 02 CLASS 03

TECHNICAL TERMS USED

- **THE VERTICAL CIRCLE:**

  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 plate rotates.

- **THE HORIZONTAL CIRCLE:**

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

- **THE LINE OF SIGHT:**

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

- **THE AXIS OF LEVEL TUBE:**

  The axis of 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.

- **CENTRING:**

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

- **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.

- **SWINGING THE TELESCOPE:**

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

- **FACE LEFT OBSERVATION:**

  If the face of the vertical circle is to the left side of the observer, then the observation is termed as face left observation.

- **FACE RIGHT OBSERVATION:**

  If the face of the vertical circle is to the right side of the observer, then the observation is termed as face right observation.

- **CHANGING FACE:**

  The operation of bringing the face of the telescope from left to right or vice versa is termed as changing face.

- **TELESCOPE NORMAL:**

  A telescope is said to normal or direct when the face of the vertical circle is to the left and the bubble of the telescope up.

- **TELESCOPE INVERTED:**

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

TEMPORARY ADJUSTMENT

Temporary adjustments or station adjustments are those which are made at every instrument setting and preparatory to taking observations with the instrument. The temporary adjustments are

a. Setting over the station.
b. Levelling up
c. Elimination parallax.

a. **Setting up:** The operation of setting up includes:

   a. Centring of the instrument over the station mark by a plumb bob or by optical plummet.
   b. Approximate levelling with the help of tripod legs. Some instruments are provided with shifting head with the help of which accurate centring can be done easily. By moving the leg radially, the plumb bob is shifted in the direction of the leg while by moving the leg circumferentially or sideways considerable change in the inclination is effected without disturbing the plumb bob. The second movement is, therefore, effective in the approximate levelling of the instrument. The approximate levelling is done either with reference to a small circular bubble provided on tribranch or is done by eye judgment.

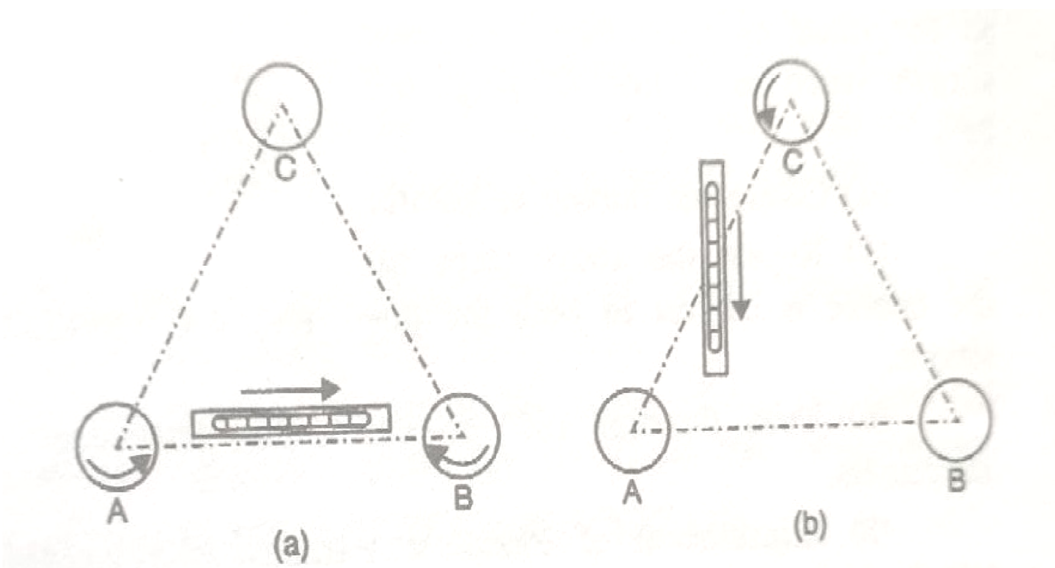
b. **Levelling up:** After having centered and approximately levelled the instrument. accurate levelling is done with the help of foot screws and with reference to the plate levels. The purpose of the levelling is to make the vertical axis truly vertical. The manner of levelling the instrument by the plate levels depends upon whether there are three levelling screws or four levelling screws.

   Three Screw Head

- Turn the upper plate until the longitudinal axis of the plate level is roughly parallel to a line joining any two (such as A and B) of the levelling screws.

- Hold these two levelling screws between the thumb and first finger of each hand and turn them uniformly so that the thumbs move either towards each other or away from each other until the bubble is central. It should be noted that the bubble will move in the direction of movement of the left thumb.
- Turn the upper plate through 90 °, i.e., until the axis of the level passes over the position of the third levelling screw C.
- Turn this levelling screw until the bubble is central.
- Return the upper plate through 90 ° to its original position and repeat step 2 till the bubble is central.
- Turn back again through 90 ° and repeat step 4.
- Repeat steps 2 and 4 till the bubble is central in both the positions.
- Now rotate the instrument through 180 °. The bubble should remain in the centre of its run, provided it is in correct adjustment. The vertical axis will then be truly vertical. If not, it needs permanent adjustment.

Note. It is essential to keep to the same quarter circle for the changes in direction and not to swing through the remaining three quarters of a circle to the original position. If two plate levels are provided in the place of one, the upper plate is not turned through 90 ° as is done in step (2) above. In such a case, the longer plate level is kept parallel to any two foot screws, the other plate level will automatically be over the third screw. Turn the two foot screws till the longer bubble is central. Turn now the third foot screw till the other bubble is central. The process is repeated till both the bubbles are central. The instrument is now rotated about the vertical axis through a complete revolution. Each bubble will now traverse, i.e., remain in the centre of its run, if they are in adjustment.



c. **Elimination of Parallax:** Parallax is a condition arising when the image tor by the objective is not in the plane of the cross-hairs. Unless parallax is eliminated, accur sighting is impossible. Parallax can be eliminated in two steps:

a. **By Focusing the eye piece for distinct vision of the cross-hairs:** To focus the eye-piece for distinct vision of the cross-a point the telescope towards the sky (or hold a sheet of white paper in front of the object and move eye-piece in or out till the cross-hairs are seen sharp and distinct. In et telescopes, graduations are provided at the eye-piece end so that one can always remember the particular graduation position to suit his eyes of her. This may save a lot of time.

b. **By focusing the objective to bring the image of the object in the plane of cross-hairs:** The telescope is now directed towards the object t sighted and the focusing screw is turned till the image appears clear and sharp. The so formed is in the plane of cross-hairs.

PERFORMACE CRITERIA WEEK - 02

SN	COURSE CONTENT
2.1	Measure horizontal angle between the given points
2.2	Measure Vertical angle between the given points

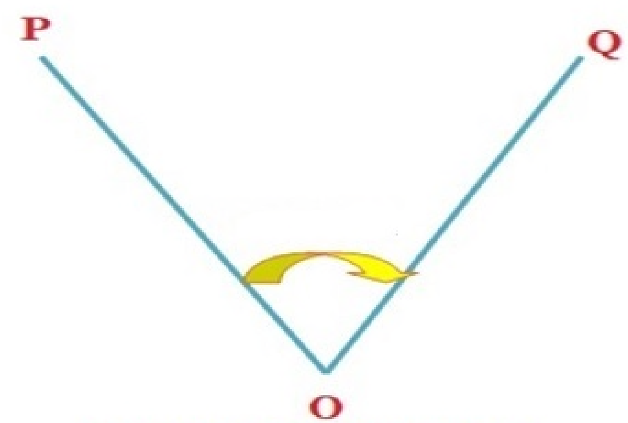
WEEK 02 PRACTICAL 01

AIM: To measure the horizontal angle between the given point with the use of Theodolite.

INSTRUMENTS USED

- Theodolite,
- Ranging rods,
- Pegs or Arrows.

OBSERVATIONS:



MEASURMENT OF HORIZONTAL ANGLE

FACE:LEFT												SWING:RIGHT	FACE:RIGHT										SWING:RIGHT	Average Horizontal Angle		
INST @	Sighted to	A			B		Mean			Horizontal Angle			A			B		Mean			Horizontal Angle					
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
O	P	0	0	0	0	0	0	0	0				0	0	0	0	0	0	0	0						
	Q																									

PROCEDURE:

- Theodolite is set over on instrument station (O) exactly and all the temporary adjustments are done. Vertical circle is placed left to the observer (face left observation).
- Vernier A is set to Zero with the help of upper clamp screw and tangent screws. Readings of Vernier A and B are noted.
- Upper clamp is clamped. Lower clamp is loosened and the telescope is turned towards “P”. Lower clamp is clamped and the point “P” is bisected exactly using tangent screws.

- Both the vernier A and B are read and noted (Must be equal to 0° and 180° respectively).Upper clamp is unclamped and the telescope is turned clockwise and "Q" is bisected.
- Upper clamp is clamped and "Q" is bisected exactly using tangent screws. Both the verniers are read. Mean of the readings provide an approximate included angle of POQ.
- The reading of vernier A gives directly the angle POQ and 180° is subtracted by the reading of vernier B. The mean value of two readings gives the angle POQ with one face.
- The face is changed and the whole process is repeated. (Face right observations).
- Average value of two horizontal angles obtained with face left and face right observations is determined.

RESULT:

The horizontal angle measured at O between P and Q = _______

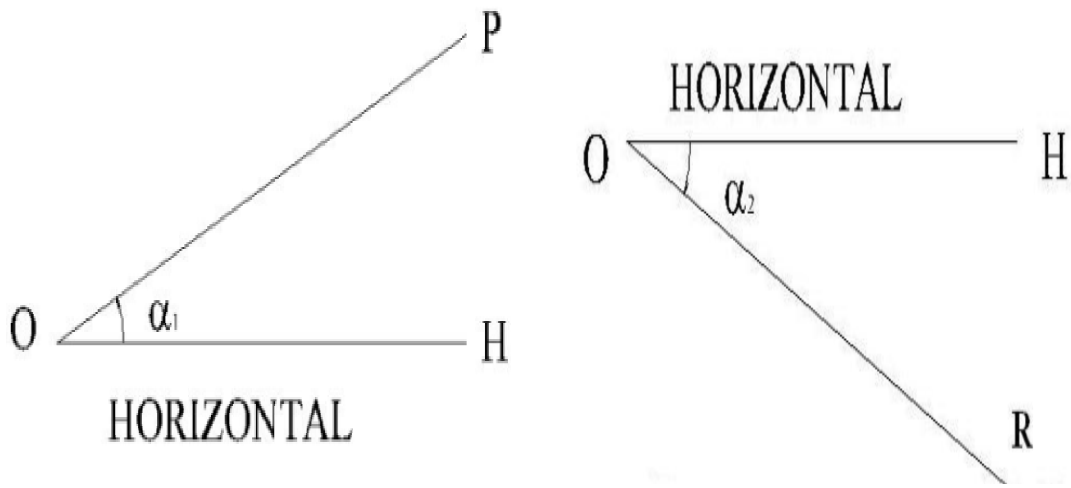
WEEK 02 PRACTICAL 02

AIM: To measure the Vertical angle between the given two points with the use of Theodolite.

INSTUMENTS USED

- Theodolite,
- Ranging rods,
- Pegs or Arrows.

OBSERVATION



ANGLE OF ELEVATION

FACE:LEFT												SWING:RIGHT	FACE:RIGHT											SWING:RIGHT	Average		
INST @	Sighted to	C			D		Mean			Vertical Angle			C			D		Mean			Vertical Angle			**Vertical Angle**		
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
O	H	0	0	0	0	0	0	0	0				0	0	0	0	0	0	0	0						
	P																									

ANGLE OF DEPRESSION

FACE:LEFT												SWING:RIGHT	FACE:RIGHT											SWING:RIGHT	Average		
INST @	Sighted to	C			D		Mean			Vertical Angle			C			D		Mean			Vertical Angle			**Vertical Angle**		
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
O	H	0	0	0	0	0	0	0	0				0	0	0	0	0	0	0	0						
	Q																									

PROCEDURE

- Theodolite is set up, centered and levelled with reference to the plate bubble.
- Telescope is placed horizontally by setting the reading of 0°0’0”in the verniers C and D.
- Levelling process is carried out with the help of foot screws and the altitude bubble is brought in its central run.
- Vertical circle clamp is loosened and the telescope is directed upwards to bisect P.
- Vertical circle clamp is clamped and the point P is exactly bisected using vertical tangent screws.
- Both the verniers of C and D are read and noted. Mean of the two verniers provide the vertical angle HOP.
- Face is changed and all the above steps are repeated to get one more vertical angle HOP.
- Average of the vertical angles taken to get an accurate vertical angle.
- The same procedure may be adopted to determine the angle of depression HOR by directing the telescope downwards.

RESULT:

- The observed angle of elevation is = ________
- The observed angle of depression is = ________

KNOWLEDGE CRETERIA WEEK - 03

SN	COURSE CONTENT
3.1	Theodolite traversing. Open and Closed Traverse.
3.2	Theodolite traversing by included angle method and deflection angle method.
3.3	Checks for open and closed traverse, Calculation of bearing from angles. Traverse computation - Latitude, Departure.

WEEK 03 CLASS 01

THEODOLITE TRAVERSING:

A traverse is a series of connected lines whose lengths and directions are measured in the field work. In this Theodolite traverse for linear measurements the tape is generally used and the directions are measured with a Theodolite. The field work in Theodolite traverse consists of Reconnaissance, Selection and marking of stations, Measurement of traverse line, Angular measurement and Picking up available details.

PURPOSE OF THEODOLITE TRAVERSING:

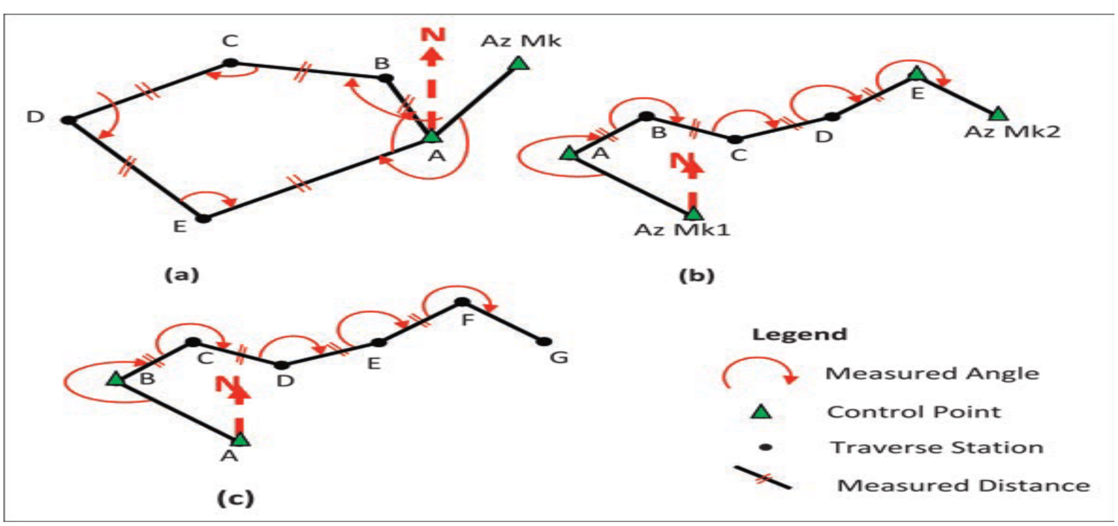
- To establish the position of new boundary lines.
- To determine the position of existing boundary marks.
- To provide control points for locating highways, railways.
- To fix the alignment of roads, canals & rails.
- To calculate the area enclosed within the boundary.
- To have the ground control for photogrammetric survey.

OPEN TRAVERSE:

If the circuit ends elsewhere other than starting point it is said to be open traverse. An open traverse is generally used for surveying a long strip of the area for a road, railway, canal etc. an open traverse cannot be properly checked and corrected.

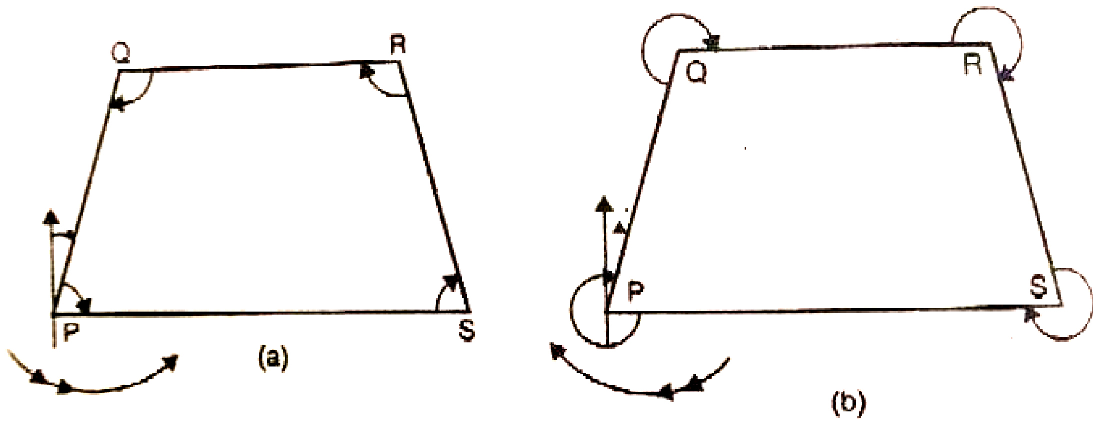
CLOSED TRAVERSE:

When the line forms a circuit which ends at the starting point is known as closed traverse. An closed traverse is generally used in control survey, construction survey and topographic survey. As for as possible the link type closed traverse should be used to have a complete check on the measured angles and distances.



WEEK 03 CLASS 02

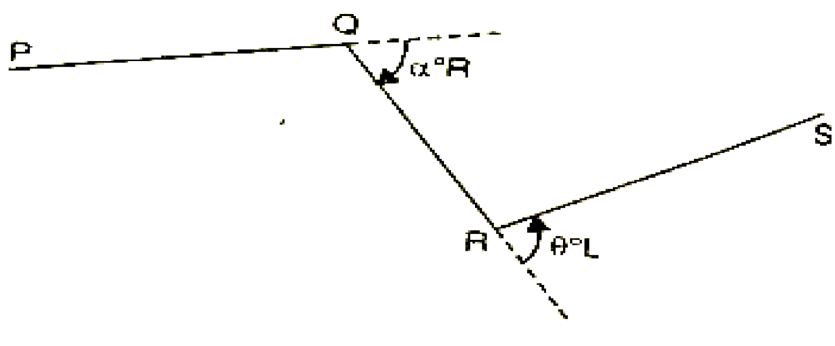
THEODOLITE TRAVERSING BY INCLUDED ANGLE METHOD



An included angle at a station is either of the two angles formed by the two survey lines meeting measuring there. The method consists simply in measuring each angle directly from a back sight on the preceding station. The angles may also be measured by repetition, if so desired. Both face observations must be taken and both the verniers should be read. Included angles can be measured either clockwise or counter-clockwise but it is better to measure all angles clockwise, since the graduations of the Theodolite circle increase in this direction.

The angles measured clockwise from the back station may be interior or exterior depending upon the direction of progress round the survey. Thus in Figure (a) direction of progress is counter-clockwise and hence the angles measured clockwise are directly the interior angles. In Figure (b) the direction of progress around the survey is clockwise and hence the angles measured clockwise are exterior angles

THEODOLITE TRAVERSING BY DEFLECTION ANGLE METHOD



A deflection angle is the angle which a survey line makes with the prolongation of the preceding line. It is designated as Right (R) or Left (L) according as it is measured to the clockwise or to anticlockwise from the prolongation of the previous line. Its value may vary from 0^0 to 180^0. The deflection angle at Q is α^0 Right and that at R is θ^0 Left as shown in figure.

- Setup the instrument at starting station P, level it & Center it accurately and measure the magnetic bearing of line PQ.
- Set the instrument at station Q and level it accurately.
- Set the vernier to read Zero by using upper clamp and tangent screw.
- Unclamp the lower plate and rotate the telescope to take a back sight on P with both plates clamped.
- Plunge the telescope. Thus the line of sight is in the direction PQ produced when the reading on vernier A is zero.
- Unclamp the upper plate and turn the telescope in clockwise direction to take a fore sight on R by reading both verniers.
- Unclamp the lower plate and turn the telescope to sight P again. The verniers still read the same reading as in the above point, plunge the telescope.
- Unclamp the upper plate and turn the telescope to sight R. read both verniers. Since the deflection angle is doubled by taking both face readings, half of the final reading gives the deflection angle at Q.
- Repeat the process for all the stations.

WEEK 03 CLASS 03

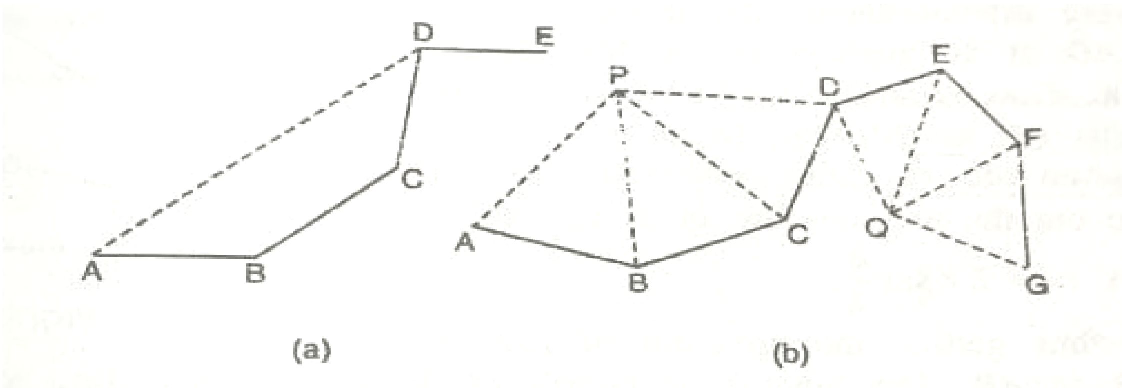
CHECKS FOR OPEN AND CLOSED TRAVERSE:

CHECKS IN OPEN TRAVERSE:

No direct check of angular measurement is available. However indirect checks can be made as illustrated in figure.

As illustrated in figure (a), in addition to the observation of bearing of AB at station A, bearing of AD can also be measured. If possible similarly at D bearing of DA can be measured and check is applied. If the two bearings differ by 180^0 the work up to D may be accepted as correct. If there is small discrepancy, it can be adjusted before proceeding further.

Another method which furnishes a check when the work is plotted is as shown in figure (b) and consists in reading the bearings to any prominent point P from each of the consecutive stations. The check in plotting consists in laying off the lines AP, BP, CP and noting whether the lines pass through one point. In case of long and precise traverse the angular errors can be determined by astronomical observations for bearings at regular intervals during the progress of the traverse.



CHECKS IN CLOSED TRAVERSE:

The errors involved in traversing are two kinds: linear and angular. For important work the most satisfactory method of checking the linear measurements consists in chaining each survey line second time, preferably in the reverse direction on different dates and by different parties. The following are the checks for the angular work.

A. Travers by Included Angles:

a. The sum of measured interior angles should be equal to $(2N - 4)$right angles, where N is Number of sides of the traverse.

b. The sum of measured exterior angles should be equal to $(2N + 4)$right angles, where N is Number of sides of the traverse.

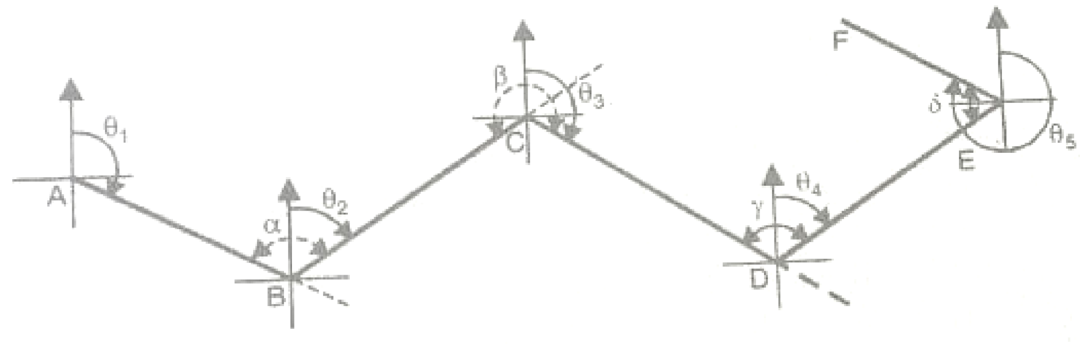
B. Travers by deflection angles:

The algebraic sum of the deflection angles should be equal to 360^0, taking the right hand deflection angles as positive and left hand angles as negative.

C. Traverse by direct observation of bearings:

The fore bearing of the last line should be equal to its back bearing $\pm 180^0$ measured at the initial station.

CALCULATION OF BEARINGS FROM ANGLES:



In the case of traverse in which included angles between successive lines have been measured, the bearings of the lines can be calculated provided the bearing of any one line is also measured. Referring to the figure let $\alpha, \beta, \gamma, \delta$ are the included angles measured clockwise from back stations and θ_1 be the measured bearing of the line AB.

$$\text{The bearing of the next line BC} = \theta_2 = \theta_1 + \alpha - 180°$$

$$\text{The bearing of the next line CD} = \theta_3 = \theta_2 + \beta - 180°$$

$$\text{The bearing of the next line DE} = \theta_4 = \theta_3 + \gamma - 180°$$

$$\text{The bearing of the next line EF} = \theta_5 = \theta_4 + \delta + 180°$$

As a evident from figure **$(\theta_1+ \alpha)$, $(\theta_2+ \beta)$ and $(\theta_3+ \gamma)$** are more than 180^0 while **$(\theta_4+ \delta)$** is less than 180^0. Hence in order to calculate the bearing of the next line, the following statement can be made:

"Add the measured clockwise angles to the bearing of the previous line. If the sum is more than 180^0, deduct 180^0. If the sum is less than 180^0, add 180^0"

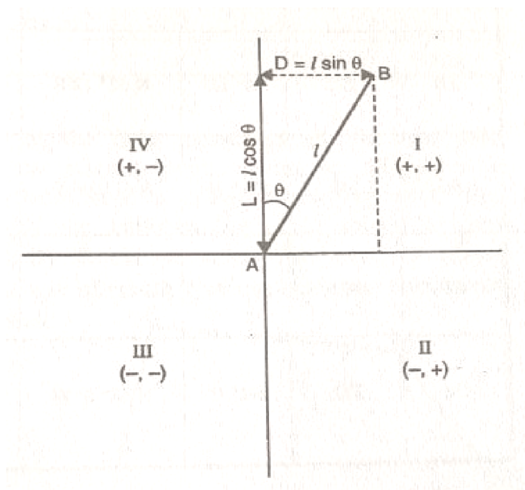
In a closed traverse, clockwise angles will be obtained if we proceed round the traverse in the anticlockwise direction.

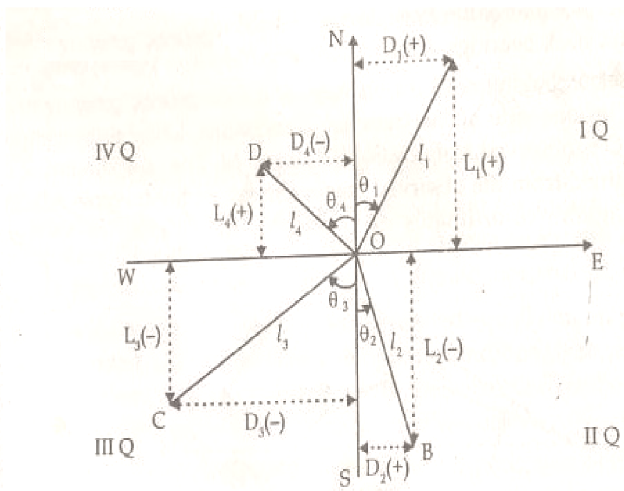
TRAVERSE COMPUTATION:

CONSECUTIVE COORDINATES: LATTITUDE AND DEPARTURE

The latitude of survey line may be defined as its coordinate length measured parallel to an assumed meridian direction (true north or magnetic north or any other reference direction). The latitude of the line is positive when measured northward (Upward) and is termed as northing. The latitude of the line is negative when measured southward (Downward) and is termed as southing.

The Departure of survey line may be defined as its coordinate length measured right angles to an assumed meridian direction. The departure of the line is positive when measured eastward and is termed as easting. The departure of the line is negative when measured westward and is termed as westing.





In above figure the latitude and departure of the line AB of the length ***l*** and reduced bearing **θ** are given by $\boldsymbol{Lattitude = L = l\,\mathrm{Cos}\,\theta}$ and $\boldsymbol{Departure = D = l\,\mathrm{Sin}\,\theta}$

To calculate the latitudes and departure of the traverse lines, it is first essential to reduce the bearing in the quadrant system. The sign of latitudes and departures will depend upon the reduced bearing of a line. Following table gives signs of latitude and departures.

Whole Circle Bearing	**Reduced or Quadrant Bearing**	**Sign of**	
		Latitude	**Departure**
0^0 to 90^0	**N θ E : I**	+	+
90^0 to 180^0	**S θ E : II**	-	+
180^0 to 270^0	**S θ W : III**	-	-
270^0 to 360^0	**N θ W : IV**	+	-

PERFORMANCE CRITERIA WEEK - 03

SN	COURSE CONTENT
3.11	Measure horizontal angle by repetition method.
3.12	Measure horizontal angle by reiteration method.
3.2	Carry out survey project for closed traverse for minimum 5 sides by locating details using Theodolite.

WEEK 03 PRACTICAL 01

AIM: To measure the horizontal angle by Repetition method with the use of Theodolite.

INSTRUMENTS USED

- Theodolite,
- Ranging rods,
- Pegs or Arrows.

OBSERVATIONS:

MEASURMENT OF HORIZONTAL ANGLE BY REPETITION METHOD

FACE:LEFT										SWING:RIGHT	FACE:RIGHT										SWING:RIGHT	Average Horizontal Angle				
INST @	Sighted to	A			B		Mean			Horizontal Angle			A			B		Mean			Horizontal Angle					
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
O	P																									
	Q																									
O	P																									
	Q																									
O	P																									
	Q																									

PROCEDURE:

- Theodolite is set over on instrument station (O) exactly and all the temporary adjustments are done. Vertical circle is placed left to the observer (face left observation).
- Vernier A is set to Zero with the help of upper clamp screw and tangent screws. Readings of Vernier A and B are noted.
- Upper clamp is clamped. Lower clamp is loosened and the telescope is turned towards “P”. Lower clamp is clamped and the point “P” is bisected exactly using tangent screws.
- Both the vernier A and B are read and noted (Must be equal to 0° and 180° respectively).Upper clamp is unclamped and the telescope is turned clockwise and “Q” is bisected.
- Upper clamp is clamped and “Q” is bisected exactly using tangent screws. Both the verniers are read. Mean of the readings provide an approximate included angle of POQ.
- The reading of vernier A gives directly the angle POQ and 180° is subtracted by the reading of vernier B. The mean value of two readings gives the angle POQ with one face.
- Lower clamp is unclamped and the telescope is turned anticlockwise to sight P again. Lower clamp is clamped and P is bisected exactly using tangent screws.
- Upper clamp is loosened and the telescope is turned clockwise and Q is bisected. Upper clamp is clamped and Q is bisected exactly using tangent screws. The vernier now read twice the value of angle POQ.
- Last two steps (7&8) are repeated once again to get the thrice value of angle POQ.
- Finally obtained reading is divided by 3 to get the mean value of angle POQ.
- The face is changed and the whole process is repeated. (Face right observations).
- Average value of two horizontal angles obtained with face left and face right observations is determined.

RESULT:

The horizontal angle measured at O between P and Q = _______

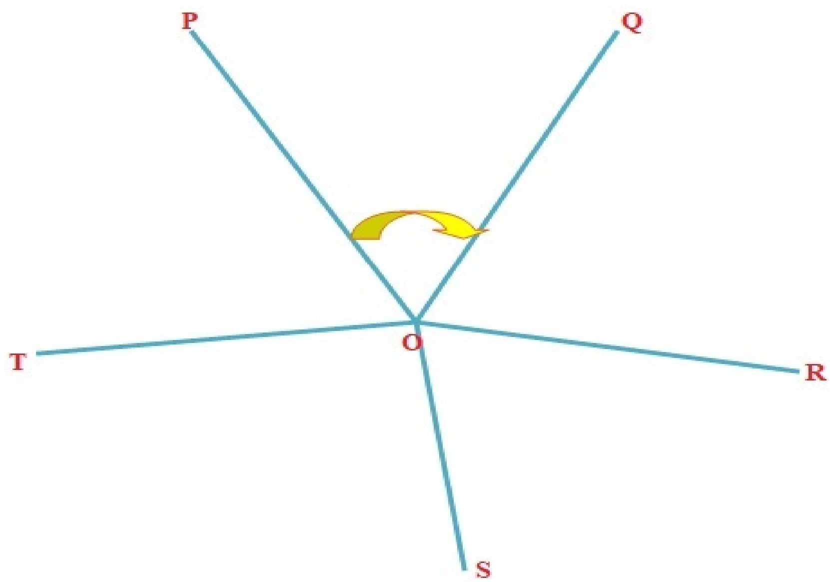
WEEK 03 PRACTICAL 02

AIM: To measure the horizontal angle by Reiteration method with the use of Theodolite.

INSTUMENTS USED

- Theodolite,
- Ranging rods,
- Pegs or Arrows.

MEASURMENT OF HORIZONTAL ANGLE BY REITERATION METHOD



FACE:LEFT												SWING:RIGHT	FACE:RIGHT											SWING:RIGHT	Average Horizontal Angle		
INST @	Sighted to	A			B		Mean			Horizontal Angle			A			B		Mean			Horizontal Angle					
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
O	**P**																									
	Q																									
	R																									
	S																									
	T																									
	P																									

PROCEDURE

- Theodolite is set over an instrument station (O) exactly and all the temporary adjustments are done. Vertical circle is placed left to the observer (face left observation).
- Vernier A is set to Zero with the help of upper clamp screw and tangent screws. Readings of Vernier A and B are noted.
- Upper clamp is clamped. Lower clamp is loosened and the telescope is turned towards “P”. Lower clamp is clamped and the point “P” is bisected exactly using tangent screws.
- Upper clamp is loosened and the telescope is turned clockwise to bisect R. Lower clamp is clamped and R is bisected exactly using tangent screws. Both the verniers are read and noted.
- The same procedure is repeated for all other points.
- The face is changed and all the above steps are repeated. (Face right observations).
- Reading from Q is subtracted by reading R to get included angle QOR. Reading from R is subtracted by reading S to get included angle ROS.
- The same procedure is followed to get readings of all other included angles.

RESULT:

The horizontal angle between the points

WEEK 03 PRACTICAL 03

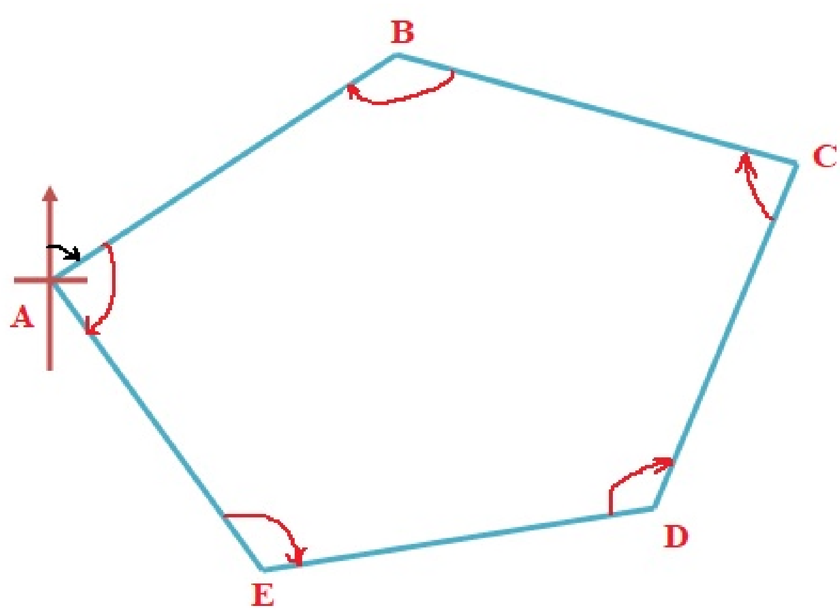
AIM: To carryout survey for closed traverse for minimum 5sides by locating details using Theodolite.

INSTRUMENTS USED

- Theodolite with Tripod.
- Prismatic Compass with stand
- Ranging rods.
- Pegs or Arrows.

THEORY

A traverse survey is one in which the framework consists of a series of connected lines, the lengths and direction of which are measured with the help of a tape or chain and an angle measuring instrument. When the lines form a circuit which ends at the starting point the traverse is called a closed traverse. If the circuit ends elsewhere it is called an open traverse. The close traverse is suitable for wide areas and for locating the boundaries of lakes, forests etc. where as an open traverse is carried out in long strips of the country as in the case of canal, road ,railways etc, In Theodolite traversing Theodolite is used for measurement of angles or tape or chain preferably steel tape.



FACE:LEFT												SWING:RIGHT	FACE:RIGHT										SWING:RIGHT	Average Horizontal Angle		
INST @	Sighted to	A			B		Mean			Horizontal Angle			A			B		Mean			Horizontal Angle					
		0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“	‘	“	0	‘	“	0	‘	“	0	‘	“
A	**B**																									
	E																									
B	**C**																									
	A																									
C	**D**																									
	B																									
D	**C**																									
	E																									
E	**A**																									
	D																									

PROCEDURE

- Magnetic bearing of line AB is measured.
- Theodolite is set up over A. Vernier A is set to zero using upper clamp and its tangent screw.
- Telescope is back sighted to B. Vernier A still read zero.
- Upper clamp is loosened and the telescope is turned clockwise to bisect E. Upper clamp is clamped.
- Both the verniers are read and noted. Mean of the two verniers A and B give the direct angle BAE.
- Face is changed and again the direct angle BAE is measured.
- Average of two values is the required direct angle BAE.
- Similarly all the other direct angles are measured.
- The lengths of traverse lines are measured and the details are noted.

RESULT:

- The closed traverse is conducted with the use of Theodolite and balanced by Bowditch or Transit Rule.

KNOWLEDGE CRITERIA WEEK - 04

SN	COURSE CONTENT
4.1	Trigonometrical Survey and its applications.
4.2	Elevations and Distances of accessible points whose base is accessible Single plane method - Simple problems.
4.3	Elevations and Distances of inaccessible points whose base is inaccessible Single plane method - Simple problems.

WEEK 04 CLASS 01

TRIGNOMETRICAL SURVEY AND ITS PRACTICAL APPLICATIONS

Trigonometrical levelling 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 angels may be measured by means of an accurate Theodolite and the horizontal distances may either be measured (in case of plane surveying) or computed (in case of geodetic surveying). We shall discuss the Trigonometrical levelling under two heads.

- Observations for heights and distances.
- Geodetic observations.

In the first case, the principles of plane surveying will be used. It is assumed that the distances between two 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 differences 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.

In the geodetic observations of Trigonometrical levelling, the distances between the points are measure 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.

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

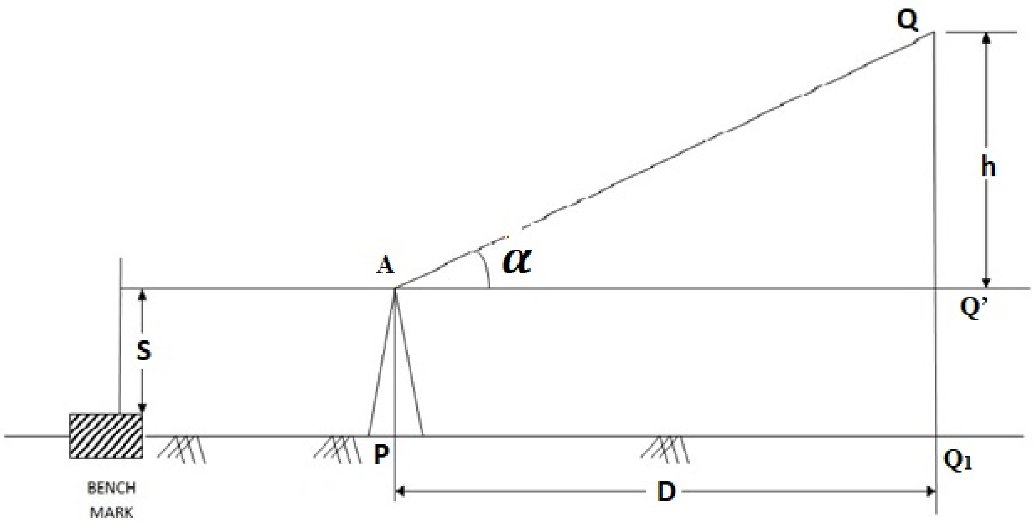
CASE 01: Base of the object is accessible.

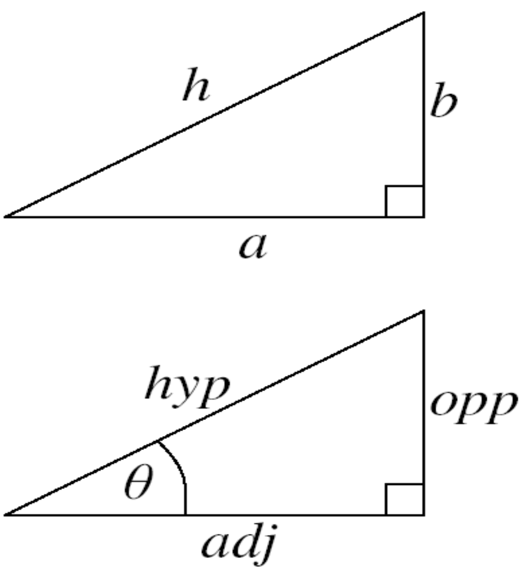
CASE 02: Base of the object is inaccessible (Instrument stations in the same vertical plane as the elevated object)

CASE 03: Base of the object is inaccessible (Instrument stations not in the same vertical plane as the elevated object)

WEEK 04 CLASS 02

ELEVATIONS AND DISTANCES OF ACCESSIBLE POINTS WHOSE BASE IS ACCESSIBLE SINGLE PLANE METHOD





Pythagoras's Theorem

$$a^2 + b^2 = h^2$$

Trigonometric Ratios

$$\sin(\theta) = \frac{opp}{hyp}$$

$$\cos(\theta) = \frac{adj}{hyp}$$

$$\tan(\theta) = \frac{opp}{adj}$$

Let it be assumed that the horizontal distance (D) between the instrument station and the object station can be measured accurately.

P = Instrument Station.

Q = Point to be observed.

A = Centre of the instrument.

Q' = Projection of Q on horizontal plane through A

D = PQ_1 = AQ' = Horizontal distance between P & Q

S = Reading of staff kept @ Bench mark with line of sight horizontal.

α = Angle of elevation from A to Q.

From Triangle AQQ',

$$h \ = D \tan \alpha$$

$$RL\ OF\ Instrumental\ axis = RL\ of Bench\ mark + S$$

$$RL\ OF\ Q = RL\ of\ Instrumental\ axis + h$$

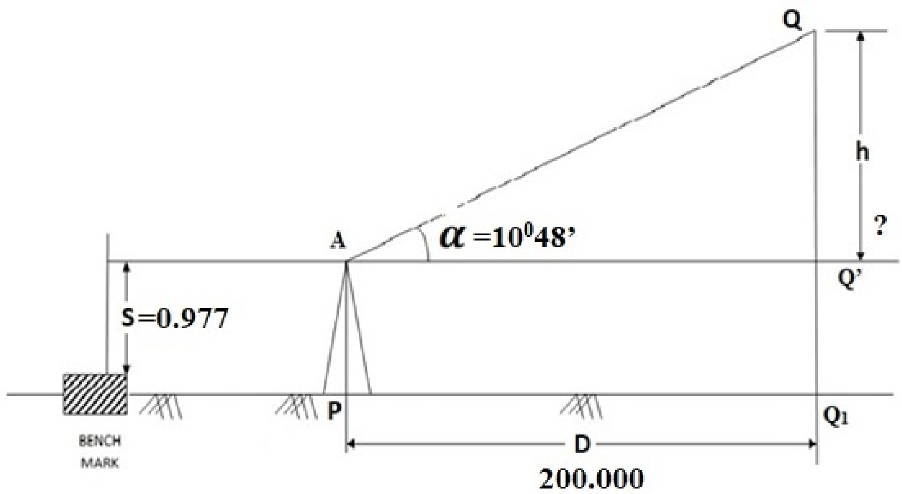
This method is usually employed when the distance is small. However if distance is large then the combined correction for curvature and refraction can be applied. Hence we conclude that if the combined correction for curvature and refraction is to be applied linearly. Its sign is positive for angles of elevation and negative for angles of depression.

Combined Correction = $C = 0.06728\text{D}^2$ When D is in Kilometres

PROCEDURE:

- Set up the Theodolite over instrument station P and level it accurately with reference to the altitude bubble.
- Take staff reading S on bench mark with horizontal line of sight to determine the elevation of line of sight.
- Direct the telescope towards Q and observe the vertical angle.
- Face left and face right observations must be taken to eliminate the instrumental errors.

A transit Theodolite was set up at a distance of 200.00M from a chimney and angle of elevation to its top was $10^0 48'$. The staff reading on benchmark of RL 70.25M with the telescope horizontal was 0.977m. Find the RL of the top of chimney.



$h = D \tan \alpha$

$h = 200 \tan 10°48'$

$h = 38.15M$

$RL\ OF\ Instrumental\ axis = RL\ of\ Bench\ mark + S$

$RL\ OF\ Instrumental\ axis = 70.25 + 0.977$

$RL\ OF\ Instrumental\ axis = 71.227M$

$RL\ OF\ Q = RL\ of\ Instrumental\ axis + h$

$RL\ OF\ Q = 71.227 + 38.15$

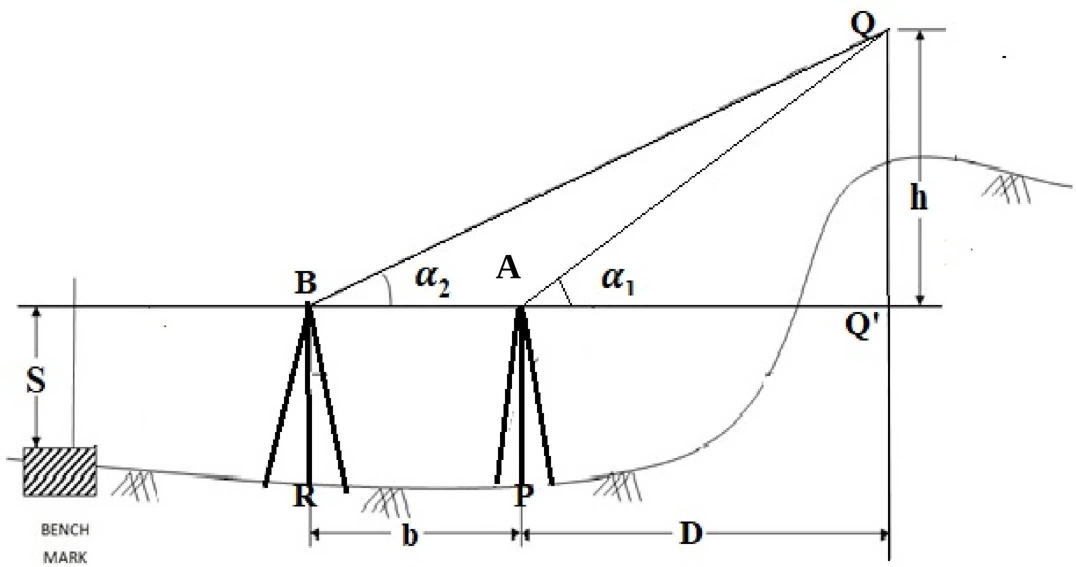
$RL\ OF\ Q = 109.377M$

WEEK 04 CLASS 03

ELEVATIONS AND DISTANCES OF INACCESSIBLE POINTS WHOSE BASE IS INACCESSIBLE SINGLE PLANE METHOD

If the horizontal distance between the instrument and the object can be measured due to obstacles, two instrument stations are used so that they are in the same vertical plane as the elevated object.

1. INSTRUMENT AXES AT THE SAME LEVEL.



P & R = Instrument Station.

Q = Point to be observed.

A & B = Centre of the instrument.

Q' = Projection of Q on horizontal plane through A & B

D = PQ_1 = AQ' = Horizontal distance between P & Q

S = Reading of staff kept @ Bench mark with line of sight horizontal.

α_1 = Angle of elevation from A to Q.

α_2 = Angle of elevation from B to Q.

From Triangle AQQ', $h = D \tan \propto_1$ ----→1

From Triangle BQQ', $h = (b + D) \tan \propto_2$ ----→2

By Equating 1 & 2 we get, $D\ tan \propto_1 = (b + D)\ tan \propto_2$

$$D\ (tan \propto_1 - tan \propto_2) = b\ tan \propto_2$$

$$D = \frac{b \tan \propto_2}{\tan \propto_1 - \tan \propto_2}$$

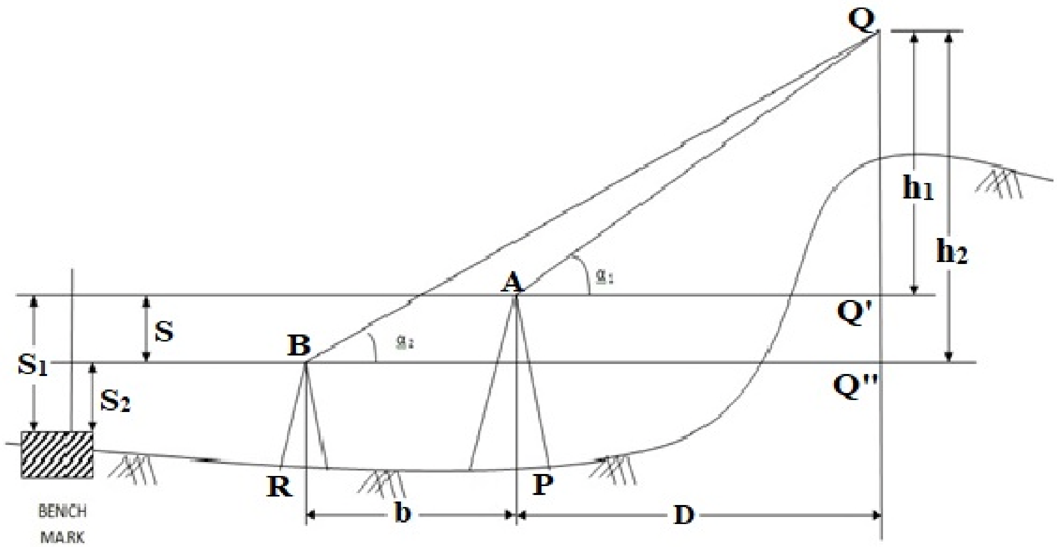
RL of Q = RL of BM + S + h

PROCEDURE:

Let A & B are the two instrument stations. Q be the elevated object whose elevation is required. A,B & Q are in same vertical plane.

1) Set up the Theodolite at Instrument station P and level it accurately with respect to altitude bubble.
2) Direct the telescope with left face towards the Q and bisect it accurately. Clamp both the plates and read the vertical angle α_1.
3) Transit the telescope so that the line of sight is reversed. Mark the second instrument station R on the ground. Measure the horizontal distance RP accurately.
4) Repeat the step 2 & 3 for right face observations. The mean values should be adopted.
5) Set the vertical vernier to read zero reading and the altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark.
6) Shift the instrument to station R and set up the Theodolite there and measure the vertical angle α_2 to Q with both face observation.
7) With the vertical vernier set to zero reading and altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark.
8) When the instrument axes at A & B are at the same level the staff reading on bench mark from both instrument station R & P should be same.

2. INSTRUMENT AXES AT DIFFERENT LEVEL (A HIGH THAN B)



P & R = Instrument Station.

Q = Point to be observed.

A & B = Centre of the instrument.

b = Horizontal distance between instrument stations.

S_1 = Reading of staff kept @ Bench mark with line of sight horizontal from A.

S_2 = Reading of staff kept @ Bench mark with line of sight horizontal from B.

S = Staff intercept = S_1 - S_2 = h_2 - h_1

α_1 = Angle of elevation from A to Q.

α_2 = Angle of elevation from B to Q.

h_1 = Height of Q above the horizontal plane through A.

h_2 = Height of Q above the horizontal plane through B.

Q' = Projection of Q on horizontal plane through A.

Q" = Projection of Q on horizontal plane through B.

D = PQ" = AQ' = Horizontal distance between P & Q

From Triangle AQQ', $h_1 = D \tan \propto_1$ ----→1

From Triangle BQQ", $h_2 = (b + D) \tan \propto_2$ ----→2

By subtracting equation 1 from 2 We get,

$$h_2 - h_1 = (b + D) \tan \propto_2 - D \tan \propto_1$$

$$S = b \tan \propto_2 - D (\tan \propto_1 - \tan \propto_2)$$

$$D (\tan \propto_1 - \tan \propto_2) = b \tan \propto_2 - S$$

$$D = \frac{b \tan \propto_2 - S}{\tan \propto_1 - \tan \propto_2}$$

RL of Q = RL of BM + S_1 + h_1

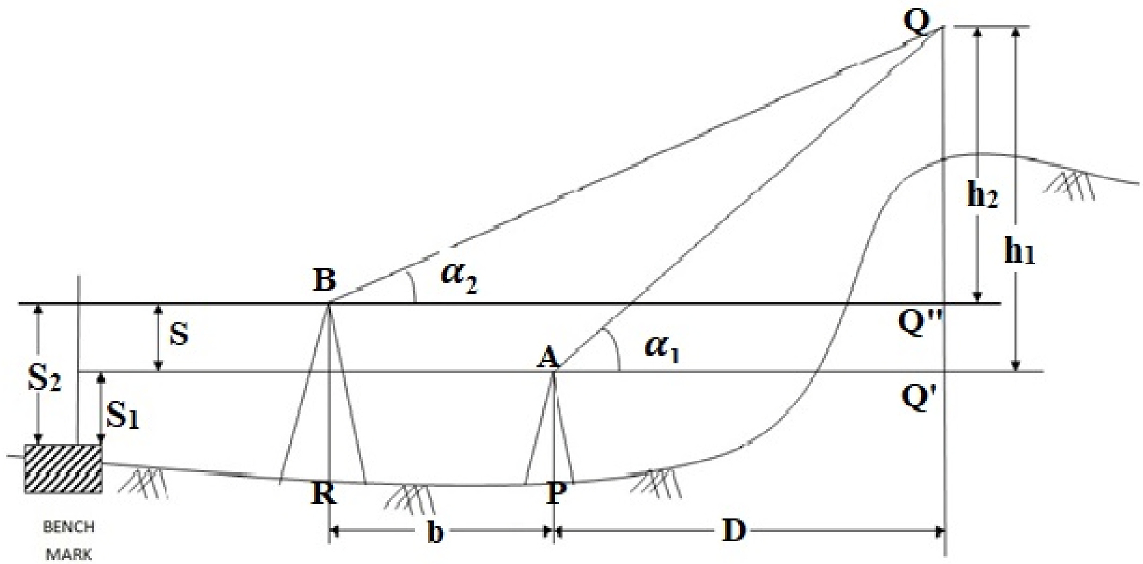
RL of Q = RL of BM + S_2 + h_2

PROCEDURE:

Let A & B are the two instrument stations. Q be the elevated object whose elevation is required. A,B & Q are in same vertical plane.

1) Set up the Theodolite at Instrument station P and level it accurately with respect to altitude bubble.
2) Direct the telescope with left face towards the Q and bisect it accurately. Clamp both the plates and read the vertical angle α_1.
3) Transit the telescope so that the line of sight is reversed. Mark the second instrument station R on the ground. Measure the horizontal distance RP accurately.
4) Repeat the step 2 & 3 for right face observations. The mean values should be adopted.
5) Set the vertical vernier to read zero reading and the altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_1.
6) Shift the instrument to station R and set up the Theodolite there and measure the vertical angle α_2 to Q with both face observation.
7) With the vertical vernier set to zero reading and altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_2.

3. INSTRUMENT AXES AT DIFFERENT LEVEL (B HIGH THAN A)



P & R = Instrument Station.

Q = Point to be observed.

A & B = Centre of the instrument.

b = Horizontal distance between instrument stations.

S_1 = Reading of staff kept @ Bench mark with line of sight horizontal from A.

S_2 = Reading of staff kept @ Bench mark with line of sight horizontal from B.

S = Staff intercept = $S_2 - S_1 = h_1 - h_2$

α_1 = Angle of elevation from A to Q.

α_2 = Angle of elevation from B to Q.

h_1 = Height of Q above the horizontal plane through A.

h_2 = Height of Q above the horizontal plane through B.

Q' = Projection of Q on horizontal plane through A.

Q" = Projection of Q on horizontal plane through B.

D = PQ" = AQ' = Horizontal distance between P & Q

From Triangle AQQ', $h_1 = D \tan \propto_1$ ----→1

From Triangle BQQ", $h_2 = (b + D) \tan \propto_2$ ----→2

By subtracting equation 2 from 1 We get,

$$h_1 - h_2 = D \tan \propto_1 - (b + D) \tan \propto_2$$

$$S = D \tan \propto_1 - b \tan \propto_2 - D \tan \propto_2$$

$$D\ (tan \propto_1 - \ tan \propto_2) = \ b\ tan \propto_2 + \ S$$

$$D = \frac{b \tan \propto_2 + \ S}{\tan \propto_1 - \tan \propto_2}$$

RL of Q = RL of BM + S_1 + h_1

RL of Q = RL of BM + S_2 + h_2

PROCEDURE:

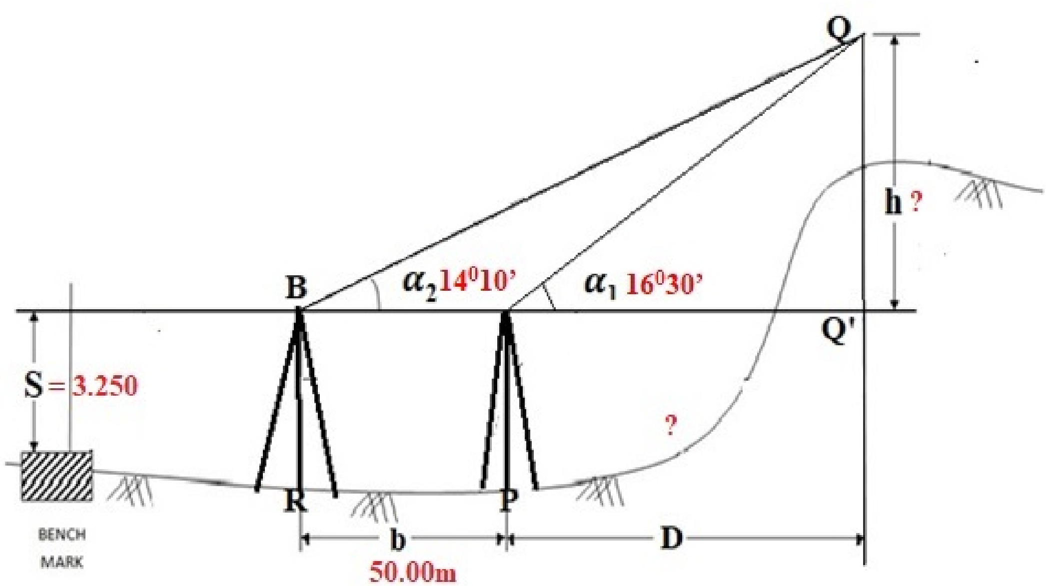
Let A & B are the two instrument stations. Q be the elevated object whose elevation is required. A,B & Q are in same vertical plane.

1) Set up the Theodolite at Instrument station P and level it accurately with respect to altitude bubble.
2) Direct the telescope with left face towards the Q and bisect it accurately. Clamp both the plates and read the vertical angle $\alpha_{1.}$
3) Transit the telescope so that the line of sight is reversed. Mark the second instrument station R on the ground. Measure the horizontal distance RP accurately.
4) Repeat the step 2 & 3 for right face observations. The mean values should be adopted.
5) Set the vertical vernier to read zero reading and the altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_1.
6) Shift the instrument to station R and set up the Theodolite there and measure the vertical angle α_2 to Q with both face observation.
7) With the vertical vernier set to zero reading and altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_2.

SIMPLE PROBLEMS

1. **Determine the reduced level of top of tower from the following observations. Station A & B are in line with tower. Station A & B is 50.00M Apart.**

Inst station	Reading on BM	Vertical angle	Remarks
A	3.250	$16^0 30'$	RL of Ground
B	3.250	$14^0 10'$	200.000 M



$$D = \frac{b \tan \propto_2}{\tan \propto_1 - \tan \propto_2}$$

$$D = \frac{50 \tan 14°10'}{\tan 16°30' - \tan 14°10'}$$

$$D = \mathbf{288.17m}$$

$$h_1 = D \tan \propto_1 = 288.17 \tan 16°30' = \mathbf{85.36m}$$

$$h_2 = (b + D) \tan \propto_2 = (50 + 288.17) \tan 14°10' = \mathbf{85.36m}$$

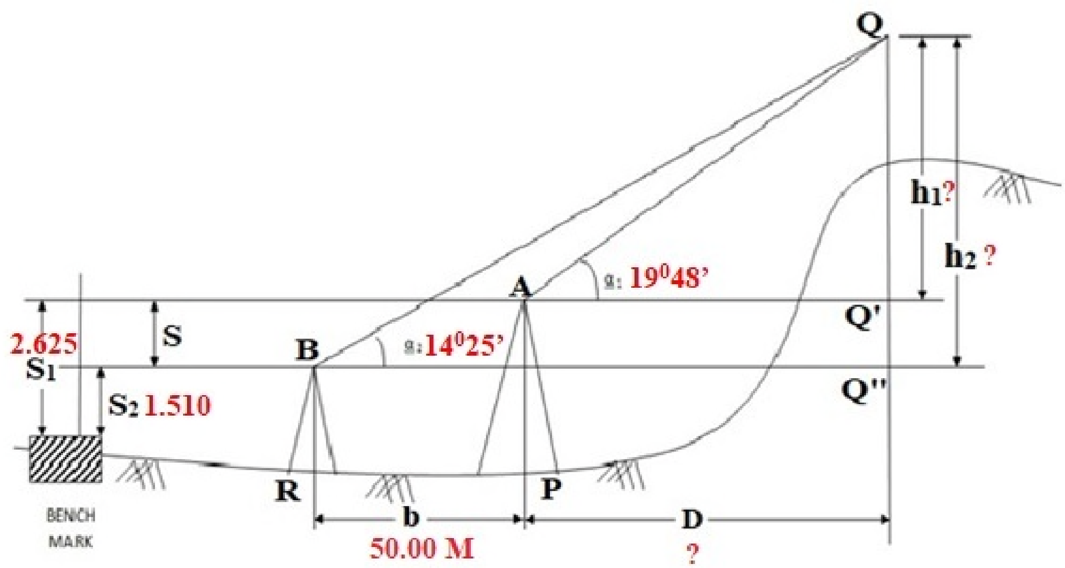
$$Rl\ of\ Q = RL\ of\ BM + S_1 + h_1 = 200.000 + 3.250 + 85.36 = \mathbf{288.610m}$$

Check

$$Rl\ of\ Q = RL\ of\ BM + S_2 + h_2 = 200.000 + 3.250 + 85.36 = \mathbf{288.610m}$$

2. **Determine the reduced level of top of a multi storied building from the following observations. Station A & B are in line with BM on multi storied building. Station A & B is 50.00M Apart.**

Inst station	Reading on BM	Vertical angle	Remarks
A	2.625	$19^0 48'$	RL of Ground 500.000 M
B	1.510	$14^0 25'$	



$$S = S_1 - S_2 = 2.625 - 1.510 = \mathbf{1.115m}$$

$$D = \frac{b\tan\propto_2 - S}{\tan\propto_1 - \tan\propto_2}$$

$$D = \frac{50\tan 14°25' - 1.115}{\tan 19°48' - \tan 14°25'}$$

$$D = \mathbf{144.01m}$$

$$h_1 = D\ tan\propto_1 = 114.01\ tan\ 19°48' = \mathbf{41.046m}$$

$$h_2 = (b + D)tan\propto_2 = (50 + 114.01)tan14°25' = \mathbf{42.161m}$$

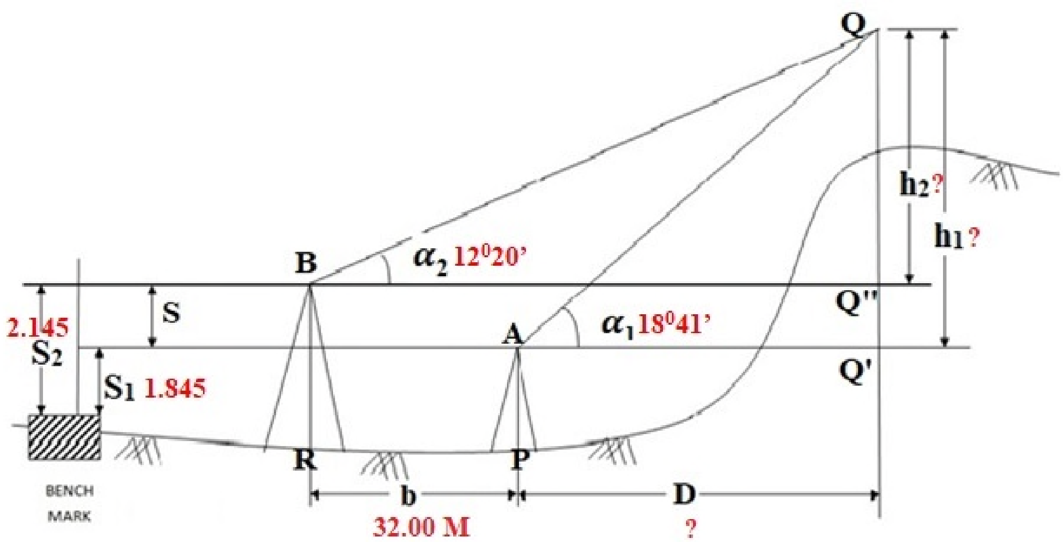
$$Rl\ of\ Q = RL\ of\ BM + S_1 + h_1 = 500.000 + 2.625 + 41.046 = \mathbf{543.671m}$$

Check

$$Rl\ of\ Q = RL\ of\ BM + S_2 + h_2 = 500.000 + 1.510 + 42.161 = \mathbf{543.671m}$$

3. **Determine the elevation of top of the mobile tower from following data. Station A & B and top of the tower are in the same vertical plane. The distance between A&B is 32.00M.**

Inst station	Reading on BM	Vertical angle	Remarks
A	1.845	$18^0 41'$	RL of Ground 165.950 M
B	2.145	$12^0 20'$	



$S = S_2 - S_1 = 2.145 - 1.845 = \mathbf{0.300m}$

$$D = \frac{b \tan \propto_2 + S}{\tan \propto_1 - \tan \propto_2}$$

$$D = \frac{32 \tan 12°20' + 0.300}{\tan 18°41' - \tan 12°20'}$$

$$D = \mathbf{61.05m}$$

$h_1 = D\ tan \propto_1 = 61.05\ tan\ 18°41' = \mathbf{20.64m}$

$h_2 = (b + D) tan \propto_2 = (32 + 61.05) tan 12°20' = \mathbf{20.34m}$

$Rl\ of\ Q = RL\ of\ BM + S_1 + h_1 = 160.950 + 1.845 + 20.64 = \mathbf{183.435m}$

Check

$Rl\ of\ Q = RL\ of\ BM + S_2 + h_2 = 160.950 + 2.145 + 20.34 = \mathbf{183.435m}$

PERFORMANCE CRITERIA WEEK - 04

SN	COURSE CONTENT
4.1	Measure height of the an object whose base is accessible
4.21	Measure height of the an object whose base is inaccessible (A higher than B)
4.22	Measure height of the an object whose base is inaccessible (B higher than A)

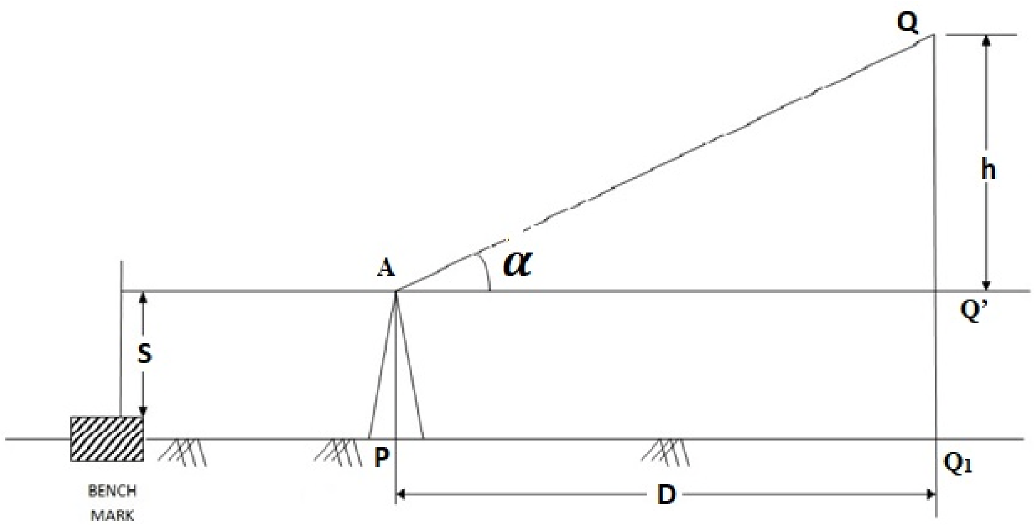
WEEK 04 PRACTICAL 01

AIM: To determine the height of the object whose base in accessible by Trigonometrical levelling.

APPARATUS USED

- Theodolite.
- Tripod.
- Levelling staff.
- Tape.

OBSERVATION



FORMULA USED

$h = D \tan \alpha$

$RL\ OF\ Instrumental\ axis = RL\ of\ Bench\ mark + S$

$RL\ OF\ Q = RL\ of\ Instrumental\ axis + h$

PROCEDURE

- Set up the Theodolite over P and level it accurately with reference to the altitude bubble.
- The staff reading S on bench mark is taken with horizontal line of sight to determine the elevation of line of sight.
- Direct the telescope towards Q and observe vertical angle.
- Face left & Face right observations must be taken to eliminate the instrumental error.

RESULT:

The reduced level of the top of the given object is ________ meters.

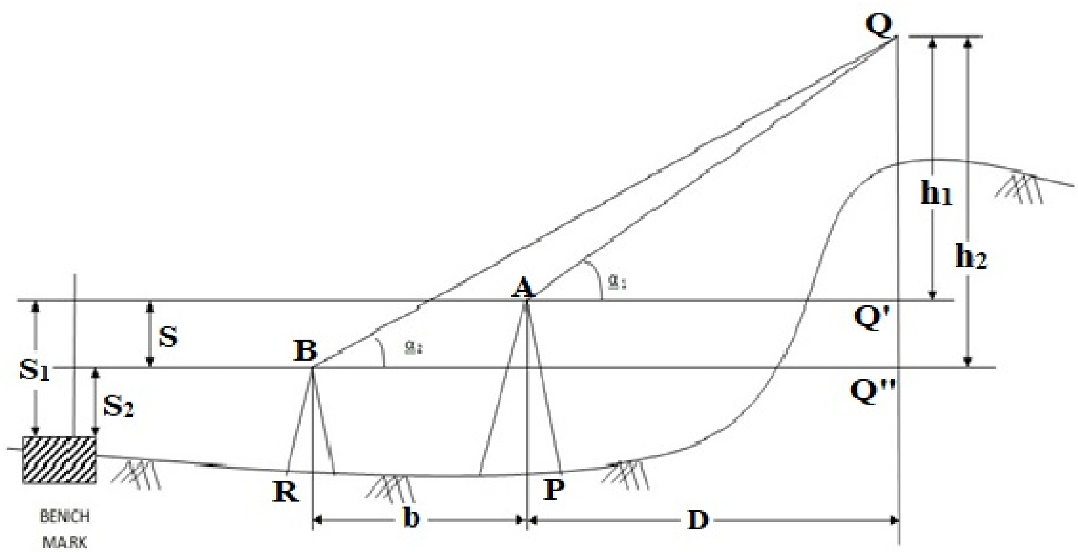
WEEK 04 PRACTICAL 02

AIM: To determine the height of an object whose base is inaccessible by single plane method instrument axes at different levels when instrument A is higher than instrument B.

INSTRUMENTS USED

- Theodolite.
- Tripod.
- Levelling staff.
- Tape.
- Ranging rod.

OBSERVATION:



Inst @	Sighted to	Face Left			Face Right			Vertical Angle
		C	D	Mean	C	D	Mean	
A	Q							
B	Q							

RL of BM =

S_1=

S_2=

$S = S_1 - S_2$ =

b =

FORMULA USED

$$D = \frac{b \tan \propto_2 - S}{\tan \propto_1 - \tan \propto_2}$$

$$h_1 = D \tan \propto_1$$

$$h_2 = (b + D) \tan \propto_2$$

RL of Q = RL of BM + S_1 + h_1

RL of Q = RL of BM + S_2 + h_2

PROCEDURE

Let A & B are the two instrument stations. Q be the elevated object whose elevation is required. A,B & Q are in same vertical plane.

8) Set up the Theodolite at Instrument station P and level it accurately with respect to altitude bubble.
9) Direct the telescope with left face towards the Q and bisect it accurately. Clamp both the plates and read the vertical angle α_1.
10) Transit the telescope so that the line of sight is reversed. Mark the second instrument station R on the ground. Measure the horizontal distance RP accurately.
11) Repeat the step 2 & 3 for right face observations. The mean values should be adopted.
12) Set the vertical vernier to read zero reading and the altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_1.
13) Shift the instrument to station R and set up the Theodolite there and measure the vertical angle α_2 to Q with both face observation.
14) With the vertical vernier set to zero reading and altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_2.

RESULT:

- The reduced level of the top of the given object is ________ Meters.
- The horizontal distance between instrument & Object station is________ Meters.

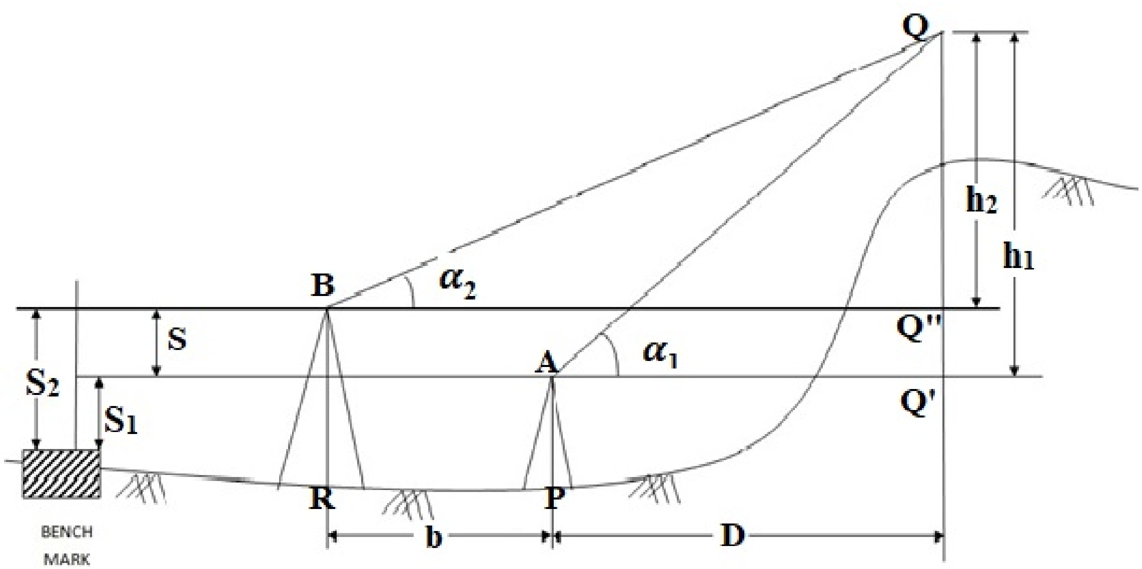
WEEK 04 PRACTICAL 03

AIM: To determine the height of an object whose base is inaccessible by single plane method instrument axes at different levels when instrument B is higher than instrument A.

INSTRUMENTS USED

- Theodolite.
- Tripod.
- Levelling staff.
- Tape.
- Ranging rod.

OBSERVATION:



Inst @	Sighted to	Face Left			Face Right			Vertical Angle
		C	D	Mean	C	D	Mean	
A	Q							
B	Q							

RL of BM =

S_1 =

S_2 =

$S = S_2 - S_1$

b =

FORMULA:

$$D = \frac{b \tan \propto_2 + S}{\tan \propto_1 - \tan \propto_2}$$

$$h_1 = D \tan \propto_1$$

$$h_2 = (b + D) \tan \propto_2$$

RL of Q = RL of BM + S_1 + h_1

RL of Q = RL of BM + S_2 + h_2

PROCEDURE:

Let A & B are the two instrument stations. Q be the elevated object whose elevation is required. A,B & Q are in same vertical plane.

8) Set up the Theodolite at Instrument station P and level it accurately with respect to altitude bubble.
9) Direct the telescope with left face towards the Q and bisect it accurately. Clamp both the plates and read the vertical angle α_1.
10) Transit the telescope so that the line of sight is reversed. Mark the second instrument station R on the ground. Measure the horizontal distance RP accurately.
11) Repeat the step 2 & 3 for right face observations. The mean values should be adopted.
12) Set the vertical vernier to read zero reading and the altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_1.
13) Shift the instrument to station R and set up the Theodolite there and measure the vertical angle α_2 to Q with both face observation.
14) With the vertical vernier set to zero reading and altitude bubble in the centre of its run, take the reading on the staff kept near Bench mark S_2.

RESULT:

- The reduced level of the top of the given object is ________ Meters.
- The horizontal distance between instrument & Object station is________ Meters.

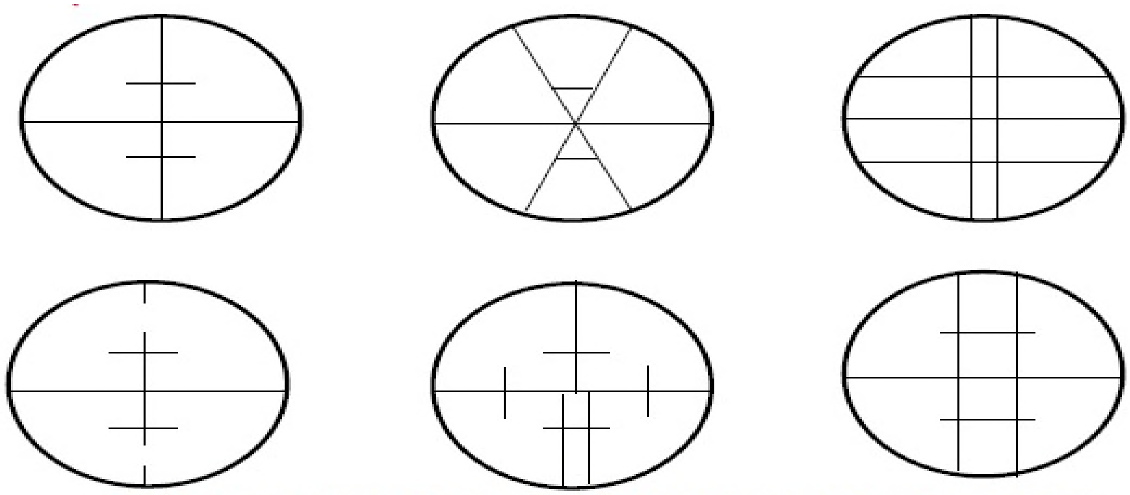
KNOWLEDGE CRITERIA WEEK - 05

SN	COURSE CONTENT
5.1	Tachometer: Principle of tachometry and component parts. Analytic lens
5.2	Tachometric formula for horizontal distance with telescope horizontal and staff vertical.
5.3	Method of determining Horizontal and vertical distances with tachometer by fixed hair method and staff held vertical.

WEEK 05 CLASS 01

TACHOMETER:

An ordinary transit Theodolite fitted with a stadia diaphragm is generally used for tachometric survey. The stadia diaphragm essentially consists of one stadia hair above and the other an equal distance below the horizontal cross hair, the stadia hairs being mounted in the same ring and in the same vertical plane as the horizontal and vertical cross hairs. The below figure shows the different forms of stadia diaphragm commonly used.



Different forms of stadia diaphragm commonly used

The telescope used in stadia surveying are of three kinds.

- The simple external focusing telescope.
- The external focusing anallactic telescope (Porro telescope)
- The internal focusing telescope.

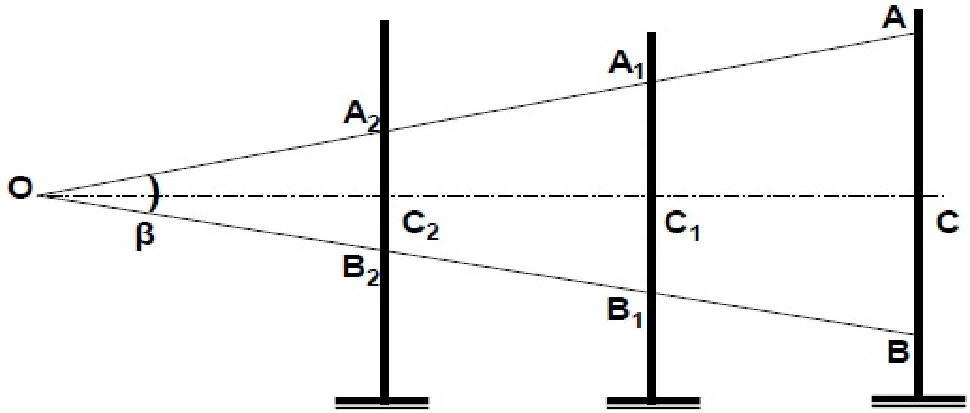
The first type is known as stadia Theodolite, while second type is called Tachometer.

The tachometer has the advantage over the first and the third type due to the fact that the additive constant of the instrument is zero. However, the internal focusing telescope is becoming more popular, thought it has a very small additive constant. Some of the latest patterns of internal focusing telescope may be regarded as strictly anallactic.

A tachometer must essentially incorporate the following features

- The multiplying constant should have a nominal value of 100 and the error contained in this value should not exceed 1 in 1000.
- The axis horizontal line should be exactly midway between the two other lines.
- The telescope should be truly anallactic.
- The telescope should be powerful having a magnification of 20 to 30diameters.

PRINCIPLE OF TACHOMETRY:

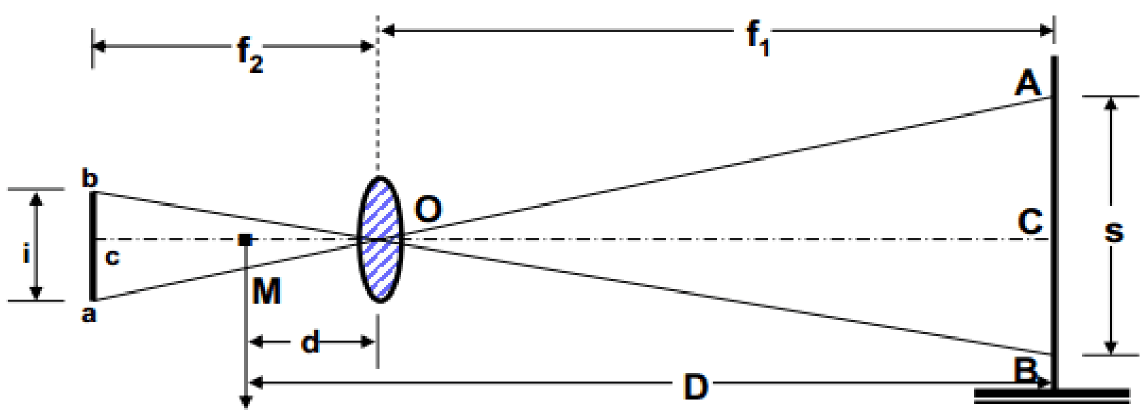


The stadia method is based on the principle that the ratio of the perpendicular to the base is constant in similar isosceles triangles. In above figure, let two rays OA & OB be equally inclined to the central ray OC. Let A_2B_2, A_1B_1 & AB be the staff intercepts. Evidently

$$\frac{OC_2}{A_2B_2} = \frac{OC_1}{A_1B_1} = \frac{OC}{AB} = CONSTANT = K = \frac{1}{2}\,Cot\,\frac{\beta}{2}$$

The constant K entirely depends upon the magnitude of the angle β. If β is made equal to 34’ 22.64”, the constant$K = \frac{1}{2}\,Cot\,17'.32" = 100$. In this case, the distance between the staff and the point O will be 100 times the staff intercept.

In actual practice, observations may be made either horizontal line of sight or with inclined line of sight. In the latter case, the staff may be kept either vertically or normal to the line of sight. We shall first derive the distance elevation formulae for the horizontal line of sight.



Consider the above figure in which O is the optical center of the objective of an external focusing telescope.

Let

- A, C & B = The points cut by the three lines of sight corresponding to the three wires.

- b, c & a = Top, Axial & bottom hairs of the diaphragm.
- ab = I = Interval between the stadia hairs
- AB = s = Staff intercept.
- f = Focal length of the objective.
- f_1 = Horizontal distance of the staff from the optical center of the objective.
- f_2 = Horizontal distance of the cross wires from O
- d = Distance of the vertical axis of the instrument from O.
- D = Horizontal distance of the staff from the vertical axis of the instrument.
- M = Center of the instrument corresponding to the vertical axis.

Since the rays Bob and Aoa pass through the optical center, they are straight so that △AOB & △aOb are similar,

Hence $\frac{f_1}{f_2} = \frac{s}{i}$

Again, since f_1 & f_2 are conjugant focal distances, we have from lens formula

$$\frac{1}{f} = \frac{1}{f_2} = \frac{1}{f_1}$$

Multiply throughout by ff_1, we get $f_1 = \frac{f_1}{f_2} f + f$

Substituting the value of $\frac{f_1}{f_2} = \frac{s}{i}$ in above equation, we get $f_1 = \frac{s}{i} f + f$

The horizontal distance between the axis and the staff is $D = f_1 + d$

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

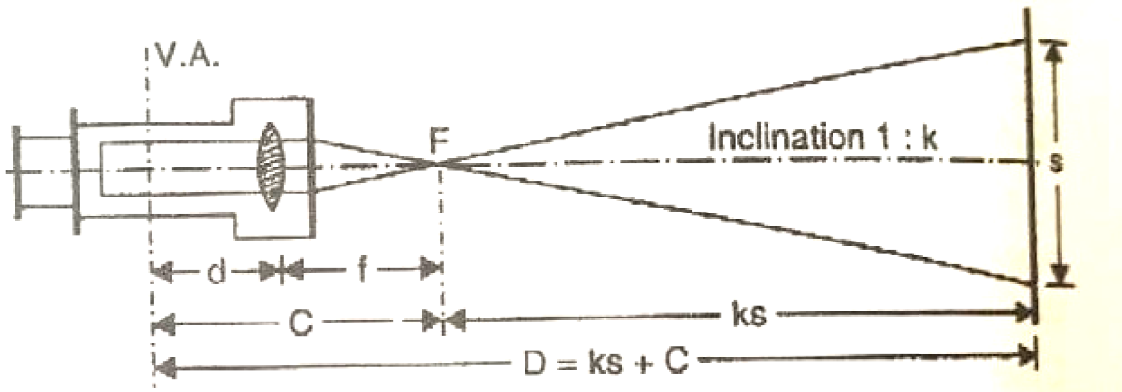
$$D = k\ s + c$$

This equation is known as Distance equation. In order to get the horizontal distance, therefore the staff intercept s is to be found out by subtracting the staff readings corresponding to the top & bottom stadia hairs.

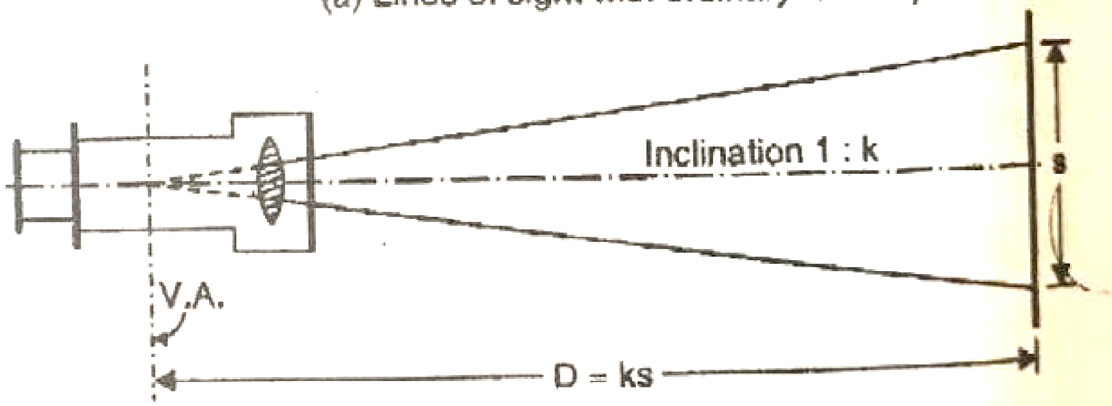
The Constant K = f/i is known as multiplying constant or stadia interval factor. The Constant C = f+d is known as additive constant of the instrument

ANALETIC LENS:

In the distance formula D= KS+C, the staff intercept s is proportional to (D-C) which is the distance between the staff and the exterior principal focus of the objective. This is because the vertex of the measuring triangle (anallactic point) falls at the exterior principal focus of the objective and not at the vertical axis of the instrument.



(a) Lines of sight with ordinary telescope



(b) Lines of sight with anallactic lens

In the distance formula D= KS+C, the staff intercept s is proportional to (D-C) which is the distance between the staff and the exterior principal focus of the objective. This is because the vertex of the measuring triangle (anallactic point) falls at the exterior principal focus of the objective and not at the vertical axis of the instrument.

In 1880 Porro devised the external focusing anallactic telescope, the special feature of which is an additional (convex) lens, called an anallactic lens placed between the diaphragm and the objective at the fixed distance from the latter. In above figure (a) shows the line of sight with ordinary telescope and figure (b) shows the line of sight with an anallactic lens.

The word anallactic means unalterable or invariable, by the provision of anallactic lens the vertex is formed at the vertical axis and its position is always fixed irrespective of the staff position. The anallactic lens is generally provided in external focusing telescope only and not in internal focusing telescope since the latter is virtually anallactic due to very small additive constant.

MERITS

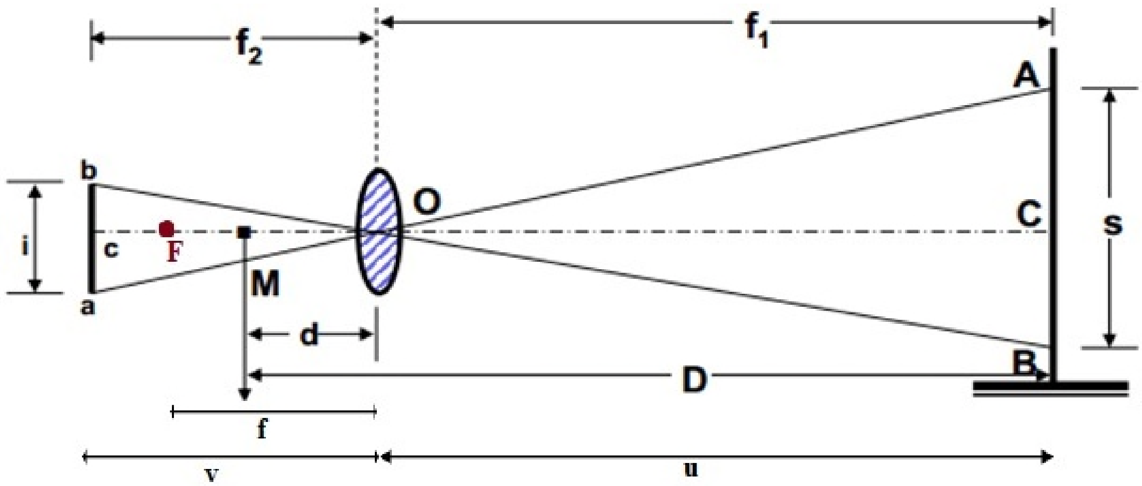
- Due to anallactic lens the addictive constant deletes and computation can be made easier.
- An anallactic lens is sealed against dust and moisture.
- With the use of larger object glass, loss of light may be compensated.
- The error in it can be compensated by changing the distance between the object glass and the anallactic lens so as to give the correct horizontal distance D.

DEMERITS

- Anallactic lens absorbs much of the incident light.
- It cannot be cleaned easily.
- It decreases the magnification.
- It increases the cost of instrument due to extra lens.

WEEK 05 CLASS 02

TACHOMETRIC FORMULA FOR HORIZONTAL DISTANCE WITH TELESCOPE HORIZONTAL AND STAFF VERTICAL.



Consider the above figure in which O is the optical center of the objective of an external focusing telescope.

Let

- A, C & B = The points cut by the three lines of sight corresponding to the three wires.
- b, c & a = Top, Axial & bottom hairs of the diaphragm.
- ab = I = Interval between the stadia hairs
- AB = s = Staff intercept.
- f = Focal length of the objective.
- f_1 = Horizontal distance of the staff from the optical center of the objective.
- f_2 = Horizontal distance of the cross wires from O
- d = Distance of the vertical axis of the instrument from O.
- D = Horizontal distance of the staff from the vertical axis of the instrument.
- M = Center of the instrument corresponding to the vertical axis.

From Figure $D = u + d$ ----→1

From △AOB & aob $\frac{u}{v} = \frac{S}{i}$ or $\frac{1}{v} = \frac{S}{iu}$ ----→2

U and v are conjugate focal lengths, hence from the lens principle

$$\frac{1}{f} = \frac{1}{u} + \frac{1}{v}$$ ----→3

Substituting Equation 2 in 3

$$\frac{1}{f} = \frac{1}{u} + \frac{S}{iu}$$

$$\frac{1}{f} = \frac{1}{u}\left(1 + \frac{S}{i}\right)$$

Or $u = f\left(1 + \frac{S}{i}\right) = f + \frac{f}{i}S$ ----→4

Substituting Equation 4 in 1

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

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

$$D = KS + C$$

This equation is known as Distance equation. In order to get the horizontal distance, therefore the staff intercept s is to be found out by subtracting the staff readings corresponding to the top & bottom stadia hairs.

The Constant K = f/i is known as multiplying constant or stadia interval factor. The Constant C = f+d is known as additive constant of the instrument

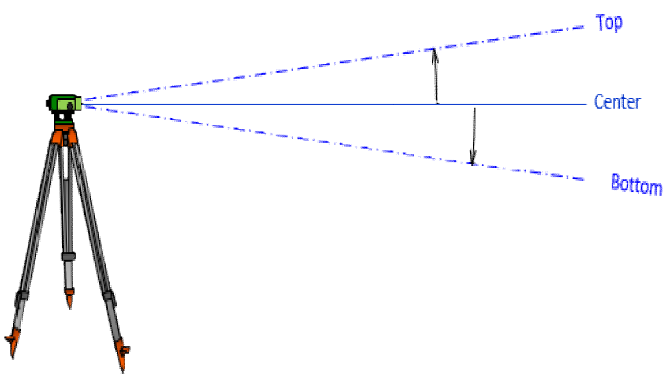
Note: The value of multiplying constant (K) varies from 50 to 200. But it is generally made 100 by suitably selecting the values of f & i. The usual value of additive constant (C) is 0.30 to 0.60 in external focusing telescope and 0.08 to 0.2 in internal focusing telescopes. But this is eliminated or made zero by introducing an additional convex lens called anallactic lens in between the objective and diaphragm

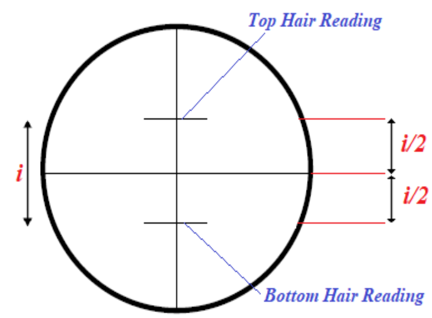
WEEK 05 CLASS 03

METHOD OF DETERMINING HORIZONTAL AND VERTICAL DISTANCES WITH TACHOMETER BY FIXED HAIR METHOD AND STAFF HELD VERTICAL.

FIXED HAIR METHOD:

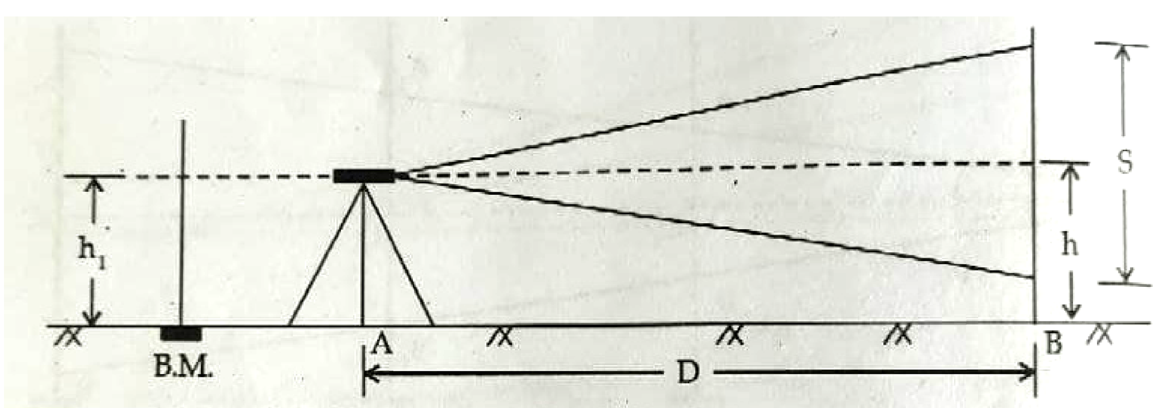
In this method observations are made with the help of a stadia diaphragm having wires at fixed or constant distance apart. The readings on the staff corresponding to all the wires are taken. The intercept (the difference of the readings corresponding to top and bottom stadia wires) depends on the distance of the staff from the instrument. When the staff intercept is more than the length of the staff, only half intercept is read. For inclined sights, readings may be taken by keeping staff either vertical or normal to the line of sight. This is the most common method in tachometry and the name stadia method generally bears reference to this method.





To calculate the horizontal distance and elevation of various points, the tachometer can be set up at a point and an ordinary levelling staff or stadia rod is held at various points whose distances and elevations are required. For observations line of sight may be horizontal or inclined according to the position of the staff station.

CASE 01: Horizontal line of sight with staff held vertically.



Let

D = Horizontal distance between instrument & Staff station.

S = Staff intercept.

h_1=Staff reading at Bench Mark

$$Horizontal\ Distance = D = KS + C$$

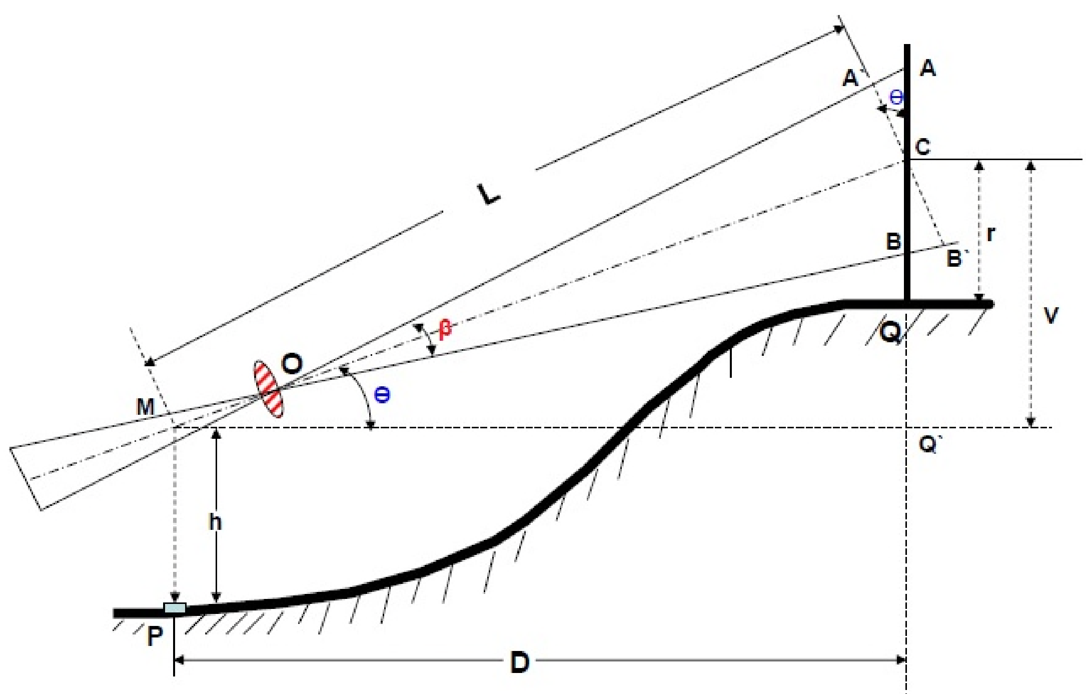
$$RL\ of\ Instumental\ Axis = Rl\ of\ Benchmark + h_1$$

$$RL\ of\ B = RL\ of\ Instrumental\ axis - h$$

PROCEDURE:

- Setup the instrument @ station A and level it accurately.
- Set the vertical vernier to read Zero.
- Direct the telescope towards the benchmark and note down the staff reading.
- Direct the telescope to staff station B and note down the stadia hair readings.
- By using distance formula calculate the horizontal distance between instrument station & staff station.
-

CASE 02: Inclined line of sight with staff held vertically (Upward).



P = Instrument Station.

Q = Staff Station.

M = Position of instrumental Axis.

O = Optical centre of the objective.

AB = s = Staff Intercept

i = Stadia Interval.

L = Length MC measured along the line of sight.

θ = Inclination of the line of sight from the horizontal.

D = MQ' = Horizontal distance between the instrument and the staff.

V = Vertical intercept at Q, between the line of sight and the horizontal line.

h = Height of the instrument

r = Central hair reading

β = Angle between two extreme rays corresponding to stadia hairs.

A, C & B = Points corresponding to the readings of the stadia hairs.

Draw the line A'CB' normal to the line of sight OC.

$\Delta AA'C = 90° + \frac{\beta}{2}$ Being the exterior angle of the △COA'.

Similarly, From △COB', Angle OB'C = Angle BB'C $90° - \frac{\beta}{2}$

Since β/2 is very small (its value being equal to 17'11.32"), Angle OB'C & Angle BB'C may be approximately equal to 90^0.

Therefore Angle OB'C = Angle BB'C = 90^0.

From △ACA' $A'C = AC \cos\theta$ or $A'B = AB \cos\theta = s \cos\theta$

Since the line A'B' is perpendicular to the line of sight OC the equation is directly applicable

We have, $MC = L = k \times A'B' + C = ks\cos\theta + C$

The horizontal distance $D = L \cos\theta$

$$D = (ks\cos\theta + C)\cos\theta$$

$$\boldsymbol{D = ks\cos^2\theta + C\cos\theta}$$

Similarly, $V = L \sin\theta$

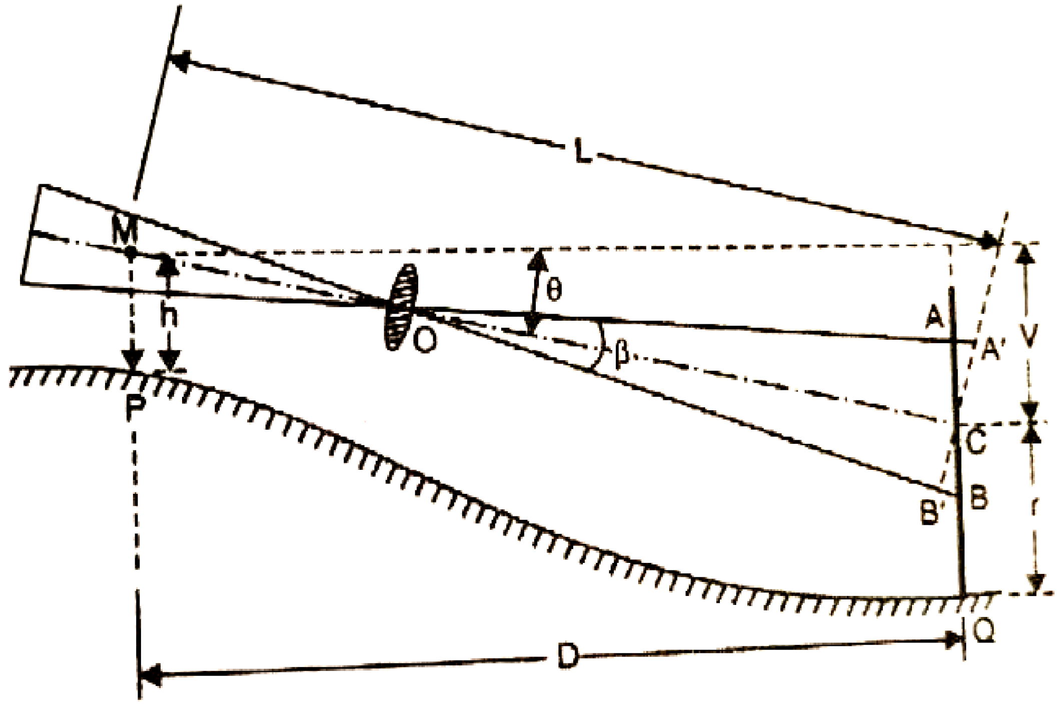
$$V = (ks\cos\theta + C)\sin\theta$$

$$V = ks\cos\theta \times \sin\theta + C\sin\theta$$

$$\boldsymbol{V = ks\frac{\sin 2\theta}{2} + C\sin\theta}$$

$\boldsymbol{Elevation\ of\ the\ staff\ station = Elevation\ of\ Instrument\ Station + h + V - r}$

CASE 02: Inclined line of sight with staff held vertically (Downward).



P = Instrument Station.

Q = Staff Station.

M = Position of instrumental Axis.

O = Optical centre of the objective.

AB = s = Staff Intercept

i = Stadia Interval.

L = Length MC measured along the line of sight.

θ = Inclination of the line of sight from the horizontal.

D = MQ' = Horizontal distance between the instrument and the staff.

V = Vertical intercept at Q, between the line of sight and the horizontal line.

h = Height of the instrument

r = Central hair reading

β = Angle between two extreme rays corresponding to stadia hairs.

A, C & B = Points corresponding to the readings of the stadia hairs.

Draw the line A'CB' normal to the line of sight OC.

$\Delta AA'C = 90° + \frac{\beta}{2}$ Being the exterior angle of the △COA'.

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From △ACA’ $A'C = AC \cos\theta$ or $A'B = AB \cos\theta = s \cos\theta$

Since the line A’B’ is perpendicular to the line of sight OC the equation is directly applicable

We have, $MC = L = k \times A'B' + C = ks\cos\theta + C$

The horizontal distance $D = L \cos\theta$

$$D = (ks\cos\theta + C) \cos\theta$$

$$\boldsymbol{D = ks\cos^2\theta + C \cos\theta}$$

Similarly, $V = L \sin\theta$

$$V = (ks\cos\theta + C) \sin\theta$$

$$V = ks\cos\theta \times \sin\theta + C\sin\theta$$

$$\boldsymbol{V = ks\frac{\sin 2\theta}{2} + C\sin\theta}$$

$$\boldsymbol{Elevation\ of\ the\ staff\ station = Elevation\ of\ Instrument\ Station + h - V - r}$$

PERFORMANCE CRITERIA WEEK - 05

SN	COURSE CONTENT
5.1	Determine tachometric constants.
5.21	Calculate the reduced levels and horizontal distances of given points using tachometer.(Line of Sight inclined Upwards)
5.22	Calculate the reduced levels and horizontal distances of given points using tachometer.(Line of Sight inclined Downwards)

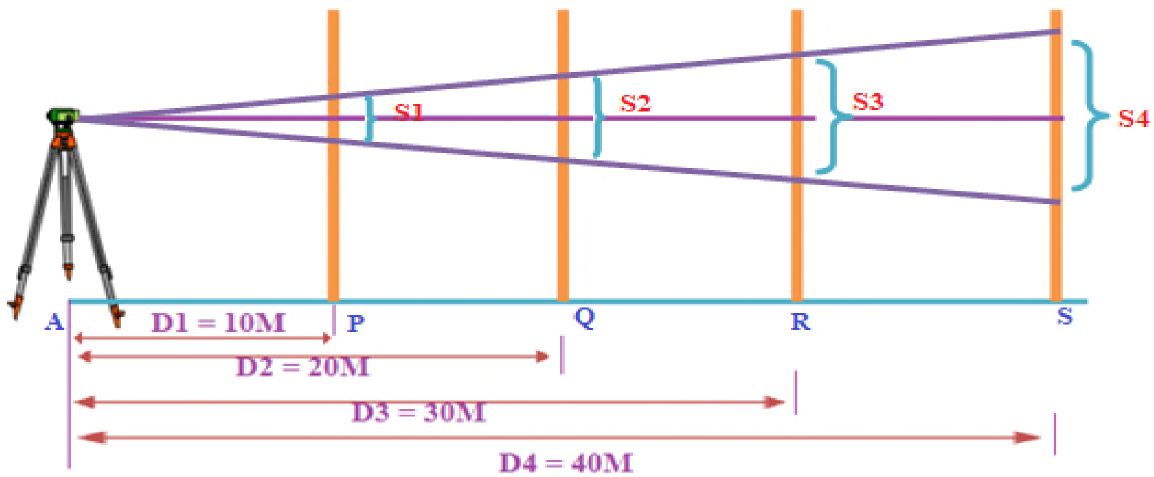
WEEK 05 PRACTCAL 01

AIM: To determine the Multiplying & Additive constant of given tachometer.

INSTRUMENTS USED

- Tachometer.
- Tripod.
- Levelling staff.
- Tape.
- Ranging rod.

OBSERVATION:



Inst @	Staff @	Distance	Stadia Readings			Staff Intercept	K	C
			Bottom	Central	Top			
A	P	10.00						
	Q	20.00						
	R	30.00						
	S	40.00						

FORMULA USED

$$Horizontal\ distance\ AP = D_1 = KS_1 + C$$

$$Horizontal\ distance\ AQ = D_2 = KS_2 + C$$

$$Horizontal\ distance\ AR = D_3 = KS_3 + C$$

$$Horizontal\ distance\ AS = D_4 = KS_3 + C$$

PROCEDURE

- Set up the Tachometer @ station A, level it & Centre it accurately with respect to altitude bubble.
- Select staff station point S say 40m away from Instrument station A.
- Drive pegs at uniform interval say 10m along AB. Mark the peg points as P, Q, R and S.
- Hold the staff @ station P and obtain the staff intercept **(S_1)** with line of sight kept horizontal.
- Similarly obtain the staff intercept **(S_2)** by holding the staff @ station Q.
- Form simultaneous equations using distance formula.

  $\boldsymbol{Horizontal\ distance\ AP = D_1 = KS_1 + C}$ **----→ 1**

  $\boldsymbol{Horizontal\ distance\ AQ = D_2 = KS_2 + C}$ **----→ 2**
- Solve equation 1 & 2 to determine the values of K & C.
- Form another pair of simultaneous equations after obtaining staff intercepts S_3 & S_4 on pegs R & S respectively.
- Solve second pair of simultaneous equations to obtain value of K & C.
- Determine the mean value of K & C obtained will give the value of Constant K & C.

RESULT:

The multiplying constant of given Tachometer = K =

The Additive constant of given Tachometer = C =

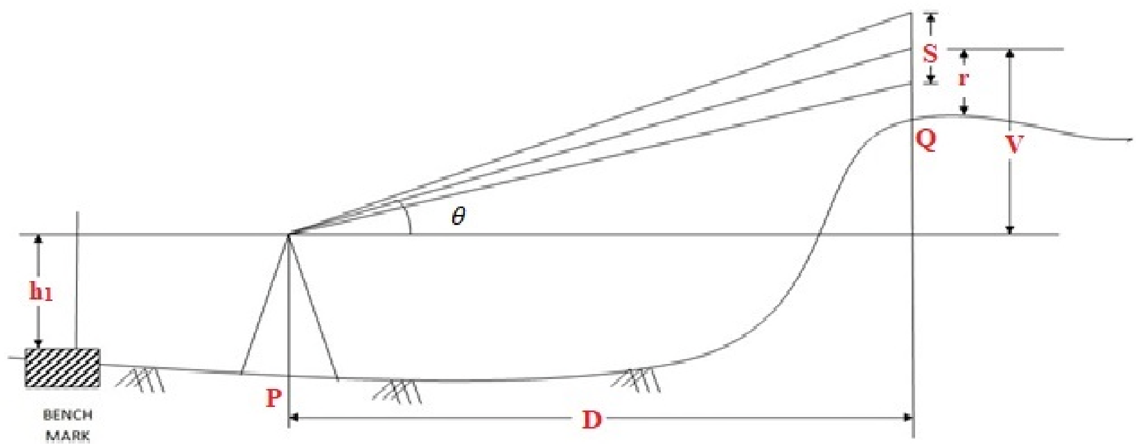
WEEK 05 PRACTCAL 02

AIM: To determine the horizontal distance and elevation for inclined upward sight with staff held vertical by Stadia hair method

INSTRUMENTS USED

- Tachometer.
- Tripod.
- Levelling staff.

OBSERVATIONS



Inst @	Staff @	Stadia Readings			Staff Intercept	Vertical Angle	K	C
		Bottom	Central	Top				
P	BM	-----		-----	-----	-----		
	Q							

FORMULA

$$D = ks\cos^2\theta + C\cos\theta$$

$$V = ks\frac{\sin 2\theta}{2} + C\sin\theta$$

$$Elevation\ of\ the\ Instrumental\ Axis = Elevation\ of\ Bench\ Mark + h_1$$

$$Elevation\ of\ the\ staff\ station = Elevation\ of\ Instrument\ Station + V - r$$

PROCEDURE

- Set up the Tachometer @ station P, level it & Centre it accurately with respect to altitude bubble.
- Hold the staff @ bench mark and obtain the staff reading by observing central hair reading (h_1) with line of sight kept horizontal.
- Direct the telescope & Hold the staff @ staff station Q and obtain the staff intercept reading (S) by observing stadia hairs with line of sight inclined upwards.
- Measure the vertical angle with both face left & Face right.
- By using distance formula determine the Horizontal distance & elevation of object.

RESULT:

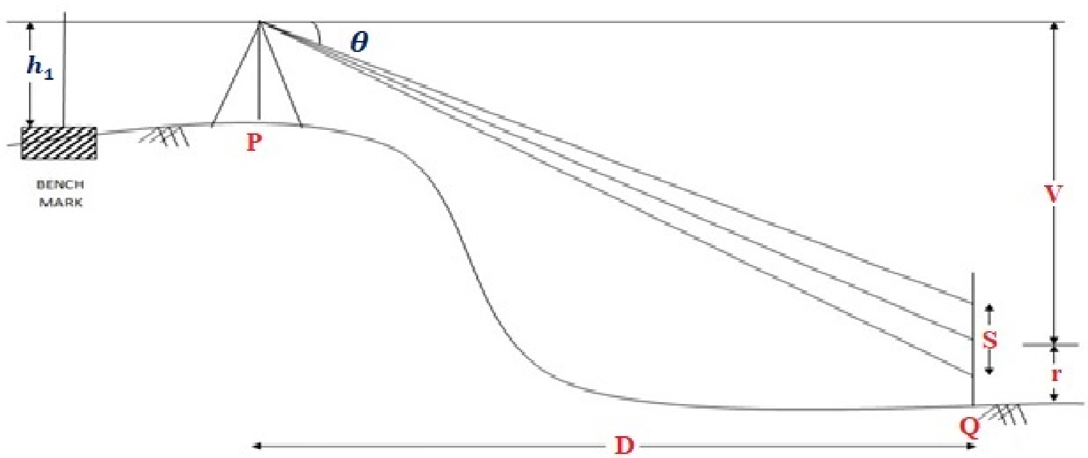
- The horizontal distance by inclined sight by using tachometer is __________
- The elevation of object Q __________

WEEK 05 PRACTCAL 03

AIM: To determine the horizontal distance and elevation for inclined downward sight with staff held vertical by Stadia hair method

INSTRUMENTS USED

- Tachometer.
- Tripod.
- Levelling staff.



Inst @	Staff @	Stadia Readings			Staff Intercept	Vertical Angle	K	C
		Bottom	Central	Top				
P	BM	-----		-----	-----	-----		
	Q							

FORMULA

$$D = ks\cos^2\theta + C\ \cos\theta$$

$$V = ks\frac{\sin 2\theta}{2} + C\sin\theta$$

$$Elevation\ of\ the\ Instrumental\ Axis = Elevation\ of\ Bench\ Mark + h_1$$

$$Elevation\ of\ the\ staff\ station = Elevation\ of\ Instrument\ Station - V - r$$

PROCEDURE

- Set up the Tachometer @ station P, level it & Centre it accurately with respect to altitude bubble.
- Hold the staff @ bench mark and obtain the staff reading by observing central hair reading (h_1) with line of sight kept horizontal.
- Direct the telescope & Hold the staff @ staff station Q and obtain the staff intercept reading (S) by observing stadia hairs with line of sight inclined downwards.
- Measure the vertical angle with both face left & Face right.
- By using distance formula determine the Horizontal distance & elevation of object.

RESULT:

- The horizontal distance by inclined sight by using tachometer is __________
- The elevation of object __________

KNOWLEDGE CRITERIA WEEK - 06

SN	COURSE CONTENT
6.1	Total Station: Introduction, Integral Parts, Applications
6.2	Working Principle, Advantages & Disadvantages.
6.3	Use of function keys, precautions to be taken while using a total station.

WEEK 06 CLASS 01

INTRODUCTION TO TOTAL STATION

The total station in surveying is an instrument that is primarily designed as a combination of electronic transit Theodolite, an electronic distance meter (EDM), and software that runs on an external computer which is referred to as the data collector. The surveyor can use the total station for various purposes such as to determine the angles, measurement of distances, etc.

Total Station is widely used in modern surveying, archaeology, mining, private accident reconstructions, etc. By the use of triangulations and trigonometric calculations, the measured angles and the distances can be used to determine the actual position of the required points or even the position of the total station from the known points in absolute terms.

AN OVER VIEW ON TOTAL STATION

Most of the total stations these days consist of a GPS interface that combines the two aforementioned technologies. The interface design is done such that the use of the advantage of both the technologies is ensured (i.e. In GPS; the line of sight is not required between the consecutive measurement points. While in the traditional total station; high precision measurement is required especially in the vertical axis in comparison to the GPS).

The total station is the survey instrument that measures the angles by making use of the electro-optical scanning of the extremely precise digital bar-codes that are etched on rotating glass cylinders or discs present within the instrument. The high-quality total station is even capable of measuring the angles down to 0.5 arc-second whereas the inexpensive total stations can mostly measure angles down to 5 or 10 arc-seconds.

In the total station, measurement of distance is accomplished utilizing a modulated microwave or infrared carrier signal which is generated by a small solid-state emitter within the instrument’s optical path which is bounced back to the object to be measured. Then, the onboard computer provided in the total station reads and interprets the returning signal. The distance is then duly determined by receiving and emitting the multiple frequencies.

Most of the total stations consist of a purpose-built glass **Porro Prism** that serves as a reflector for the EDM signal and is capable of measuring the distance up to a few kilometres. A typical total station’s EDM is capable of measuring the distance accurately up to about 3 millimetres or 1/100th of the foot.

Nowadays, the robotic type of total station is also in use. Such a type of total station allows the operator to control the instrument from a distance utilizing a remote control. This helps to avoid the need for an assistant staff member to hold the reflector prism over the point to be measured. It allows the operator to hold the reflector himself or herself and to control the total station from the observed point.

INTEGRAL PARTS OF TOTAL STATION

A total station comprises an EDM, Theodolite, and an inbuilt microprocessor. For storing the data, a memory card is also provided. It consists of a battery that is provided in the battery socket.

A typical total station consists of the following components:

1. Handle
2. Handle Screwing Screw
3. Data Input/ Output Terminal
4. Instrument Height Mark
5. Battery Cover
6. Operation Panel
7. Tribrach Clamp/ Shifting Clamp
8. Base Plate
9. Levelling Foot Screw
10. Circular Level Adjusting Screws
11. Circular Level
12. Display
13. Objective Lens
14. Tubular Compass Slot
15. Optical Plummet Focusing Ring
16. Optical Plummet Reticle Cover
17. Optical Plummet Eyepiece
18. Horizontal Clamp
19. Horizontal Fine Motion Screw
20. Data Input/ Output Connector
21. External Power Source Connector
22. Plate Level
23. Plate Level Adjusting Screw
24. Vertical Clamp
25. Vertical Fine Motion Screw
26. Telescope Eyepiece
27. Telescope Focusing Ring

28. Peep Sight

29. Instrument Center Mark

In addition to the major components, the total station also requires some additional accessories. Such accessories include the keyboard, control panel, data collectors, reflectors, memories, etc which have been described in brief below.

- **Keyboard:** The keyboard is an essential accessory required for giving the command to the microprocessor inbuilt in the total station as it contains various keys. Such keys include command keys, switching keys, lighting keys, power keys, etc. The different types of keys incorporated in the keyboard with their function have been listed as follows:
  Power Key: To switch on or off the total station.
  Star Key: To switch from one mode to another and to change the settings of the total station.
  Illuminator Key: To turn on/off the laser pointer and the guiding light, to light the reticle as well as to select the screen backlight brightness.
  Esc Key: To return to the previous screen or cancel the input data.
  Tab Key: To switch to another item.
  B.S Key: To delete an item on the left side.
  S.P Key: To input a blank space.
  FUNC Key: To switch between the observations.
  ENT Key: Select or Accept any input value or observation.
  Shift Key: To switch between lowercase and uppercase characters.
  Target Keys: To switch between the different targets.
  PRG Key: To switch between different modes of the program.
- **Data Collector:** As the name itself implies, the data collector collects and stores the observed data or observations. The entire operation of taking in and storing the data is controlled by the data collector. The data collector stores the data either in binary form or ASCII. Data collectors can be further divided into external data collectors and internal data collectors.
- **Memory:** The total station is equipped with inbuilt memory cards to process and store the data or observations. The memory card provided in the total stations is generally up to the PMCIA standards. The memory card can have a capacity ranging from 5000 to 10,000 coded points. The data held by the memory card can be unloaded on any computing device.

- **Reflector**: The reflector is one of the most essential accessories of the total station. This is because the total station functions or takes the measurements utilizing the reflected rays. The reflector comprises of a specially built reflecting prism made up of cubes or blocks of reflecting glasses.

APPLICATIONS OF TOTAL STATION

1. **Distance Measurement:** One of the most important uses of the total station is a distance measurement. It can be used for the measurement of distance employing a modulated carrier infrared signal. This signal can calculate the distance after it is bounced back or reflected by the object under consideration. Such infrared carrier signal is generated by the total station employing a solid-state emitter and the interpretation of the reflected signal is carried out by using the computing chip inbuilt in the total station itself.
2. **Angle Measurement:** Another important use of the total station is the measurement of angles between the points. Such angular determinations are accomplished using the electrical and optical scanning system of the telescope. This system is provided with rotating glass that comprises of the bar codes. Such glass makes it easier to read the angles. In the case of digital total stations, the readings are directly recorded and stored in the main memory of the total station. The data stored can also be transferred to a computer.
3. **Coordinate Determinations:** An essential application of the total station is to determine the location of the required points i.e. it can be used to compute the X, Y, Z coordinate of the survey points.
4. **Land Surveying & Alignment Surveying:** Land surveying can be done easily using a total station. In the land surveying, the total station can be used to measure the distances, angles as well as the location coordinates which can be further used for the preparation of topographical maps and plans. Total stations can also be used for demarcating the property lines and the boundary lines. It can also be used for determining the alignment of canals, tunnels, roads, bridges etc.
5. **Mining:** Before the commencement of any mining work, a mining survey has to be done. The total station is an essential instrument used for this purpose. It can also be used to determine the mining points and to prepare the mining maps. Such points and maps can be used by the miners to locate the points with abundant minerals. The total station is also used for determining the location and the alignment of the mining tunnels.
6. **Civil Engineering Construction Works:** Almost all types of civil engineering construction and related works can be done using the total station. The total station can be

used for the measurement of construction parameters such as distances, angles, elevations of points, coordinates, etc. It can be used for the preparation of maps and plans, demarcating the property lines, land surveying, determining the alignment of roads, bridges, tunnels, etc.

7. **Automatic Target Aiming:** Most of the modern total stations are provided with automatic target systems to increase the degree of efficiency of the total station. Such a system is provided to ensure better and faster targeting and locating of any point on the surface of the earth. The system is so quick that the entire process of measurement, processing, and computing of the distance, angles and coordinates takes place very quickly within seconds. Thus, the entire work can be completed easily and fastly.
8. **Electrical and Mechanical Construction:** The layouts for the electrical as well as the mechanical constructions can be determined easily utilizing the total station. Various works such as underground pipe laying, laying of utilities and cables, etc can be done using the total station.

- General purpose of angle and distance measurements.
- Plotting of contours
- Illustration of detailed maps
- Carrying out controlled surveys
- Archaeologists use total station to record excavations
- Police use it in crime scene investigations to take measurements of the scene
- Used to fix the missing pillars
- Remote Distance Measurement (RDM)
- Missing Line Measurement (MLM)
- Remove Elevation Measurement (REM)

TYPES OF TOTAL STATIONS

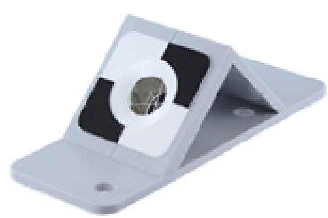
In the early days, three classes of total stations were available—manual, semiautomatic and automatic.

- **Manual Total Stations** It was necessary to read the horizontal and vertical angles manually in this type of instrument. The only value that could be read electronically was the slope distances.
- **Semiautomatic Total Stations** The user had to manually read the horizontal circle for these instruments, but the vertical circle readings were shown digitally. Slope distances

were measured electronically and the instruments could, in most cases, be used to reduce the values to horizontal and vertical components.

- **Automatic Total Stations** This type is the most common total station used now-a-days. They sense both the horizontal and vertical angles electronically and measure the slope distances, compute the horizontal and vertical components of those distances, and determine the coordinates of observed points. To compute the coordinates of observed points, it is necessary to properly orient the instrument to some known directions such as true north, magnetic north or to

TYPES OF TOTAL STATIONS PRISMS

		
360^0 MINI PRISM	**180^0 MINI PRISM**	**L BAR PRISM**
		
MAGNETIC PRISM	**MINI PRISM**	**PEANUT PRISM**
		
SPECIALITY PRISM	**SECO PRISM**	**NODAL PRISM**

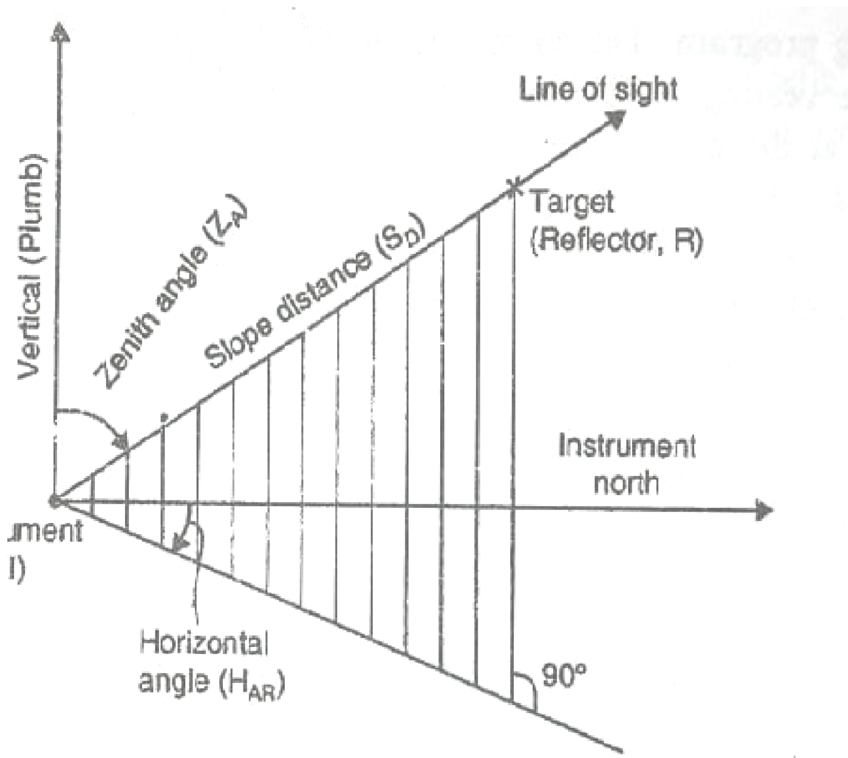
WEEK 06 CLASS 02

WORKING PRINCIPLE OF TOTAL STATION

The basic principle of the total station is that the distance between any two points can be known once the velocity and the time taken by the light to travel are known.

Distance = Velocity * Time

The following relation is already programmed in the memory of the total station along with the correction factors that are used to calculate the required horizontal distance and is finally displayed on the LCD screen of the instrument.



ADVANTAGES OF TOTAL STATION

The total station offers the following major advantages:

- The total station helps to complete the fieldwork quickly in a lesser period.
- The setting up of the total station is also easier as it can be done easily by using the laser plummet.
- The degree of accuracy of the total station is relatively higher than other survey instruments.
- The computed values can be directly stored in the total station and can also be easily transferred to a computing device.

- Greater accuracy in area computation because of the possibility of taking arcs in area computation
- On-board area computation programme to compute the area of the field.
- Graphical view of plots and land for quick visualization
- The total station is also advantageous in the computerization of old maps.
- Since all the observations and computations are done digitally, error due to omission of data, wrong reading or noting of observation etc is omitted.
- It can also help in contouring and map preparation.
- When the atmospheric pressure and temperature are provided, the pressure and temperature corrections can also be applied directly.
- The entire survey work and office work can be completed easily.
- Integration of database (exporting map to GIS packages)
- Automation of old maps
- Full GIS creation (using MapInfo software)
- Local language support

DIS ADVANTAGES OF TOTAL STATION

Some of the disadvantages of the total station can be listed as follows:

- While using the total station, it may be difficult for the surveyor to recheck the work.
- Skilled manpower or experienced personnel is required for operating the instrument.
- The total station is costlier than other conventional survey equipment.
- The total station is incorporated with several electronic accessories and parts which may be affected by moisture.
- Their use does not provide hard copies of field notes. Hence, it may be difficult for the surveyor to look over and check the work while surveying.
- For an overall check of the survey, it will be necessary to return to the office and prepare the drawings using appropriate software.
- They should not be used for observations of the sun, unless special filters, such as the Troelof’s prism, are used. If not, the EDM part of the instrument will be damaged.
- The instrument is costly, and for conducting surveys using total station, skilled personnel are required.

WEEK 06 CLASS 03

USE OF FUNCTION KEYS, GENERAL COMMANDS USED (BASIC KEY FUNCTIONS)

Keys	Description
F1~F4	Select the functions matching the soft keys
0~9	1.Input number when numeric input 2.Input characters when alphabetic input
●	Input a decimal point
±	Input plus/minus sign
Power	Power on/off
★	Enter into setting mode directly
ESC	ESC Escape to the previous menu or mode
SFT	1.Shift between number and alphabetic when inputting 2.Shift targets model when measuring
BS	1. Delete the character at the left of the cursor when inputting 2. Open electronic level menu
Space	1. Input a black space when inputting. 2. Input the target or instrument height.
Func	Turn page
ENT	1. Select/Accept input data. 2. Accept the option when selecting.
DIST	Start distance measure

SHV1	Display switching between SD/HAVA, HD/HA/VA and VD/HA/VA
SHV2	Display switching between SD/HD/VD and SD/HA/VA
0SET	Set horizontal angle to 0
CORD	Enter coordinate measurement menu
MENU	Enter program menu
HOLD	Hold the horizontal angle
HSET	Set horizontal angle
EDM	Enter distance setting menu
OCC	Setting the station point
OFST	Enter offset measurement menu
REC	Enter points collection menu
RES	Enter resection program menu
REMS	Enter angle repeat measurement menu
MLM	Enter missing line measurement menu
S.O.	Enter stake out measurement menu
TILT	Display electronic level
REM	Enter remote elevation measurement menu
HARL	Horizontal angle display switching between HR and HL
ZA/%	Vertical angle display switching grade and zenith
OUT	Output the current measurement data via RS-232C port
AREA	Enter area measurement menu
ROAD	Enter road measurement menu
IHT	Enter instrument height setting menu
LSO	Enter line stake out measurement menu
PROJ	Enter point projection measurement menu

PRECAUTIONS TO BE TAKEN WHILE USING A TOTAL STATION

The precautions that must be taken while using a total station can be listed as follows:

- The total station must be handled properly and held by both hands.
- The tripod must be held on a stable surface as far as practicable.
- Always carry a total station in a locked hard case even for a very short distance. Take the total station out of the hard case only for fixing it firmly on a tripod for taking observations.
- The battery pack must be stored with the battery discharged.
- The clamping screws must not be tightened extremely.
- Do not move or carry a tripod with the total station fixed on it, except for centering.
- Care must be taken when the tribrach has been removed from the total station.
- Never release the handle before the total station is fixed with the tripod’s fixing screw.

- Always keep the top of the tripod, the bottom and top of the tribrach and the bottom of the total station clean and away from any shock and impact.
- Take maximum care when the tribrach is removed from the total station.
- Do not make the total station wet.

PERFORMANCE CRITERIA WEEK - 06

SN	COURSE CONTENT
6.1	Component parts and General commands used.
6.2	Instrument preparation and setting up.

WEEK 06 PRACTICAL 01

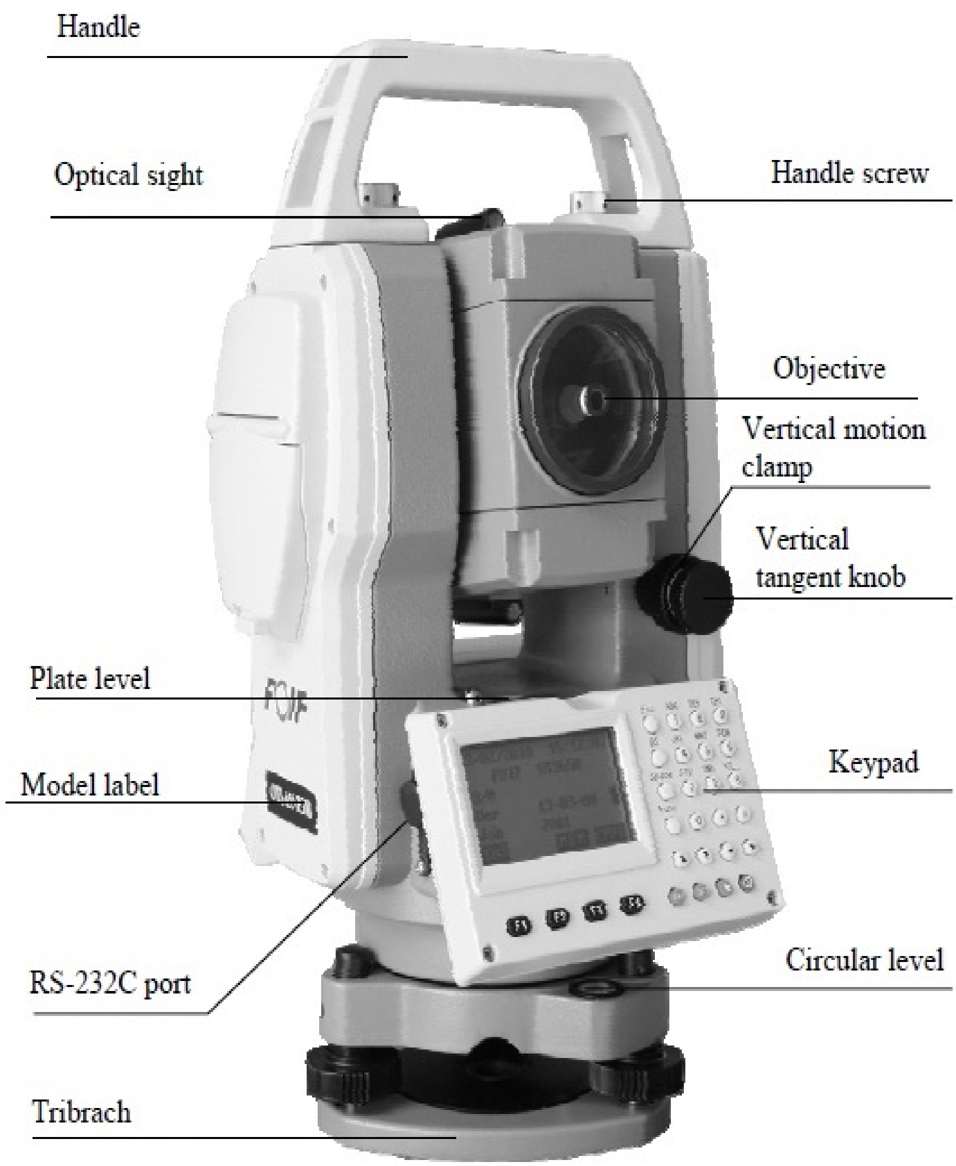
COMPONENT PARTS AND GENERAL COMMANDS USED.

AIM: To know the instrument & general commands used in total station (MODEL: FOIF RTS105R5)

APPARATUS USED

- Total Station
- Tripod.
- Prism with stand.

COMPONENT PARTS



NOMENCLATURE


Prism
Mainframe
Tripod
Prism pole

GENERAL COMMANDS USED (BASIC EY FUNCTIONS)

Keys	Description
F1~F4	Select the functions matching the soft keys
0~9	1.Input number when numeric input 2.Input characters when alphabetic input
●	Input a decimal point
±	Input plus/minus sign
Power	Power on/off
★	Enter into setting mode directly
ESC	ESC Escape to the previous menu or mode
SFT	1.Shift between number and alphabetic when inputting 2.Shift targets model when measuring
BS	1. Delete the character at the left of the cursor when inputting 2. Open electronic level menu
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ENT	1. Select/Accept input data. 2. Accept the option when selecting.
DIST	Start distance measure
SHV1	Display switching between SD/HAVA, HD/HA/VA and VD/HA/VA
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0SET	Set horizontal angle to 0

CORD	Enter coordinate measurement menu
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TILT	Display electronic level
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HARL	Horizontal angle display switching between HR and HL
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AREA	Enter area measurement menu
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IHT	Enter instrument height setting menu
LSO	Enter line stake out measurement menu
PROJ	Enter point projection measurement menu

WEEK 06 PRACTICAL 02

INSTRUMENT PREPARATION AND SETTING UP

AIM: To know the instrument preparation before measurement and setting up of total station. (MODEL: FOIF RTS105R5)

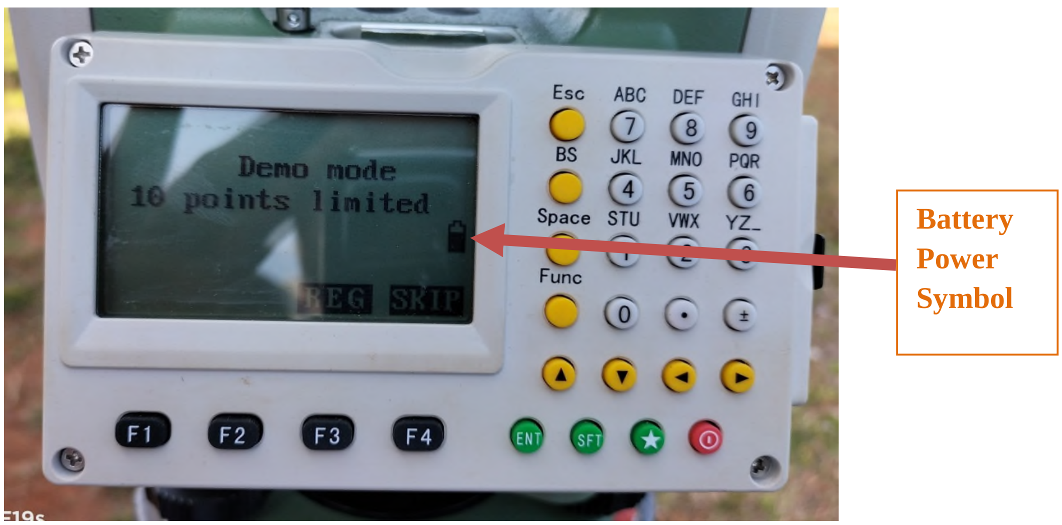
APPARATUS USED

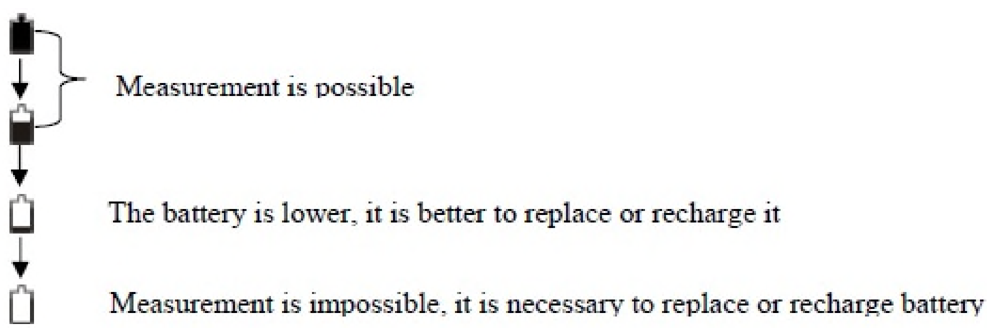
- Total Station
- Tripod.
- Tape.
- Arrows.
- Prism with stand.

INSTRUMENT PREPARATION BEFORE MEASURMENT

1. ABOUT BATTERY:

a. BATTERY POWER SYMBOL:

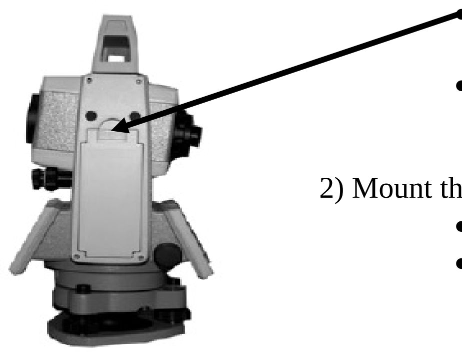




- The working time of battery will be effected by many factors, such as ambient temperature, recharging time, recharging and discharging times. On the data safe side, we suggest the users recharge the battery full or prepare several full batteries before operation.

- The battery symbol only indicates power capability for current measurement mode. The power consumption in distance measurement mode is more than in angle mode, if the instrument enters into distance measurement mode from angle mode, the power maybe auto-off because of lower battery.
- The symbol only indicates the supply power but not the instantaneous power change. And if the measurement mode changes, the symbol will not show the power's decrease or increase immediately.
- It is suggested to check every battery power before field work.

b. REPLACE THE BATTERY:



1) Remove the battery
   - Press the button downward as shown left.
   - Remove the battery by pulling it toward you

2) Mount the battery
   - Insert the battery to the instrument
   - Press the top of the battery until you hear a Click.

c. RECHARGE THE BATTERY:

As above figures show, connect the charger and the battery, then plug the charger into the outlet of 100V-240V AC power supply, recharging will begin.

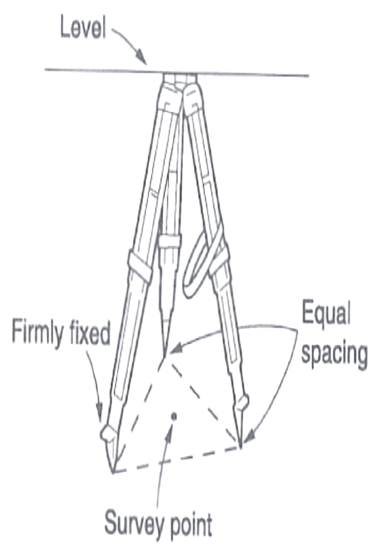
The indicator light on the charger will illuminate three separate colours for varies mode conditions:

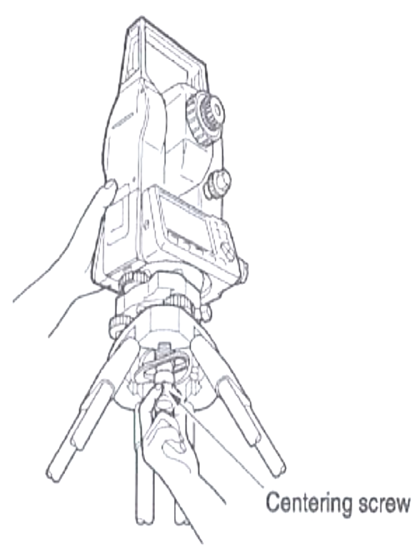
- Solid Red Light—indicates that the charger is working;
- Solid Green Light— indicates that the charge has finished;
- Flashing Red Light—indicates no battery on charging, poor connection or some problems exist.

- For a new (or long time no use) battery, in order to fully extend its capacity, it is absolutely necessary to carry out 3 to 5 complete charging/discharging cycles, and the charging time must be 10 hours at least each time.
- It is recommended to continue charging for 1 or 2 hours after the light turn green.
- Once the red light flashes constantly after the charger is plugged into the outlet of 100V-240V AC power supply please remove the battery and reconnected it after 3 or 5 min.

2. SETTING UP THE INSTRUMENT:

- Mount the battery in the instrument before performing this operation because the instrument will tilt slightly if the battery is mounted after leveling.
- Set up the tripod first: extend the extension legs to suitable lengths and tighten the screws on the midsections. Make sure the legs are spaced at equal intervals and the head is approximately level. Set the tripod so that the head is positioned over the surveying point. Make sure the tripod shoes are firmly fixed in the ground.
- Mount the instrument on the tripod head. Supporting it with one hand, tighten the centering screw on the bottom of the unit to make sure it is secured to the tripod.





3. CENTRING AND LEVELLINF UP:

- Position tripod legs so that the plummet is aimed to the ground mark point. Turn the focusing ring of the optical plummet to focus.
- Turn three foot screws of the tribrach till the centre of reticle exactly coincides with the surveying point in any position.

- Move the tripod legs to centre the circular level. The instrument is now roughly levelled-up.
- Centre the bubble in the circular level.
- Centre the surveying point again Loosen the centering screw slightly. Looking through the optical plummet eyepiece, slide the instrument over the tripod head until the surveying point is exactly centered in the reticle. Re-tighten the centering screw securely.
- Check again to make sure the bubble in the plate level is centered. If not, repeat procedure 4.