GOVERNMENT OF TAMILNADU
DIRECTORATE OF TECHNICAL EDUCATION

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STATE PROJECT COORDINATION UNIT

Diploma in Civil Engineering

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e-TEXTBOOK

on

SURVEYING – II

for

IV Semester DCE

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1.0 THEODOLITE SURVEYING

Introduction - Types of Theodolites : Transit and non-transit Theodolite, Vernier and Micrometer Theodolites – Electronic Theodolite (Principles and description only) – Component parts of a transit Theodolite – Functions – Technical terms used in Theodolite surveying – Temporary adjustments – Fundamental lines and relationship between them – Measurement of horizontal angle by method of repetition and reiteration – Measurement of vertical angle and deflection angle – Reading bearing of a line – Theodolite traversing – Methods – Field checks in closed traverse - Latitude and departure – Consecutive coordinates - Independent coordinates – Problems on computation of area of closed traverse – Balancing the traverse - Omitted measurements – Problems.

1.1 Introduction

A theodolite is an instrument which is used primarily to measure angles, both horizontal and vertical. It is also used for many other subsidiary work during surveying such as setting up of intermediate points between inter visible points, establishment of inter visible points, prolonging a line, laying out traverse etc.

1.2 Types of Theodolite:

There are different types of theodolite available. It may be classified into four broad categories.

- Transit Theodolite
- Non-transit theodolite
- Vernier theodolite
- Micrometer theodolite
- Electronic Theodolite

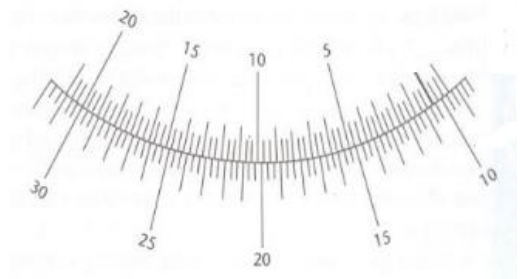
Theodolite

	
Transit	**Non transit**

1.2.1. Transit theodolite: A transit theodolite is one in which the telescope can be revolved through a complete revolution about is horizontal axis. (Trunnion axis or Transverse axis) in a vertical plane.

1.2.2. Non- Transit theodolite: A Non- transit theodolite is one in which the telescope cannot be revolved through a complete revolution about is horizontal axis in a vertical plane. In non-transit theodolite the telescope is mounted in such a manner that the line of sight cannot be reversed by revolving the telescope. Such theodolite has become obsolete now and transit theodolite are only used widely.

1.2.3 Vernier theodolites: This is most commonly used. In this type of instrument, observations are taken by using the principle of vernier caliper. The precision of this type of instrument varies in the order of 10" to 20".



Vernier theodolite

The vernier is an auxiliary graduation, placed alongside the main one with the purpose of determining fractions of the graduation unit. The vernier is typically used with theodolites of medium or low accuracy, as well as for simple angle measuring devices.

1.2.4. Micrometre theodolites: Micrometres fitted to read the horizontal and vertical angles.

The circles are graduated to 10 minute divisions and the readings are taken through two micrometer eyepieces placed 180 degrees apart on the alidade. The degrees and minutes can be read directly off the circles, while for greater accuracy the micrometer will give readings directly to 2 seconds and estimations to 0.5 of a second.

Micrometer theodolite

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1.2.5. Electronic Theodolite: In an electronic theodolite, absolute angle measurement instrument is provided by a dynamic system with Opto-Electronic scanning. The electronic theodolites are provided with control panel with key board and LCD panel displays

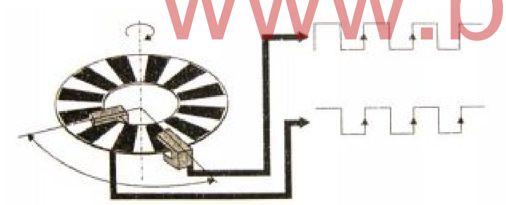
Electronic Theodolite

Opto-Electronic scanning principle

Electronic theodolite has micro-processor controlled angle measurement system of high accuracy. Absolute angle measurement is provided by a dynamic system with opto-electronic scanning.

As the graduations around the full circle are scanned for every reading, circle graduation error cannot occur. Scanning at diametrically opposite positions eliminates the effect of eccentricity. Circle readings are corrected automatically for index error and horizontal collimation error. Thus angle measurement can be taken in one position to a far higher accuracy than with conventional theodolites.

Description:



Microprocessor controlled angle measurement system

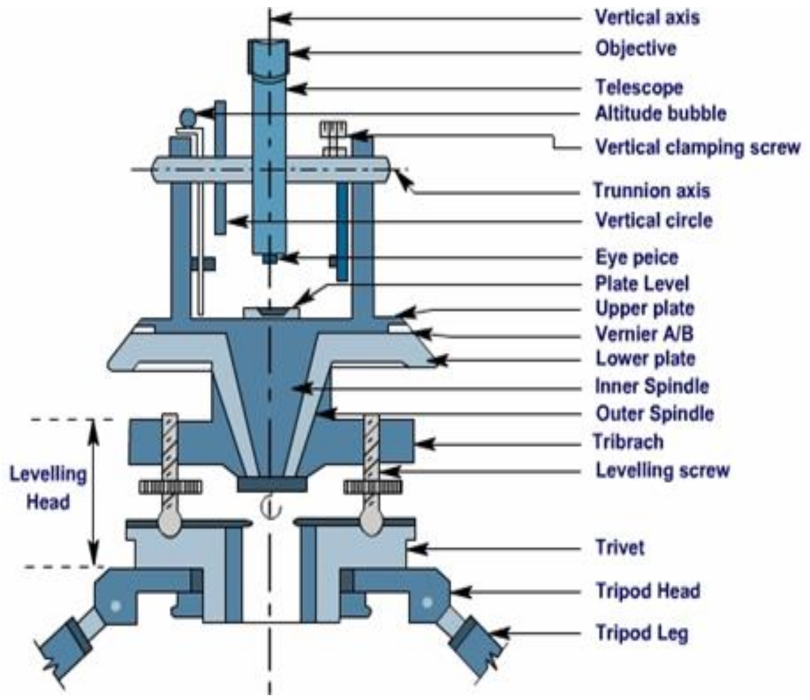
It has 30 x telescopes which gives a bright, high-contrast image. The course and fine focusing ensures that the target is seen sharp and clear. Pointing is fast and precise even in poor observing conditions. Theodolite has electronic clamp for circle setting to angle measurements. The whole instrument is controlled through the keyboard. The theodolite has two control panels, each with keyboard and two LCDs displays. It can be used easily and quickly in both positions. The main operation requires only as single Keystroke. Accepted keystrokes are acknowledged by a beep.

Characteristics of electronic theodolites

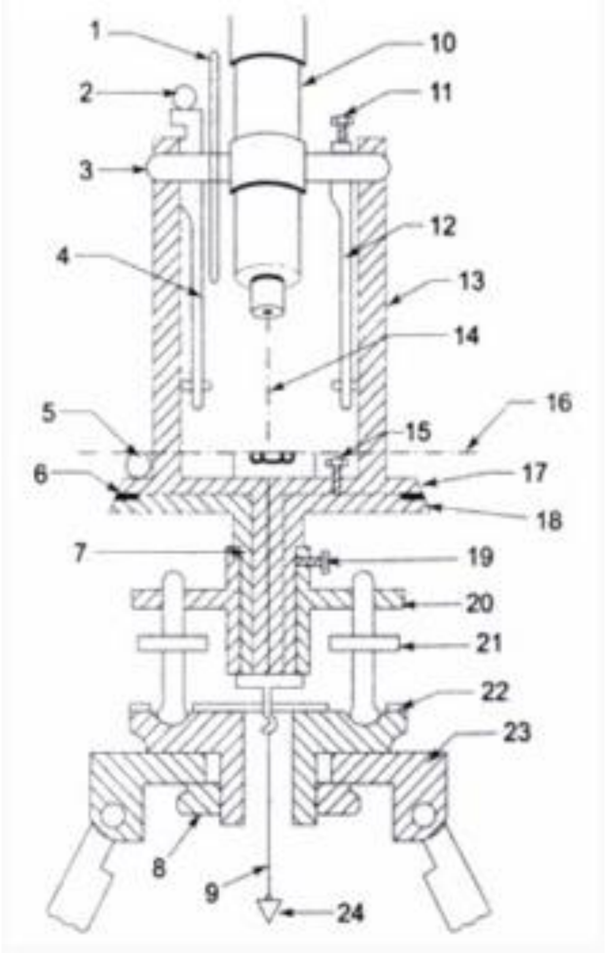
- Angle least count can be 1" with precision ranging from 0.5" to 20"
- Digital readouts eliminate the personal error associated with reading and interpolation of scale and micrometer settings.
- Display window/unit for horizontal and vertical angles available at either one or both ends.
- Some digital theodolites have modular arrangement where they can be upgraded to be a total station or have an EDMI attached for distance measurements.
- Vertical circles can be set to zero for horizon or zenith along with the status of battery shown in the display window.
- Typical specifications for digital theodolites are generally given as follows:
    - Magnification: 26X to 30X
    - Field of view (FOV) 1.5^0.
    - Shortest viewing distance 1.0 m
    - Angle readouts, direct 5" to 20"
    - Level sensitivity: plate level vial 40"/2 mm, circular level vial 10"/2 mm

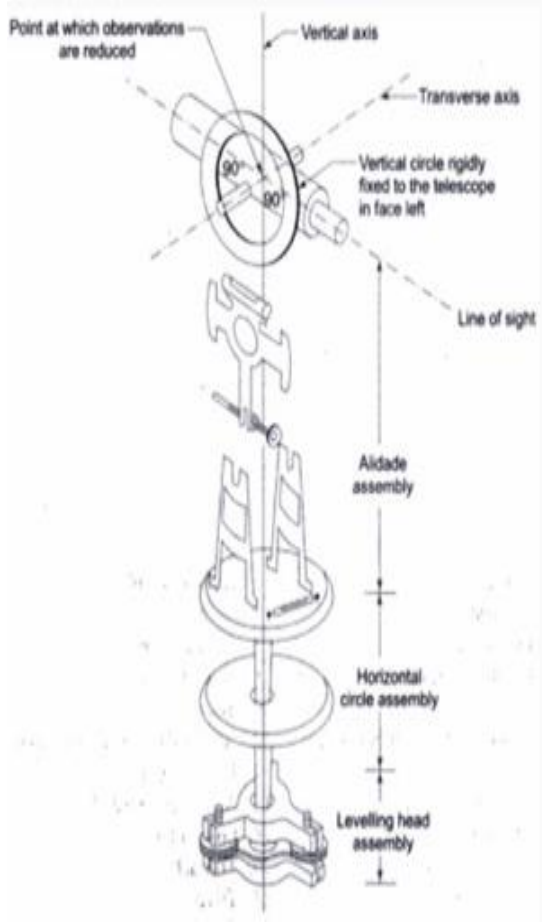
1.3 Component Parts of Theodolite - Functions The salient parts of a vernier theodolite have been discussed below:

➢ Levelling Head	➢ Plate Levels	➢ Vertical Circle
➢ Shifting Head	➢ Standard (or a Frame)	➢ Altitude Bubble
➢ Lower Plate	➢ Vernier Frame	➢ Clamp Screws and
➢ Upper Plate	➢ Telescope	➢ Tangent Screw



Sectional view of a theodolite





Parts of theodolite		
1.Vertical circle	2.ltitude bubble	3.Horizontal axis
4.Vernier arm	5.Plate bubble	6.Graduate arc
7.Levelling head	8.Clamping jut	9.Vertical axis
11.Telescope	12.Arm of 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.Tribrach	21.Foot screw	22.Trivet
23.Tripod top	24.Plumb bob	

1.2.7 Function:

Leveling Head: It is the lowermost part of a theodolite. It consists of two parallel horizontal plates separated by three levelling screws. The lower plate with a large threaded hole in its centre is called trivet or foot plate. It provides a means to place the instrument on (tripod) stand and get it screwed. Its central aperture provides a way for suspending a plumb bob. The upper plate of the levelling head is called the tribrach. It contains a tapered bearing at the centre. It has three arms each carrying a levelling screw. It provides a support for the upper part of the instrument. The principal use of levelling head is to provide a means for levelling the instrument.

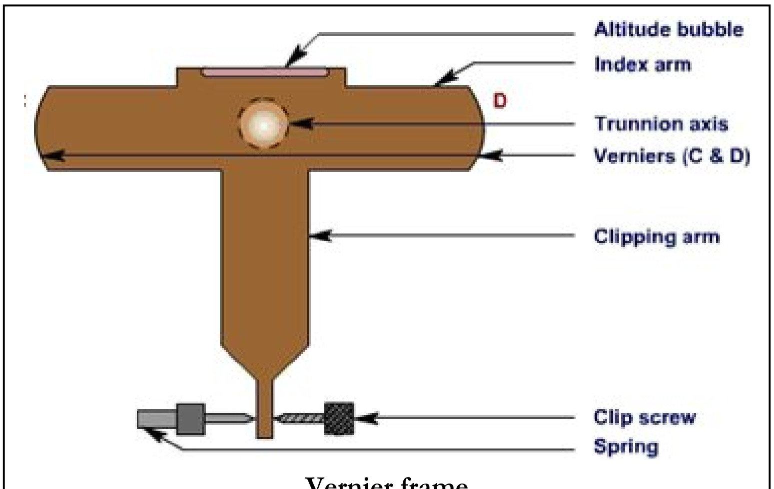
Shifting Head: It consists of a pair of horizontal plates and an annular treaded ring. One of the plates is placed below the lower plate but above the tribrach and the other below the tribrach. The annular treaded ring is placed in between lower plate and the tribrach which is used to tighten/untighten the whole of the instrument. The shifting head is used for exact centering of the instrument after leveling has been completed.

Lower Plate: It is a horizontal circular plate monolithically constructed with the outer spindle. A scale is engraved at its bevelled edge with divisions in degrees and minutes increasing in clockwise direction. It provides the main scale reading of a horizontal angle and a means to fix / unfix the whole of the instrument.

Upper Plate: It is a horizontal circular plate monolithically constructed with the inner spindle. It is fitted with two diametrically opposite vernier scales designated as A and B. Functions of upper plates are to support a pair of magnifiers for the verniers, a pair of plate levels, a pair of support "A" frames for telescope and a means to fix / unfix the upper plate of the instrument with its lower plate.

Plate Levels: A pair of level tubes are placed at right angles on the upper plate. These are used to make the vertical axis of the instrument truly vertical i.e., for levelling of the instrument.

Standard (or A Frame): Two standards resembling the letter A are attached on the upper plate. These provides the bearings of the pivots of the telescope allowing it to rotate on its trunnion axis in vertical plane. The vernier frame and arm of vertical circle clamp are also attached to it.



Vernier frame

Vernier Frame: Also called T -frame or index frame, consists of a vertical leg known as clipping arm and a horizontal bar called the index arm engraved with verniers C and D at its ends. Each of the verniers at C and D are having two scales which increases in opposite directions.

It is used as seat for altitude bubble and also provides vernier reading for vertical angle measurement.

Telescope: The telescope of a theodolite is identical in structure and uses, as in case of a dumpy level. But, in theodolite, the telescope is mounted on a horizontal spindle called the horizontal axis or the trunnion axis to rotate it also in vertical plane.

Vertical Circle: The vertical circle is attached with the trunnion axis. It is engraved with a scale reading vertical angle in degrees and minutes. The vertical circle is divided into four quadrants each reading 0° to 90° with 0° - 0° either along vertical or in horizontal. It provides the main scale reading for vertical angle.

Altitude Bubble: A sensitive level tube placed on vernier frame is called altitude bubble. It is used to make horizontal axis truly horizontal.

Screws: A theodolite instrument has number of screws as its component parts. These are classified into different types depending on their functions.

1. Levelling Screws 2. Clamp Screws 3. Tangent Screws

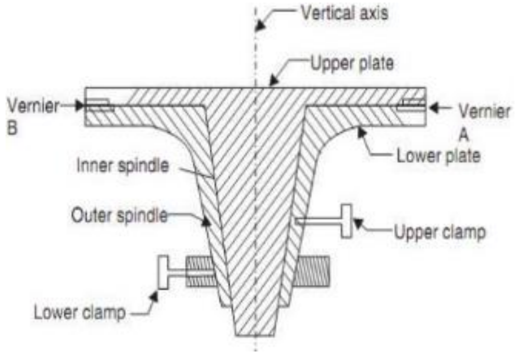
Leveling Screws: These are present in the levelling head of a theodolite in between trivet and tribrach. These work in threaded holes in the tribrach arms and their lower ends rest in recesses in the trivet. These screws are used for levelling the instrument i.e., to make plate level axis truly horizontal.

Clamp screws: These are used to fix the parts of a theodolite with which these are attached.

1. Lower Plate Clamp Screw 2. Upper Plate Clamp Screw 3.Vertical plate Clamp Screw

1. Lower plate Clamp Screw: The clamp screw attached to the lower plate of a theodolite is called lower plate clamp screw. When it is tightened, the outer spindle gets fixed with the tribrach, and, thus, the lower plate gets fixed in position.

2.Upper plate Clamp Screw: The clamp screw attached with the upper plate of a theodolite is called upper plate clamp screw. When it is tightened, the inner spindle gets fixed with the outer spindle and, thus, the upper plate gets fixed in position. The manipulation of the upper plate



Details of upper and lower plate

and lower plate clamp screws provide three conditions:

- When both the upper plate clamp screw and the lower plate clamp screw are tightened, the instrument gets fully fixed.
- When the upper plate clamp screw is tightened and the lower plate clamp screw is opened, the instrument rotates on its outer axis, there is no relative motion between the two plate and the readings in the horizontal vernier scales do not change.
- When the lower plate clamp screw is tightened, and the upper plate is opened, the instrument rotates on the inner axis with outer axis fixed. The readings in the horizontal vernier scales change.

3. Vertical plate Clamp Screw

It is present on a frame fixed with standard and above the shaft of trunnion axis. It is used to clamp the telescope in any plane and hence at any desired vertical angle.

Tangent Screws

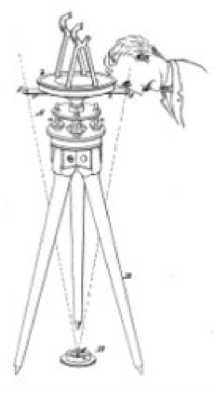
With each clamping screw, there is a tangent screw present in the instrument to provide fine movement. The tangent screws work only after its clamping screws get tightened. Thus when the upper clamp screw has been tightened, small movement of the upper plate can be made by the upper tangent screw; when the lower clamp screw has been tightened, small movement of the lower plate can be made by the lower tangent screw and similarly for vertical clamp screw.

Tripod Stand

The theodolite is mounted on a strong tripod when being used in the field. The legs of the tripod are solid or framed. At the lower ends of the legs, pointed steel shoes are provided to get them pushed into ground. The tripod head has screws on which the trivet of the leveling head is screwed

1.4 Technical Terms Used in Theodolite Survey

1. Centering: setting up the theodolite exactly over a station mark by means of either a plumb bob or optical plummet available with the instrument.



Centering

2. Transiting: It is also known as plunging or reversing. It is the process of turning the telescope about its horizontal axis through 180^0 in the vertical plane thus bringing it upside down and making it point exactly in opposite direction.

3. Swinging the telescope: It means turning the telescope about its vertical axis in the horizontal plane. A swing is called right or left according as the telescope is rotated clockwise or counter clockwise.

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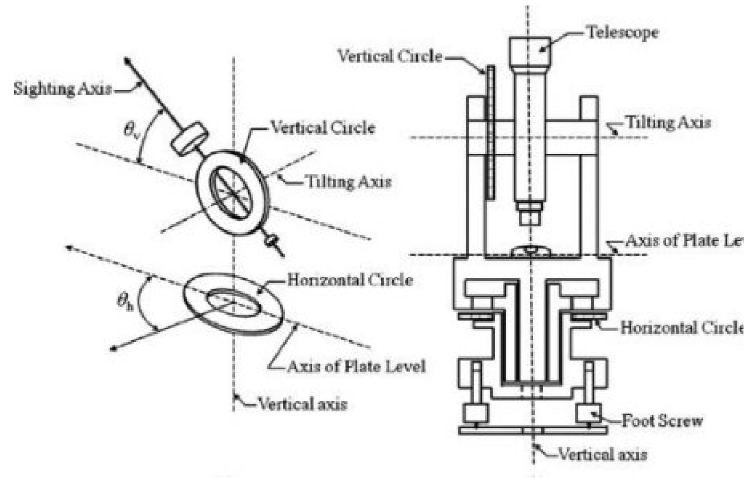
4. Face left: If the vertical circle of the instrument is on the left of the observer while taking a reading, the position is called the face left and the observation taken on the horizontal or they are vertical circle in this position, is known as the face left observation.

5. Face Right: If the vertical circle of the instrument is on the left of the observer while taking a reading, the position is called the face right and the observation taken on the horizontal or they are vertical circle in this position, is known as the face right observation.

6. Changing Face: It is the operation of bringing the vertical circle to the right of the observer, if originally it is to the left, and vice versa. It is done in two steps: firstly, revolve the telescope through 180^0 in a vertical plane then rotate it through 180^0 in the horizontal plane i.e. first transit the telescope and then swing it through 180^0.

7. Line of collimation: It is also known as the line of sight. It is the imaginary line joining the intersection of the cross hairs of the diaphragm to the optical centre of the object- glass and in its continuation.

8. Axis of the telescope: It is also an imaginary line joining the optical center of object-glass to center of the eye-piece.



Fundamental axis of a theodolite

9. Axis of the level Tube: It is also called the bubble line. It is a straight line tangential to the longitudinal curve of the level tube at the centre of the tube. It is horizontal when the bubble is central.

10. Vertical axis: It is the axis about which the telescope can be rotated in the horizontal plane.

11. Horizontal axis: It is also called the trunnion axis or the transverse axis. It is the axis about which the telescope can be revolved in the vertical plane.

1.5 Temporary Adjustment of Vernier Theodolite: At each station point, before taking any observation, it is required to carry out some operations in sequence. The set of operations those are required to be done on an instrument in order to make it ready for taking observation is known as temporary adjustment. Temporary adjustment of a vernier theodolite consists of following operations:

1. Setting, 2. Centering, 3. Leveling and 4. Focusing.

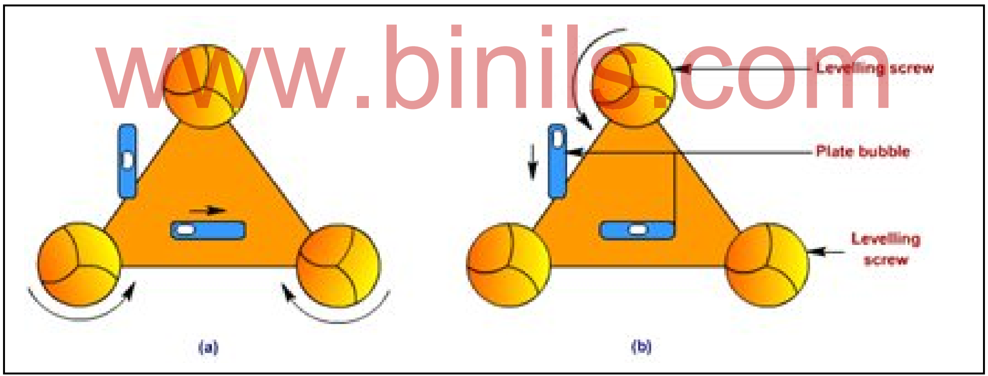
Temporary adjustment

1. Setting

The setting operation consists of fixing the theodolite with the tripod stand along with approximate leveling and centring over the station. For setting up the instrument, the tripod is placed over the station with its legs widely spread so that the centre of the tripod head lies above the station point and its head approximately level (by eye estimation). The instrument is then fixed with the tripod by screwing through trivet. The height of the instrument should be such that observer can see through telescope conveniently. After this, a plumb bob is suspended from the bottom of the instrument

2. Centering: The operation involved in placing the vertical axis of the instrument exactly over the station mark is known as centring. First, the approximate centring of the instrument is done by moving the tripod legs radially or circumferentially as per need of the circumstances. It may be noted that due to radial movement of the legs, plumb bob gets shifted in the direction of the movement of the leg without seriously affecting the level of the instrument. On the other hand, when the legs are moved sideways or circumferentially, the plumb does not shift much but the level gets affected. Sometimes, the instrument and the tripod have to be moved bodily for centring. It must be noted that the centering and leveling of instrument is done recursively. Finally, exact centering is done by using the shifting head of the instrument. During this, first the screw-clamping ring of the shifting head is loosened and the upper plate of the shifting head is slid over the lower one until the plumb bob is exactly over the station mark. After the exact centering, the screw clamping ring gets tightened.

3. Leveling



Levelling of an instrument is done to make the vertical axis of the instrument truly vertical. Generally, there are three levelling screws and two plate levels are present in a theodolite instrument. Thus, levelling is being achieved by carrying out the following steps

<u>Step 1:</u> Bring one of the level tube parallel to any two of the foot screws, by rotating the upper part of the instrument.

<u>Step 2:</u> The bubble is brought to the centre of the level tube by rotating both the foot screws either inward or outward. The bubble moves in the same direction as the left thumb. [Figure 2.5 (a)]

Step 3: The bubble of the other level tube is then brought to the centre of the level tube by rotating the third foot screw either inward or outward [Figure 2.5 (b)]. [In step 1 itself, the other plate level will be parallel to the line joining the third foot screw and the centre of the line joining the previous two foot screws.]

Step 4: Repeat Step 2 and step 3 in the same quadrant till both the bubble remain central.

Step 5: By rotating the upper part of the instrument through 180°, the level tube is brought parallel to first two foot screws in reverse order. The bubble will remain in the centre if the instrument is in permanent adjustment.

Otherwise, repeat the whole process starting from step1 to step5.

4. Focusing: To obtain the clear reading, the image formed by the objective lens should fall in the plane of diaphragm and the focus of eye-piece should also be at the plane of diaphragm. This is being carried out by removing parallax by proper focusing of objective and eye-piece. Thus, focusing operation involves two steps:

a. Focusing of the eye-piece lens b. focusing of the objective lens.

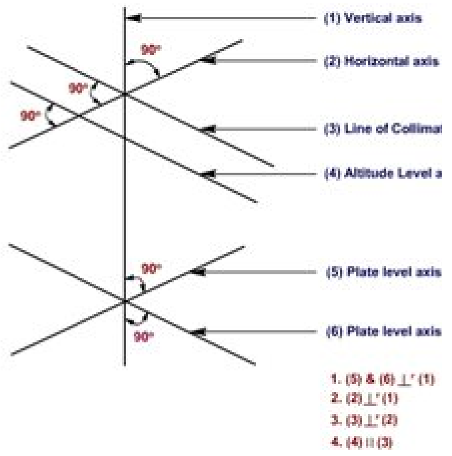
a. **Focusing of Eye-piece:** The eye-piece is focused to make the appearance of cross hairs distinct and clear. This is being carried out in steps: First, point the telescope towards the sky or hold a sheet of white paper in front of the objective; next, move the eye-piece in or out by rotating it gradually until the cross hairs appear quite sharp and clear. Focusing of eye-piece depends on the eye-sight of observer and so for each observer it needs to adjust accordingly.

b. Focusing of Objective: It is done for each independent observation to bring the image of the object in the plane of cross hairs. It includes following steps of operation: First, direct the telescope towards the object for observation. Next, turn the focusing screw until the image of the object appears clear and sharp as the observer looks through properly focused eye-piece. If focusing has been done properly, there will be no parallax i.e., there will be no apparent movement of the image relative to the cross hairs if the observer moves his eye from one side to the other or from top to bottom.

1.6.1 Fundamental Lines of a Theodolite

- The fundamental lines are imagined in a theodolite instrument (Adjacent Figure) are

1. Vertical Axis
2. Horizontal axis
3. Line of collimation
4. Axis of the altitude level tube
5. Axis of the plate level



Fundamental Lines of a Theodolite and their relationship

1.6.2 Relations among Fundamental Lines

In a perfectly adjusted instrument, the fundamental lines bear relations as follows:

1. The vertical cross hair should lie in a plane perpendicular to the horizontal axis.

2. The axis of each plate level should lie in a plane perpendicular to the vertical axis

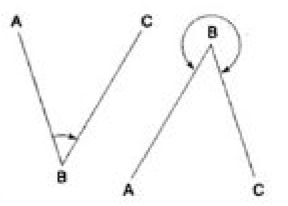
3. The horizontal axis should be perpendicular to the vertical axis.

4. The axis of the telescope level should be parallel to the line of sight

5. The line of sight should be perpendicular to the horizontal axis at its intersection with the vertical axis. Also, the optical axis, the axis of the objective slide, and the line of sight should coincide.

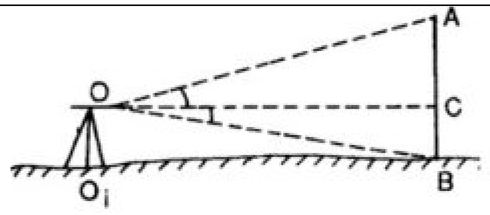
1.7 Measurement of Direction using Theodolite

The primary observation during surveying are the horizontal angles and the vertical angles measurements. These quantities can be observed directly in the field using theodolite.



Interior &Exterior

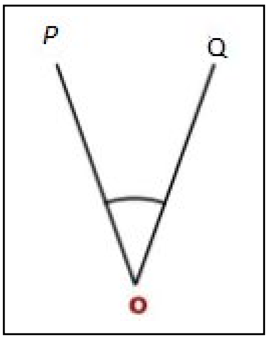
Horizontal angel



Vertical angle

1.7.1Measurement of Horizontal Angle by general method

To represent the direction of a line, the horizontal angle of the line from a reference line is to be measured. The steps required to be adopted are as follows:



1. Two points ne on each of the lines, say P and Q, are to be marked.
2. A transit theodolite is to be set at the point of intersection of the lines, say at O. Initially, the instrument is in the face left condition and its temporary adjustment is to be done over the point O.

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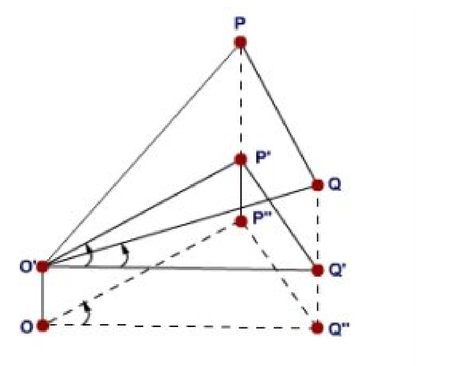
3. Both the lower and upper plate main screws are to released and get the vernier A set to 0° (or 360°) mark on the main scale. After clamping the upper main screw, index of vernier A is to be brought exactly to the zero of the main scale using the upper plate tangent screw.
4. At this stage the reading of the vernier B should be 180°.



5. Swing the telescope in the horizontal plane and point it to the left station, say P. Tighten the lower plate clamp screw, and bisect the signal at P exactly using the lower plate tangent screw. Record the readings in the form of Table 1.1.
6. Loosen the upper plate main screw and turn the telescope the signal at Q is sighted. Tighten the upper clamp screw and bisect the ranging pole at Q exactly using the upper plate tangent screw.
7. Read both the verniers A and B and record the readings. The reading of the vernier A is the angle POQ. The vernier B gives the value of angle POQ after deducting from it 180°. The mean of two values of the angles obtained from the verniers A and B is the required angle P'O'Q'.
8. Change the face of the instrument to the face right by transiting the telescope and swinging it by 180°.

Table 1.1 Measurement of a horizontal angle

Inst. at	Sighted to	Face left								Swing Right				Face right									Swing left				Average horizontal angle		
		A			B		Mean			No. of repetition	Horizontal angles			A			B		Mean			No. of repetition	Horizontal angles					
		°	'	"	'	"	°	'	"		°	'	"	°	'	"	'	"	°	'	"		°	'	"	°	'	"
O	P	00	00	00	00	00	00	00	00					00	00	00	00	00	00	00	00							
	Q	58	43	20	4	20	58	43	20	1	58	43	20	58	43	40	43	40	58	43	40	1	58	43	40	58	43	3

9. Repeat steps 3 to 8 and determine another value of the angle P'O'Q'.
10. The mean of the face left and face right observations is the final required angle P'O'Q'.

1.7.2 Method of Repetition

When the precision of measurement of a horizontal angle is desired to be more than the least count of the instrument, repetition method is used. In this method, the desired angle is measured several times, and average of the observed values is considered as the value of the angle. The precision thus attained is to a much finer degree than the least count of the vernier. The steps involved in the measurement of the horizontal angle, say POQ at (Figure 1.1) by method of repetition are as follows:

Repetition method: In surveying, the repetition method is used to improve precision and accuracy of **measurements of horizontal angles**. The same **angle** is **measured** multiple times, with the survey instrument rotated so that systematic errors tend to cancel. The arithmetic mean of these observations gives true value of an **angle**.

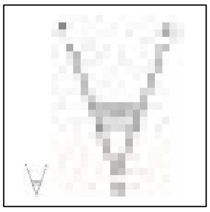


Fig. 1.1

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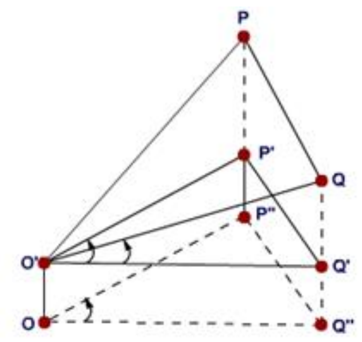
Steps 1 to 7 is same as given in method of measurement of horizontal angle by general method but record readings in the form of Table 1.2



Horizontal angle between P and Q is
∠P'O'Q' or ∠P"OQ"

8. Unclamp the lower plate, and turn the telescope to sight the signal P again. Tighten the lower clamp. Use the lower plate tangent screw for exact bisection of the signal P. (The vernier readings should be as it was during previous reading).

9. Release the upper clamp and turn the telescope to sight the signal Q. Tighten the upper clamp. Bisect the signal Q exactly using the upper tangent screw. The vernier A will give the value which is about twice the angle POQ.

10. Repeat steps (8) and (9) once again. The final reading of the vernier A will be approximately thrice the angle POQ. If necessary, more repetitions can be done.

11. Divide the final reading by the number of repetition to obtain the value of the angle POQ. For every completed revolution of the circle to the final reading, if necessary, add 360°.

12. Change face of the instrument to the face right. The telescope will be in the inverted condition. Repeat steps (2) to (9), with the face right, and determine another value of the angle POQ.

13. Determine the average value of the angles obtained with the face left and face right.

The method of repetition eliminates different errors present in measurement of horizontal angle. These are as follows:

1. The errors due to eccentricity of verniers and centres get eliminated as readings from both the verniers are taken.
2. The errors due to inaccurate graduations get eliminated as the readings are observed at different parts of the circle.
3. The errors due to lack in adjustment of line of collimation and the horizontal axis of the instrument get eliminated for considering both faces readings.
4. Errors due to inaccurate bisection of the object, eccentric centering etc are eliminated partially as these get counter-balanced in different observations.

However, the errors due to slip, due to displacement of station or its signal do not get eliminated and moreover, these errors are of cumulative in nature.

Measurement of a horizontal angle by repetition

Table 1.2

		Face left									Swing Right			Face right										Swing left		Average horizontal angle		
Inst. at	Sighted to	A			B		Mean			No. of repetition	Horizontal angles			A			B		Mean			No. of repetition	Horizontal angles					
		°	′	″	′	″	°	′	″		°	′	″	°	′	″	′	″	°	′	″		°	′	″	°	′	″
O	P	00	00	00	00	00	00	00	00					00	00	00	00	00	00	00	00							
	Q	58	43	20	43	20	58	43	20	1	58	43	20	58	43	40	43	40	58	43	40	1	58	43	40			
	Q	168	19	40	19	40	168	19	40	3	58	43	33	168	19	20	19	20	168	19	20	3	58	43	27	58	43	30

`

1.7.3 Method of Reiteration

Method of reiteration for measurement of horizontal angle is usually adopted in case several angles of well distributed points/ objects are to be measured from the same instrument station with high precision. In this method, angles are measured successively starting from a point termed as initial station (Figure 1.2). The angle between the terminating station and the initial station is the last observation during a set of measurement of horizontal angle by method of reiteration. This process of measuring the angles at an instrument station round the point is to obtain a check on their sum being equal to 360° and is called closing the horizon. When the horizon is closed, the final reading of the vernier should be the same as its initial reading if there is no discrepancy. Figure 1.2 shows a instrument station O where the angles POQ, QOR and ROS have to be measured by method of reiteration. The steps involved in the measurement of the horizontal angles by method of reiteration are as follows:

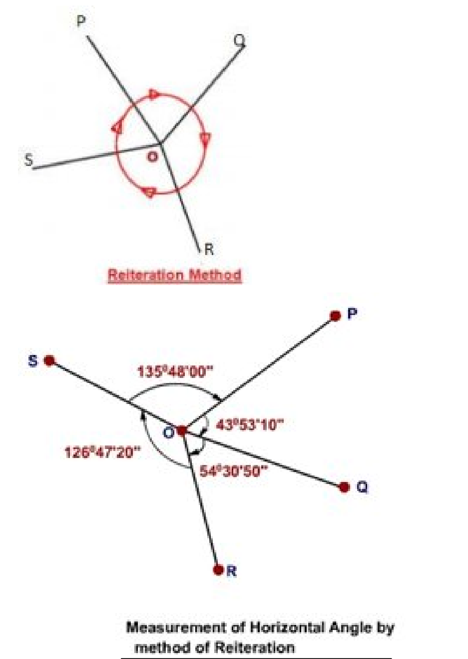


Fig.1.2

Steps 1 to 7 are same as given in measurement of horizontal angle by general method and record readings in Table 1.3.

8. Loosen the upper plate clamp screw and turn the telescope clockwise until the station R is sighted. Tighten the upper clamp screw. Use the upper tangent screw for placing the object R on the vertical cross hair. Read both the verniers, and record readings in the Table 1.3. Compute the angle QOR. And note down in the table.

9. Likewise, determine the angle ROS.

10. Finally, close the horizon by sighting the reference object P again. Note down the readings. The vernier A should now read zero (or 360°).

Table.1.3 Measurement of a horizontal angle by reiteration method

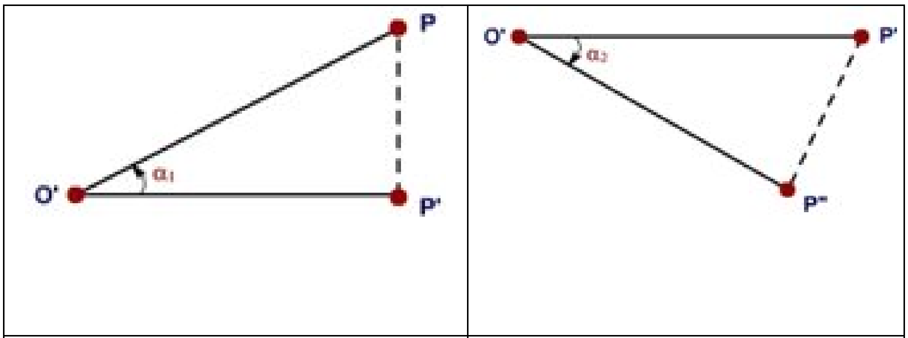
Inst. at	Sighted to	Face left									Swing Right		Face right									Swing left		Average horizontal angle		
		A			B		Mean			Horizontal angles			A			B		Mean			Horizontal angles					
		°	'	"	'	"	°	'	"	°	'	"	°	'	"	'	"	°	'	"	°	'	"	°	'	"
O	P	00	00	00	00	00	00	00	00				00	00	00	00	00	00	00	00						
										43	53	10									43	53	10	43	53	10
	Q	43	53	10	53	10	43	53	10				43	53	00	53	00	43	53	10						
										54	30	50									54	30	50	54	30	50
	R	98	24	00	24	00	98	24	00				98	24	00	24	00	98	24	00						
										126	47	20									126	47	20	126	47	20
	S	225	11	20	11	20	225	11	20				225	11	20	11	20	225	11	20						
										134	46	40									134	48	40	134	48	40
	P	360	00	00	00	00	360	00	00				360	00	00	00	00	360	00	00						

`

11. Now change the face left of the instrument to the face right by transiting (plunging) the telescope and swinging it through 180°. Repeat steps 3 to 10 in the anti-clockwise direction.

12. The average value of each angle obtained with the face left and the face right provides the observed values of the angles.

1.8 Measurement of Vertical Angle



Elevation angle **Fig.1.3** **Depression angle**

A vertical angle is the angle between the inclined line of sight and the horizontal plane through the trunnion axis of the instrument. Prior to the measurement of vertical angle, instrument is required to be leveled with reference to the altitude level. Figure 1.3 shows vertical angles.

The procedure for measuring a vertical angle is as follows:

1. The temporary adjustment of the instrument is to be done on the station.
2. Then, leveling of theodolite is to be done using altitude level (the operations involved are same as leveling using plate level).

3. Loosen the vertical circle clamp, and direct the telescope towards the object whose vertical angle is required to be measured. Clamp the vertical circle, and bisect the point by turning the vertical tangent screw.
4. Read and record the scale with vernier C and D in Table 1.4
5. Change the face of the instrument and read the vertical angle again.
6. The required vertical angle is the average of the values in steps 4 and 5.

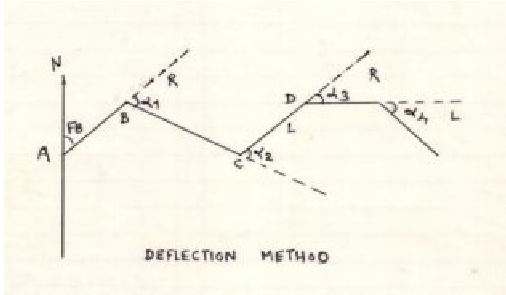
Table 1.4 Measurement of vertical angle

		Face left													Face left													Average vertical angle		
Inst. at	Sighted to	C			D		Mean			Vertical angles			C			D		Mean			Vertical angles					
		°	'	"	'	"	°	'	"	°	'	"	°	'	"	'	"	°	'	"	°	'	"	°	'	"
O	P	+28	24	20	24	00	+28	24	10	+28	24	10	+28	24	40	24	20	+28	24	30	+28	24	30	+28	24	30
	P"	-24	35	40	35	20	-24	35	40	-24	35	40	-24	36	00	35	40	-24	35	50	-24	35	40	-24	35	35

Angle POR = 53° 59' 55"

1.9 Measurement of deflection angle:

Theory: The term deflection angle is used to mean the angle made by survey line to the extension of the preceding line.



Guide sketch

Instrument required

1. Theodolite
2. Ranging rods

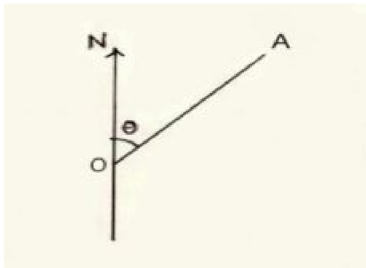
Procedure

1. Set up the instrument at station A and measure the bearing of AB.
2. Set up the theodolite at B and level it accurately.
3. Set up the vernier A to read 360^0 or 0^0 and read the vernier.
4. Loosen the lower clamp and bisect A accurately. Again, check the vernier readings.
5. Transit the telescope.
6. Release the upper plate and bisect the point C exactly by using upper tangent screw.
7. Read both the verniers. The mean of the two vernier readings gives the value of the deflection angel at B.
8. Loosen the lower plate and turn the telescope in a horizontal plane and again bisect A with verniers still reading the same approximate value of the deflection angle and the telescope inverted.
9. Transit the telescope and release the upper plate and take a fore sight again on C. Read both the verniers.
10. Find out the mean of the final vernier readings which gives twice the value of the deflection angle. One -half of this mean value will give the value of deflection angle at B.
11. Repeat the whole procedure to find the deflection angles at C, D, etc.

1.10 Reading bearing of a line

Instrument required

1. Theodolite
2. Ranging rods
3. Trough compass



Guide sketch:

Procedure

1. Set up the instrument at station o and level it
2. Insert the trough compass in the groove available in the instrument
3. Set the A vernier to read 0^o /360^o
4. Loose the lower clamp, rotate the telescope in clockwise direction until the North of the magnetic needle comes to its centre. Now the telescope is in magnetic North direction.
5. Loose the upper clamp, rotate the telescope to bisect the object or station A

6. Note down the vernier readings
7. The measured angle will be the magnetic bearing of the survey line OA

1.11 Theodolite traversing

→ The traverse survey consist of a continuous series of lines from which offsets may be taken to locate the positions of objects to be located.

→ The lengths of survey lines are measured by chaining or any other suitable method

→ The relative directions of the survey lines are found by the ∟measurement of angels or bearings.

Types of traverse

The following are the two types of traverse

1. Closed traverse
2. Open or unclosed traverse.

Closed traverse survey by theodolite

→ "The closed traverse is one in which the point of start is the same as the point of finish of the survey". (starting and end point are same)

→ In this case the measurements can be checked and adjusted or balanced.

→ This type of traverse is used to locate the perimeter or area of any moderately large survey.

→ But particularly it is suitable for the determination of the boundaries of lakes or woods across which tie lines cannot be measured conveniently.

Open or unclosed traverse survey by theodolite

→ "The open traverse is one in which the point of start is not same as the point of finish of the survey" (starting and end point are not same)

→ In this case the measurements cannot be checked and adjusted or balanced.

→ The open traverse is useful when the survey of a long narrow strip of country like a valley, a river, etc.

1.12 Methods of traversing by theodolite

The following methods by which the relative directions of "lines" may be obtained are as follows

A. By measurement of angles between two successive lines (where accuracy is required)
B. By direct observation of bearings of the survey lines (where accuracy is not required)

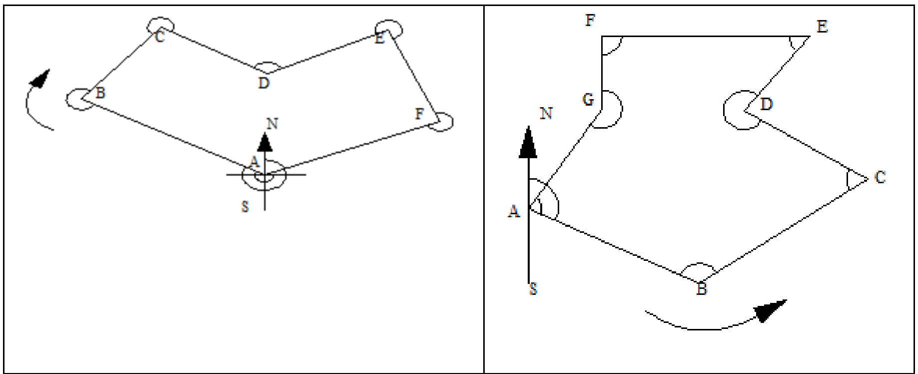
A. Traversing by direct observation of angles.

Direct observation of angles can be done b

A1 Traversing by the method of included angles,
A2. Traversing by the method of direct angles, and
A3. Traversing by the method of deflection angles.

A1 Traversing by included angles

→ This method is specially used in land surveying. The bearing of the initial line and the included angles between the successive survey lines are measured.
→ It is suitable where great accuracy is required.



Traverse running in clockwise direction	Traverse running in counter clockwise direction

AIM: To conduct a closed traverse of ABCDEFG, the following procedure is to be followed.

→ The theodolite is set up over station A and the bearing of line AB is observed.
→ Angle GAB is measured by taking a back sight on the preceding station G and a foresight on the forward station B, rotating the telescope clockwise as below.
→ Set the A vernier to read 0°or 360°, rotate telescope and bisect station G.
→ Loose the tangent screw, rotate telescope and bisect the object A.
→ Both the vernier are read; the mean of the two vernier readings gives the required angles GAB.
→ To eliminate the instrumental errors, face left and face right observations should be taken.
→ The theodolite is then moved to each of the successive stations B, C, D etc., and angles ABC, BCD, CDE, etc. are measured in the usual way and recorded in the field book.
→ For very precise works, the angles must be measured by repetition method, in facelift and face right conditions.

A2. Traversing by direct angles

This method of traverse survey is generally used in open traverses.

→ The transit is set up at the starting station A and the bearing of the line AB is taken.

→ The theodolite is then moved to station B. At station B, with vernier A at zero, a back sight is taken on the preceding station A.

→ Loosening the upper plate and turning the telescope in a clockwise direction, a foresight is taken on the following station C, and both the verniers are read.

→ The average reading gives the required angle ABC. Similarly, all the angles are measured at stations C, D etc.

→ The traverse lines AB, BC, etch are measured and the necessary offsets are taken from the traverse lines in the usual way.

A3. Traversing by deflection angles:

This method is usually used for open traverses in places like survey of canals, highways, railways, etc. when the survey lines make small deflection angles with each other.

→ For conducting a an open traverse ABCDE the following procedure is to be followed.

→ The theodolite is set up at station A and the bearing of the initial line AB is observed.

→ The transit is then moved to station Band with vernier A set at zero, a back sight is taken on the preceding station A.

→ The telescope is then plunged and a foresight is taken in the counter-clockwise direction (towards left) by means of the upper clamp and the tangent screw.

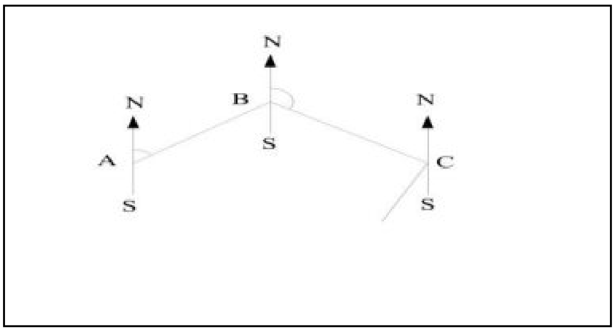
→ Both the verniers are then read. the mean of the two readings gives the defalcation angle of BC from AB.

→ The direction, as already described, should be carefully noted in the field book. In the similar manner, the other deflection angles are measured and recorded.

→ From the traverse lines, offsets can be taken in the normal way. To eliminate the errors of adjustment, the angles should be measured from both face left and face right.

B. Traversing by direct observation of bearings of the survey lines:

This method of traversing by direct observation of bearings (directions) is also called the "fast needle method of traversing"



Traversing by fast needle method:

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The "fast needle" is a term which is generally confined to "dialing" but the principle remains the same when it is applied to a theodolite traverse. There are mainly three methods of observing the bearing in the field, viz:

B1. The direct method in which the telescope is transited

B2. The direct method in which the telescope is not transited; and

B3. The back bearing method.

B1.Direct method in which the telescope is transited:

→ Set up the theodolite at station A and level it accurately) Fig.

→ Set vernier A at zero. Direct the telescope to the magnetic north as indicated by the needle of the tubular compass by using the lower clamp and tangent screw.

→ Loosen the upper plate and sight point B exactly by upper clamp and tangent screw. Read vernier A which gives the bearing of line AB, say x

→ Move the transit and set it up at station B, check the vernier reading which should still read x. If not, correct the reading exactly by means of upper slow motion screw.

→ Read again vernier A. The reading β gives the bearing of line BC.

→ Again with vernier A clamped at β, transfer the instrument to station C and repeat the whole process.

1.13 Field checks in closed traverse

(i) Traversing by included angles

In case of closed traverse, the sum of the measured angles is very rarely equal to the theoretical sum. The theoretical sum means:

1. The sum of the measured interior angles should be equal to (2 n-4) right angles; and
2. The sum of the measured exterior angles should be equal to 2 n+4) right angles.

There may be a small discrepancy which in case of angular measurements is called the "angular error of closure". This error should not exceed the least count of the instrument X. √n (where n is equal to number of sides of traverse) and should be equally distributed among all the angles. The sum of the corrected angles should satisfy the above conditions.

Note:

If some of the survey lines are short, the correction should be mostly applied to angles adjacent to them because they are more liable to errors.

(ii) Traversing by deflection angles:

In this case, the check is that the algebraic sum of the deflection angles should be equal to 360^0 assuming right-hand deflection angles as positive and the left-hand deflection angles as negative.

(iii) Traversing by direct observation of bearings:

In this method, the check is that he fore and back bearings of the same line taken from the first and the last stations should differ by 180^0.

Plotting of traverse

There are two principal methods of plotting a traverse survey:

1. The angle and distance method and

2. The Co-ordinate method.

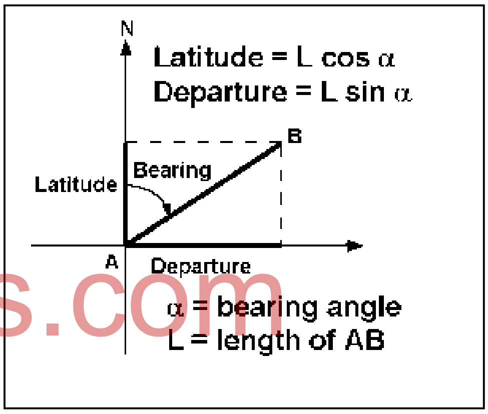
1.12 Latitude and departure

If the length and bearing of a survey line are known, it can be represented on plan by two rectangular co-ordinates.

The axes of the co-ordinates are the North and South line, and the East and West line.

Latitude

- "The latitude of survey line may be defined as its co-ordinate length measured parallel to the meridian direction".
- The latitude (L) of the line is positive when measured northward (or upward) and is termed as northing.
- The latitude is negative when measured southward (or downward) and is termed as southing.



Departure

- "The departure of survey line may be defined as its co-ordinate length measured at parallel angles to the meridian direction".
- The departure (D) of the line is positive when measured eastward and termed as Easting.
- The departure is negative when measured westward and is termed as westing.

To calculate the latitudes and departures of the traverse lines.

The latitude and departure of the line OA of length t_1 and reduced bearing O_1 is given by

$\rightarrow L = +t_1 \cos \theta_1$

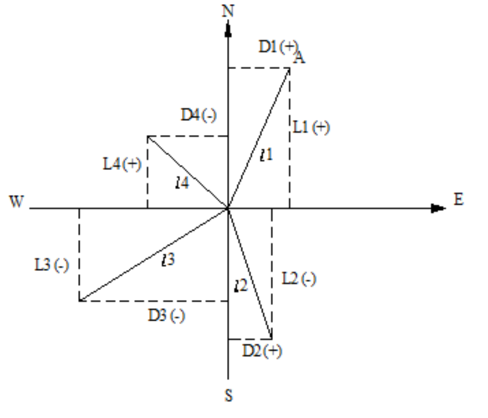
$\rightarrow D = +t_1 \sin\ \theta_1$

Note:

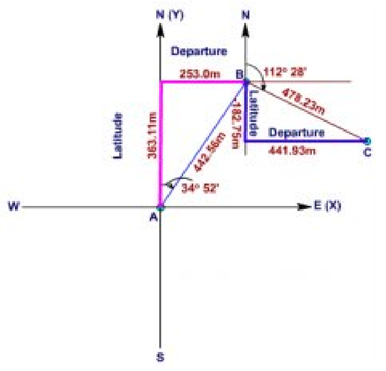
Convert the whole circle bearing to the reduced bearing system

The sign of latitude and departures will depend upon the reduced bearing of line.

The following table gives the signs of latitudes and departures.



Sign of Latitude and Departure



Latitude and departure of traverse sides AB

W.C.B.	R.B. and Quadrant	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	+	-

CONSECUTIVE CO –ORDINATES:

The latitudes and departure co-ordinates of any point with reference to the preceding point are equal to the latitude and departure of the line joining the preceding point to the point under consideration are called as consecutive co-ordinate points

INDEPENDENT CO-ORDINATES:

The co-ordinates of traverse station can be calculated with respect to a common origin. "The total latitude and departure of any point with respect to a common origin are known as independent co-ordinates or total co-ordinates of the point".

The two reference axes in this case may be chosen to pass through any of the traverse stations but generally a most westerly station is chosen for this purpose.

The independent co-ordinates of any point may be obtained by adding algebraically the latitudes and the departure of the lines between the point and the origin. The two reference axes in this case may be chosen to pass through any of the traverse stations but generally a most westerly station is chosen for this purpose.

Thus, total latitude (or departure) of end point of a traverse = total latitudes (or departures) of first point of traverse plus the algebraic sum of all the latitudes (or departures)

Advantages of Latitudes/Departures:

1. Accuracy of linear measurements is better over long distances than angular measurements.
2. Latitudes and Departures are easily adjustable for closure using several methods.
3. Easily plotted on computer using Relative X/Y (@ x, y)
4. Area calculation by some methods requires coordinates or latitudes and departures

Ex1: Calculate the latitude and departure for the following observations of a closed traverse ABCD.

Line	Length (m)	W.C.B
AB	235	$338^0\ 22'$
BC	317.5	$82^0\ 20'$
CD	215	$167^0 10'$
DA	218.5	$259^0 42'$

Given data: As per given table

To calculate: Latitude and Departure

Solution

Convert W.C.B into reduced bearing

Line	W.C.B	Conversion	R.B(θ)
AB	$338^0\ 12'$	360^0-$338^0 12'$	N 21^0 48' W
BC	$82^0\ 20'$	W.C.B	N 82^0 20' E
CD	$167^0 10'$	180^0 -167"10'	S 12^0 50' E
DA	$259^0 42'$	$259^0 42'$-180^0 00'	S 79^0 42' W

Latitude of AB, BC, CD and DA

Latitude of a line = t cos θ

Latitude of AB = +235 x cos 21^0 48' = + 218.19

Latitude of BC = +317.5 x cos 82^0 20' = + 42.36

Latitude of CD = -215 x cos 12^0 50' = - 209.63

Latitude of DA = -218.5 x cos 79^0 42' = -39.07

Departure of AB, BC, CD and DA

Departure of a line = l sin θ

Departure of AB = -235 x sin 21^0 48' = - 87.27

Departure of BC = +317.5 x sin 82^0 20' = + 314.66

Departure of CD = -215 x sin 12^0 50' = + 47.75

Departure of DA = -218.5 x sin 79^0 42' = -214.98

Result

Latitude of AB = + 218.19, BC =+ 42.36

CD= - 209.63, DA =- 39.07

Departure of AB = - 87.27, BC = +314.66

`

Ex2: A closed-loop traverse was run among stations A, B, C and D having following observation

Sides	Length (l_i) m	Azimuth (q_i)
AB	372.222	0° 42'
BC	164.988	94° 42'
CD	242.438	183° 04'
DA	197.145	232° 51'

Find the consecutive coordinates of the stations.

Solution: The consecutive coordinates of the stations A, B, C and D are as calculated in Table 1.1

Table 1.1 Consecutive Coordinates of stations of a closed-loop traverse

Sides	Length (l_i) m	Azimuth (q_i)	Stations	Consecutive coordinates of stations, (m)			
				Departure (X_i), l_i sin q_i		Latitude (Y_i), l_i cos q_i	
				East (+)	West (-)	North (+)	South(-)
AB	372.222	0° 42'					
			B	4.547		372.194	
BC	164.988	94° 42'					
			C	164.576			-11.653
CD	242.438	183° 04'					
			D		-12.970		-242.091
DA	197.145	232° 51'					
			A		-157.136		-119.056

Thus, consecutive coordinates of station are A (-157.136m, -119.056m), B (4.547m, 372.194m), C (164.576m, -11.653m) and D (-12.970m, -242.091m)

Ex3: Calculate the Independent Coordinates for the traverse defined in Example 29.1 . Given that the independent Coordinates of the stations A as (7200.054, 7640.842).

Solution: **Table 1.2 Computation of Independent Coordinates of a closed-loop traverse**

Stations (1)	Sides (2)	Length (l_i) m (3)	Azimuth (q_i) (4)	Consecutive Coordinates, (m) (5)		Independent Coordinates (m) (6)	
				Departure (a)	Latitude (b)	X (a)	Y (b)
A						7200.054	7640.842
	AB	372.222	0° 42'	4.547	372.194		
B						7204.601	8013.036
	BC	164.988	94° 42'	164.576	-11.653		
C						7369.177	8001.383
	CD	242.438	183° 04'	-12.970	-242.091		
D						7356.207	7759.292
	DA	197.145	232° 51'	-157.136	-119.056		
A							

Ex4: The following are the latitudes and departures of the sides of the traverse ABCD. Calculate the Consecutive and Independent Co- ordinates of A, B, C and D

LINE	Latitude(m)	Departure(m)
AB	+107.40	+62.00
BC	-122.60	+102.90
CD	-77.90	-45.00
DA	+93.10	-119.90

Solution: Calculation of Consecutive Co- ordinates

Consecutive co-ordinates of any station
= Latitude and Departure w.r.t. proceeding point
Consecutive co-ordinates w.r.t. Latitude (w.r.t. means with respect to)

For A = Latitude of DA = +93.10

B = Latitude of AB = +107.40

C = Latitude of BC = -122.60

D = Latitude of CD = -77.90

Consecutive X – Coordinates w.r.t departures

For A = Departure of DA = -199.90

B = Departure of AB = +62.00

C = Departure of BC = +102.90

D = Departure of CD = -45.00

Calculation of Total Co- ordinates

Total latitude (or) Independent coordinates of any point = Y Co-ordinate of origin + algebraic sum of latitudes of all the lines up to that point.

The origin is so chosen that all the points lie in the north-east quadrant.

Assume the independent coordinates of origin (A) as 400,400 (y.x)

North Coordinate of	A	=	400
Add northing of	B	=	+107.40
North Coordinate of	B	=	507.40
Deduct southing of	C	=	- 122.60
North Coordinate of	C	=	384.80
Deduct southing of	D	=	- 77.90
North Coordinate of	D	=	306.90
Add northing of	A	=	93.10
North Coordinate of	A	=	400.00

This north coordinate of A is same as assumed co – ordinate. **It is a check.**

Total departure of any point = X coordinate of origin +algebraic sum of departures of all the lines up to that point.

East Coordinate of	A	=	400
Add easting of B	=		+62.00
East Coordinate of	B	=	462.00
Add easting of C		=	+102.90
East Coordinate of	C	=	564.90
Deduct westing of	C	=	-45.00
East Coordinate of	C	=	519.90
Deduct westing of	A	=	-119.90
East Coordinate of	A	=	400.00

Same as assumed co – ordinate. **It is a check**

Result: The results are tabulated as below.

Point	Line	Latitude	Departure	Consecutive Coordinates		Independent Coordinates	
				Latitude Y	Departure X	Y	X
A	AB	+107.40	+62.00	+93.10	-119.90	400	400
B	BC	-122.60	+102.90	+107.40	+62.00	507.40	462
C	CD	-77.90	-45.00	-122.60	+102.90	384.80	546.60
D	DA	+93.10	-119.90	-77.60	-45.00	306.90	519.90
A	-	-	-	-	-	400	400

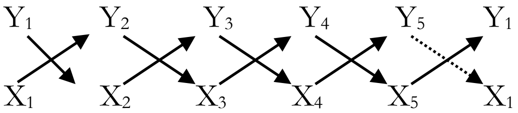
1.13 Problems on computation of Area of closed traverse problems

The area of the traverse is computed from independent co-ordinate If ABCDE be the given traverse to find the area, each station point has the independent coordinates X_1Y_1, X_2Y_2, X_3Y_3, X_4Y_4, and X_5Y_5 respectively from the common origin O.

The area of the closed traverse ABCDE are computed from the known values of the independent coordinates.

Arrange the independent co-ordinates s shown. The first coordinate is repeated in the last also.

Find the sum of the products of the co-ordinate joining by full lines and the sum of the products of the co-ordinates joined by broken lines.



Let $P= Y_1X_2+ Y_2X_3+ Y_3X_4+ Y_4X_5+ Y_5X_1$ (ie Y_n X_n)

$Q= X_1Y_2 + X_2Y_3 + X_3Y_4+ X_4Y_5 + X_5Y_1$ (ie X_n Y_n)

Find the difference between two sum (ie subtract the lower number from higher number) P~ Q

Find the half of the above difference to find the area

Area of the traverse ABCED ½ (P~ Q)

Ex5: Fine the area of the closed traverse having the following data by the co-ordinate method

Side	Latitude	Departure
AB	+225.5	+120.5
BC	-245.0	+210.0
CD	-150.5	-110.5
DA	+170.0	-220.0

Solution:

I. The consecutive coordinates are arranged in independent coordinate form as follows
The independent coordinate of station 'A' are assumed as +400, +400

Station	Side	Consecutive coordinate		Independent coordinate	
		Latitude (Y)	Departure(X)	Latitude (Y)	Departure(X)
A				+ 400	+ 400
B	AB	+225.5	+120.5	+625.5	+520.5
C	BC	-245.0	+210.0	+380.5	+730.5
D	CD	-150.5	-110.5	+230.0	+620.0
A	DA	+170.0	-220.0	+400	+400

II. The independent coordinates are arranged in determinant form

A	B	C	D	A
400	625.5	380.5	230.0	400
400	520.5	730.5	620.0	400

Sum of products of coordinates joined by single arrow lines

$\sum p$=((400X520.5) +(625X730.5) +(380.5X620) +(230X400)) = 993037.75

Sum of products of coordinates joined by double arrow lines

$\sum Q$=((400X625.5) +(520.5X380.5) +(730.5X230) +(620X400)) =864265.25

Required area= ($\sum P \sim \sum Q$)/2 = (993037.75 ~ 864265.25)/2

=**64,386.25 m².**

1.14 BALANCING THE TRAVERSE

The term 'balancing' is generally applied to the operation of applying corrections to latitudes and departures so that EL=0 and ED=0. This applies only when the survey forms a closed polygon. The following are common methods of adjusting a traverse:

1. Bowditch's method
2. Transit method
3. Graphical method
4. Axis method

1. Bowditch's Method.

The basis of this method is on the assumptions that the error in linear measurements are proportional to and that the errors in angular measurements are inversely proportional to where l is the length of a line. The Bowditch's rule, also termed as the compass rule, is mostly used to balance a traverse where linear and angular measurements are of equal precision.

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

The Bowditch Rule is :

Correction to latitude (or departure) of any side =

Total error in latitude (or departure) x $\frac{length\ of\ that\ side}{Perimeter\ of\ trafverse}$

Thus, if C_L= correction to latitude of any side

C_D = correction to departure of any side

EL=total error in latitude

ED = total error in departure

El=length of the perimeter

l=length of any side

We have $C_{L = EL} \cdot \frac{l}{\sum l}$

and $C_{D = ED} \cdot \frac{l}{\sum l}$

2. Transit Method

The transit rule may be employed where angular measurement are more precise that the linear measurements. According to this rule, the total error in latitudes and in departures is distributed in proportion to the latitudes and departures of the sides. It is claimed that the angles are less affected by corrections applied by transit method than by those by Bowditch's method.

The transit rule is:

Correction to latitude (or departure) of any side

$$= \frac{Latitude\ (or) departure)\ of\ that\ line}{Arithmetic\ sum\ of\ latitudes\ (or\ departures)}$$

Thus, if L = latitude of any line

D = departure of any line

L_r =arithmetic sum of latitudes

D_r=arithmetic sum of departure

We have $C_{L = EL} \cdot \frac{L}{L_r}$

and $C_{D = ED} \cdot \frac{D}{D_r}$

3. Graphical Method

For rough survey, such as a compass traverse, the Bowditch rule may be applied graphically without doing theoretical calculations. Thus, according to the graphical method, it is not necessary to calculate latitudes and departures etc. However, before plotting the traverse directly from the field notes, the angles or bearings may have adjusted to satisfy the geometric conditions of the traverse.

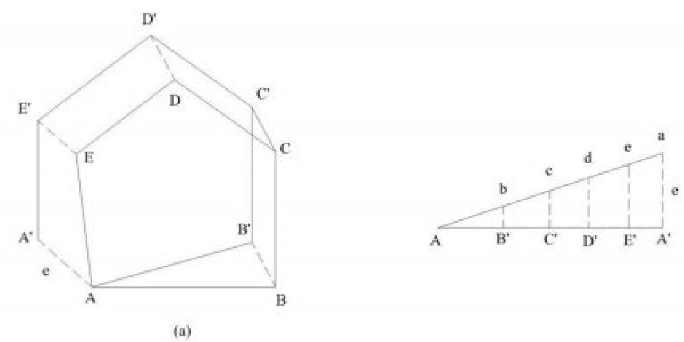


Fig.No.1.15 Graphical Method

Thus, in Fig. 1.15 a. polygon, AB'C'D'E'A' represents an unbalanced traverse having a closing error equal to A'A since the first point A and the last point A' are not coinciding. The total closing error AA' is distributed linearly to all the sides in proportion to their length by a graphical construction shown in Fig. 1.15 b.in fig. 1.15 b, AB', C'C, C'D etc. represent the length of the side of the traverse either to the same scale as that of Fig. 1.15 (a) or to a reduced scale. The ordinate aA' is made equal to the closing error A'A(of fig. 1.15 (a)] by constructing similar triangles, the corresponding error bB', cC', dD', eD' are found. In fig. 1.15 a. lines E'E, D'D, C'C, B'B are drawn parallel to the closing error A'A and made equal to eE, dD', cC', bB' respectively. The polygon ABCDE so obtained represents the adjusted traverse. It should be remembered that the ordinates bB',cC'dD.eD'aA', of fig. 1.15 (b) represent the corresponding errors in magnitude only but not in direction.

Ex6: The following data pertains to a theodolite traverse:

Line	Length(m)	Bearing
AB	111.64	45°10’
BC	274.70	72°05
CD	188.98	161°52
DE	198.88	228°43’
EA	290.33	300°42

Balance the traverse by both Bowditch and transit rule.

Solution:

III. Bowditch rule

A. Calculation of latitude and departure

Side	Latitude L X cosΘ	Departure L X sinΘ
AB	+78.71	+79.17
BC	+84.51	+261.38
CD	-179.59	+58.82
DE	-131.22	-149.45
EA	+148.23	-249.64

B. Calculation error in latitude and departure

€l=0

= +78.71+84.51-179.59-131.22+148.23=+0.64

€d=0

= +79.17+261.38+58.82-149.45-249.64=+0.28

C. Perimeter: 111.64+274.70+188.98+290.33=1064.53 m

D. Apply Bowditch rule

Correction for latitude	Correction for departure
$\frac{\text{Length of line AB}}{\text{Perimeter}}$ X €l	$\frac{\text{Length of line AB}}{\text{Perimeter}}$ X €d
For line AB	
$\frac{111.64}{1064.53}$ X -0.64 =-0.07	$\frac{111.64}{1064.53}$ X -0.28 -0.03
Note: correction is always opposite sign to error	

Side	Latitude L X cosΘ	Departure L X sinΘ	Correction for latitude	Correction for Departure	Corrected latitude	Corrected Departure
AB	+78.71	+79.17	-0.07	-0.03	+78.64	+79.14
BC	+84.51	+261.38	-0.17	-0.07	+84.34	+261.31
CD	-179.59	+58.82	-0.11	-0.05	-179.70	+58.77
DE	-131.22	-149.45	-0.12	-0.05	-131.34	-149.50
EA	+148.23	-249.64	-0.17	-0.08	+148.06	-249.72

Check: corrected €l=0 and corrected €d=0 Hence OK

IV. Transit rule:

A. Calculation of latitude and departure

`

Side	Latitude L X cosΘ	Departure L X sinΘ
AB	+78.71	+79.17
BC	+84.51	+261.38
CD	-179.59	+58.82
DE	-131.22	-149.45
EA	+148.23	-249.64

B. Calculation error in latitude and departure

€l=0

= +78.71+84.51-179.59-131.22+148.23=+0.64

€d=0

= +79.17+261.38+58.82-149.45-249.64=+0.28

C. Calculation of arithmetic sum of

(a) Latitude:

=78.71+84.51+179.59+131.22+148.23 = 633.26 m

(b) Departure

= 79.17+261.38+58.82+149.45+249.64=798.46

D. Apply transit rule

Correction for latitude	Correction for departure
$\frac{\text{Latitude of line AB}}{\text{Arithmetic sum of all latitude}}$ X €l	$\frac{\text{Departure of line AB}}{\text{Arithmetic sum of all latitude}}$ X €d
Line AB	
$\frac{78.71}{622.26}$ X -0.64 =-0.08	$\frac{79.17}{798.46}$ X -0.28 =-0.03
Note: correction is always opposite sign to error	

Side	Latitude L X cosΘ	Departure L X sinΘ	Correction for latitude	Correction for Departure	Corrected latitude	Corrected Departure
AB	+78.71	+79.17	-0.08	-0.03	+78.63	+79.14
BC	+84.51	+261.38	-0.08	-0.09	+84.43	+261.29
CD	-179.59	+58.82	-0.19	0.02	-179.78	+58.80
DE	-131.22	-149.45	-0.14	-0.05	-131.36	-149.50
EA	+148.23	-249.64	-0.15	-0.09	+148.08	-249.73

Check: corrected €l=0 and corrected €d=0 Hence OK

`

1.15 OMITTED MESASUREMENTS

In order to have a check on field work and in order to balance a traverse, the length and direction of each line is generally measured in the field. There are times, however, when it is not possible to take all measurements due to obstacles or because of some over-sight. Such omitted measurements or missing quantities can be calculated by latitudes and departures provided the quantities required are not more than two. In such cases, there can be no check on the field work nor can the survey be balanced. All errors propagated throughout the survey are thrown into the computed values of the missing quantities.

Generally, in a traverse survey, length, angle and bearings are observed. Some time it is not possible to take all measurements due to observations or due to oversight. These missed readings are called **omitted measurements**.

Since for a close traverse, EL and ED are zero, we have

→ EL= $+l_1 \cos\theta_1 + l_2 \cos\theta_2 + l_3 \cos\theta_3 + \cdots = 0$ 1

→ ED= $+l_1 \sin\theta_1 + l_2 \sin\theta_2 + l_3 \sin\theta_3 + \cdots = 0$ 2

where l_1, l_2, l_3,......etc., are the lengths of the lines and $\theta_1, \theta_2, \theta_3$...etc. their reduced bearings. With the help of the above two equations, the two missing quantities can be calculated. Table below gives the trigonometric relations of a line with its latitude and departure, and may be used for the computation of omitted measurements:

There are four general cases of omitted measurements

I. a. When the bearing of one side is omitted.
b. When the length of one side is omitted
c. When the bearing and length of one side is omitted.

II. When the length of one side and the bearing of another side are omitted.

III. When the lengths of two sides are omitted.

IV. When the bearings of two sides are omitted.

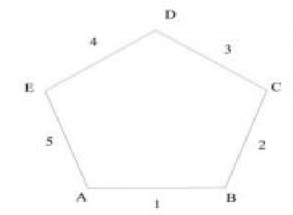
Given	Required	Formula
1, θ	L	L = l cos θ
l, θ	D	D=1 sin θ
L,D	Tan θ	tanθ=D/L
L, θ	l	l-lsecθ
D, θ	l	L=D cosec θ
L,l	Cos θ	Cos θ = L/I
D,l	Sin θ	Sin θ = D/l
L,D	L	l=$\sqrt{L^2+D^2}$

In case (I) only one side is affected, In case II, III and IV two sides are affected both of which may either be adjacent or may be away.

`

CASE I: BEARING, OR LENGTH, OR BEARING AND LENGTH OF ONE SIDE OMITTED

In Fig: 1.16, let it be required to calculate either bearing or length or both bearing and length of the line EA. Calculate EL, and ED' of the four known sides AB, BC, CD, and DE. Then



One sided omitted measurement

EL=Latitude of EA + EL] = o

or

Latitude EA + ED'=0

Similarly,

ED=Departure of EA+ED'=0

or

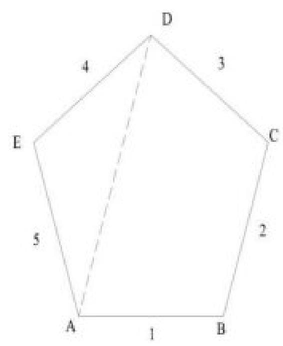
Departure EA==ED'

knowing latitude and departure of EA, its length and bearing can be calculated by proper trigonometrical relations.

CASE II: LENGTH OF ONE SIDE AND BEARING OF ANOTHER SIDE OMITTED

In Fig, let the length of DE and bearing of EA be omitted. Join DA which becomes the closing line of the traverse ABCD in which all the quantities are known. Thus the length and bearing of DA can be calculated as in case I.

In ADE, the length of sides DA and EA are known an angle ADE (a) is known. The angle β and the length DF can be calculated as under:



Two side omitted adjacent to each other

$$\sin \beta = \frac{DA}{EA} \sin \alpha$$

$$\gamma = 180^0 - (\beta + a)$$

$$DE = EA \frac{\sin y}{\sin \alpha} = DA \frac{\sin y}{\sin \beta}$$

knowing γ, the bearing of EA can be calculated.

CASE III : LENGTHS OF TWO SIDES OMITTED

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In Fig.:1.17, let the length of DE and E4 be omitted. The length and bearing of the closing line DA can be calculated as in the previous case. The angles α, β and γ can then be computed by the known bearing. The lengths of DE and EA cn be computed by the solution of the triangle DEA.

Thus, $DE = \frac{\sin\gamma}{\sin\beta} DA$

and $EA \frac{\sin\alpha}{\sin\beta} DA$

CASE IV : BEARING OF TWO SIDES OMITTED

In Fig: 1.17 let bearing of DE and EA be omitted. The length and baring of the closing line DA can be calculated. The angles can be computed as under:

The area $= \sqrt{s)s-a)(s-d)(s-e)}$ 1

where s=half the perimeter = 1/2 (a+d+e)

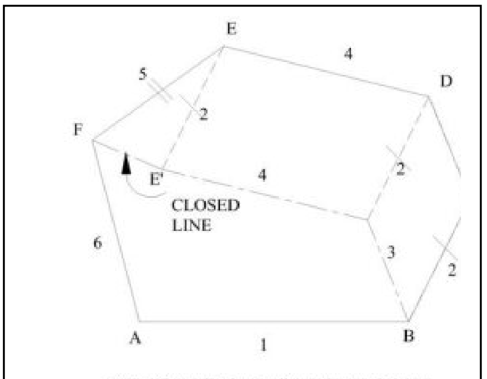
a=ED, e=AD and d=AE.

Also, 1/2 ad sin β = 1/2 de sin γ=1/2 ae sin α 2

Equating (1) and (2), α, β and γ can be calculated. Knowing the bearing of DA and the angles α, β, γ, the bearings of DE and EA can be calculated.

CASE II, III, IV: WHEN THE AFFECTED SIDES ARE NOT ADJACENT

If the affected sides are not adjacent, one of these can be shifted and brought adjacent to the other by drawing lines parallel to the given lines. Thus in Fig let BC and EF be the affected sides. In order to bring them adjacent, choose the starting point (say B) of any one affected side (say BC) and draw line BD' parallel and equal to CD. Through D', draw line D'E' parallel and equal to ED. Thus evidently, EE\=BC and FE and BC are brought adjacent. The line E\F becomes the closing line of the traverse ABD'E'F. The length and bearing of E'F can be calculated. Rest of the procedure for calculating the omitted measurements is the same as explained earlier.



Two side omitted not adjacent to each other

T

1. 16 problems:

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Ex7: The following are the data of a closed traverse ABCDA. Calculate the Consecutive and Independent Co- ordinates of A, B, C and D.

LINE	Latitude(m)	Departure(m)
AB	-300	+450
BC	+640	+110
CD	+100	-380
DA	-440	-180

Given data: As per given table

To Calculate: To calculate the area of the traverse

Solution: 1. Calculation of Consecutive Co- ordinates

LINE	Latitude(y)	Departure(x)
A	-440	-180
B	-300	+450
C	+640	-110
D	-100	-380

2. Calculate the Independent Co- ordinates

Assume coordinates of origin (A) as 400,400(y, x)

North Coordinate of	A	=	400
Deduct northing of	B	=	-300
North Coordinate of	B	=	100
Add northing of	C	=	640
North Coordinate of	C	=	740
Add northing of	D	=	100
North Coordinate of	D	=	840
Deduct southing of	A	=	-440
North Coordinate of	A	=	400

East Coordinate of	A	=	400
Add easting of B		=	450
East Coordinate of	B	=	850
Add easting of C		=	110
East Coordinate of	C	=	960
Deduct westing of	D	=	- 380
East Coordinate of	D	=	580

`

Deduct westing of A = -180

North Coordinate of A = 400

LINE	Independent	Coordinates
A	400	400
B	100	850
C	740	960
D	840	580
A	400	400

Ex8: The following table gives the lengths and bearings of the lines of a traverse ABCDE, the length and bearing of EA having been omitted. Calculate the length and bearing of the line EA.

LINE	Length(m)	Bearing
AB	204.00	$87^0 30'$
BC	226.00	$20^0 20'$
CD	187.00	$280^0 00'$
DE	192.00	$210^0 30'$
EA	?	?

Given data: As per given table

To Calculate: To calculate length and bearing of the line EA

Solution: Case (i)

LINE	WCB	R.B
AB	$87^0 30'$	$N87^0 30'E$
BC	$20^0 20'$	$N20^0 20'E$
CD	$280^0 00'$	$N80^0 00'W$
DE	$210^0 30'$	$S30^0 30'W$
EA	?	?

Calculation of latitude of EA

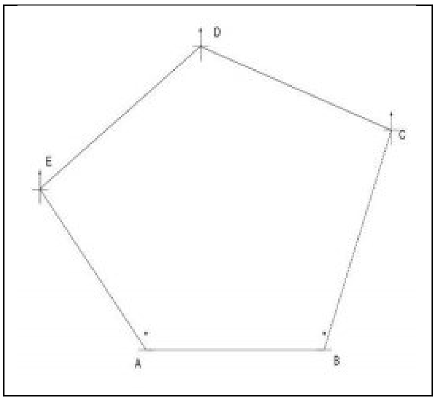
Let l_1, l_2, l_3, l_4 and l_5 be the lengths of AB, BC, CD, DE and EA respectively.

Let θ_1, $_2$, θ_3, θ_4 and θ_5 be the reduced bearings of AB, BC, CD, DE and EA respectively.

In a closed traverse, ΣL=0

Therefore, $l_1 \cos\theta_1, l_2 \cos\theta_2, l_3 \cos\theta_3, l_4 \cos\theta_4, l_5 \cos\theta_5 = 0$

$204 \times \cos 87^0 30' + 226 \times \cos 20^0 20' + 187 \cos 80^0 00' - 192 \times \cos 30^0 30' + l_{5,} \cos \theta_5 = 0$



$253.30 - 165.44 + l_{5,} \cos \theta_5 = 0$

$8.90 + 211.92 + 32.48 - 165.44 + l_{5,} \cos \theta_5 = 0$

$253.30 - 165.44 + l_{5,} \cos \theta_5 = 0$

$l_{5,} \cos \theta_5 = -87.86$ (Southing)

Calculation of departure of EA

In a closed traverse, ΣD∟ = 0

Therefore, $l_{1,} \sin \theta_{1,} l_2 \sin \theta_{2,} l_3 \sin \theta_{3,} l_4 \sin \theta_{4,\ 15} \sin \theta_5 = 0$

i.e., $204 \times \sin 87^0 30' + 226 \times \sin 20^0 20' - 187 \sin 80^0 00' - 192 \times \sin 30^0 30' + l_5 \sin \theta_5 = 0$

$203.80 + 78.52 - 184.16 - 97.44 + l_5 \sin \theta_5 = 0$

$282.32 - 281.60 + l_5 \sin \theta_5 = 0$

$l_{5,} \sin \theta_5 = -0.72$ (Westing)

Calculation of length of EA

Length of EA, $l_5 = \sqrt{(l_{5,} \sin \theta_5)^2 + (l_{5,} \cos \theta_5)^2}$

$= \sqrt{(0.72)^2 + (87.86)^2}$

= 87.85m

Calculation of bearing of EA

$$\tan \theta_5 = \frac{\text{Departure of EA}}{\text{latitude of EA}} = 0.72/87.86$$

Therefore, Bearing of EA, $\theta_5 = \tan^{-1} 0.72/87.86$

$= S\ 0^0 28' W = 180^0 28'$

Result: Length of EA = 87.85 m; Bearing of EA = $180^0 28'$

Ex9: In a closed traverse ABCDE, the length of line AB and the bearing of line EA could not be measured in the field. From the field measurements the following information is available.Calculate the missing measurements.

LINE	Length(m)	Bearing
AB	?	95^0
BC	140	$27^028'$
CD	163	$317^030'$
DE	173	$260^000'$
EA	201	?

Given data: A per given table

To calculate: Length of one line and bearing of another line

Solution: Calculation of latitudes and departures of the lines for which both bearings and length were available will be made first.

Side	Length m	RB	Latitude	Departure
BC	140	N $27^021'$ E	124.21	+64.57
CD	163	N $42^030'$ W	+120.17	-110.12
DE	173	S $80^000'$ W	-30.04	-170.37
BE	-	-	+214.35	-215.92

Length of BE = $\sqrt{(214.35)^2 + (215.92)^2}$ = 304.25m

If θ is RB of BE, $\tan\theta = \frac{-215.92}{+214.35}$ = tan ($-45^013'$)

Bearing of BE = 3600- 45^0 13' = $314^047'$

Bearing of BA = 950 + 180^0 = 275^0

Bearing of BE = 314047'

Angle of ABE = Bearing of BE – Bearing of BA

= $314^047'$ - 275^0

Angle of ABE = $39^047'$

In ΔABE we have EA = 201m

BE = 304.252m and aB = $39^047'$

Using sine rule

$$\frac{AE}{\sin B} = \frac{BE}{\sin A} = \frac{AB}{\sin E}$$

`

Sin A = $\frac{BE \sin B}{AB}$ = 304.252 x sin39^{0}47' / 201

Sin A = sin 75^{0}37'

aEAB = 75^{0}37'

Bearing of AB = 95^0

Less aA = 75^0 37'

Bearing of AE = 95^0 - 75^{0}37' = 19^{0}37'

Bearing of EA = 180^0 + 19^{0}23' = 199^{0}23'

aAEB = 180^0 – (39^{0}47' + 75^{0}37') = 64^{0}36'

AB = AE sin E/sin B = 201 x sin64^{0}36'/sin39^{0}47' = 283.754m

Result: Bearing of EA = 199^{0}23' Length of AB = 283.754m

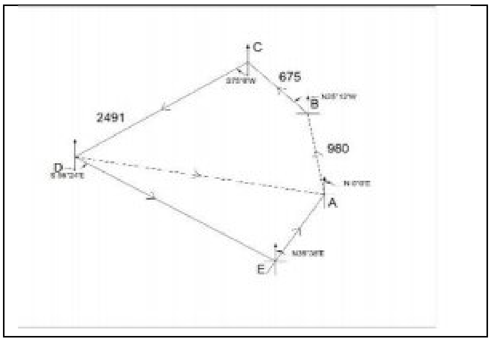
Ex10: In a closed traverse ABCDE, the bearings of the sides AB and CD could not be observed. The lengths of the lines and the bearings of the other lines are given below. Compute the missing bearings.

LINE	Length(m)	Bearing
AB	150	?
BC	175	30^0
CD	185	?
DE	170	240^0
EA	210	172^0

Given data: A per the given table

To calculate: Bearing of two sides.

Solution: Referring to fig complete parallelogram ABCC' and join C'D.



Length and Bearing of AC' will be those of BC.

Length and Bearing of C'C will be those of AB.

Now consider closed traverse DE A C' of which C'D can be treated as the closing line. Tabulating latitude and departures of this traverse.

Latitude of C'D = +141.402 Departure of C'D = +30.498

C'D = $\sqrt{lat^2 + dep^2}$ = $\sqrt{30.498^2 + 141.402^2}$ = 144.654m.

If θ is the bearing of C'D,

Tan = Departure/Latitude = 30.498/141.402

= $12^0 10'$

In Δ CDC', we have

CD = 185m, DCΔ CDC" = 144.654m and CC' = AB = 150m

Line	Length m	Bearing	Latitude	Departure
DE	170	240^0	-85.000	-147.224
EA	210	172^0	-207.956	+29.226
AC=BC	175	30^0	+151.554	+87.500
Total	-	-	-141.402	-30.498

S = 185+144.654+150/2 = 239.287m

Δ CDC' = √s(s-c) (s-d) (s-c')

= √239.837 x 95.173 x 89.827 x 54.827 = 10602.459 m^2

Sin CDC' = 2Δ/ CD x CC' = 2 x 10602.459/185x150 = sin$49^0 50'$

a CDC' = $49^0 50'$

Bearing of DC = Bearing of D C' - a C'DD

= $192^0 10'$ - $49^0 50'$ = $142^0 20'$

Bearing of CD = $142^0 20'$ - 180^0 = $322^0 20'$

Latitude of CD = 185 cos $322^0 20'$ = 146.442m

Departure of CD = 185 sin $322^0 20'$ = -113.047m

Line	Latitude	Departure
BC	+151.554	+87.500
CD	146.442	-113.047
DE	-85.000	-147.224
EA	-207.956	+29.226
Sum	+5.040	-143.545

Latitude of AB = - 5.040 Departure of AB = + 143.545

tan(RB of AB) = $\frac{143.545}{-5.040}$ = tan (180^0 - 88^0)

Bearing of AB = 180^0 - 88^0 = 92^0

Result: Bearing of BA = 92^0 Bearing of CD = $322^0 20$

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Ex11: Given the following observed lengths and bearings of the sides of a closed traverse ABCDE. The bearings of BC and CD having been omitted. Find the bearing of BC and CD.

LINE	Length(m)	Bearing
AB	217.5	S $59^0 45'$ E
BC	318.0	?
CD	375	?
DE	283.5	S $55^0 18'$ W
EA	173.15	S $2^0 40'$ W

Given data: A per the given table

To calculate : Bearing of two sides.

Solution:

1. Join the line DB excluding the affected sides BC and CD.
2. Consider the traverse ABDE.
3. Calculate the bearings of BC and CD.
4. Calculate the latitude and departures of AB, DE and EA.

Latitude of AB = 217.5 cos $59^0 45'$ = -109.57

Latitude of DE = 283.5 cos $55^0 18'$ = -161.39

Latitude of EA = 173.15cos $2^0 40'$ = -172.96

Departure of AB = 217.5 sin $59^0 45'$ = +187.88

Departure of DE = 283.5 sin $55^0 18'$ = - 233.08

Departure of EA = 173.15 sin $2^0 40'$ = - 8

Line	Latitude		Departure	
	Northing (+)	Southing (-)	Easting (+)	Westing (-)
AB	-	109.57	187.88	-
DE	-	161.39	-	233.08
EA	-	172.96	-	8.07
Sum	0	443.92	187.88	241.15

$\Sigma L = 0 - 443.92 = -$ **443.92**

$\Sigma D = 187.88 - 241.15 = -$ **53.27**

In a closed traverse $\Sigma L = 0$ and $\Sigma D = 0$

$\Sigma L = 0 \quad -443.92 + \text{Latitude of BD} = 0$

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Latitude of BD $= 443.92$

$\Sigma D = 0$ $\quad -53.27 + \text{Departure of BD} = 0$

Departure of BD $= 53.27$

Length of BD $= \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$

$= \sqrt{(443.92)^2 + (53.27)^2} \quad = 447.1$ m

Bearing of BD

$\tan\theta = \frac{Departure}{Latitude} = \frac{+53.27}{+443.92}$

Bearing = N $6^0 50.4'$ E

Calculate the value of = s

$S = \frac{447.1+318+375}{2} = 570.05$

Find the a B =

$\tan B/2 = \sqrt{\frac{(s-c)(s-d)}{s(s-b)}} = \frac{(570.05-447.1)(570.05-318)}{570.05(570.05-375)}$

a B/2 = $27^0 49.8'$ = a B = $55^0 39.6'$

Bearing of BC

Bearing of BD = N $6^0 50.4'$ E

Add AB = $\underline{\quad 55^0 39.6'}$

Bearing of BC = $\underline{\text{N } 62^0 30' \text{ E}}$

Calculate a D

$\tan D/2 = \sqrt{\frac{(s-c)(s-b)}{s(s-d)}} = \frac{(570.05-447.1)(570.05-375)}{570.05(570.05-318)}$

a D/2 = $22^0 13'$ = a D = $44^0\ 26'$

Bearing of CD

Bearing of DB = S $6^0 54.4'$ W

Bearing of DC = $44^0\ 26' - 6^0\ 54.4'$ = S $37^0 31.6'$ E

Bearing of CD = N $37^0 31.6'$ W

Result : Bearing of BC = N $62^0 30'$ E **Bearing of CD = N $37^0 31.6'$ W**

Ex12: The measured length and bearing of the sides of a closed traverse ABCDEA are tabulated below. Find the lengths sides DE and EA.

LINE	Length(m)	Bearing
AB	980	S $0^0 0'$ E
BC	675	N $25^0 12'$ W
CD	2491	S $75^0 6'$ W
DE	?	S $56^0 24'$ E
EA	?	N $35^0 36'$E

Given data: A per the given table

To find : length of DE and EA.

Solution : Consider the closed traverse ABCDA

(i)Calculation of latitude of line DA

In the closed traverse ABCDA, $\Sigma L = 0$

$$l_1 \cos \theta_1 + l_2 \cos \theta_2 + l_3 \cos \theta_3 + l_4 \cos \theta_4 = 0$$

$$980 \times \cos 0^0 0' + 675 \times \cos 25^0 12' - 2491 \times \cos 75^0 6' + l_4 \cos \theta_4 = 0$$

$$950.24 + l_4 \cos \theta_4 = 0$$

$$l_4 \cos \theta_4 = -950.24$$

Latitude of DA = - 950.24 (Southing)

(ii)Calculation of Departure of line DA

In the closed traverse, $\Sigma D = 0$

$$l_1 \sin\theta_1 + l_2 \sin \theta_2 + l_3 \sin \theta_3 + l_4 \sin \theta_4 = 0$$

$$980 \times \sin 0^0 0' - 675 \times \sin 25^0 12' - 2491 \times \sin 75^0 6' + l_4 \sin \theta_4 = 0$$

$$-2694.64 + l_4 \sin \theta_4 = 0$$

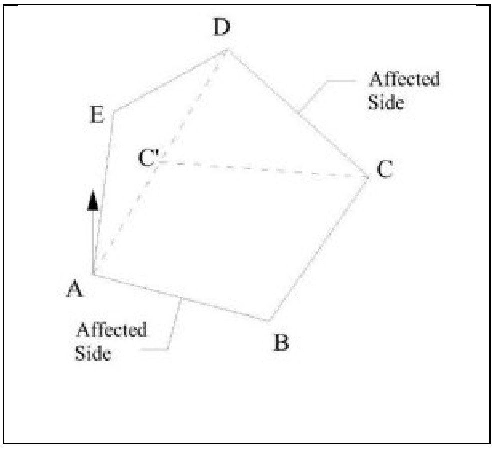
$$l_4 \sin \theta_4 = -2694.64 \text{ (Easting)}$$

(iii)Calculation of length of line DA

length of that line DA = √(Latitude of DA)^2 + (Departure of DA)^2

= √(950.24)^2 + (2694.64)^2

L_4 = 2857.28m



(iv)Calculation of bearing of line DA

Bearing of line DA = $\tan^{-1} \frac{Dep.of\ DA}{Lat.of\ DA}$

$= \tan^{-1} \frac{2694.64}{950.24} = 70^0 34'31''$

Bearing of line DA = $70^0 34'31''$

(v) To find the angle ADE, DEA and EAD (α,β, γ)

a ADE = α = Bearing of DA - Bearing of DE

= $70^0 34'31''$ - $56^0 24'$ = **$14^0 10'31''$**

a ADE = β = Bearing of EA - Bearing of DE

= $35^0 36'$ - $56^0 24'$ = **$92^0 0'0''$**

$\gamma = (180^0 - (\alpha + \beta))$

$= (180^0 - (14^0 10'31'' + 92^0 0'0'')) = 73^0 49'29''$

(vi) To find the length of line DE and EA

Using sine rule, $\frac{DA}{\sin\beta} = \frac{DE}{\sin\gamma} = \frac{EA}{\sin\alpha}$

DE $= \frac{DA\ x\ \sin\gamma}{\sin\beta} = \frac{2857.28\ x \sin 73°49'29''}{\sin 92°}$ = 2745.84m

EA $= \frac{DA\ x\ \sin\alpha}{\sin\beta} = \frac{2857.28\ x \sin 14° 10' 31''}{\sin 92°}$ = 700.14m

Result:

Length of line DE = 2745.84m **Length of line EA = 700.14m**

Ex13: The following are the length and bearings of close traverse ABCDE. Calculate the length and bearing of EA

Line	length	W.C.B
AB	458.0	198°59'
BC	262.5	282°14'
CD	160.0	320°13'
DE	398.5	35°13'
EA	-	-

Solution

To find

i. length of line EA

ii. Bearing line EA

1. Conversion of given W.C.B in to Q.B or R.B. reduced nearing

R.B. of line AB= 198°59' -180° =S18° 59'W

R.B. of line BC= 360°00' -282° 14'=N77° 46'W

R.B. of line CD= 360°00' -320° 13'=N39° 47'W

R.B. of line DE = 35° 13'=N35° 13'E

2. Calculation of latitude and departure

Line	Length(l)	R.B.(θ)	Latitude in (m)	Departure in (m)
	In meter		+ or - l cosθ	+or – l sinθ
AB	458	S18° 59'W	-458Xcos18° 59' = -433.09	-458Xsin18° 59' =-148.98
BC	262.5	N77° 46'W	+ 262.5 cos77° 46' =+88.62	-262.5Xsin77° 46' = -256.53
CD	160	N39° 47'W	+ 160cos39° 47' =+122.95	-160Xsin39° 47' =-102.38

`

DE	398.5	N35° 13'E	+ 398.5cos35° 13 =+325.56	+398.5sin35° 13 =+229.80
EA	-	-	L cosɵ	L sinɵ

Since the given traverse is closed one

Hence ΣL=0

= -433.00+55.62+122.95+325.56 +latitude of line EA =0

+71.04+latitude of EA=0

Latitude of EA= -71.04 m

ΣD = 0

=-148.98-256.53-102.38+229.80+ departure of line EA=0

= -278.09+departure of line EA=0

Departure of line EA = +278.09

3. Calculation of bearing

$$\tan\theta = \frac{\text{Departure of EA}}{\text{Latitude of EA}} = \frac{(+)\ 278.09}{(-)\ 71.04}$$

ɵ= S75°40'E

Since the line EA have latitude (-ve) and departure (+ve) hence the line falls in second quadrant

4. Calculation of length of line EA

Length of line EA= Sq root of $((\Sigma L)^2+(\Sigma D)^2)$

= Sq. Root of $(-71.04)^2+(278.09)^2$

= 287.02 m

Result: 1. Length of line EA= 287.02 m

2. Reduced bearing of line EA= S75° 40' E

3. Whole circle bearing of line EA= 180°+ 75° 40'=255°40'

Question bank

A. One-mark question

1. Mention the classification of theodolite
2. Name the five components of transit theodolite
3. Define transiting
4. Define swinging
5. Name the fundamental lines of theodolite
6. Define angle of elevation
7. Define angle of depression
8. Define latitude
9. Define departure

B. Three marks questions

1. Differentiate between transit and non-transit theodolite
2. Differentiate between face left and face right observation
3. Differentiate between Clamp screw and Tangent screw.
4. Write any five technical term used in theodolite surveying
5. Differentiate between telescope normal and telescope inverted
6. Differentiate between methods of repetition and reiteration for measurement of horizontal angles.
7. State the fundamental axis of a transit theodolite and mention the relationship between them
8. Differentiate between latitude and departure
9. Define independent co-ordinate and consecutive co-ordinate
10. What is omitted measurement

C. Ten marks questions

2. Draw the neat sketch of theodolite mention their parts and explain their important function

1) Difference between
   (i) Transiting and swinging of the telescope
   (ii) Face left and face right reading
   (iii) Clamp screw and tangent screw of a theodolite
   (iv) Telescope normal and telescope inverted.
2) What do you mean by temporary 'adjustment' of a theodolite? Describe in brief the steps of temporary adjustment in proper order.
3) Enumerate the fundamental lines of a theodolite instrument and state their relationship in a permanently adjusted instrument.
4) Explain how do your measure horizontal angle by Reiteration method with neat sketch , with tabulation
5) Explain the method of repetition for the measurement of horizontal angles.
6) Describe the method of measuring vertical angles using a theodolite
7) Describe the process of measuring bearing in the field
8) Fine the area of the closed traverse having the following data by the co-ordinate method

Side	Latitude	Departure
AB	+84.59	+40.20
BC	+68.88	+-80.06
CD	-23.40	-76.80
DE	-67.65	+27.67
EA	-62.42	+88.99

9) The following length and bearing were recorded in running a traverse ABCDE, the length and bearing of EA having been omitted . Calculate the length and bearing of the line EA.

Line	Length(m)	Bearings

AB	217.5	120°15’
BC	1050	62°30’
CD	1250	322°24’
DE	950	335°18’
EA	-	-

11. Calculate the latitude and departure for the following observation of a closed travers ABCDE.

LINE	LENGTH(m)	BEARING
AB	229.00	198°59'
BC	131.35	282°14'
CD	80.00	320°13'
DE	199.25	35°13'
EA	142.60	104°26'

(Ans: Latitude: - 216.22, 27.83, 61.48, 162.78, -35.54
Departure: - 74.49, -128.37, -51.19, 114.90, 138.10)

12. The following are the latitudes and Departures of a closed traverse ABCDE. Calculate the Consecutive & Independent Co-ordinates of all the stations.

LINE	Latitude(m)	Departure(m)
AB	+15.23	+193.50
BC	+194.34	+52.07
CD	+44.20	-159.38
DE	-166.32	-46.13
EA	-87.45	-40.06

(Ans: Latitude: - 216.22, 27.83, 61.48, 162.78, -35.54
Departure: - 74.49, -128.37, -51.19, 114.90, 138.10)

13. The following are the data of a closed traverse ABCDA

LINE	Latitude(m)	Departure(m)
AB	-88.00	+133.90
BC	-416.10	+356.90
CD	+7.00	-14.84
DA	-335.10	-475.96

Calculate the area of the traverse by independent co – ordinate method.

(Ans: 47502 m²)

14. The table below gives the lengths and bearings of the lines of a traverse ABCDE, the length and bearing of EA having been omitted. Calculate the length and bearing of the line EA.

LINE	Length(m)	Bearing
AB	204.0	87°30'
BC	226.0	20°20'
CD	187.0	280°0'

DE	192.0	$210^0 80'$
EA	?	?

\
(Ans: Bearing of EA = $180^0 28'$ Length of EA = 87.85m)

15. The observed length and bearing of a line of closed traverse ABCDE are given below. Calculate the bearing of BC and CD.

LINE	Length(m)	Bearing
AB	730	$S60^0 00'E$
BC	1051	?
CD	1245	?
DE	940	$S55^0 24'W$
EA	223	$S2^0 42'W$

(Ans: Bearing of EA = $180^0 28'$ Length of EA = 87.85m)

16. Following are the latitude and departure of lines AB, BC, CD and DA in a theodolite traverse

Line	Latitude(m)	Departure
AB	123..35	35.68
BC	93.82	205.86
CD	-177.44	70.11
DA	-39.21	-312.25

Adjust the traverse by applying the both Bowditch and Transit rule.

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2 TACHEOMETRIC SURVEYING

Introduction – Instruments used in tacheometry – Systems of tacheometry: Stadia and Tangential tacheometry – Principles – Fixed hair method of tacheometry – Distance and Elevation formulae – Anallactic lens (No proof) : Advantages and uses – Simple problems – Distomats (Description only) – Direct reading tachometers - Determination of constants of a tacheometer : Problems – Tacheometric traverse – Errors in tacheometric surveying.

2.1 INTRODUCTIO

Tacheometry (from Greek, quick measure), is a system of rapid surveying, by which the positions, both horizontaland vertical, of points on the earth surface relatively toone another are determined without using a chain or tapeor a separate leveling instrument.

Tacheometry or tachemetry or telemetry is a branch of angular surveying in which the horizontal and Vertical distances of points areobtained by optical means as opposed to the ordinary slower processof measurements by tape or chain.

The method is very rapid and convenient.

It is best adapted in obstacles such as steep and broken ground,deep revines, stretches of water or swamp and so on, which makechaining difficult or impossible,

The primary object of tacheometry is the preparation of contouredmaps or plans requiring both the horizontal as well as Vertical control.Also, on surveys of higher accuracy, it provides a check on distancesmeasured with the tape.

Uses of Tacheometry

The tacheometric methods of surveying are used with advantage over the direct methods of measurement of horizontal distances and differences in elevations.

Some of the uses are:

- Preparation of topographic maps which require both elevations and horizontal distances.
- Survey work in difficult terrain where direct methods are inconvenient
- Detail filling
- Reconnaissance surveys for highways, railways, etc.
- Checking of already measured distances
- Hydrographic surveys and
- Establishing secondary control.

DEFINITION: Tacheometry or telemetry is a branch of angular surveying in which the horizontal and vertical distances of points are obtained by optical means instead of using chain, tape, compass and

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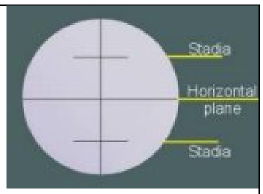
2.2 INSTRUMENTS USED IN TACHEOMETRY

The instruments employed in tacheometry are

- Tacheometer
- the leveling rod or stadia rod.

2.2.1 Tacheometer: An ordinary transit theodolite fitted with a stadia diaphragm. 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 ring and on the same vertical plane as the horizontal and vertical cross-hairs.

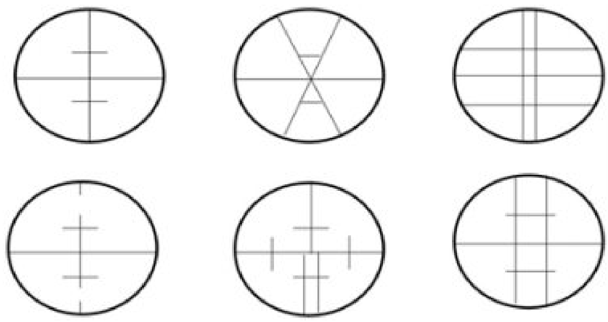
Transit theodolite



Stadia diaphragm.

Types of stadia hairs:

The different forms of stadia diaphragm commonly used are as follows

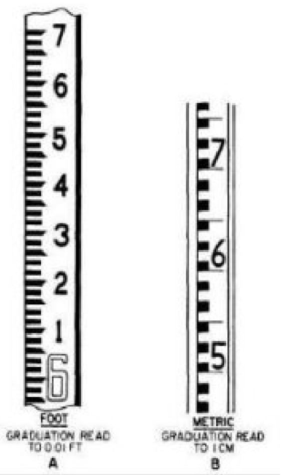


Different forms of stadia diaphragm commonly used

2.2.2 Levelling rod

A level staff, also called levelling rod, is a graduated wooden or aluminum telescopic rod, used for stadia measures at shorter distances (up to about 125 m).

2.2.3. Stadia rod. A stadia rod is usually of one piece, having 3 – 5 meters length. For smaller distances, a stadia rod graduated in 5 mm (i.e. 0.005 m) for smaller distances and while for longer distances, the rod may be graduated in 1 cm (i.e. 0.01 m).



Stadia Rods

2.3 SYSTEMS OF TACHEOMETRIC MEASUREMENT:

Depending on the type of instrument and methods/types of observations, tacheometric measurement systems can be divided into two basic types:

(i) Stadia systems and

(ii) Non-stadia systems

(i) Stadia Systems

In their systems, staff intercepts at a pair of stadia hairs present at diaphragm, are considered. The stadia system consists of two methods:

- ❖ Fixed-hair method and
- ❖ Movable-hair method

❖ Fixed - Hair method

In this method, stadia hairs are kept at fixed interval and the staff interval or intercept (corresponding to the stadia hairs) on the leveling staff varies. Staff intercept depends upon the distance between the instrument station and the staff.

❖ Movable - Hair method

In this method, the staff interval is kept constant by changing the distance between the stadia hairs. Targets on the staff are fixed at a known interval and the stadia hairs are adjusted to bisect the upper target at the upper hair and the lower target at the lower hair. Instruments used in this method are required to have provision for the measurement of the variable interval between the stadia hairs. As it is inconvenient to measure the stadia interval accurately, the movable hair method is rarely used.

(ii) Non - stadia systems

This method of surveying is primarily based on principles of trigonometry and thus telescopes without stadia diaphragm are used. This system comprises of two methods:

a) Tangential method and

b) Subtense bar method.

a) Tangential Method

In this method, readings at two different points on a staff are taken against the horizontal cross hair and corresponding vertical angles are noted.

b) Subtense bar method

In this method, a bar of fixed length, called a subtense bar is placed in horizontal position. The angle subtended by two target points, corresponding to a fixed distance on the subtense bar, at the instrument station is measured. The horizontal distance between the subtense bar and the instrument is computed from the known distance between the targets and the measured horizontal

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2.4 FIXED HAIR METHOD OF TACHEOMETRY

It is the most prevalent method for tacheometric surveying. In this method, the telescope of the theodolite is equipped with two additional cross hairs, one above and the other below the main horizontal hair at equal distance. These additional cross hairs are known as stadia hairs. This is also known as tacheometer.

Principle of Stadia method

A tacheometer is temporarily adjusted on the station P with horizontal line of sight. Let a and b be the lower and the upper stadia hairs of the instrument and their actual vertical separation be designated as i. Let f be the focal length of the objective lens of the tacheometer and c be horizontal distance between the optical centre of the objective lens and the vertical axis of the instrument. Let the objective lens is focused to a staff held vertically at Q, say at horizontal

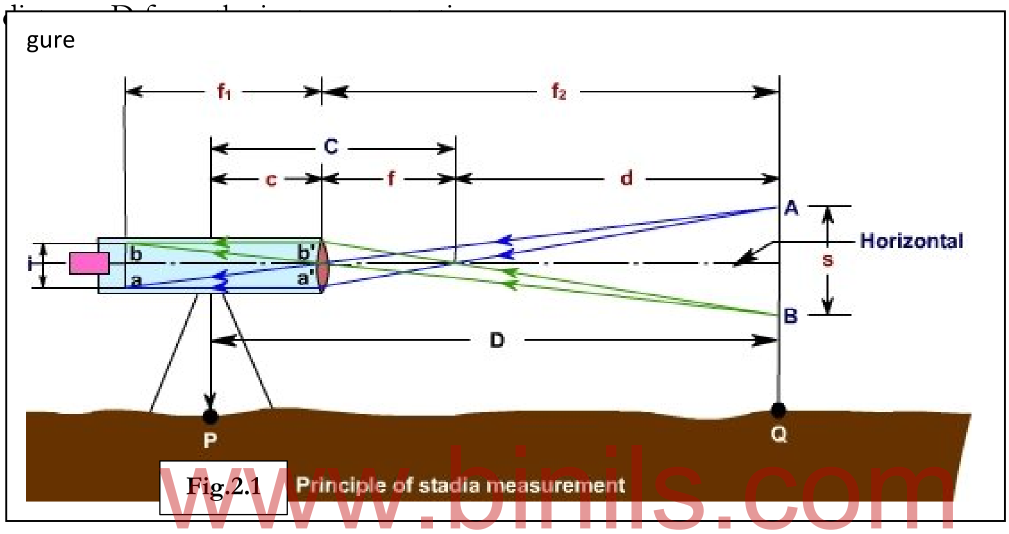


Fig.2.1 Principle of stadia measurement

By the laws of optics, the images of readings at A and B of the staff will appear along the stadia hairs at a and b respectively. Let the staff interval i.e., the difference between the readings at A and B be designated by s. Similar triangle between the object and image will form with vertex at the focus of the objective lens (F). Let the horizontal distance of the staff from F be d. Then, from the similar Ds ABF and a' b' F,(Figure2.1)

$$\frac{AB}{d} = \frac{a'b'}{f}$$

$$\text{Or, } d = \frac{AB}{a'b'} \times f = \frac{s}{i} \times f$$

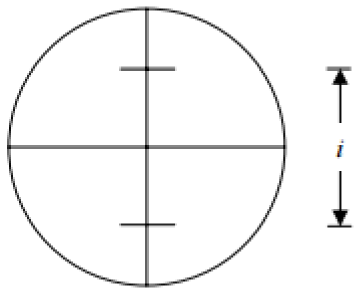
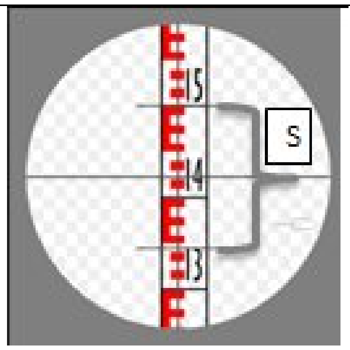
$$\therefore d = \frac{f}{i} \times s$$

as a' b' = ab = i. The ratio *(f / i)* is a constant for a particular instrument and is known as stadia interval factor, also instrument constant. It is denoted by K and thus d = K.s --------------------- Equation (2.1)

The horizontal distance (D) between the center of the instrument and the station point (Q) at which the staff is held is d + f + c. If C is substituted for (f + c), then the horizontal distance D from the center of the instrument to the staff is given by the equation

D = Ks + C ---------------------- Equation (2.2)

The distance C is called the stadia constant. Equation (2.2) is known as the stadia equation for a line of sight perpendicular to the staff intercept.

Stadia interval(i)	**Staff intercept(S)**
The distance between the upper hair and lower hair	The difference of upper hair reading and lower hair reading
	

Example1. The stadia readings with horizontal sight on vertical staff held 50m from a tacheometer were 1.285 m and 1.780 m. The focal length of the object glass was 25 cm. The distance between the object glass and the vertical axis of the tacheometer was 15 cm. Calculate the stadia interval.

Solution

C= f+d = 25+15 =40 cm = 0.4 m

S= 1.780-1.285 = 0.495

Now D=(f/i)s +(f+d)

50= (0.25/i)x 0.495 +).4

I= 2.49 x 10^{-3} m = **2.49 mm**

2.5 DISTANCE AND ELEVATION FORMULAE FOR STAFF VERTICAL: INCLINED SIGHT

(a) Consider angle of elevation (Positive)

It is usual that the line of sight of the tacheometer is inclined to the horizontal. Thus, it is frequently required to reduce the inclined observations into horizontal distance and difference in elevation.

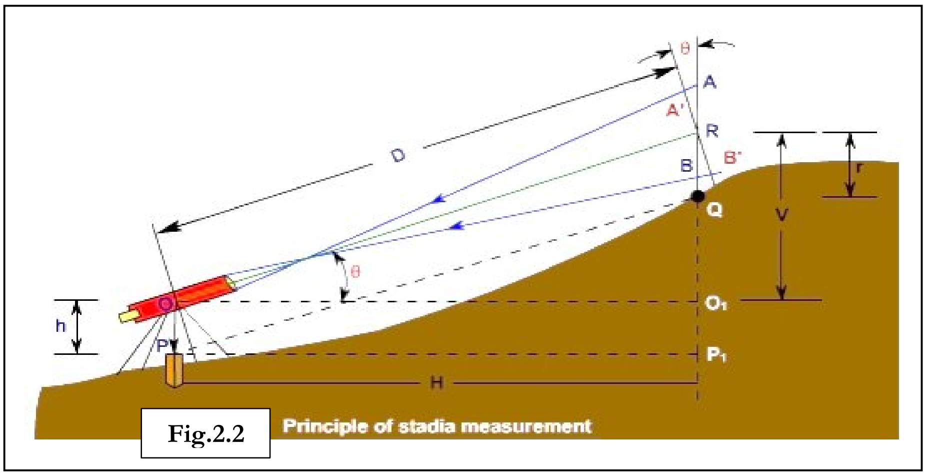


Fig.2.2 Principle of stadia measurement

Let us consider a tacheometer (having constants K and C) is temporarily adjusted on a station, say P (Figure 2.2). The instrument is sighted to a staff held vertically, say at Q. Thus, it is required to find the horizontal distance PP_1 (= H) and the difference in elevation P_1Q. Let A, R and B be the staff points whose images are formed respectively at the upper, middle and lower cross hairs of the tacheometer. The line of sight, corresponding to the middle cross hair, is inclined at an angle of elevation q and thus, the staff with a line perpendicular to the line of sight. Therefore, A'B' = AB cos q = s cos q where s is the staff intercept AB. The distance D (= OR) is C + K. scos q (from Equation 23.2). But the distance OO_1 is the horizontal distance H, which equals OR cos q. Therefore, the horizontal distance H is given by the equation.

$$H = (Ks \cos q + C) \cos q$$

Or $H = Ks \cos^2 q + C \cos q$ ---------------- Equation (2.3)

in which K is the stadia interval factor (f / i), s is the stadia interval, C is the stadia constant (f + c), and q is the vertical angle of the line of sight read on the vertical circle of the transit.

The distance RO_1, which equals OR sin q, is the vertical distance between the telescope axis and the middle cross-hair reading. Thus V is given by the equation

$$V = (K s \cos q + c) \sin q$$

$V = Ks \sin q \cos q + C \sin q$ ---------------- Equation (2.4)

$\therefore V = \frac{1}{2} Ks Sin2\theta + C sin\theta$ ---------------- Equation (2.5)

(Or) V can also be calculated by,

$$V = D\tan\theta$$

Since the line of sight has an angle of elevation θ, as shown in the figure, we have

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

Thus, the difference in elevation between P and Q is (h + V - r), where h is the height of the instrument at P and r is the staff reading corresponding to the middle hair.

b. Consider angle of depression (negative)

- Set the instrument at P
- vertical circle is set to zero and the staff reading on B.M is taken
- Rotate the telescope downward till the three hairs cut the portion of the staff zero.
- Note down the angle of depression (θ) and three hair readings.

$D = Cs \cos^2\theta + k\cos\theta$

Vertical component below the line of sight,

$V = \frac{Cs\sin 2\theta}{2} + k\sin\theta$ (Or) V can also be calculated by, $V = D\tan\theta$

Elevation of the staff station for the angle of depression:

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

Example2. In order to carry out tacheometric surveying, following observations were taken through a tacheometer set up at station P at a height 1.235m.

Staff held Vertical at	**Horizontal distance from P (m)**	**Staff Reading (m)**	**Angle of Elevation**
Q	100	1.01	0°
R	200	2.03	0°
S	?	3.465, 2.275, 1.280	5° 24' 40"

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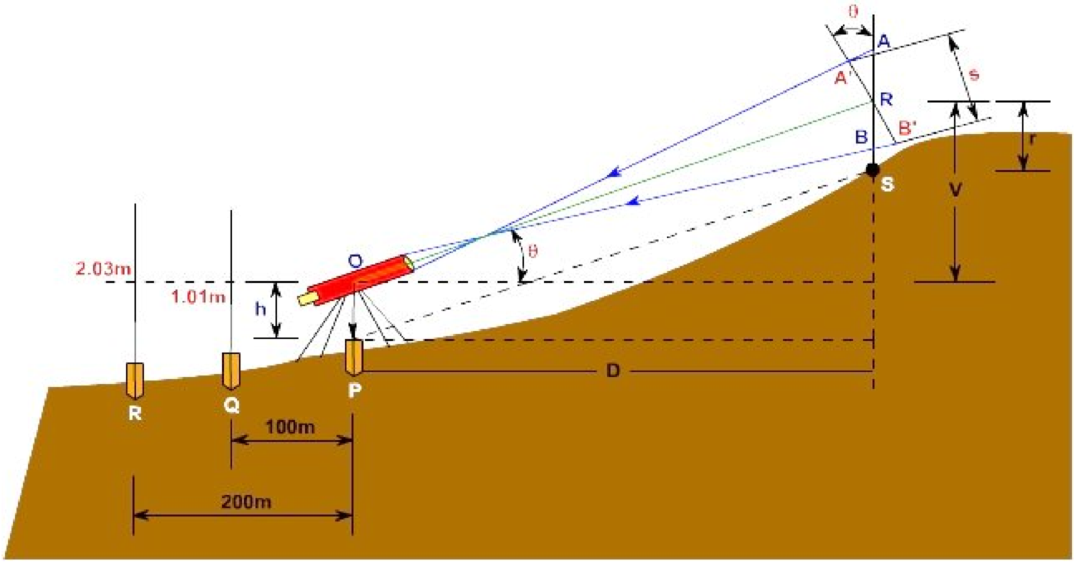
Compute the horizontal distance of S from P and reduced level of station at S if R.L. of station P is 262.575m



Solution :

Since the staff station P and Q are at known distances and observations are taken at horizontal line of sight, from equation 23.2

i.e. from D = K.s + C, we get

100 = K. 1.01 + C --------------- Equation 2.6

200 = K. 2.03 + C --------------- Equation 2.7

where K and C are the stadia interval factor and stadia constant of the instrument.

Therefore Solving equation 1 and 2 , $K = \frac{100}{1.02} = 98.04$

Substituting, value of K in Equation 1, we get

C = 100 - 1.01 x 98.04 = 0.98

Now, for the observation at staff station S, the staff intercept

s = 3.465 - 1.280 = 2.185 m;

Given, the angle of elevation (of a observation at S), q = 5° 24' 40"

Using equation 23.3 i.e., D = K s $\cos^2$ q + C.cos q, the horizontal distance of S from P is

D = 98.04 x 2.185 x $\cos^2$ 5° 24' 40" + 0.98 cos 5° 24' 40"

= 212.312 + 0.9756 = 213.288 m

`

Using equation 23.4, i.e., $V = \frac{1}{2} Ks \sin 2\theta + C.\sin\theta$ the vertical distance

$$V = \frac{1}{2} \times 98.04 \times 2.185 \sin 2 \times (5^\circ 24'40'') + 0.98 \sin 5^\circ 24'20''$$

$= (20.11 + 0.0924)\text{m} = 20.203$ m

Thus R.L. of station S = R.L. of P + h + V - r

= 262.575 + 1.235 + 20.203 - 2.275

= 281.738 m

Uses of Stadia Method

The stadia method of surveying is particularly useful for following cases:

1. In differential leveling, the back sight and foresight distances are balanced conveniently if the level is equipped with stadia hairs.
2. In profile leveling and cross sectioning, stadia is a convenient means of finding distances from level to points on which rod readings are taken.
3. In rough trigonometric, or indirect, leveling with the transit, the stadia method is more rapid than any other method.
4. For traverse surveying of low relative accuracy, where only horizontal angles and distances are required, the stadia method is a useful rapid method.
5. On surveys of low relative accuracy - particularly topographic surveys-where both the relative location of points in a horizontal plane and the elevation of these points are desired, stadia is useful. The horizontal angles, vertical angles, and the stadia interval are observed, as each point is sighted; these three observations define the location of the point sighted.

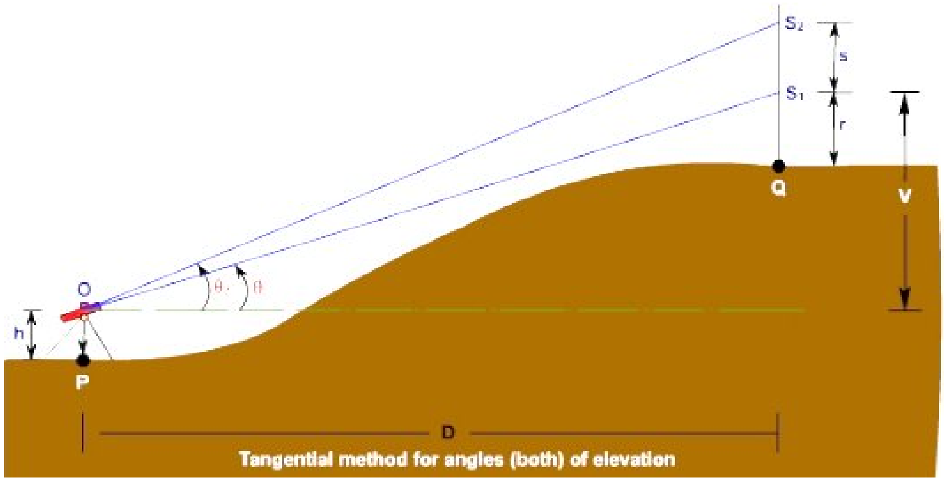
Tangential Method

The tangential method of tacheometry is being used when stadia hairs are not present in the diaphragm of the instrument or when the staff is too far to read.

In this method, the staff sighted is fitted with two big targets (or vanes) spaced at a fixed vertical distances. Vertical angles corresponding to the vanes, say q_1 and q_2 are measured. The horizontal distance, say D and vertical intercept, say V are computed from the values s (pre-defined / known) q_1 and q_2 . This method is less accurate than the stadia method.

Depending on the nature of vertical angles i.e, elevation or depression, three cases of tangential methods are there.

(i) Both vertical Angles are Angles of Elevation



$V = D \tan \phi_1$

and $V + s = D \tan \phi_2$

Thus, $s = D(\tan \phi_2 - \tan \phi_1)$

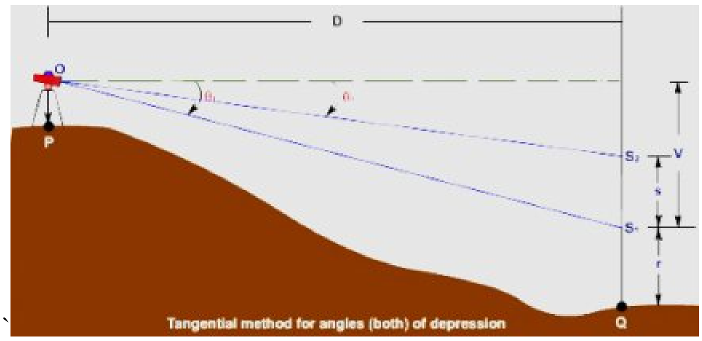
$$\text{Or, } D = \frac{s}{(\tan\theta_2 - \tan\theta_1)}$$ -------------------- Equation (2.8)

$$\text{and } V = \frac{s}{(\tan\theta_2 - \tan\theta_1)} \tan\theta_1$$ -------------------- Equation (2.9)

Therefore R.L. of Q = (R.L. of P + h) + V – r -------------------- Equation (2.10)

where, h is the height of the instrument, r is the staff reading corresponding to lower vane.

(ii) When Both Vertical Angles are Depression Angles



From above figure

$V = D \tan \phi_1$

and $V - s = D \tan \phi_2$

Thus, $s = D (\tan \phi_1 - \tan \phi_2)$

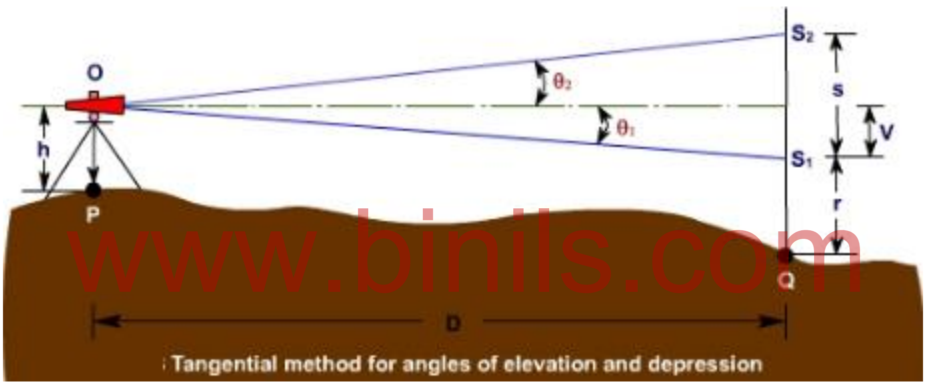
$$\text{Or } D = \frac{s}{(\tan\theta_2 - \tan\theta_1)}$$ -------------------- Equation (2.11)

$$\text{and } V = \frac{s}{(\tan\theta_1 - \tan\theta_2)}\tan\theta_1$$ -------------------- Equation (2.12)

Therefore R.L. of Q = (R.L. of P + h) - V – r -------------------- Equation (2.13)

where, h is the height of the instrument, r is the staff reading corresponding to lower vane.

(iii) When one of the Vertical Angles is Elevation Angle and the other is Depression Angle



Tangential method for angles of elevation and depression

From above Figure

$V = D \tan \phi_1$

and $s - V = D \tan \phi_2$

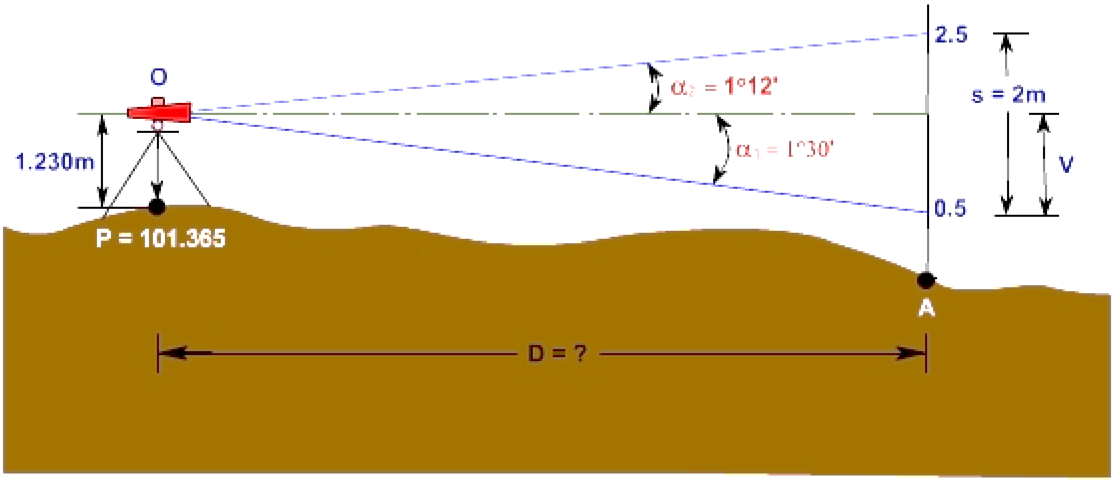
Thus, $s = D (\tan \phi_2 + \tan \phi_1)$

$$\text{Or, } D = \frac{s}{(\tan\theta_2 + \tan\theta_1)}$$ -------------------- Equation (2.14)

$$\text{and } V = \frac{s}{(\tan\theta_2 + \tan\theta_1)}\tan\theta_1$$ -------------------- Equation (2.15)

Therefore R.L. of Q = (R.L. of P + h) - V – r -------------------- Equation (2.16) where, h is the height of the instrument, r is the staff reading corresponding to lower vane.

Example3: In a tangential method of tacheometry two vanes were fixed 2 m apart, the lower vane being 0.5 m above the foot of the staff held vertical at station A. The vertical angles measured are +1° 12' and -1° 30'. find the horizontal distance of A and reduced level of A, if the R.L. of the observation station is 101.365 m and height of instrument is 1.230 m.



Solution :

Let D be the horizontal distance between the observation station P and staff point A. Then, from Figure

$V = D \tan a_1$

$s - V = D \tan a_1$

Or, $s = D \tan a_2 + D \tan a_1$

$$\therefore D = \frac{s}{\tan\alpha_1 + \tan\alpha_2}$$

Given, s = 2 m; a_1= 1° 30' & a_2 = 1° 12'

$$\therefore D = \frac{2.0}{\tan(1^\circ 30') + \tan(1^\circ 12')} = 42.43\text{m}$$

$$\text{and } V = \frac{s\tan\alpha_1}{\tan\alpha_1 + \tan\alpha_2} = \frac{s\tan 1^\circ 30'}{\tan 1^\circ 30' + \tan 1^\circ 12'} = 1.11\text{m}$$

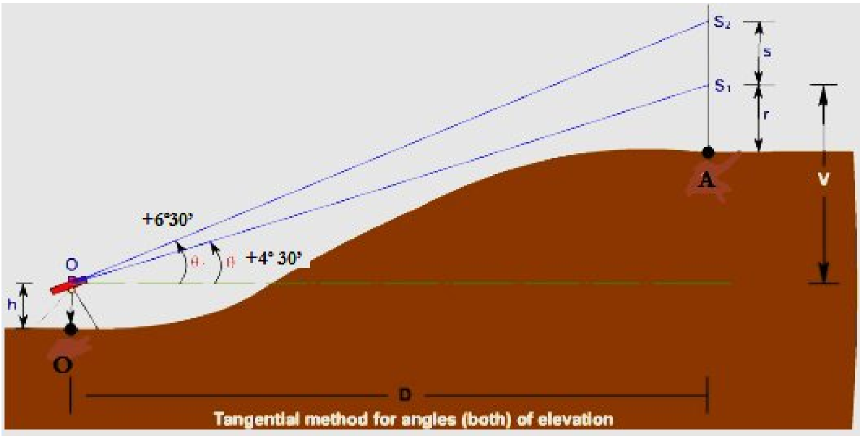
Therefore R.L. of A = 101.365 + 1.230 -1.11 - 0.5 = 100.985 m

`

Example 4. The following observations were taken with transit theodolite

Inst station	Staff station	Target	Vertical angle	Staff reading in (m)	Remarks
O	**A**	**Lower**	**+4° 30'**	**0.950**	**RL of Instrument axis=255.500m**
		Upper	**+6°30'**	**3.250**	

Solution



Guide sketch

$$\text{Or, } D = \frac{s}{(\tan\theta_2 - \tan\theta_1)}$$

D= 2.300(tan6°30'-tan4° 30')= 65.340 m

Hence V= D tanø2

= 65.340 x tan4° 30' = 5.142 m

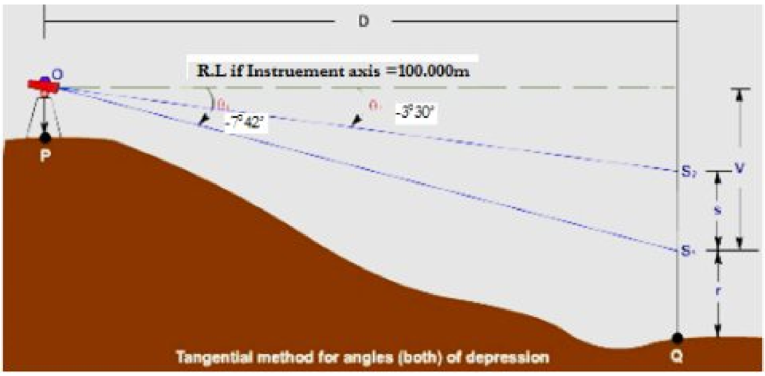
R.L of A = R.L of instrument axis +V- r

255.500 +5.142 – 0.950 = **259.692m**

Example 4.

The vertical angles to vane fixed at 1m and 3m above the foot of the staff held vertically at station Q were -3°30' and -7°42' respectively from theodolite station P. If the elevation of the instrument axis at P is 100.000m, calculate (1) Horizontal distance between P and Q and (2) The elevation of the staff station Q.

Solution



Guide sketch

S= 3-1 = 2m

Ø$_1$ = -3^{0}30'

Ø$_2$= -7^{0}42'

r=1m

Then,

$$\text{Or } D = \frac{s}{(\tan\theta_2 - \tan\theta_1)}$$

= 2/ (tan7^{0}42'-tan3^{0}30') = 27 m.

And V= D tan$\varnothing_2$

= 27 X tan7^{0}42' = 3.65 m

Elevation of staff station Q

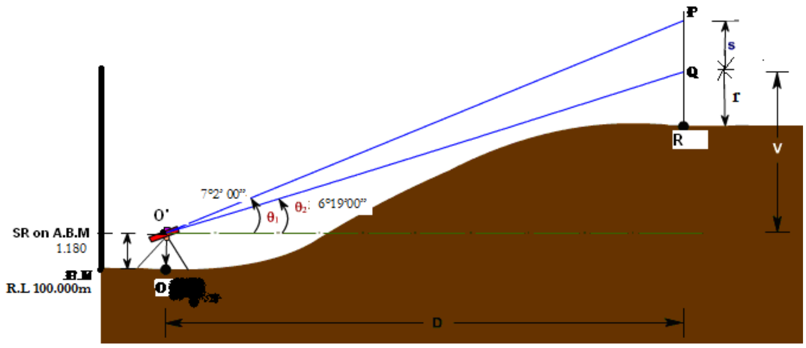
= 100 –V –r

=100-3.65 -1 = **95.35 m**

Example 5.

The vertical angles to vane fixed at 1m and 2m above the foot of the staff held vertically at station R were 6°19'00"and 7°2' 00" respectively from theodolite station O. The staff reading on B.M of R.L 100.000 with line of sight horizontal was 1.180m. calculate (1) Horizontal distance between O and R and (2) The elevation of the staff station R.

Solution:



Where $S= 2.00-1.000=1.000m$

$r=1.000m$

$\theta_1=+7°2'\ 00''$

$\theta_2=6°19'00''$

Then $$D = \frac{S}{\tan \theta_1 - \tan \theta_2}$$

$$D = \frac{1}{\tan 7°2'\ 00'' - \tan 6°19'00''}$$

$D = 78.87$ m

$$V= \frac{S \times \tan \theta_2}{\tan\alpha_1 - \tan \alpha_2}$$

$V= 78.87 X \tan 6°19'00''$

$V= 8.730m$

R.L of Instrument axis= R.L of B.M + Staff readings on B.M

= 100+1.180

=101.180m

R.L of staff station = R.L of Instrument axis +V-r

Where h= Portion of staff reading up to the lower target= 1.000m

= 101.180 +8.730-1.000

= 108.910

2.6 Anallatic lens (No Proof)

It is a special convex lens, fitted in between the object glass and eyepiece, at a fixed distance from the object glass, inside the telescope of a tacheometer. The function of the anallactic lens is to reduce the stadia constant to zero. Thus, when tacheometer is fitted with anallactic lens, the distance measured between instrument station and staff position (for line of sight perpendicular to the staff intercept) becomes directly proportional to the staff intercept. Anallactic lens is provided in external focusing type telescopes only.

In 1840, Porro devised the external focusing anallatic telescope, the special feature of which is on additional (convex) lens called an anallatic lens (or anallatic lens), placed between the diaphragm and the objective at a fixed distance from the latter.

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2.6.1 Advantages:

- As the additive constant (K) is zero, the calculations are greatly simplified.
- As the anallatic lens is properly sealed, moisture and dust cannot easily enter the telescope.

2.6.2 Disadvantages:

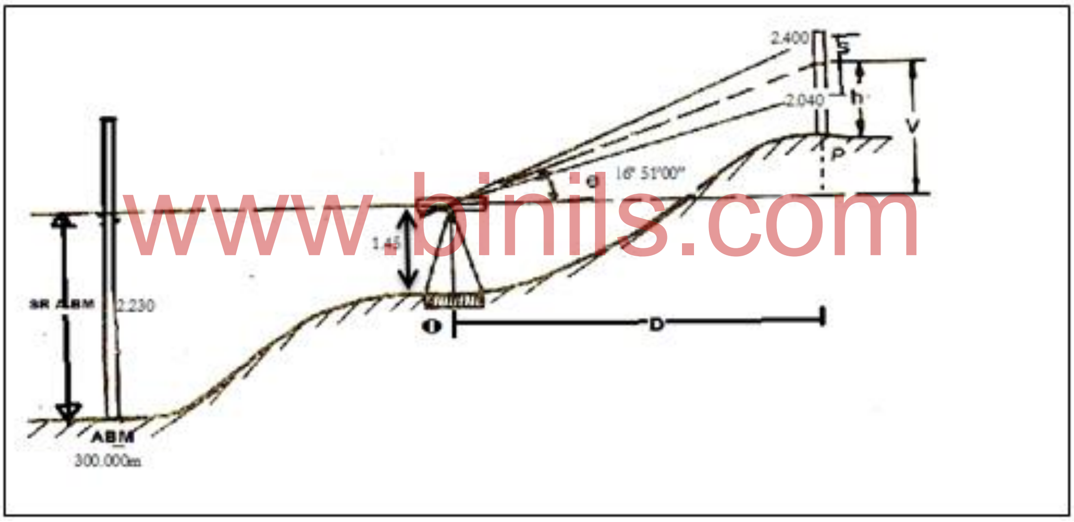
- It reduces the brilliancy of the image
- The anallatic lens cannot be cleaned easily.
- It increases the cost of the instrument because of one extra lens.

2.7 Simple Problem

Example1. To determine the elevation of station P in a tacheometer surveying following observation were made with staff hele vertical. The instrument was fitted with an anallactic lens and its multiplying constant was 100.Take R. L B.M. as 300.000m

Instrument station	H.I (m)	Staff station	Vertical angle	Staff reading in (m)		
				Top	Middle	Bottom
O	1.45	B.M.	$00^0O'0''$		2.230	
	1.45	P	$16^o\ 51'00''$	2.400		2.040

Solution:



Guide sketch

The staff is kept at point say "P" and having an inclination of (ø) $+16^o\ 51'00''$

Horizontal distance= $D_1 = CS_1 \cos^2\theta + K \cos\theta$

Where K= 100 and C= 0

$S = 2.400 - 2.040 = 0.360m$

D $= 100(0.360) \cos^2 16^o\ 51'00'' + 0 \times \cos 16^o\ 51'00''$

$= 32.975$ m

Vertical distance V $= D \tan \theta_1$

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$V = 32.975 \times \tan 16^\circ 51'00'' = 9.990$ m

Where h =Middle hair readings= 2.215

R.L. of staff station P = R.L of line of sight +V - h

Where R. L of line of sight = R.L. of B.M + Staff reading on B.M. when line of sight horizontal

= 300 +2.230=302.230m

Therefore R.L. of staff station (P) = 302.230 + 9.990-2.215 = 310.005 m

Result:

Distance between instrument station to staff station : 32.975m
R.L of staff station : 310.005

Example 2.A tacheometer having multiplying constant 100 and additive constant 0 was set up over a bench mark 250. 00m above datum and the following readings were taken

Staff station	Staff readings			Vertical angle
	Top	Middle	Bottom	
1	**1.620**	**1.035**	**0.450**	**+5° 14'**
2	1.680	1.270	0.860	-7° 23'

Calculate the distance of the points from the instrument stations and their elevation. The height of the instrument is 1.500m

Solution:

Given date

Bench Mark R. L = 250.000 m
Instrument height = 1.500 m
K = 100
C = 0
Staff station = 2 No

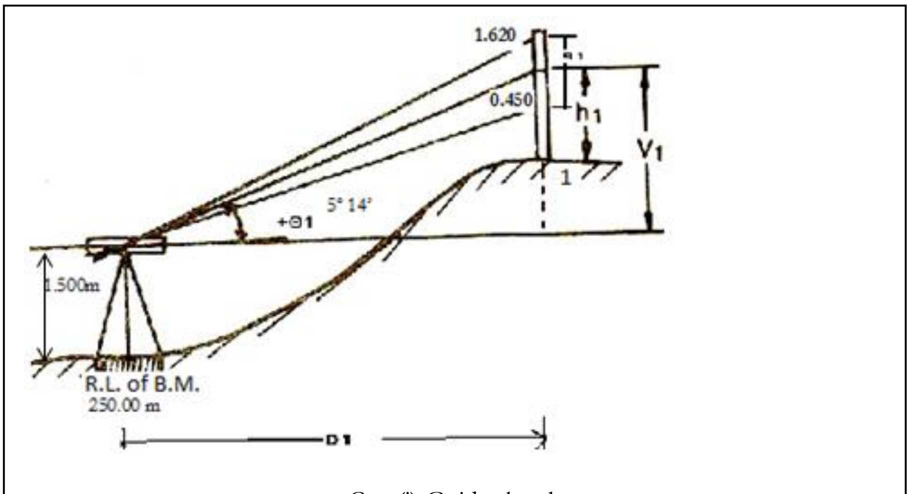
Staff station	Top	Middle	Bottom	Vertical angle
1	1.620	1.035	0.450	+5° 14'
2	1.680	1.270	0.860	-7° 23'

To find distance between staff stations to instrument & R.L if staff stations

Solution:

Case(i) The staff is kept at point "1" and having an inclination of +5° 14'

Horizontal distance= $D_1 = CS_1 \cos^2 \theta_1 + K \cos\theta_1$



Case(i) Guide sketch

Where K= 100 and C= 0

S_1 = 1.620-0.45 =1.170

D_1 = 100(1.17) $\cos^2$ 5° 14'+0

= 116.026 M

Vertical distance V1= $D_1 \tan \theta_1$

V_1 = 116.026 tan5° 14' = 10.677 m

Where h_1 =Middle hair readings= 1.035 (Given)

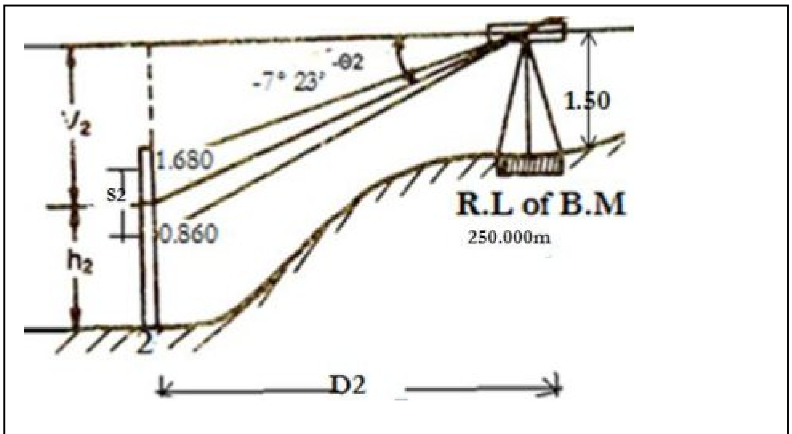
R.L. of station 1 = R.L of line of sight $+V_1-h_1$

Where R.L of line of sight = R.L. of B.M + Instrument height

= 250 +1.5=251.500m

Therefore R.L. of station 1 = 251.500+10.627 -1.035

= 261.092 m



Case(ii) Guide sketch

Case(ii) The staff is kept at point "2" and having an declination of -7° 23'

Horizontal distance= $D_2 = CS_2 \cos^2\theta_2 + K \cos\theta_2$

Where K= 100 and C= 0

S_2 = 1.680-0.860 =0.820

$D_2 = 100(0.82)\cos^2 7^\circ 23' + 0$

$= 80.645$ M

Vertical distance $V_2 = D_2 \tan \theta_2$

$V_2 = 80.645 \tan 7^\circ 23' = 10.450$ m

Where h_2 =Middle hair readings= 1.270 (Given)

R.L. of station 2 = R.L of line of sight - V_2 - h_2

Where R.L of line of sight = R.L. of B.M + Instrument height

= 250 +1.5=251.500m

Therefore R.L. of station 2= 251.500-10.450 -1.270

= 239.780m

Example 3. Find the gradient from A to B from the following observation made with a tacheometer fitted with an anallatic lens. The instrument constant being 100, 0.

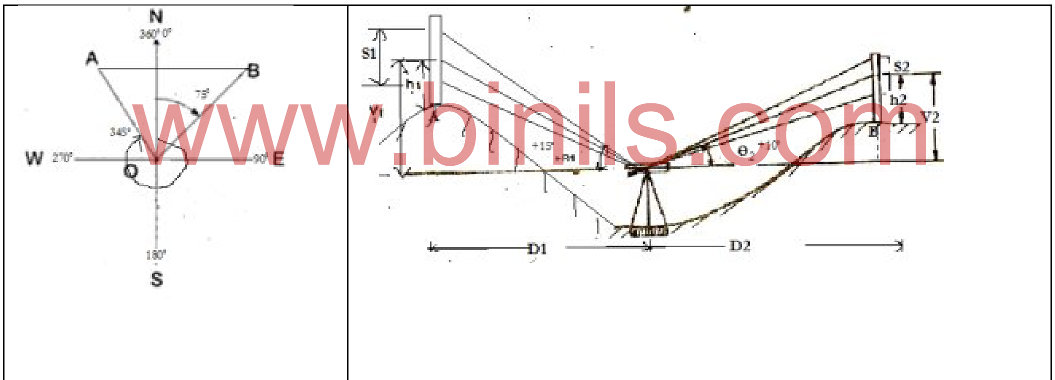
Inst. station	Staff station	Bearing	Readings on stadia hair in (m)	Vertical angle
0	A	345°	1.000,2.400,2.800	$+15^\circ$
	B	75^0	0.800,2.400,4.000	$+10^\circ$

Solution

Multiplying constant, C= 100

Additive constant, K = 0

$\angle AOB = (360^\circ - 345^\circ) + 75^0 = 90^\circ$



Guide sketch

Case(i)

Calculation of R.L of A

Staff intercept, S_1 = 2.800-1.000=1.800m

Axial hair readings h_1 = 1.900m

Horizontal distance of OA = $D_1 = CS_1 \cos^2 \theta_1 + K \cos\theta_1$

Where K= 100 and C= 0

$D_1 = 100(1.800)\cos^2 15^\circ + 0$

$= 167.94$ M

Vertical distance $V_1 = D \tan \theta_1$

$V_1 = 137.94 \tan 15^\circ = 44.99$ m say 45 m

R.L. of station A = R.L of Instrument axis $+V_1-h_1$

Assume R.L of the instrument axis as 100.000m

Therefore R.L. of station A = 100.00+45.00 -1.900

= 143.100 m

Case(i)

Calculation of R.L of B

Staff intercept, S_2 = 4.000-0.800=3.200m

Axial hair readings h_1 = 2.400m

Horizontal distance of OB = D_2= $CS_2 \cos^2 \theta_2 + K \cos\theta_2$

Where K= 100 and C= 0

$D_2 = 100(2.40) \cos^2 10^o + 0$

= 310.35 M

Vertical distance V_2= D tan θ_2

$V_2 = 310.35 \tan 10^o = 54.72$ m

R.L. of station B = R.L of Instrument axis $+V_2-h_2$

Assume R.L of the instrument axis as 100.000m

Therefore R.L. of station B = 100.00+54.72 -2.40

= 152.32 m

Calculation of gradients from A to B

L AOB = 90°

From the triangle AOB

OA= D_1 = 137.94 m

OB = D_2 = 310.35

Horizontal distance of AB = Sq. root of $(OA)^2 + (OB)^2$

Horizontal distance of AB = Sq. Root of $(167.94)^2+(310.35)^2$

Horizontal distance AB= 352.87 m

Difference in R.L between A and B = R.L of B-R. L of A

Difference in R.L between A and B= 152.32-143.100

Difference in R.L between A and B= 9.22 m

Gradients A tp B = 1 in $\dfrac{\text{Horizontal distance of AB}}{\text{Difference in R.L between A and B}}$

Gradients from A to B = 1 in (352.87/9.22)

= 1 in 38.27 m

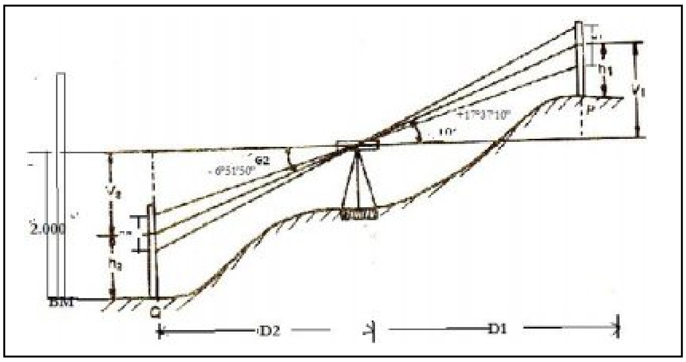
Result: Gradient of AB = 1 in 38.27 m falling from B to a

Example 4. Fine the gradient from P to Q from the following observation made with a tacheometer fitted with an anallatic lens. The instrument constant being 100, 0. The staff reading on B.M. with line of sight horizontal was 2.000m ant take R.L of B.M as 100.00m

Inst. station	**Staff station**	**Readings on stadia hair in (m)**	**Vertical angle**
0	P	2.110, 2.010, 1.910	+17°37'10"
	Q	0.925, 0.825, 0.725	- 6°51'50"

Calculation:

(i) Instrument at "O" staff held at P



Θ_{1}= 17°37'10"

Staff intercept S_1 = 0.20 m

Multiplying constant C= 100

Additive constant K = 0

Middle hair readings (h_1) = 2.010m

Staff reading on on A.B.M (S') = 2.000m

Calculation of distance

Horizontal Distance (D_1) = CSCos^2 Θ_1 +K Cos Θ_1

= 100X0.200 Cos^{2}17°37'10" + 0 X Cos 17°37'10"

= 18.168m

Vertical distance V_1 = D_1 tan Θ_1

= 18.168 X tan 17°37'10"

= 5.770m

R.L calculation:

R.L of Instrument axis at station "O" = R.L of A.B.M + Staff reading on A.B.M

= 100.000+2.000=102.000m

R.L of staff station "P" = R.L of instrument axis + V –h_1

=102.000 +5.77 -2.010 = 105.760m

(ii) Instrument at " O" staff held at Q

Θ_2= - 6°51'50"

Staff intercept S_2 = 0.20 m

Multiplying constant C= 100

Additive constant K = 0

Middle hair readings (h_2) = 0.825m

Staff reading on A.B.M (S') = 2.000m

Calculation of distance

Horizontal Distance (D_2) $= CS\cos^2 \Theta_2 + K \cos \Theta_2$

$= 100 X 0.200 \cos^2 6°51'50" + 0 X \cos 6°51'50"$

= 19.710 m

Vertical distance $V_2 = D_2 \tan \Theta_2$

$= 19.71 X \tan 6°51'50"$

= 2.370m

R.L calculation:

R.L of Instrument axis at station "O" = R.L of A.B.M + Staff reading on A.B.M

= 100.000+2.000=102.000m

R.L of staff station "Q" = R.L of instrument axis – V_2 –h_2

=102.000 -2.370 -0.825 = 98.805m

R.L of P = 105.760 m

R.L of Q = 98.805

Calculation of Gradient of the give line PQ

Distance between P and Q (D) = 18.168+19.710 = 37.878m

Vertical distance between P and Q(V) = R.L of A $\sim$ R.L of B

=105.760 $\sim$ 98.805 =6.955 m

Gradients $= \frac{V}{D} = \frac{6.955}{37.878} = \frac{(6.955/6.955)}{(37.878/6.955)}$

$= \frac{1}{5.4}$ or 1 in 5.45

`

2.8 DISTOMAT:

Distomat is the Electronic Distance Measurement (EDM) equipment which is used for measuring distance electronically between two points.

The innovative hand-held laser DISTO meter for fast and easy distance measurements of length, squares and volumes with the press of a button. Save time and money

Direct measurement of distances can be obtained by using electronic instruments that rely on propagation, reflection and reception of either light waves or radio waves. They may be broadly classified into three types:

a. Infrared wave instruments

b. Light wave instruments

c. Microwave instruments

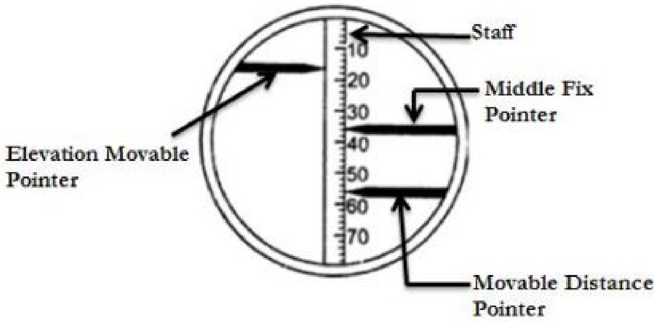
Electronic distance measurement (EDM) is a method of determining the length between two points, using phase changes, that occur as electromagnetic energy waves travels from one end of the line to the other end

Electronic distance measurement equipment's are incorporated along with theodolites that possess automatic angle readout called as total station (electronic tacheometers) also called as field to finish systems. These record distance and angles simultaneously.

2.9 Direct Reading Tacheometer:

A direct reading tacheometer in which two movables, cam-controlled, stadia points enable the horizontal distance and the vertical height to be read off from the staff intercepts directly without calculation.

The Jeffcott direct reading tacheometer



Jeffcott direct reading tacheometer

This instrument was devised by H.H. Jeffcott, and manufactured by Cooke, Troughton, and Simms of London. The horizontal distance and vertical components are read without measuring the vertical angle. The diaphragm carries three platinum pointer: the middle one is fixed: and the outer ones-one for the distance and the other for elevation are movable. The movable pointers are actuated by system of cams and levers operated by the tilting of the telescope. The right-hand movable pointer is called distance pointer. The staff intercept between this and the fixed pointer, multiplied by 100 gives the horizontal distance directly, if the telescope is anallatic. The left-hand movable pointer is called height pointer. The staff intercept between this one and the fixed pointer multiplied by 10 gives the vertical component.

The fixed pointer can be conveniently set first to read a whole metre or any convenient reading on the staff. The height pointer, moves upwards or away from the fixed pointer for elevations, and downwards or towards it for depression. A view through the telescope appears somewhat as shown in figure.

As a simple example, Let the readings of height, fixed and distance pointer be 1.310, 1.000, and 3.300, respectively. Then, the horizontal distance is 100(3.300-2.000)=1.30 m, and the vertical components is 10(2.000-1.3`0)= 6.9m. The latter is positive since the image of the staff is inverted. It is assumed that the instrument is fitted with an anallatic lens, and that the constant is 100.

2.10 Determination of constant *k* and C

The values of the multiplying constant *k* and the additive constant C can be computed by the following methods:

1st method:

In this method, the additive constant C = (f + d) is measured from the instrument while the multiplying constant k is computed from field observations:

1. Focus the instrument to a distant object and measure along the telescope the distance between the objective and cross-hairs.

$$\frac{1}{f} = \frac{1}{f1} + \frac{1}{f2}$$

Since f_1 is very large in this case, f is approximately equal to f_2, i.e., equal to the distance of the diaphragm from the objective.

2. The distance *d* between the instrument axis and the objective is variable in the case of external focusing telescope, being greater for short sights and smaller for long sights. It should, therefore be measured for average sight. Thus, the additive constant *(f + d)* is known.

3. To calculate the multiplying constant k, measure a known distance D_1 and take the intercept s_1 on the staff kept at that point, the line of sight being horizontal. Using the equation,

$$D = ks_1 + C$$

$$\text{Or} \qquad k = \frac{D_1 - C}{S_1}$$

For average value, staff intercepts, s_2, s_3 etc., can be measured corresponding to distance D_2, D_3 etc., and mean value can be calculated.

Note: *In case of some external focusing instruments, the eye-piece-diaphragm unit moves during focusing. For such instruments d is constant and does not vary while focusing.*

2nd method: Theory:

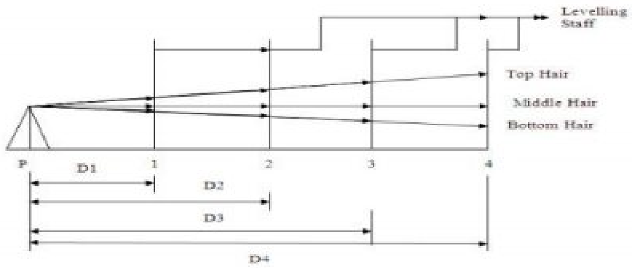
In this method, both the constants are determined by field observations as under:

Instruments Required:

Tacheometer with stand, Levelling Staff, Ranging rods, tape.

Procedure:

1. Setup the instrument at P.
2. Measure a line from P , 120metres long on a fairly level ground and fix arrows at 30m intervals.
3. Keep the vertical circle to read zero during observations.
4. Note down the stadia hair readings (top, middle, bottom) by placing the staff over the arrow stations (1234).
5. Calculate the other staff intercepts in the manner.



Field observation for tacheometer constant

Calculation:

Stadia intercept:

S = Difference of top and bottom hair readings.

Let S_1 is the staff intercept corresponding to distance D_1 and S_2 corresponding to D_2.

By using tacheometric equation.

D= f/i S + (f + d), since vertical angle is zero.

Where, f/i = Multiplying constant denoted by K

and, (f + d) = additive constant denoted by C

then, $D = KS + C$

Now we have,

$D_1 = KS_1 + C$

$D_2 = KS_2 + C$

Solving the above two equations to get the values for K and C.

Similarly find out the values for K and C, by other set of readings.

The average values of the K and C will be the tacheometric Constants.

Tabulation:-

Inst. at	Staff station	Horizontal Distance	Stadia hair readings			Stadia intercept
			Top	Middle	Bottom	
A	1	30m				
	2	60m				
	3	90m				
	4	120m				

Ex. The following observation was made with a tacheometer find out the constants of that instrument with line of sight horizontal

Int at	Staff at	Distance form station "O" in m	Top hair reading(m)	Bottom hair reading (m)
O	P1	10	1.50	1.60
	P2	20	2.00	2.200
	P3	30	2.50	2.800

Solution:

Calculation:

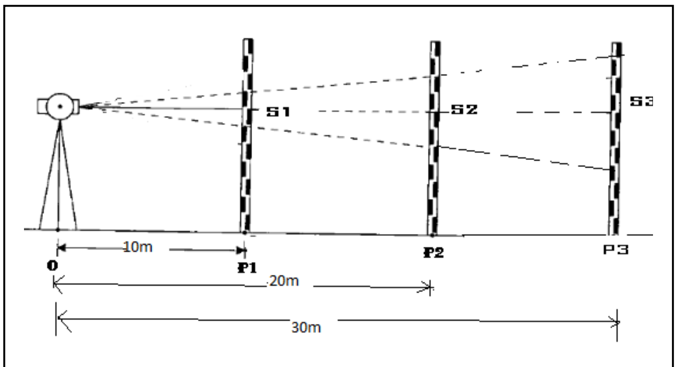
For line of sight horizontal D = CS +K

Where D=distance between instrument station to staff station 10m

C=Multiplying constant

S= Staff intercept

K = Additive constant



Rough sketch

D1 = CS1 +K --------------------(1)

10=C (0.1) +K

D2 = CS2+K

20=C(0.2) +K---------------(2)

D3= CS3 +K

30=C(0.3)+K --------------(3)

I. Subtracting equation (2) – (1)

20 = 0.2 C +K

(-) (-) (-)

10 = 0.1 C +K

10 = 0.1 C

C= 10/0.1

C= 100

Substitute Multiplying constant C= 100 in equation (1)

10= (100X0.1) + K

K= 10 - 10

K= 0

`

II. Subtracting equation (3) – (2)

30 = 0.3 C +K

(-) (-) (-)

20 m = 0.2 C +K

10 = 0.1 C

C= 10/0.1

C= 100

Substitute Multiplying constant C= 100 in equation (2)

20= (100X0.2) + K

K= 20 - 20

K= 0

III. Subtracting equation (3) – (1)

30 = 0.3 C +K

(-) (-) (-)

10 = 0.1 C +K

20 = 0.2 C

C= 20/0.2

C= 100

Substitute Multiplying constant C= 100 in equation (3)

30= (100X0.3) + K

K= 30 -30

K= 0

Average:

C = (100+100+100) /3

C= 100

K = (0+0+0)/3

K = 0

2.11 Tacheometric Traverse:

If the survey is to be carried over a long stretch of land, as in the case of alignment surveys, then it is necessary to keep the instrument at various points so that the complete area can ne covered, as one instrument position will not cover the entire area. In such a case, traverse station A,B, D, D etc., are to be set out along the stretch of land. Then the following procedure is adopted

1. Set and level the instrument station A, the first traverse station. Measure the height of instrument over that station

2. Set the horizontal plate to zero and orient the telescope towards the north using tubular compass. Clamp the instrument in the position.
3. Releasing the upper plate, sight the staff held over the BM and observe the vertical angle and stadia readings. R.L. of line collimation and station A, can be determined from the above observation
4. From station A, locate all the points around A by taking horizontal angle vertical angle and stadia readings on staff held at each of those points
5. Direct the telescope toward the staff held at the second traverse station B and note horizontal angle, vertical angle and stadia readings. From these readings the distance AB , direction of AB and R.L. of B can be determined.
6. Shift the instrument to station B. Set and level the instrument, orient the telescope towards north with horizontal circle reads zero similar to the procedure adopted at station A. Measure the height of instrument at B.
7. Releasing the upper plate swing the telescope, and sight the staff held at the previous station A. Note the horizontal angle, vertical angle and stadia readings on A. From these readings also the distance AB, direction AB and R.L of B can be determined
8. Mean of the two values fixes the location of second instrument station B exactly
9. Now from B, locate all the points around B as did at station A
10. Repeat this process until the end of traverse is reached

2.12 Errors in tacheometric Surveying:

Errors in Stadia Measurement

Most of the errors associated with stadia measurement are those that occur during observations for horizontal angles (Lesson 22) and differences in elevation (Lesson 16). Specific sources of errors in horizontal and vertical distances computed from observed stadia intervals are as follows:

1. Error in Stadia Interval factor

This produces a systematic error in distances proportional to the amount of error in the stadia interval factor.

2. Error in staff graduations

If the spaces on the rod are uniformly too long or too short, a systematic error proportional to the stadia interval is produced in each distance.

3. Incorrect stadia Interval

The stadia interval varies randomly owing to the inability of the instrument operator to observe the stadia interval exactly. In a series of connected observations (as a traverse) the error may be expected to vary as the square root of the number of sights. This is the principal error affecting the precision of distances. It can be kept to a minimum by proper focusing to eliminate parallax, by taking observations at favorable times, and by care in observing.

4. Error in verticality of staff

This condition produces a perceptible error in measurement of large vertical angles than for small angles. It also produces an appreciable error in the observed stadia interval and hence in computed distances. It can be eliminated by using a staff level.

5. Error due to refraction

This causes random error in staff reading.

6. Error in vertical angle

Error in vertical angle is relatively unimportant in their effect upon horizontal distance if the angle is small but it is perceptible if the vertical angle is large.

Questions bank

A. One-mark question

1. Define tacheometer
2. Mention the system of tacheometry
3. Draw the different pattern of stadia diaphragm
4. Define staff intercept
5. Define stadia interval
6. Name the different constant applied in stadia tacheometry
7. State the horizontal distance formula if the line of sight horizontal of stadia tacheometry system
8. State the vertical distance formula if the line of sight horizontal of stadia tacheometry system
9. Define anallatic lens
10. How errors are classified in tacheometry surveying
11. Define Distomat
12. Who was invented the direct reading tacheometer
13. What are the advantages direct reading tacheometer

B. Three marks questions

1. What is tacheometry and what are its advantages
2. What is anallatic lens? Explain their object
3. Differentiate between staff intercept and stadia interval
4. Write short notes on distomats
5. Write short notes on direct reading tacheometer

C. Ten marks questions

1. How will you determine the tacheometric constant in the field?
2. Describe the field procedure adopted for tacheometric traverse
3. List out the errors in tacheometric surveying
4. Fine the constants of tacheometer from the following data
5. Enumerate salient features of direct reading tacheometer

Horizontal distance	Reading at	
	Lower	Upper
200 m	1.50	3.46
400 m	0.40	4.33

5. A tacheometer fitted with an analatic lens having the value of constant 100 was used and the following observations were obtained. The staff was held vertical

Inst station	HI in m	Vertical angle	Staff at	Staff readings in m
P	1.45	+2°24'	M	1.20,1.83.2.46
P	1.45	-4°36'	O	1.35,1.82,2.29

R.L of station m is 50 mts. Calculate R.L of P and Q and the distance PQ

6. A tachometer fitted with an analatic lens was setup at station D. Calculate the gradient from point A to B with the following observations

Station	Berating	Staff readings	Vertical angle
A	340°20'	0.600,1.855,2.910	+6°30'
B	70°30'	0.600,2.200,3.740	-3°20'

7. A tachometer fitted with an anallatic lens was set over a B.M. 250.000 above and the following readings were taken

Staff station	Stadia readings	Vertical angle
1	0.450,1.035,1.620	+5°14'
2	0.860,1.270,1.680	-47°33'

Calculate the distance of point from the instrument station and their elevations. Take multiplying constant as 100. The height of instrument is 1.500.

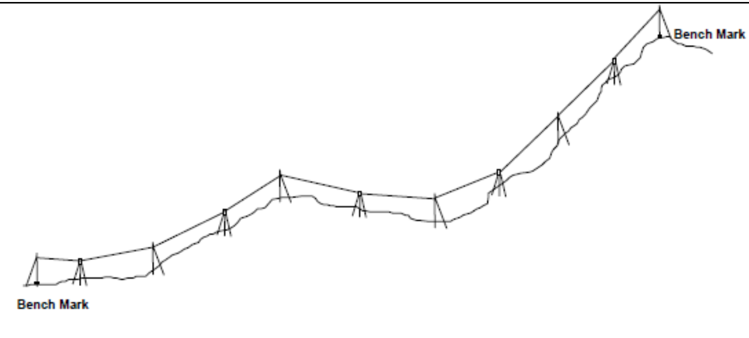
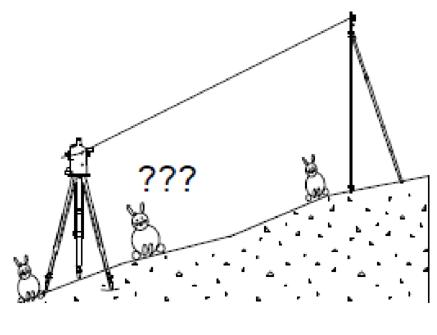
8. In the tangential method of tacheometry two vane were fixed 2m apart the lower vane being 0.50 m above the foot of the staff held vertical at station A. The vertical angles measured were $+1^0$ 12' and -1^0 30'. Find the horizontal distance and R.L of A, if the height of line of collimation is 100 m.
9. The vertical angles to vane fixed at 1m and 3m above the foot of the staff held vertically at station A were $+4^0$ 15' and $+6^0$50' respectively. Determine the horizontal distance and reduced level of A if the height of the instrument axis is 235.665 m
10. The vertical angles to vane fixed at 1m and 3m above the foot of the staff held vertically at station P were -2^0 15' and -5^0 50' respectively. Determine the horizontal distance and reduced level of P if the height of the instrument axis is 250.000 m.

Introduction – Finding elevation of objects – Base accessible - Base inaccessible: Single Plane and Double Plane methods – Problems on determination of elevation of objects

3.1 TRIGONOMETRIC LEVELLING

1.1 INTRODUCTION

Trigonometrical leveling is a branch of leveling in which the relative elevations of different stations are determined from the observed vertical angles and known horizontal distances. The vertical angles may be measured by means of an theodolite and the horizontal distance may either be measured.

	
Conventional levelling using level with levelling staff	Trigonometrical levelling using theodolite

Key point

By using the **trigonometric relationship**, the horizontal distance between instrument station and object and vertical height or elevations of the objects are determined

We shall explain the trigonometrical leveling under two heads:

- Observations for height and distances
- Geodetical observations

In the first case, the principles of the plane surveying will be used. It is assumed that the distance between the points observed are not large so that either the effect of curvature and refraction may be neglected or correction may be applied linearly to the calculated different in elevation.

In the second case, the distance between the points measured is geodetic and is large. The ordinary principles of plane surveying are not applicable. The corrections for curvature and refraction are applied in angular measure directly to the calculated angles.

3.1.2 METHODS OF TRIGNOMETRICAL LEVELLING:

They are **Base accessible and Base inaccessible.**

Base accessible

In this method the base of the object to which the vertical height required above a reference line is accessible to the surveyor. It means that the surveyor can go near the base of the object and the horizontal distance between the object and instrument. Station can be measured directly using chain and tape. In addition the vertical angle 'θ' is also measured from the instrument station

'p' be the base of the object 'p'

'o' be the instrument station.

'θ' be the vertical angle measured to the top of the object 'p'.

'D' be the horizontal.

'h' be the staff reading BM with reference to the horizontal line of sight.

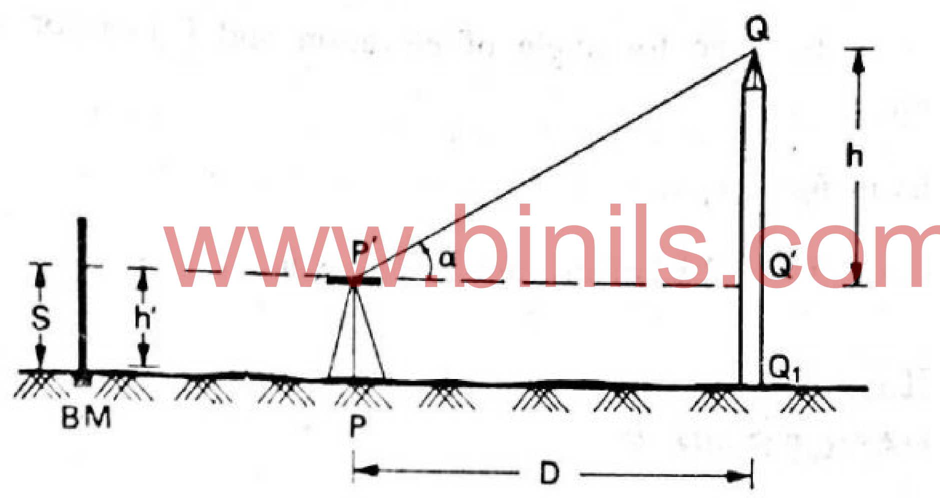


Fig 3.1.1

AB=D

PB=V

PAB=θ

PB=V/D=tanα

V=D tan α

R.L of P=R.L of BM +staff reading on BM+V

R.L of P=R.L of BM+ h+ D tanα

Base Inaccessible:

If the horizontal distance between the instrument and the object cannot be measured due to obstacles etc., two instrument stations are used so that they are in the same vertical plane as the elevated object.

- Single plane Method.
- Double plane Method.

Single plane method:

When the horizontal distance the instrument station and the object is not known, two instrument stations are chosen in line the object. The horizontal distance between these two instrument stations is measured and the vertical angles are observed to the top of the object.

a) **Nearest instrument station line of collimation is at lower level than the farthest one.**

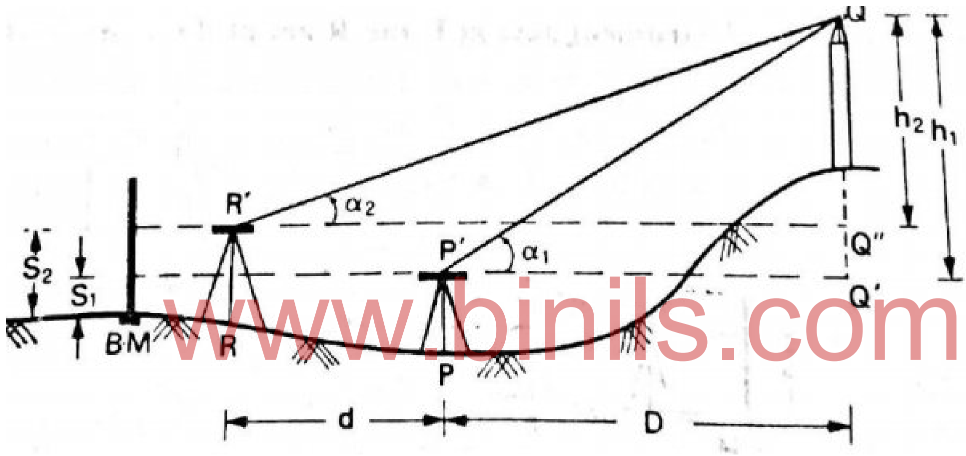


Fig 3.1.2

Let A and B are the instrument stations in line with the object P.

When the instrument at ac,

$$R.L \text{ of } P = R.L \text{ of } BM + a + D \tan\theta 1$$

When the instrument a+ DE

$$R.L \text{ of } P = R.L \text{ of } BM + b + (D+d) \tan\theta 2$$

$$V1-V2=h=b-a$$

$$R.L \text{ of } P=R.L \text{ of } BM +a+V1 \qquad -1$$

$$R.L \text{ of } P= R.L \text{ of } BM +b+V2 \qquad -2$$

`

When the instrument at 'A'

$$V1/D= \tan \theta 1 \qquad -3$$

When the instrument at 'B'

$$V2/d+D = \tan \theta 2$$

$$V2 = (d+D) \tan\theta 2 \qquad -4$$

I=II

$$R.L \text{ of } BM +a + V1 = R.L \text{ of } BM +b + V2$$

$$V1-V2 = b-a = h$$

3 in 4 sub.

$$D\tan\theta 1- (d+D)\tan\theta 2 = h$$

$$D\tan\theta 1 - d \tan\theta 2 - D\tan\theta 2 = h$$

$$D(\tan\theta 1-\tan\theta 2) = h+d\tan\theta 2$$

$$D = h+d \tan\theta 2/(\tan\theta 1-\tan\theta 2).$$

b) Nearest instrument station line of collimation is at higher level the farthest one:

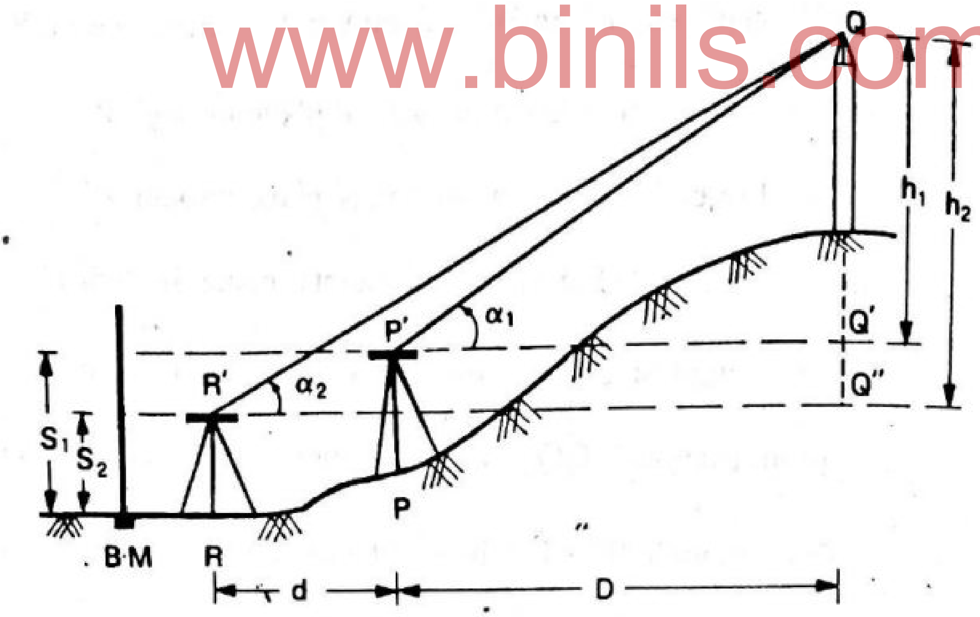


Fig.3.1.3

Let A and B are the instrument stations in line with object P.

$$V2-V1=h=a-b$$

R.L of P = R.L of BM + a + V1 -1

R.L of P = R.L of BM +b + V2 -2

When the instrument at 'A'

$$V1/D= \tan\theta 1$$

$$V1 = D \tan \theta 1 \quad -1$$

When the instrument at 'B'

$$V2/d+D = \tan\theta 2$$

$$V2 =(d+D) \tan\theta 2 \quad -2$$

$$V2-V1 =(d+D) \tan\theta 2 - D \tan\theta 1$$

$$V2-V1= d \tan\theta 2 + D \tan\theta 2- D \tan\theta 1$$

$$V2-V1=a-b=h$$

1 in 2

$d \tan\theta 2 +D \tan\theta 2 -D \tan\theta 1=h$

$D\tan\theta 2-D\tan\theta 1=h-d \tan\theta 2$

$D(\tan\theta 2-\tan\theta 1)=h-d\tan\theta 2$

$D=h-d \tan\theta 2/\tan\theta 2-\tan\theta 1$

c) Both the instrument stations lines of collimation one at same level.

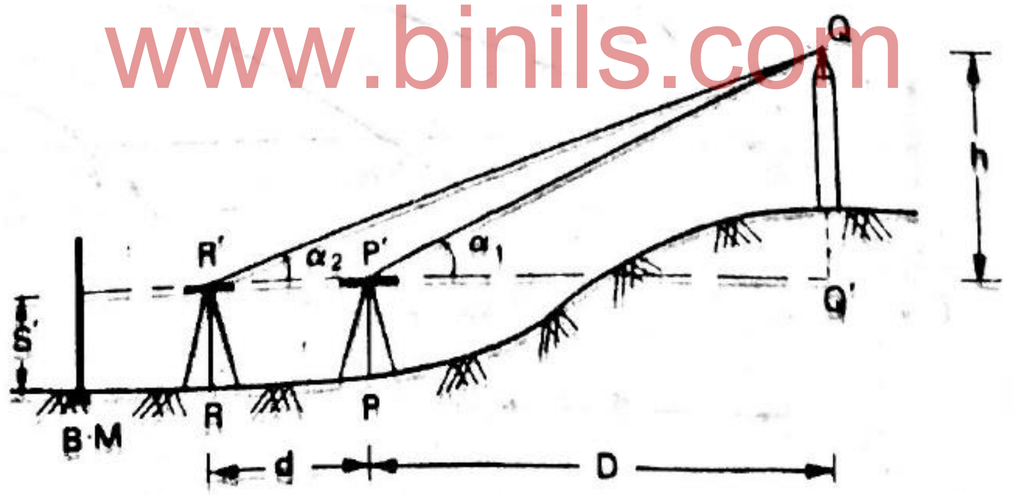


Fig.3.1.4

R.L of BM + a+ V1= R.L of BM+ a+ V2

$$V1=V2$$

$h=0, \quad V1=V2$

$$D \tan\theta 1= (d+D) \tan\theta 2$$

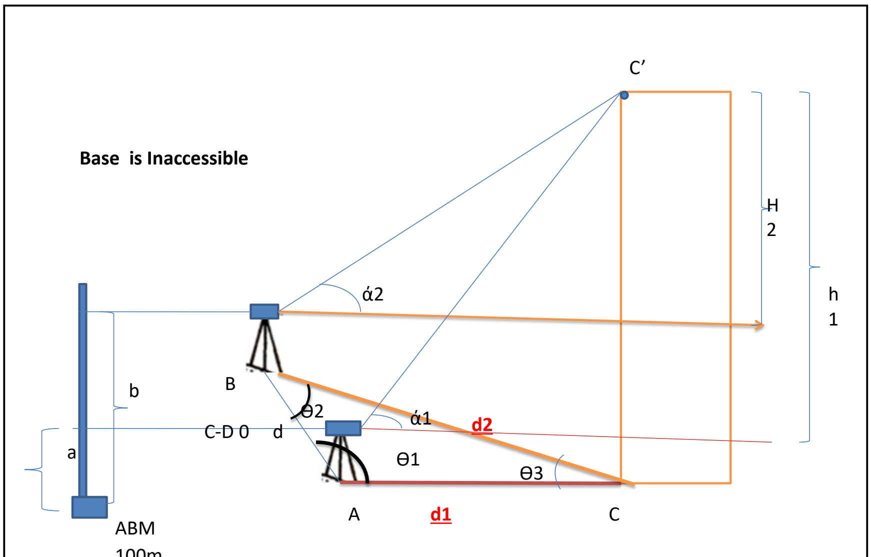
$$D \tan\theta 1= d \tan\theta 2 + D \tan\theta 2$$

$$D\tan\theta_1 - D\tan\theta_2 = D\tan\theta_2$$

$$D(\tan\theta_1 - \tan\theta_2) = d\tan\theta_2$$

$$D = d\tan\theta_2 / \tan\theta_1 - \tan\theta_2$$

Double plane method:



Double plane method

In the Method to find the horizontal distance between the instrument station and the object, two instrument stations are chosen which are not in line with the object. Hence the vertical angles observed from these instrument stations are in two vertical planes for different elevation .To the vertical angles horizontal angles are also observed at the two instrument stations .The lines joining the instrument stations and the object will from a triangle in plane. The horizontal distances are completed from the solution of Triangles as give.

Let A and B be the two instrument stations not in the line with the object b.

A and b the staff reading observed on the B.M.

θ_1 and θ_2 be the horizontal angle.

α_1 and α_2 be the vertical angle.

d be the horizontal distance between the two instrument stations.

$AB = d$

$AP' = d1$, $BP' = d$

$AP'B = 180°(\angle p'AB + \angle P'BA)$

$= 180° - (\theta_1 + \theta_2) = \theta_3$

Using sine rule we can write,

$AB/\sin\angle AP'B = AP'/\sin\angle p'BA = BP'/\sin\angle P'AB$

$d/\sin\theta_3 = d1/\sin\theta_2 = d2/\sin\theta_1$

$d1 = d1/\sin\theta_3 \times \sin\theta_2$

$d2 = d1/\sin\theta_3 \times \sin\theta_1$

`

in Δ PCD,

PD=V1, CD=AP' =d1

PD/CD=V1/d1=tanα1

V1=d1 tanα1

R.L of P= R.L of BM + a + V1

In ΔPEF,

PF= V2, EF=BP' =d2

PE/ EF = V2/d2 = tan α2

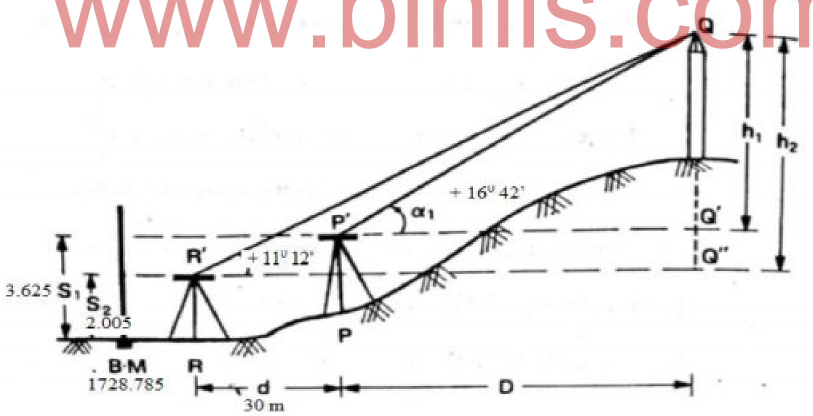
V2 = d2 tanα2

R.L of P = R.L of BM + b + V2.

Exercise: 1

. Determine the R.L of the top of the Temple from the following observation

Instrument at	Vertical angle	Reading on B.M	Remarks
A	+ 16^0 42'	3.625	R.L of B.M = 1728.785
B	+ 11^0 12'	2.005	Distance AB = 30 m



Calculation of R.L

R.L of Instrument Axis, at A = 1728.785 + 3.625=1732.41 m

R.L Instrument Axis, at B = 1728.785 +2.005=1730.79 m

Calculation of S

$S = S_1 - S_2$ = 3.625-2.005=1.62m

Calculation of D

$$D = \frac{d \tan \alpha_2 - S}{\tan \alpha_1 - \tan \alpha_2} = \frac{(30X \tan 11^0 12') - (1.62)}{\tan 16^0 42' - \tan 11^0 12' \alpha_2} = 42.35 \text{ m}$$

Calculation of h_1 and h_2

$h_1 = D\tan \alpha_1$ -------------(1)

$= 42.35 \tan 16^0 42' = 12.71$ m

$h_2 = (D+d) \tan \alpha_2$ -------(2)

$= (42.35+30) \tan 11^0 12'$

$h_2 = 14.32$ m

Calculation of R.L of Q

R.L of Q= R.L of Instrument axis, at $A_1 + h_1$

$= 1732.41+12.71=1745.12$m

Check

R.L of Q = R.L of Instrument axis, at $B_1 + h_2 = 1730.79+14.32= 1745.11$ m

Result: R.L of top of the temple = 1745.12m

Exercise: 2

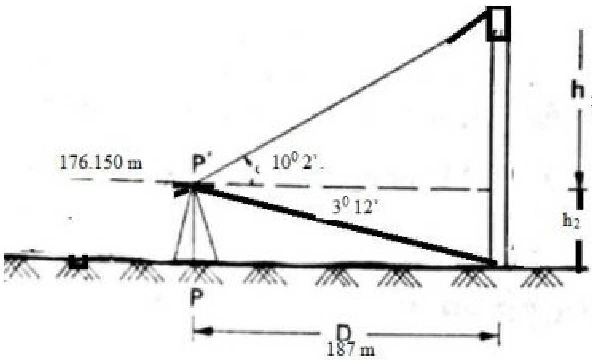
A transit was set up at a distance of 187 m from a temple. The angle of depression to the temple was 3^0 12' and the angle of elevation to its top was 10^0 2'. The elevation of the instrument axis 176.150 m. Find the height of the temple and the elevation of top.

Given data

Distance , D = 187 m

Angle of elevation, $\alpha_1 = 10^0$ 2'

Angle of depression, $\alpha_1 = 3^0$ 12'



To find: Height of the temple and R.L of temple

Solution

Height of temple above line of sight

$h_1 = D\tan\alpha_1 \quad = 187\tan 10^0\, 2' = 33.08m$

Height of temple below line of sight

$h_2 = D\tan\alpha_2 \quad = 187\tan 3^0\, 12' = 10.45$ m

Height of the temple $= h = h_{1+} h_2$

$= 33.08+10.45= 43.53$ m

Calculation of R.L.

R.L of top of temple= R.L of Instrument axis + h_1

= 176.150+33.08 = 209.23m

Result:

Height of temple = 43.53 m

R.L of top of temple = 209.23m

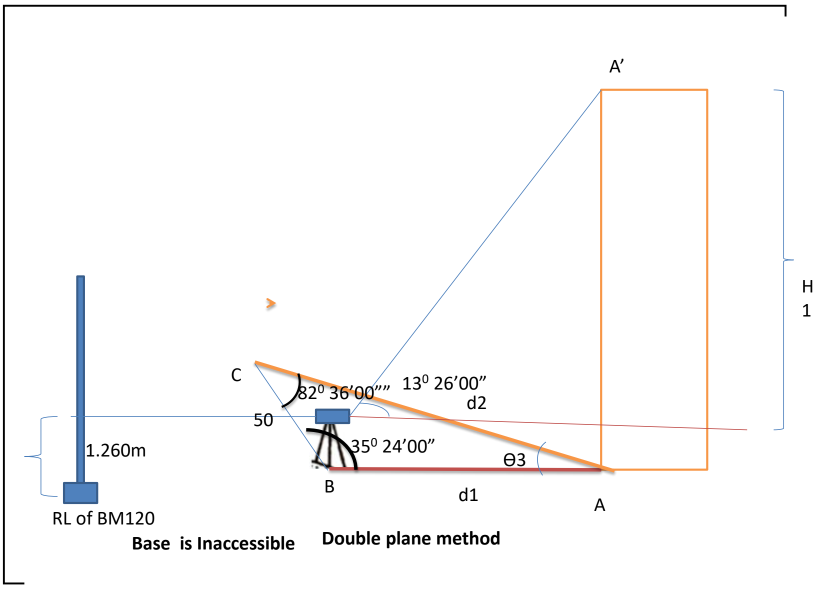
Exercise: 3

In order to determine the R.L of the top of an observation tower A. the following theodolite observation s were made at two stations B and C at a horizontal distance 120 m apart/

Inst at	**Sight to**	**Left face horizontal angle**	**Vertical angle**	**Staff readings**
	B.M	-	0^0 0'00"	1.260m
B	A	360^0 00'00"	13^0 26'00"	
	C	35^0 24'00"		
C	B	360^0 00'00"		
	A	82^0 36'00"		

`

Determine the R.L of A if the R.L of B.M = 152.260m. Instrument station B and C mare not in the same vertical plane



Solution:

In triangle ABC

BC= 120m

∟B= 35^0 24'00"

L C= 82^0 36'00"

L A= (180^0 00'00"-(35^0 24'00"+ 82^0 36'00")

LA = 62^0 00'

Applying sine rule

$$AB = \frac{BC}{\sin A} \times \sin C$$

$$= \frac{120}{\sin 62^0\ 00'} \times \sin 82^0\ 36'00" = 134.776\text{m}$$

h = Height of A above line of collimation at B = D tan α_1

= 134.776 tan13^0 26'00" = 32.191m

R.L of top of tower = R.L of B.M +h+S_1

= 152.260+32.191+1.260

= 185.711m

Result:

R.L of Tower A = 185.711m

`

Exercise: 4

Calculate the RL of the top of the chimney from the following observation

Instrument at	Sight to	Vertical angle	Staff reading on A.B.M
A	B.M	$00^0\ 00'00''$	2.500
	Top of chimney	$30^0\ 18'00''$	
B	B.M	$00^0\ 00'00''$	0.550
	Top of chimney	$29^0\ 12'00''$	

Horizontal distance between A and B is 50 m. Horizontal angle at A is $60^0\ 48'00''$ and Horizontal angle at B is $50^0\ 36'00''$. Take RL of BM as 120.00m

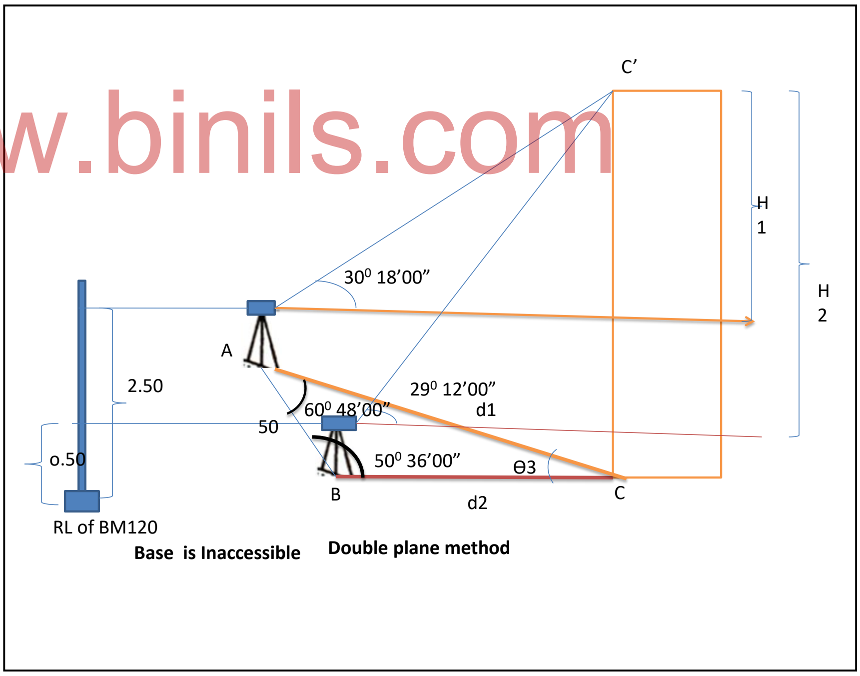
Given data

A and B are the instrument station and P is the top of the chimney

Horizontal distance between A and B is d= 50 m.

Horizontal angle at A $\theta_{1=}$ $60^0\ 48'00''$

Horizontal angle at B
$\theta_{2=}$ $50^0\ 36'00''$



Angle of elevation at A
$\alpha_1 = 30^0\ 18'00''$

Angle of elevation at B
$\alpha_{2=}$ $29^0\ 12'00''$

Staff reading on B.M from the instrument station A S_1 =2.500

Staff reading on B.M from the instrument station B S_2 = 0.550

From the Triangle ABP

LP= ($180^0\ 00'00''$-LA +LB)

= ($180^0\ 00'00''$- ($60^0\ 48'00''$ +$50^0\ 36'00''$))

θ_3= $68^0\ 36'00$

`

Using sin rule

$$\frac{AP}{\sin \theta_2} = \frac{BP}{\sin \theta_1} = \frac{AB}{\sin \theta_3}$$

$$AP = D_1 = \frac{AB}{\sin \theta_3} \times \sin \theta_2$$

$$= \frac{50}{\sin 68^0\, 36'00''} \times \sin 50^0\, 36'00''$$

$$= \frac{38.637}{0.931} = 41.50 \text{ m}$$

$D_1 = 41.50$ m

$$BP = D_2 = \left(\frac{AB}{\sin \theta_3} \times \sin \theta_1\right)$$

$$= \frac{50}{\sin 68^0\, 36'00''} \times 60^0\, 48'00''$$

$$= \frac{43.646}{0.931} = 46.88 \text{ m}$$

$D_2 = 46.88$ m

R.L of instrument axis at A = R.L of B.M + S_1

R.L of instrument axis at A = 120+ 2.5= 122.50

R.L of instrument axis at A== 122.50

`

R.L of instrument axis at B= R.L of B.M + S_2

R.L of instrument axis at B = 120+ 0.55= 120.55

R.L of instrument axis at B== 120.55

Height of top of chimney above the instrument axis at A = $h_{1=}$ D_1 tan α_1

$h_{1=}$41.50 tan30^0 18'00"

$h_{1=}$ 24.25m

Height of top of chimney above the instrument axis at B = $h_{2=}$ D_2 tan α_1

$h_{2=}$46.88 tan29^0 12'00"

$h_{2=}$ 26.20m

R.L of top of Chimney = R.L of Instrument axis at A + h_1

= 122.50+**24.25= 146.75m**

Check:

R.L of top of Chimney = R.L of Instrument axis at B + h_2

= 120.55+26.20m

= 146.75m

Result: R.L of top of chimney = **146.75m**

Exercise: 5

To determine the elevation of the top of the aerial pole the following observation were made(October-04)

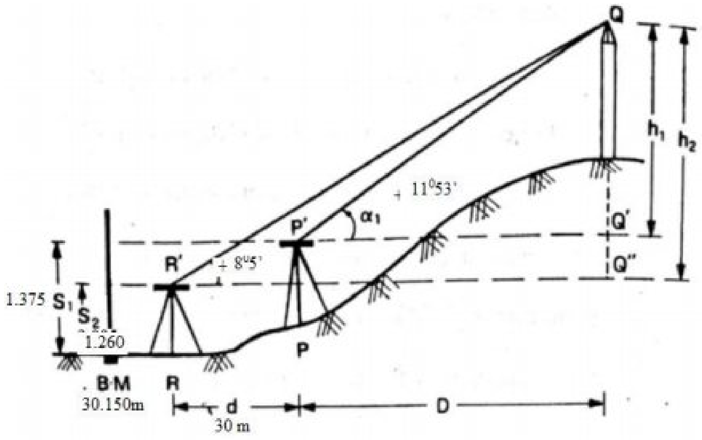
Station A and B are the top of the aerial pole are in the same vertical plane

Find the elevation of the top of the aerial pole if the distance between A and B was 30m

Instrument station	**Reading on B.M**	**Angle of elevation**	**remarks**
A	1.375	$11^0 53'$	R.L of B.M.=30.150m
B	1.260	$8^0 5'$	

`

Given data



R.L. of B.M = 30.150m

Staff readings on B.M from the instrument station A, $S_1 = 1.375$m

Staff readings on B.M from the instrument station B, $S_2 = 1.260$m

Angle of elevation at instrument station A, $\alpha_1 = 11^0 53'$

Angle of elevation at instrument station B, $\alpha_2 = 8^0 5'$

Horizontal distance between tow instrument station A and B is d= 30 m.

Note: Station A, B and the top of the axial pole (Q) are in the same vertical plane

Solution (single Plane method)

Difference between two staff readings $S = S_1 - S_2 = 1.375 - 1.260 = 0.115$m

Distance of aerial pole Q, from the instrument station A,

$$D = \frac{d \tan \alpha_2 - S}{\tan \alpha_1 - \tan \alpha_2} = \frac{(30X \tan 8^0 5') - (0.115)}{\tan 11^0 53' - \tan 8^0 5'}$$

$$= \frac{4.261 - 0.115}{0.210 - 0.142}$$

$= \mathbf{D=60.97\ m}$

From Δle A'QQ',

$h_1 = D \tan \alpha_1$

$= 60.97 \tan 11^0 53'$

$h_1 = 12.83$ m

`

From Δle B'QQ"

$h_2 \quad = (D+d)\tan \alpha_2$

$\quad = (60.97+ 30) \tan 8^0 5'$

$h_2 \quad = 12.92$ m

R.L of top of aerial pole = R.L. of B.M. + staff reading on B.M from instrument station A + h_1

$= \text{R.L. of B.M.} + S_1 + h_1$

$= 30.150+1.375+12.83$

= 44.355 m

Check:

R.L of top of aerial pole = R.L. of B.M. + staff reading on B.M from instrument station B + h_2

$= \text{R.L. of B.M.} + S_2 + h_2$

$= 30.150+1.260+$**12.92**

= 44.33 m

Result: R.L. top of aerial pole= 44.355 m

===

Exercise: 6

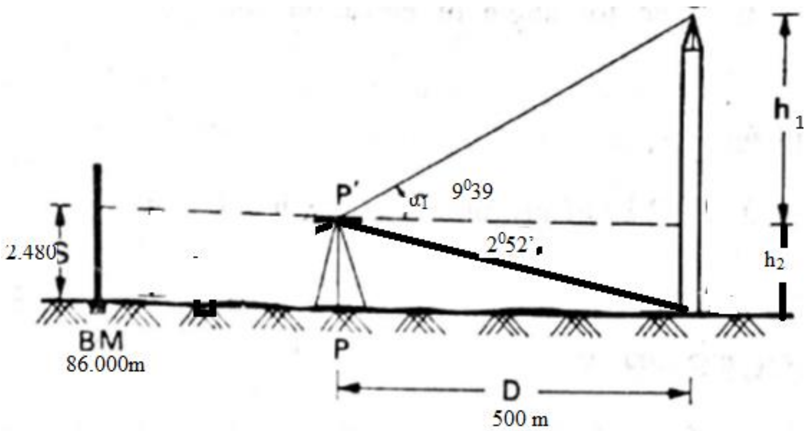
A theodolite was setup at a distance of 500 m from a tower and the angle of elevation to the top was $9^0 39'$, while the angle of depression to the foot of the tower was $2^0 52'$. The staff reading on the bench mark of R.L. 86.000m was 2.480. What is the height of the tower and R.L. of its top and its foot. (October-2006)

Given data

Horizontal Distance between the tower and the instrument , D = 500 m

Angle of elevation to the top of the tower, $\alpha_1 = 9^0 39'$

Angle of depression to the foot of the tower , $\alpha_2 = 2^0 52'$



Staff reading on B.M.= 2.480

R.L. of B.M= 86.000m

To find

1. Height of the tower
2. R.L of top of tower
3. R.L. of foot of tower

Solution

Height of tower above line of sight

h_1 = Dtan α_1 = 500tan9^{0}39' = 85.018m

Height of temple below line of sight

h_2 = Dtan α_2 = 500tan2^{0}52' = 25.037 m

Height of the tower h = h_1+ h_2

= 85.018+25.037

Height of the tower= 110.055 m

Calculation of R.L.

R.L of line of collimation=R.L of B.M + staff reading on B.M

= 86.000m+2.480

=88.480m

R.L of top of tower= R.L of line of collimation + h_1

= 88.480 +85.018

R.L. of top of tower = 173.498 m

R.L of foot of tower= R.L of line of collimation - h_2

= 88.480 - 25.037 =

R.L. of foot of tower = 63.443 m

Result:

1. **Height of tower = 110.055 m**
2. **R.L of top of tower = 173.498 m**

`

<u>**Questions bank**</u>

A. One-mark question

1) Define trigonometric levelling
2) Write the different methods of determining the elevation of an object
3) What do you mean base accessible
4) What do you mean base in accessible
5) What are the methods available for determining the elevation of a point, when the elevation of a point, when the base is inaccessible?
6) What are the Two heads. Trigonometrical leveling discussed.

B. Three marks questions

1. Define the objects trigonometric levelling
2. Briefly explain the procedure to determine the elevation of the top of tower whose base is accessible
3. Differentiate between single plane and double plane method

C. **Ten marks questions**

1. Explain single plane method
2. Explain double plane method

3. An instrument was set up at a point 200m away from a transmission tower. The angle of elevation to the top of the tower was 30°42'. Where the angle of depression was 2°30'. Calculate the total height of the transmission tower (April1992)
4. Determine the R.L. of top of the top of the flag from the following observation:

Inst. Station	Reading on BM in m	R.L of B.M in m	Vertical angle
A	1.57	148.92	+21°52'
B	1.26	148.92	+21°00'

5. Distance between A and B is 60m and A and B are not in the same plane with
6. Determine the R.L. of top of the top of the chimney from the following observation:

Inst. Station	Reading on BM in m	R.L of B.M in m	Vertical angle
A	1.965	435.065	+10°48'

`

B	2.005	435.065	+10°00'

7. Distance between A and B is 60m and A and B are not in the same plane with top f chimney . Horizontal angle at A between B and top of chimney is 68°18'. Horizontal angle between A and top of chimney at B is 52°28'.
8. To determine the elevation of top of the flag mast over a building the following observations were made in a single plane.

Inst.at	Reading BM	Vertical angle	R.L of BM
A	1.085	10°48'	R.L of BM is 150.00 m
B	1.265	7°12'	AB=50m

Find the evaluation:

9. Determine the R.L of the tpo a chimney from the following observations.

Inst.at	Reading on BM	Vertical angle	R.L of BM
A	1.020	12°48'	450.00
B	1.020	9°32'	

Stations A and B are in line with top of the chimney, and they are speed at a distance of 70 m.

3.2 Remote sensing, Photogrammetric surveying and Hydrographic surveying

Remote sensing – Definition – Basic Process – Methods of remote sensing – Applications - Photogrammetric Surveying – Definition – Terrestrial and Aerial photographs – Applications - Hydrographic surveying – Definition- Uses – Sounding: Definition, Purpose, Instruments needed – Steps in hydrographic surveying.

3.2.1 Remote Sensing

3.2.1.1 Introduction

Humans apply remote sensing in their day-to-day activity, through vision, hearing and sense of smell. Reading the newspaper, watching cars driving in front of you are all remote sensing activities. Most sensing devices record information about an object by measuring an object's transmission of electromagnetic energy from reflecting and radiating surfaces. It finds extensive applications in civil engineering including watershed studies, hydrological states and fluxes simulation, hydrological modelling, disaster management services such as flood and drought warning and monitoring, damage assessment in case of natural calamities, environmental monitoring, urban planning etc.

Family members are enjoying the televison programme with the help of Remote sensing organs

3.2.1.2 Definition

There are many possible definitions about what Remote Sensing actually is.

Here's a collection of quotes.

F.F. Sabins in his book "Remote sensing: principles and interpretation" defines it as follows:

"Remote Sensing is the science of acquiring, processing and interpreting images that record the interaction between electromagnetic energy and matter."

Lillesand and Kiefer in their book "Remote Sensing and Image Interpretation" even define it as an art:

"Remote Sensing is the science and art of obtaining information about an object, area, or phenomenon through the analysis of data acquired by a device that is not in contact with the object, area, or phenomenon under investigation."

Charles Elachi in "Introduction to the Physics and Techniques of Remote Sensing":

"Remote Sensing is defined as the acquisition of information about an object without being in physical contact with it."

3.2.1.3 Basic Process

Concept of Remote Sensing

Remote sensing is the most prominent technique of collecting information from the distance. This is done by sensing and recording reflected or emitted energy and processing, analysing and applying that information

The Remote Sensing is basically a multi-disciplinary science which includes a combination of various disciplines such as optics, spectroscopy, photography, computer, electronics and telecommunication, satellite launching etc. All these technologies are integrated to act as one complete system in itself, known as Remote Sensing System.

Table: 3.2.1 Segment of remote sensing

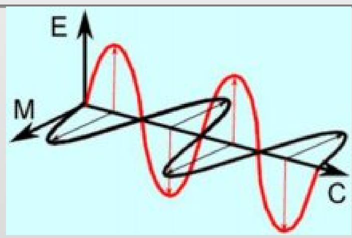
Sensor	Platform
A device to detect this reflected or emitted electro-magnetic radiation from an object is called a "sensor" (e.g., cameras and scanners	A vehicle used to carry the sensor is called a "platform" (e.g., aircrafts and satellites)

Figure 1: Illustration of the IRS-P6 spacecraft (image credit: ISRO)

Principles of Remote Sensing

Different objects reflect or emit different amounts of energy in different bands of the electromagnetic spectrum. The amount of energy reflected or emitted depends on the properties of both the material and the incident energy (angle of incidence, intensity and wavelength). Detection and discrimination of objects or surface features is done through the uniqueness of the reflected or emitted electromagnetic radiation from the object.

- ✓ Amount of radiation emitted from an object depends on its temperature



Electromagnetic energy is emitted in waves

Elements in remote sensing: There are eight of elements in a Remote Sensing process, and each of them is important for successful operation.

Eight elements in remote sensing are the following.

A. Emission of electromagnetic radiation: The Sun or an EMR (Electro Magnetic Radiation) source located on the platform

B. Transmission of energy from the source to the object: Absorption and scattering of the EMR while transmission

C. Interaction of EMR with the object and subsequent reflection and emission

D. Transmission of energy from the object to the sensor

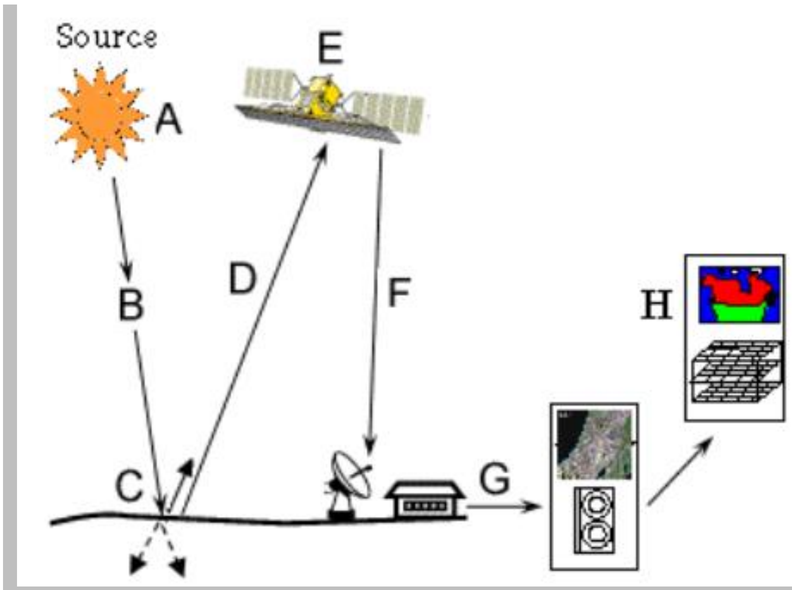
E. Recording of energy by the sensor: Photographic or non-photographic sensors

`

F. Transmission of the recorded information to the ground station

G. Processing of the data into digital or hard copy image

H. Analysis of data is shown in fig.



Stages in remote sensing

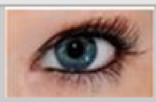
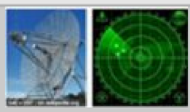
3.2.1.4 Methods of remote sensing

Depending on the source of electromagnetic energy, remote sensing can be classified a passive and active remote sensing

- Passive remote sensing:

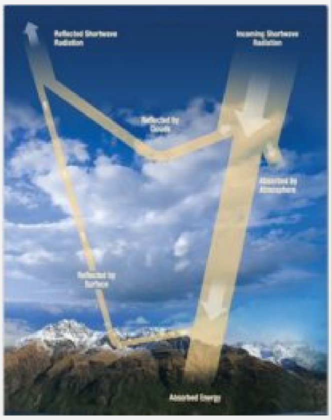
In the case of passive remote sensing, source of energy is that naturally available such as the Sun. Most of the remote sensing systems work in passive mode using solar energy as the source of EMR. Solar energy reflected by the targets at specific wavelength bands are recorded using sensors on board air-borne or space borne platforms. In order to ensure ample signal strength received at the sensor, wavelength / energy bands capable of traversing through the atmosphere, without significant loss through atmospheric interactions, are generally used in remote sensing

Example

Passive RS	Active RS
	
Human eye	Radar

Facts

Incoming ultraviolet, visible, and a limited portion of infrared energy (together sometimes called "shortwave radiation") from the Sun drive the Earth's climate system. Some of this incoming radiation is reflected off clouds, some is absorbed by the atmosphere, and some passes through to the Earth's surface.



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Active remote sensing

In the case of active remote sensing, energy is generated and sent from the remote sensing platform towards the targets. The energy reflected back from the targets are recorded using sensors on board the remote sensing platform. Most of the microwave remote sensing is done through active remote sensing.

A simple comparison

Passive remote sensing is similar to taking a picture with an ordinary camera

The active remote sensing is analogous to taking a picture with camera having built-in flash

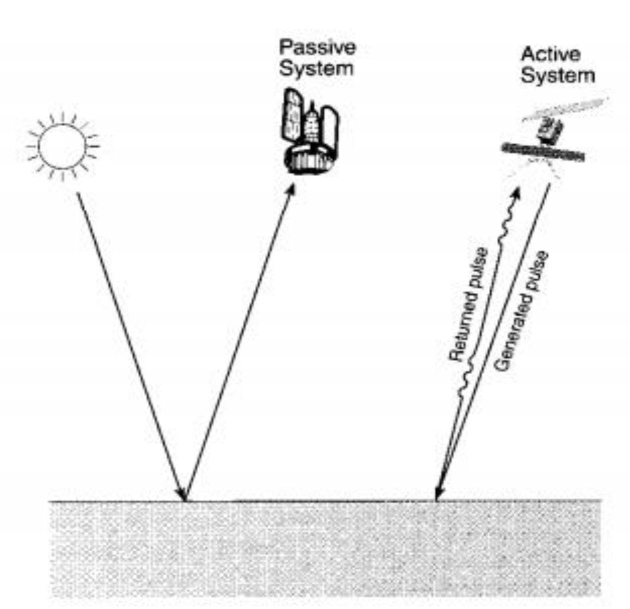


Fig. Schematic representation of passive and active remote sensing

3.2.1.5 Advantages and Disadvantages of Remote Sensing

Advantages of remote sensing are:

a) Provides data of large areas

b) Provides data of very remote and inaccessible regions

c) Able to obtain imagery of any area over a continuous period of time through which the any anthropogenic or natural changes in the landscape can be analysed

d) Relatively inexpensive when compared to employing a team of surveyors

e) Easy and rapid collection of data

f) Rapid production of maps for interpretation

Disadvantages of remote sensing are:

a) The interpretation of imagery requires a certain skill level

b) Needs cross verification with ground (field) survey data

c) Data from multiple sources may create confusion

d) Objects can be misclassified or confused

e) Distortions may occur in an image due to the relative motion of sensor and source.

Name of the satellite	
Indian	Foreign
RESOURCESAT-1	LANDSAT
(IRS-P6)	ERS-1/2
EDUSAT	IKONOS
HAMSAT	QUICKBIRD
CARTOSAT-1	SPOT

National Remote Sensing Centre has the campus at Balanagar,Shadnagar and Jeedimetla in Hyderabad . The Earth Station complex for receiving satellite data is at Shadnagar, about 60 km from Hyderabad. The aerial data acquisition and processing facilities are located at Hyderabad airport and at Jeedimetla in Hyderabad. Regional Remote Sensing Centres are located at Kolkata, Jodhpur, Nagpur and Bengaluru

For more Information visthttp://www.nrsc.gov.in/

Earth Observation Programme In India

National Remote Sensing Centre (NRSC), is a full-fledged centres of ISRO. NRSC was functioning as an autonomous body called National Remote Sensing Agency (NRSA) under Department of Space (DOS) till August, 2008. The Centre is responsible for remote sensing satellite data acquisition and processing, data dissemination, aerial remote sensing and decision support for disaster management.

The Indian Remote Sensing satellite (IRS) programme commissioned with the launch of IRS-1A in 1988 presently includes twelve satellites that continue to provide imageries in a variety of spatial resolutions from better than one metre ranging up to 500 metres. The data is used for several applications covering agriculture, water resources, urban development, mineral prospecting, environment, forestry, drought and flood forecasting, ocean resources and disaster management.

The Indian Remote Sensing satellite system has one of the largest constellations of remote sensing satellites in operation in the world today. The IRS series of satellites provide data in a variety of spatial, spectral and temporal resolutions which are effectively used for resource management purposes.

Starting with IRS-1A in 1988, ISRO has launched many operational remote sensing satellites. Today, India has one of the largest constellations of remote sensing satellites in operation. Currently, eleven operational satellites are in orbit – RESOURCESAT-1 and 2, CARTOSAT-1, 2, 2A, 2B, RISAT-1 and 2, OCEANSAT-2, Megha-Tropiques and SARAL. Varieties of instruments have been flown onboard these satellites to provide necessary data in a diversified spatial, spectral and temporal resolutions to cater to different user requirements in the country and for global usage.

Foreign Satellite Data Distribution

Part from supplying the data for the Indian Remote Sensing Satellites, they also supplement the data requirements of users through other Remote Sensing Satellites. NRSC acquires and distributes data from a number of foreign satellites.NRSC has been acquiring microwave Synthetic Aperture Radar (SAR) data from the ERS-1/2 satellites from 1992 onwards. Products can be supplied from archived data or through programming for future acquisitions. ERS data have been acquired against user requests from 1991 and Tandem data from ERS-1 and ERS-2 during 1995-96.

3.2.1.5 Applications:

A remote sensing application is a software application that processes remote sensing data. Remote sensing applications are similar to graphics software, but they enable generating geographic information from satellite and airborne sensor data.

Remote sensing is being used to collect the information about agriculture, forestry, geography, and archeology, weather and climate, marine environment, hydrology, water resources management and assessment, engineering, etc.

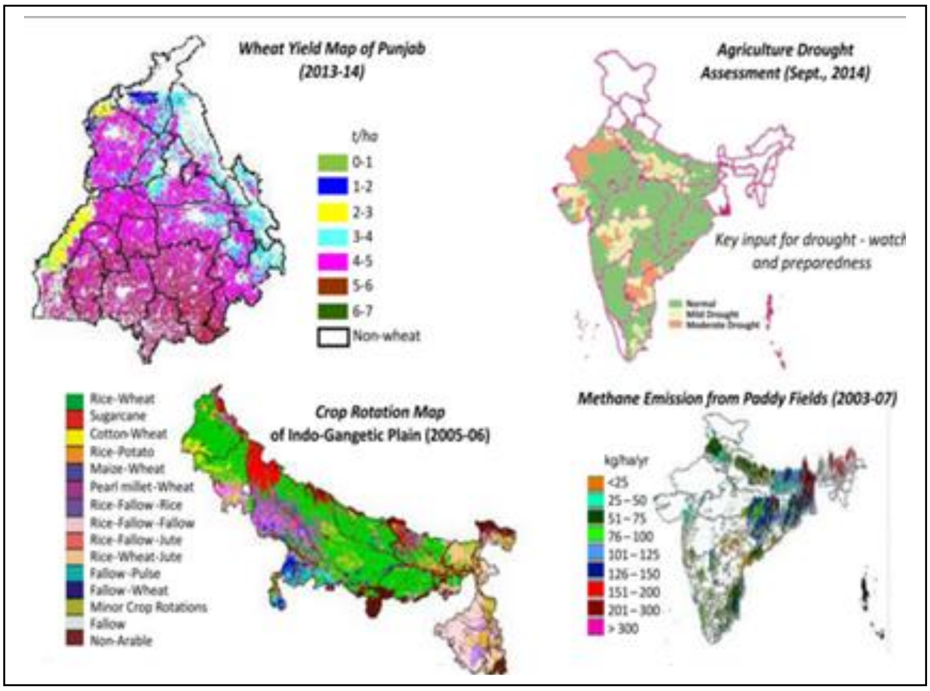
General application

It has vast applications in exploration of natural resources, analysis of land use and land cover, information about environments, natural hazard studies such as earthquake, land slide, land subsidence, flood, etc

Specific allocations in Civil Engineering

1. Construction material (sand and gravel) inventories within economic hauling distance in site investigations for dams. Reservoir, bridge, pipeline, crossing of rivers, airstrips, staging and docking facility, etc.
2. utility for mapping and assessing soil erosion conditions
3. Watershed management like drainage network mapping and assessment by remote sensing technique.
4. It can be of great assistance in environment impact assessment, especially where the areas to be studied are large.
5. It can be of great assistance in environment impact assessment, especially where the areas to be studied are large.
6. For detection and monitoring of the water pollution, remote sensing prove useful.
7. Remote sensing is applicable in acquiring information regarding offshore engineering activities, fisheries surveillance, ocean features, coastal regions and storm forecast operations.
8. Remote sensing is very useful in identifying Potential Fishing Zones (PFZ). This data is very useful for fishermen because they came to know likely occurrence of fish shoals which helps them for getting more catch.
9. The major agricultural applications of remote sensing include the following:
   - Vegetation
   - crop type classification
   - crop condition assessment (crop monitoring, damage assessment)
   - crop yield estimation
10. The major soil applications of remote sensing include the following:
    - mapping of soil characteristics

- mapping of soil type
- soil erosion
- soil moisture
- mapping of soil management practices
- compliance monitoring (farming practices)



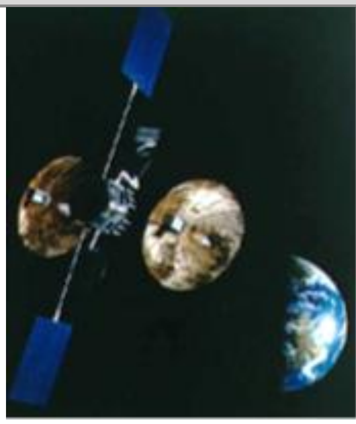
11. In civil engineering projects, RS and GIS techniques can become potential and indispensable tools. Various civil engineering application areas include regional planning and site investigation, terrain mapping and analysis, water resources engineering, town planning and urban infrastructure development, transportation network analysis, landslide analysis, et

Remote Sensing Platforms


Ground based


Aircraft


Space shuttle


Satellite

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3.2.2 Photogrammetric Surveying

3.2.2.1 Introduction:

Photogrammetry is the practice of determining the geometric properties of objects from photographic images. It helps in the making of precise measurements from photographs for the preparation map. It is also called science of using aerial photography to obtain measurement of natural and man-made features on the earth. In this method too, like remote sensing objects are measured without being touched

3.2.2.1 What is Photogrammetry

Process of making surveys and maps using photographs.

The science of deducing the physical dimensions of objects from the measurement on images (usually photographs) of the objects

The making of maps by photographs from the air using reference point of known level and position which can be identified on the photographs

The art and science of obtaining reliable quantitative and qualitative measurements through the use of photographs

Is the Art, Science and technology of obtaining reliable information about physical and the environment through process of recording and interpreting photographs images.

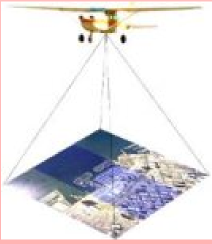
3.2.2.3 Uses of Photogrammetry

- Photogram try is used in preparing topographic maps, advantages include.
- Speed of coverage of an area
- Relatively low cost
- Ease of obtaining topographic details
- Reduce likely wood of omitting date due to tremendous amount of detail shown in photographs.
- It is used in land surveying to compute co-ordinates of section corner, body corner or pts. of Evidence that help locate these corner
- Photogram try is used to make shore line in hydrographic surveying.
- It is used to preside ground co-ordinates of pts in control surveying.
- Used to develop maps and cross-section route (road or railways) and survey.

1851: The french officer Aime Laussedat develops the first photogrammetrical devices and methods. He is seen as the initiator of

photogrammetry.

1858-Gasper Felix Tournachon"Nadar" took the first aerial photograph from a captive ballon from an altitude of 1,200 feet over Paris.



1980ies: Due to improvements in computer hardware and software, digital photogrammetry is gaining more and more importance.

- Photogram try is also used successfully in many non-engineering fields' e.g, geology, archeology, forestry, agriculture, conservation, military intelligence, traffic management accident investigation.
- It is particularly suitable for inaccessible regions, forbidden properties (restricted arrears) etc.

3.2.2.4 Types of Photographic Survey

Photogrammetry may be divided into two classes depending upon the camera position at the time of photography:

1. Terrestrial or ground photogrammetery.
2. Aerial photogrammetry

1. Terrestrial or ground photogrammetery.

Fig 3.2Phototheodolit

When the photographs are taken with phototheodolit(fig.3.2.) Having camera station on the ground and axis of the camera horizontal or nearly horizontal, the photographs are called the terrestrial photographs. These photographs present familiar view of the front view or elevation. They are generally used for the survey structures and architectural or archaeological monuments

The terrestrial photographic surveying considered as the further development of plane table surveying.

A photo theodolite is a form of ground camera. It is a combination of ca... is used for taking photographs and measuring the angles which the pla... with base line.

Terrestrialphotographics – Taken with ground based cameras

The optical axis of the camera is arrange parallel to the line of collimati... the theodolite allows the determination of the orientation of the camera

2. Aerial Photogrammetry

The photographs taken with an aerial camera having camera station i... camera vertical or nearly vertical are called aerial photographs.

Depending on the angle between the axis of the camera and the vertical, the aerial photographs may be classified as:

1. Vertical photographs	3. Convergent photographs
2. Oblique photographs	4. Trimertrogen photographs

1. **Vertical Aerial Photograph**

These are taken when with the optical axis of the camera is vertical or nearly vertical (Fig) True vertical photograph closely resembles a map. These are utilized for compilation of topographical and engineering survey on various scale A deviation up to 4° is acceptable. This gives the map of the earth detail somehow in same scale.

Characteristics:

(1) The lens axis is perpendicular to the surface of the earth.

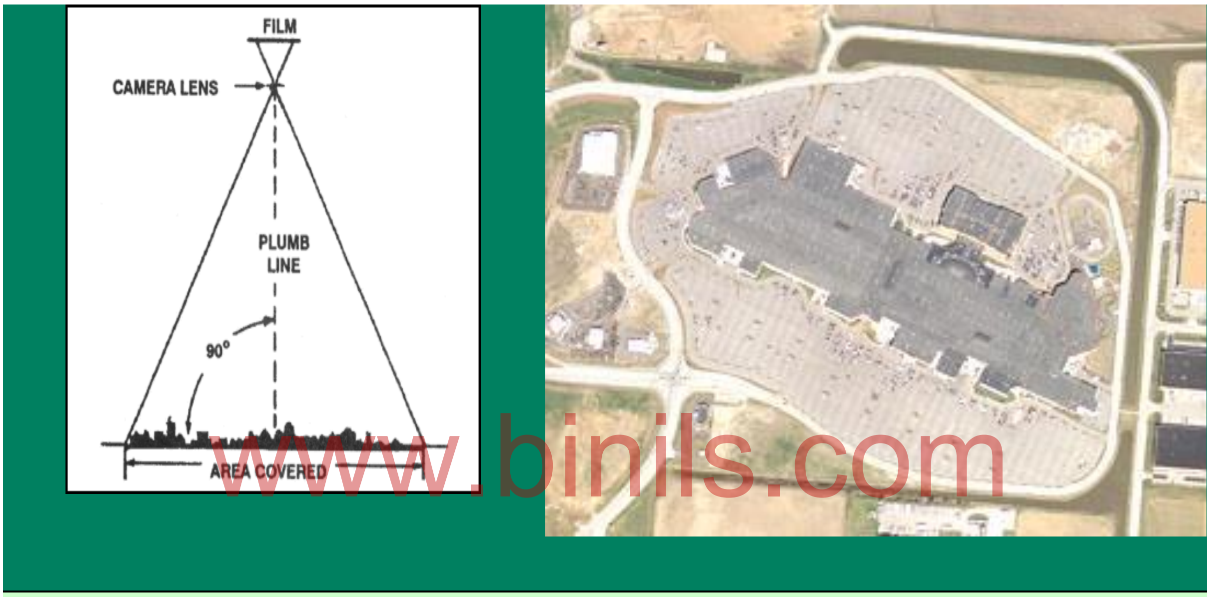
(2) It covers a relatively small area.

(3) The shape of the ground area covered on a single vertical photo closely approximates a square or rectangle.

(4) Being a view from above, it gives an unfamiliar view of the ground.

(5) Distance and directions may approach the accuracy of maps if taken over flat terrain.

(6) Relief is not readily apparent.



Camera angle Section

Plan of view

2.Oblique Aerial Photograph

These are obtained when the optical axis of the camera intentionally inclined from vertical (F It cover larger area of ground. It oblique photograph which does not show the horizon is known as low- oblique photographs.(Fig) complies reconnaissance map of inaccessible areas.

The aerial photographs which shown the horizon is called high-oblique photograph (Fig) used for military intelligence

An oblique photographic one which has been taken with the optical axis of the camera intentionally tilted from perpendicular position obliquely. The degree of tilt from the perpendicular further classifies oblique photographs into high oblique photograph and low oblique photograph. A high oblique photograph is one which is taken with the optical axis of the camera making an angle >30° with the vertical axis and which shows the apparent horizon on the photograph.

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Low Oblique. This is a photograph taken with the camera inclined about 30° from the vertical

Characteristics:

(1) It covers a relatively small area.

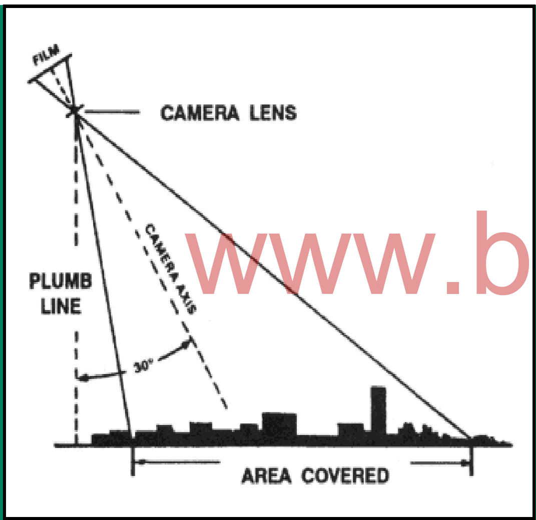
(2) The ground area covered is a trapezoid, although the photo is square or rectangular.

(3) The objects have a more familiar view, comparable to viewing from the top of a high hill or tall building.

(4) No scale is applicable to the entire photograph, and distance cannot be measured. Parallel lines on the ground are not parallel on this photograph; therefore, direction (azimuth) cannot be measured.

(5) Relief is discernible but distorted.

(6) It does not show the horizon.



Camera angle Section

Plan of view

High Oblique. The high oblique is a photograph taken with the camera inclined about 60° from the vertical

Characteristics:

(1) It covers a very large area (not all usable).

(2) The ground area covered is a trapezoid, but the photograph is square or rectangular.

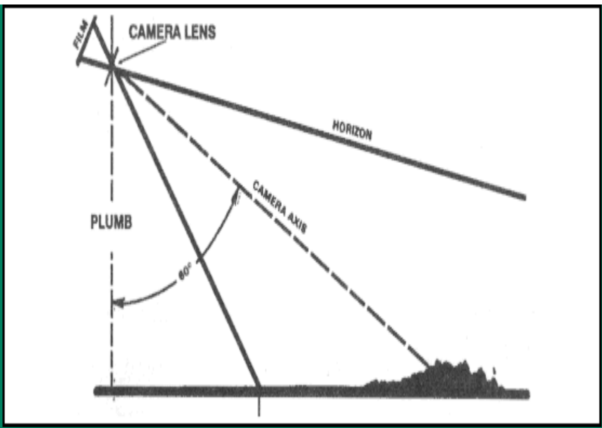
`

(3) The view varies from the very familiar to unfamiliar, depending on the height at which the photograph is taken.

(4) Distances and directions are not measured on this photograph for the same reasons that they are not measured on the low oblique.

(5) Relief may be quite discernible but distorted as in any oblique view. The relief is not apparent in a high altitude, high oblique.

(6) The horizon is always visible.



Camera angle Section

Plan of view

Convergent photographs: These are low-oblique photographs with two cameras exposed simultaneously at successive exposure station, with titled at a fixed inclination form the vertical in the direction of flight. So that the forward exposure of the first station forms stereo pair with the backward exposure of the next station.

Trimertrogen photographs: This types of photographs is a combination of vertical and low-oblique photographs exposed simultaneously from the air station from two cameras

3.2.2.5 Applications

Nature of field	Uses
• Soil mapping	▪ It involves detailed soil survey
• Land use and Land cover mapping	▪ Use of land related human activates
• Forestry application	▪ Identification of tree species, timer movement and assessment of diseases and insects infection

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• Water resource	▪ Monitor the quality, quantity and geographic distribution of water, reservoir site selection , shore line erosion studies
• Urban and regional planning	▪ Population estimation, housing quality studies , parking studies, site selection process
• Wetland mapping	▪ Deduction of areas where the water tables at near or above the land surface
• Wild life ecological	▪ Wildlife habit napping and wildlife censusing
• Archeological	▪ Analyze the remains of historic human antiquities
• Environmental assessment	▪ The adverse environmental effects of human activity can be assessed
• Geographic Information system	▪ Preparation of topographic maps and
Project Idea 	
• Rainfall	Measurement of precipitations is made of representative points distributed over the watershed and each such measurement is multiplied by the total units of the watershed area to which the measurement applied.
• Garbage	Volume of garbage can be measured directly from aerial photograph

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3.2.3 Hydrographic Surveying

3.3.3.1 General

Hydrographic survey crew with accessories

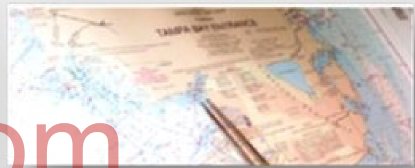
A hydrographic survey, also known as a bathymetric survey, is used to produce a bathymetric contour map, which is a plan of the depth of the sea bed for the primary purpose of safety of navigation and in support of all other marine activities, including economic development, security and defense, scientific research, and environmental protection.

3.2.3.2 Definition:

It is the branch of surveying which deals with measurement of any water body of still or running water such as a lake, harbor, stream or river. Hydrographic surveys are used to define shore line and under water features.

3.2.3.3 Uses:

The most common uses of the hydrographic survey can be listed as



A nautical chart is one of the most fundamental tools available to the mariner. It is a map that depicts the configuration of the shoreline and seafloor. It provides water depths, locations of dangers to navigation, locations and characteristics of aids to navigation, anchorages, and other features.

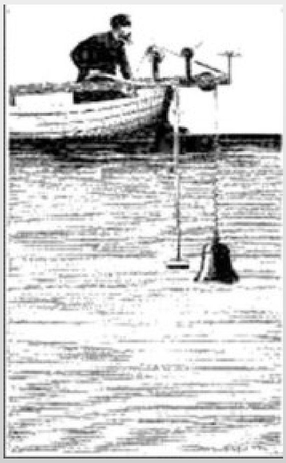
2. Measurement of tides sea coast work E.g. construction of sea defense works, harbors etc.,

3. Determination of bed depth by soundings for location of rocks, sand bars, navigation light.
4. Determination of scour and irregularities of the bed
5. Determination of shore lines
6. Establishment of the mean-sea level
7. Preparation of navigation charts
8. Measurement of discharge of rivers and streams
9. Determination of direction of current to locate sewer fall
10. Providing help in the planning of projects like bridges, dam and reservoirs etc.,
11. Measurement of quantity of water and flow of water in connection of water schemes, Power scheme and flood controls.

3.2.3.4 Soundings:

The measurement of depth below the water surface is called sounding. Sounding is the main operation in hydrographic surveying.

Soundings are analogous to levelling on land, and therefore, the reduced level of any point on the bottom of water is obtained by subtracting the sounding from the mean sea-level. To take the sounding a vertical control is necessary. A horizontal is necessary to locate the sounding.



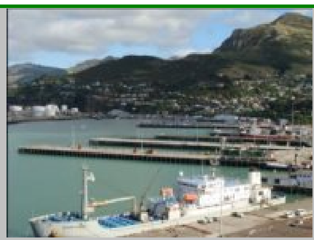
The first measurement of sounding 1826

3.2.3.5 Purpose

Sounding are essential for the following purpose:

1. Preparation of accurate chart for navigation
2. Deamination of the quantity of the material to be dredged
3. Location of the areas from where material to be dredged, and where to be dumped
4. Obtaining information for the design of breakwater, wharves, etc.,

Breakwater, artificial offshore structure protecting a harbour, anchorage, or marina basin from water waves.



Wharves a structure built on the shore of or projecting into a harbor, stream, etc., so that vessels may be moored alongside to load or unload or to lie at rest

3.2.3.6 Instruments needed

The essential equipment and instrument employed for taking the sounding ar

1. Sounding boat
2. Sounding pole or rod
3. Lead line
4. Sounding machine

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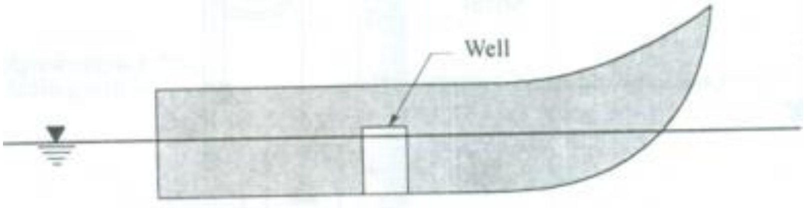
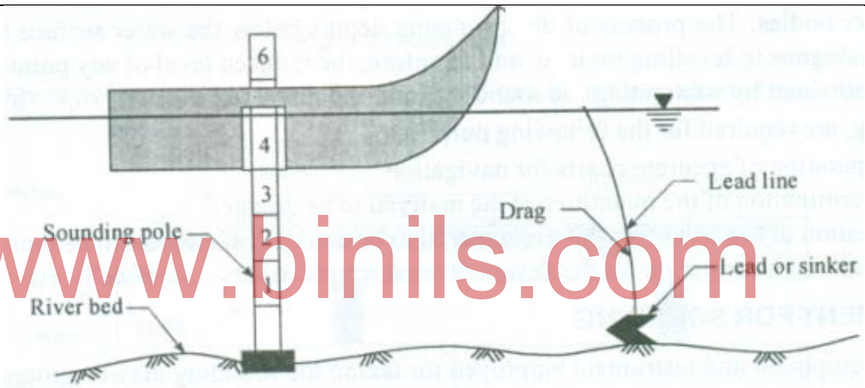
1.Sounding boat:

It is a flat –bottom boat of low draft. These are generally provided with opening, called wells through which soundings are taken. The sounding platform is provided and it extended far enough over the side so that the sounding line or sounding poles to do not strike the boat.

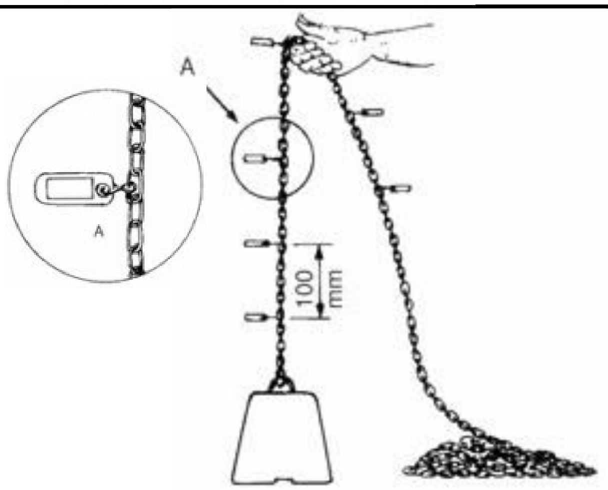


2.Sounding pole or rod:

These are made of strong well-seasoned timber usually 5 to 10cm in diameter and 5 to 8m on length. The sounding rods consist of two or three length screwed together so that unnecessary length may be removed when not in required for sounding in shallow water. A lead shoe of sufficient weight is fitted at the bottom, to keep the rod vertical in flowing water and avoid sinking in mud or sand. The graduation on the rod a marked from bottom upward. Hence the reading on the rod corresponding to the water surface , is directly the depth of wate.



3. A **lead line** is graduated rope of hemp or brass chain attached to the lead or sinker. The mass of the lead generally between 5 to 10kg, depending upon the strength of the current and depth of water. Lead lines are used for depth over 6 meters. A correction is required to be applied to the measured length to get true depth when using the lead line in the deep and swift-flowing water. Due to the drag, the measured length will be greater than the true length



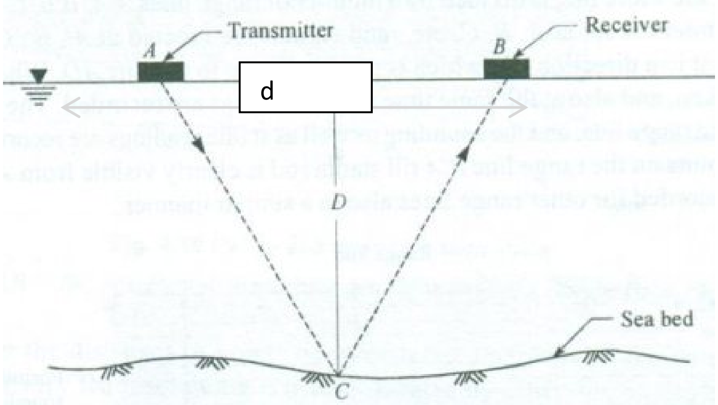
4. Sounding machine/Fathometer(Echo-sounding)

Fathometer

A fathometer is an echo-sounding instrument used to determine the depth indirectly. It works the principles of recording time travel by sound waves. Knowing the time of travel from a point of the surface of water to the bottom of ocean and back and the velocity of sound waves, the depth can be calculated.

Working principle

The sound waves are transmitted from A as shown in fig , and the reflected waves from the bed say C are received at B. The total time of travel of the waves from A to C and C to B is recorded. The depth D an be calculated knowing the velocity of sound and the distance AB.



Advantages of fathometer

1. It gives truly vertical accurate depth
2. It is more sensitivity than lead line
3. It gives a continuous profile by recording the measurement on a drum
4. The sounding can be made with greater speed
5. The rocks underlying soft oils are also recorded
6. The velocity of sound waves varies with the density of water, adjustments can be made to read the depth of water of any type

3.2.3.7 Steps in hydrographic surveying:

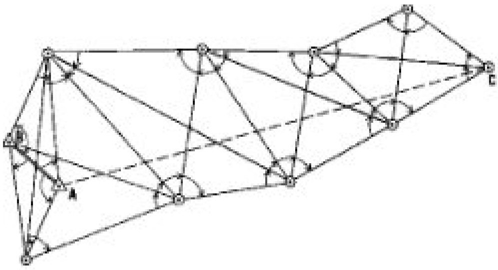
Before locating the details in hydrographic survey, the following preliminary steps are required to be carried out

1. Reconnaissance	2. Establishing horizontal control
3. Establishing vertical control	4. Shore-line survey

1. Reconnaissance: It is the first step in hydrographic survey by careful reconnaissance of the area to be surveyed in order to select the most-expeditious manner of performing the survey, and to plan all operation so that the survey is satisfactorily completed in accordance with the requirements and specifications governing such work. The use of aerial photographs can be considerable help in this preliminary study.

Reconnaissance survey

2. The horizontal control: It is a framework by which land and marine features are in their true relative positions. It usually consists of series of connected lines whose lengths and azimuths are determines accurately. Triangulation and, traverse are most commonly executed to provide horizontal control,. For rough works, a tacheometric traverse or plane –table traverse may be run. However, for more precise control , a theodolite and tape traverse is usually run. For extended survey, where greater precision is required, horizontal control is provided by triangulation. A combined triangulation and traverse system is shown in figure.



A combined Framework system

3. Establishing vertical control: A number of bench marks are established near the shore line at close spacing to serve as vertical control. The bench marks are used for setting and checking the levels of gauge to which the sounding refer to

Levelling for vertical control

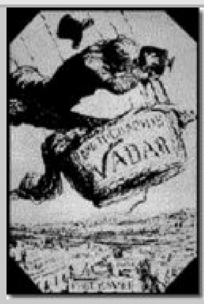
4. Shore-line surveys are conducted for delineation of shore lines, determination of high and low-water lines, and the location of prominent points on the shore to which the horizontal position of soundings may be referred. Normal methods of chain survey and plane-table survey are adopted to survey irregularities in the shoreline. Points which are clearly visible from the water surface, are selected as reference points such as churches, temple. Light houses, wind mills, transmission pylons, etc., and their locations plotted easily.Determination of high-water line may be easily judged roughly from the marks on permanent rocks. For accurate, method of contouring may be adopted which a number of points are established at the time of high tide. The line connecting these points so obtained is the required high-water line.

Shore line

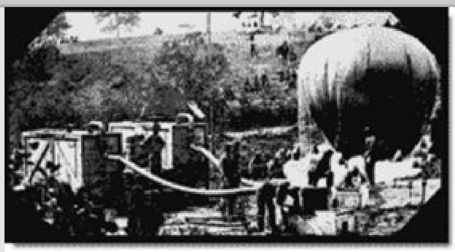
Pioneer user of Remote Sensing


Galileo

1609 - Invention of the telescope by Galileo



1859 - First aerial photographer Gaspard Felix Tournachon, also known as Nadar



1862 - US Army balloon corp

1903 - The Bavarian Pigeon Corps

1909 - Dresden International Photographic Exhibition

1908 - First photos from an airplane(First flight, Wright Bros., Dec. 1903)

1914-1918 - World War

Class Exercise

A. Fill in the blanks.

1. Most sensing devices record information about an object by measuring an object's transmission of ----------------------------------- from reflecting and radiating surfaces.
2. The Remote Sensing is basically a ----------------------- science
3. A device to detect this reflected or emitted electro-magnetic radiation from an object is called a -----------------
4. A vehicle used to carry the sensor is called a ------------------
5. There are ----------------- stages in a Remote Sensing process
6. National Remote Sensing Agency (NRSA) under Department of --------------------
7. Photogram try is used in preparing ---------------------------------------
8. A photo theodolite is a combination of ------------------- and ----------------------------- and is used for taking photographs and measuring the angles
9. A vertical photograph is one which has been taken with optical axis of camera -------- --------------------------------- to the horizontal plane.
10. An oblique photographis one which has been taken with the optical axis of the camera -- from perpendicular position obliquely.
11. A high oblique photograph is one which is taken with the optical axis of the camera making an angle --------------- with the vertical axis
12. -------------------------is the branch of surveying which deals with anybody of still or running water such as a lake
13. Soundings are analogous to ------------------------- on land

Question bank

B. Answer the following questions in one or two sentence/ one mark questions

1. What is remote sensing
2. Can you name all the remote sensing organs of a human being
3. Give an example for human remote sensing organ
4. What is sensor
5. What is platform used in remote sensing
6. What are the two methods of remote sensing
7. What is passive remote sensing
8. What is active remote sensing
9. Give one example for passive type of remote sensing
10. Give one example for active type of remote sensing
11. Abbreviate the term NRSC
12. Name any tow Indian remote sensing satellite
13. Name any two foreign remote sensing satellite
14. What is Photogrammetry
15. What are the two classes photogrammetry depending upon the camera position

16. What is the other name of Terrestrial photogrammetry
17. Name the instrument used in terrestrial photogrammetry
18. Draw camera position of
19. What is Terrestial or Ground Photogrammetry
20. What is aerial photogrammetry
21. What is sounding
22. What are the equipment and instrument employed for taking the sounding
23. What are the steps are required to be carried out in hydrographic surveying.

C. Answer the following questions in short /three marks questions

1. Differentiate between sensor and platform
2. Differentiate between active and passive types of remote sensing
3. Explain the application of remote sending in agriculture sector
4. Mention any three uses of Photogrammetry
5. Write short notes on Terrestrial photogrammetry
6. Write short notes on Aerial photogrammetry
7. Differentiate between terrestrial and aerial photogrammetry
8. What are the different classification of aerial photogrammetry
9. Mention any three application of photogrammetry
10. Write any three uses of hydrographic surveying
11. Mention any three purpose of sounding
12. What are the essential equipment and instrument employed for taking the sounding
13. Explain the salient features of sounding boat
14. Explain the salient features of sounding rod
15. Explain the salient features of lead line
16. Explain the salient features of Fathometer
17. Mention any three advantages of fathometer

D. Answer the following questions in details/Ten marks questions

1. Explain the basic process of remote sensing with sketch in sequential stage
2. Summaries the advantages and disadvantages of remote sensing
3. Describe the various field application of remote sensing with example
4. Enumerate the various uses of Photogrammetry
5. What are the different classification of aerial photographs and explain any three of them
6. Describe the various field application photogrammetry
7. Enumerate the various uses of Hydrographic survey
8. Write short notes on a) Sounding boat b) sounding pole c) Lead line
9. Explain the salient features and working principles of Fathometer
10. What are the different steps involved in hydrographic surveying and explain it?

UNIT-IV

4. CURVES

Introduction – Types of curves – Designation of curves – Elements of simple circular curve – Setting out simple circular curve by: Offsets from long chords, Offsets from tangents, Offsets from chords produced and Rankine's method of deflection angles – Simple problems – Transition curves : Objectives – Vertical curves : Definition and types.

4.1 Introduction:

While aligning roads, railways, canals, etc., the alignment cannot be straight throughout because of various types of obstacles like marshy land, crossing, key points etc. Hence the change of direction from straight reach to another straight is achieved by connecting them with a smooth curve.

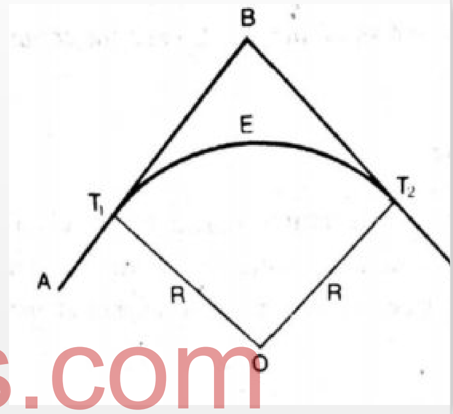


Fig.4.1

4.2 Types of curves:

a) Simple curve
b) Compound curve
c) Reverse curve
d) Transition curve

a. Simple curve:

When two straight AB and BC are connected by a single arc of a circle T1 ,T2, it is known as simple curve. This curve joints the two straight at T1,and T2 tangentially and it has only are center . This curve is used, when the direction of communication route suddenly changes to the right or to the left.

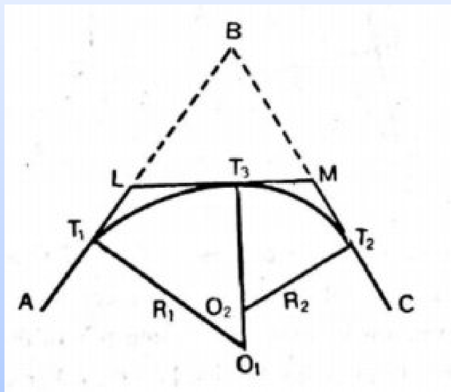


Fig.4.2

b. Compound curve:

When the direction of communication route changes as shown in figure ,a compound curve is used commenting the straights AB,BC and CD. A compound curve consists of two or more simple curve.

c. Reverse curve:

Reverse curve consists of two arcs of same or different radii bending in the opposite direction . This curve connects the straights AB and CD are joined by two circular arcs. T1,T3 and T3,T2 of radii R1 and R2 with centre's at O1 and O2 bending in the opposite direction

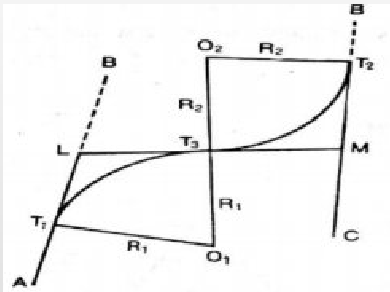


Fig.4.3

4.3 Designation of curve:

A simple curve is designated either by its radius or by its degrees of curvature . In India a curve is designated by the angle. subtended at the center by chord of 30 meters length . This 30 meter length is fixed either on the length of are of on the length of chord , which depends on the departments uses the curve.

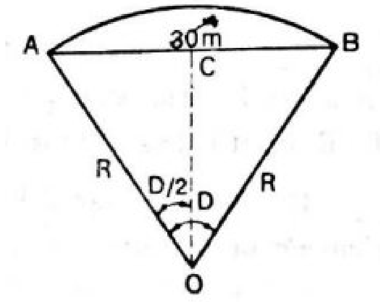


Fig 4.4

Relation between the radius and the degrees of a simple curve

AO=R, AE=15m

∟AOE =D/2

Sin D/2=15/R

R= 15/sin D/2

R=15/D/2 x π/180 = 15x2x180/ D x π = 1718.9/D = 1719/D

Chord Definition

The degree of a curve is defined as the angle subtended at the centre of the curve by a chord of 30 m length.

Let D be the degree of a curve i.e., it is the angle subtended at its centre O by a chord C_1C_2 of 30 m length as shown Figure 4.5. Thus

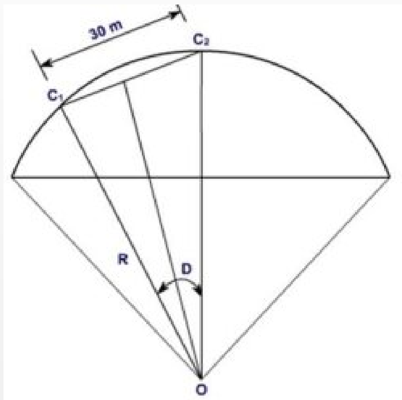


Fig 4.5

$$\sin\frac{D^{\circ}}{2} = \frac{(30/2)}{R}$$, where R is the radius of the circular curve

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$$\therefore R = \frac{15}{\sin\left(\frac{D^\circ}{2}\right)} \quad \text{--------------(1)}$$

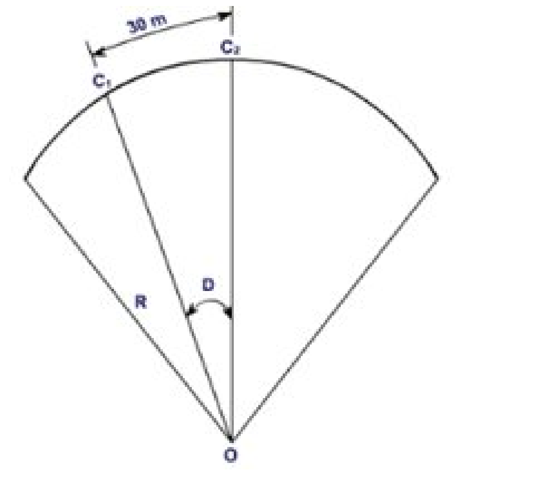


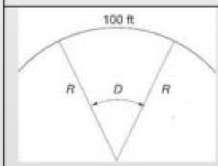
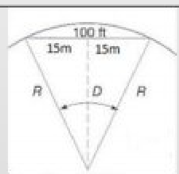
Fig 4.6

Arc Definitio

From Figure 4.6,

$$\frac{D^\circ}{30} = \frac{360^\circ}{2\pi R}$$

$$\text{Or, } D^\circ = \frac{1718.9}{R} \text{ degrees} \quad \text{-------------- (2)}$$

The degree of curve	
Arc definition	**Chord definition**
The central angle subtended by a circular arc of 30m (100-ft).	The angle at the center of a circular arc subtended by a chord of 30m (100 ft).
	

4.4 Elements of simple circular curve:

PI POINT OF INTERSECTION. The point of intersection is the point where the back and for- ward tangents intersect. Sometimes, the point of intersection is designated as V (vertex).

I INTERSECTING ANGLE. The intersecting angle is the deflection angle at the PI. Its value is either computed from the preliminary traverse angles or measured in the field.

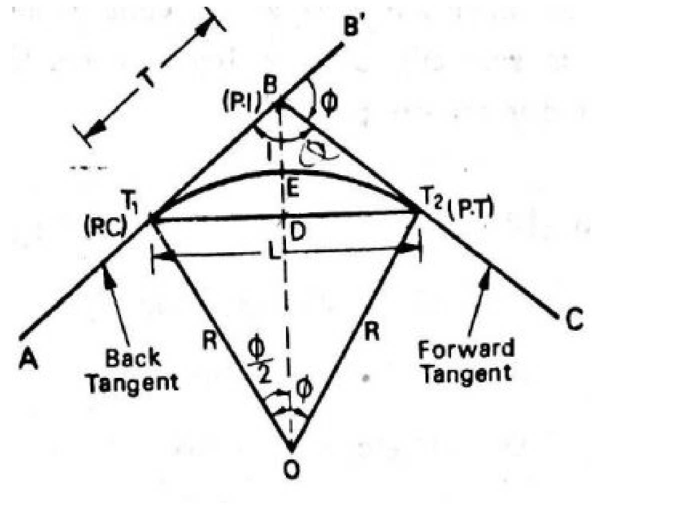


Fig4.7

CENTRAL ANGLE. The central angle is the angle formed by two radii drawn from the center of the circle (O) to the PC and PT. The value of the central angle is equal to the I angle. Some authorities call both the intersecting angle and central angle either I or

R RADIUS. The radius of the circle of which the curve is an arc, or segment. The radius is always perpendicular to back and forward tangents.

`

PC POINT OF CURVATURE. The point of curvature is the point on the back tangent where the circular curve begins. It is sometimes designated as BC (beginning of curve) or TC (tangent to curve).

PT POINT OF TANGENCY, The point of tangency is the point on the forward tangent where the curve ends. It is sometimes designated as EC (end of curve) or CT (curve to tangent).

POINT OF CURVE. The point of curve is any point along the curve.

LENGTH OF CURVE. The length of curve is the distance from the PC to the PT, measured along the curve. $\pi R\varphi/180^{\circ}$

TANGENT DISTANCE. The tangent distance is the distance along the tangents from the PI to the PC or the PT. These distances are equal on a simple curve. R $\tan(\varphi/2)$

LONG CHORD. The long chord is the straight-line distance from the PC to the PT. Other types of chords are designated as follows: $2R \sin(\varphi/2)$

C The full-chord distance between adjacent stations (full, half, quarter, or one- tenth stations) along a curve.

The sub chord distance between the PC and the first station on the curve

The sub chord distance between the last station on the curve and the PT

E EXTERNAL DISTANCE. The external distance (also called the external secant) is the distance from the PI to the midpoint of the curve. The external distance bisects the interior angle at the PI. $R(\mathrm{Sec}(\varphi/2)-1)$

M MIDDLE ORDINATE. The middle ordinate is the distance from the midpoint of the curve to the midpoint of the long chord. The extension of the middle ordinate bisects the central angle. $R(1-\cos(\varphi/2)$

D DEGREE OF CURVE. The degree of curve defines the sharpness or flatness of the

curve.

The degree of a curve is defined as the angle subtended at its centre of the curve by an arc of 30 m length.

OT1 = OT2 = R (Radius of the curve)

∟T1OT2) = φ (Central Angle)

∟B'BT2 = φ (Deflection angle)

∟OT1B = ∟OT2B = 90° (BT, and BT2 are tangents to the curve)

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Example 1. Two straights meet at an intersection angle of 75°.Calculate the following elements of the curve

i) Tangent length

ii) Length of curve

iii) Length of the long chore

iv) Apex distance

As the degree of the curve as 6°

Solution

As the intersection angle I is given as 75°

Central angle φ=180°-I =180°-75° = 105°

Radius of the curve ® = 1719/6=286.5 m

Elements of the curve:

1) Tangent length = R tan(φ/2)

= 286.5 tan(105°/2)

=973.374 m

2) Length of the curve = πRφ/180°

= (πX286.5X105°)180°

= 525.039m

3) Length of long chord = 2R sin(φ/2)

= 2X286.5 sin(105°/2)

=454.595 m

4) Apex distance = R(Sec(φ/2)-1)

= 286.5X(Sec(105°/2)-1) =184.128 m

Result:

i) Tangent Length =373.374 m

ii) Length of the curve =535.039m

iii) Length of long chord =453.592m

iv) Apex distance =184.128m

Example 2. The deflection angle at the point of intersection (chainage 1256.260 m) of two straights is 43°. Find the elements of a 3° circular curve to be set out between the straights.

Solution

The radius of the circular curve,

R = 1718.9 / D = 1718.9 / 3 = 572.97 m ~ 573 m

Let us consider, the radius of the circular curve to be set be, R = 575 m

From given data,

the deflection angle, D = 43°

Thus, from Equation 37.2, tangent length T = R tan (D / 2) = 575 tan (43° / 2)

= 226.498 m = 226.50 m

Length of the curve, l = $\frac{\pi R \Delta}{180^\circ}$

$$= \frac{\pi \times 575 \times 43}{180^\circ} = 431.533 \text{ m}$$

Chainage of T_1 (the point of curve) = Chainage of P.I. – T

= 1256.260 – 226.50 = 1029.676 m

And Chainage of T_2 (the point of tangency) = Chainage of P.C. + l

= 1029.76 + 431.533 = 1461.293 m

Length of the long chord, L = 2R Sin (D / 2)

= 2 x 575 x Sin (43° / 2) = 421.476 m

External distance, E = $R\left[\text{Sec.}\left(\frac{\Delta}{2}\right) - 1\right]$

$$= 575\left[\text{Sec.}\left(\frac{43}{2}\right) - 1\right] = 43.00\text{ m}$$

Mid-ordinate distance, $M = R\left(1 - \text{Cos}\frac{\Delta}{2}\right)$

$$= 575\left[1 - \text{Cos}\left(\frac{43}{2}\right)\right] = 40.00\text{ m}$$

4.5 Setting out simple curve:

Simple curves may be set out either by linear methods or by instrumental methods.

Linear methods are used when a high degree of accuracy is not required and the curve is short in length. In the method only chain and tape are used.

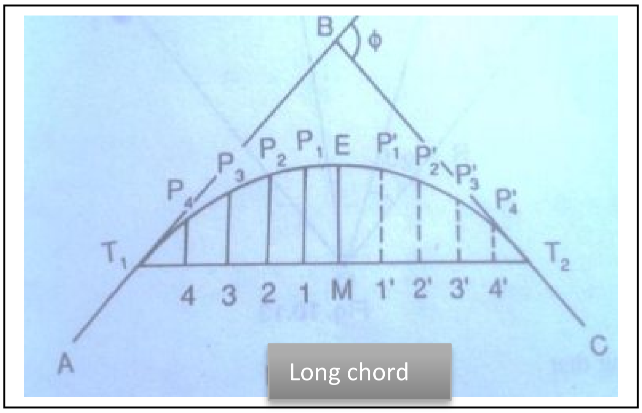
Linear method of setting out simple curves:

- By Offsets from long chord .
- By Offsets from tangents
- By Offsets from chords produced.

4.6 By Offsets from long chord

Procedures of setting out the curve

1. Set out the straight and fix up the tangent points
2. Join the tangent point T_1 and T_2 which gives the long chord
3. Locate the midpoint (M) of the long chord and erect a perpendicular ME. Mark off the offsets O_o (i.e., mid-ordinate) along that perpendicular and obtain the point E, the apex point of the curve



4. Divide the half of the long chord T_1M and DT_2 into number of equal parts as shown in figure and measure these distance from M

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5. At each of these set out the offset O_x, calculated from the equation O_x= (Sq.Root of ((R^2-X^2)(R-O_o)). These point gives the points on the curve

6. Repeat this procedure on either side of M until all the points are covered.

2. By Offsets from tangents:

Procedure for setting out the curve

In setting out curve the point on the curve can be obtained by marking offsets from the tangents. These offsets may be either radial or perpendicular to the tangents.

Procedure for setting out the Curve:

1. Set out the straights and fix up the tangent points

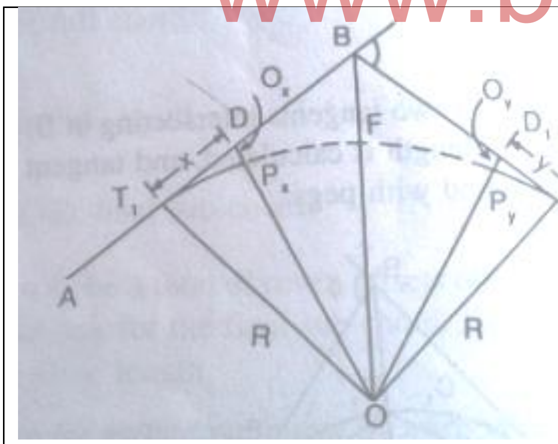
2. Along the tangent T_1B and T_2B drive pegs at regular intervals say 15m.

3. Calculate the offsets along these points using either radial or perpendicular offsets formula depending on the method use for setting out the curve.

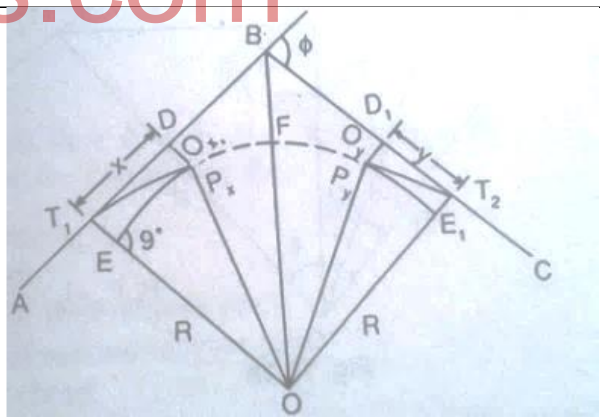
   [Note: X used in these formulae is the distance always measured from the tangents points to the point where the offsets is to be set out]

4. Set out the radial or perpendicular line at each of the peg point and measure the respective offset distance to fix up the point on the curve.

5. Similarly fix up all the points on the curve until the apex of the curve is reached from both the tangents. Joining all these points a smooth curve is obtained.



Radial or perpendicular offset



Perpendicular offsets

Example: 3

Two straight meets at angle of 130°. Calculate the necessary data for setting out a circular curve of 15 chain radius between the roads by the following methods. The length Chain is 20m.od

(i) Perpendicular offset method

(ii) Radial offsets method.

Solution

The radius of curve= R= 15X 20= 300 m

The deflection angle Delta = 180° - ϕ

= 180°-130°=50°

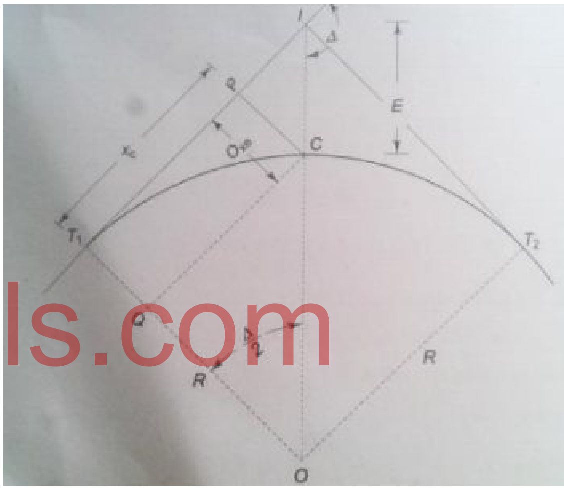
(i) Perpendicular offset method

The value X_c of x for fixing the apex C of the curve is determined from the

X_c = R sin (ϕ/2)

= 300 X sin(50°/2)

= 126.79m



O_{xc} = R-SQRT(R^2-X_c^2)

=300-SQRT(300^2-126.79^2)

=28.11m

Since the half of the curve is set out from T_1 and the other half from T_2, the value of x from which the offsets are to be calculated will be X= 20,40,60,12080,100,anherd 126.79m.

Therefore , the offsets are

O_{20}= 300- SQRT (300^2-20^2) =0.67 m

O_{40}= 300- SQRT (300^2-40^2) =2.68 m

O_{60}= 300- SQRT (300^2-60^2) =6.06 m

O_{80}= 300- SQRT (300^2-80^2) =10.86 m

O_{100}= 300- SQRT (300^2-100^2) =17.16 m

$O1_{20}$= 300- SQRT (300^2-120^2) =25.06 m

$O_{126.79}$= 300- SQRT (300^2-126.79^2) =28.11 m(Already computed) (ii)Radial offsets method

In case of the distance x for the last point to locate the apex of the cure is equal to the tangent length

. Thus for x equal to x_c= T, the offset O_{xc} will be equal to IC which is the external distance E

X_c= T = Rtan(ϕ/2)

= 300Xtan(50°/2)

= 139.89m

O_{xc}= E= R(Sec ϕ/2-1)

=31.01m

The value of x for which the offsets are to be calculated are

X= 20,40,60,80,100,120,139.89m

O_{xc} =SQRT $(R^2+X_c^2)$-R

O_{20} =SQRT(300^2+20^2)-300 = 0.67 m

O_{40} =SQRT(300^2+40^2)-300 = 2.65 m

O_{60} =SQRT(300^2+60^2)-300 = 5.49 m

O_{80} =SQRT(300^2+80^2)-300 = 10.48 m

O_{100} =SQRT(300^2+100^2)-300 = 16.23 m

O_{120} =SQRT(300^2+120^2)-300 = 23.11m

$O_{139.89}$ =SQRT$(300^2+139.89^2)$-300= 31.01 m (already computed

Example: 4

Calculates at ate the ordinate at 10m interval for setting out a circular curve for of 200m radius for a deflection angle of 60°. Use the method of offsets from the long chord.

Solution

Length of long chord = 2R sin(ϕ/2)

= 2 X 200 x sin (60°/2)

=200.00m

Mid ordinate O_o= R(1-cos(ϕ/2))

= 200(1-cos(30°)) = 26.79m

The value of x for half of the curve from D are as

X= 10,20,30,40,50,60,70,80,90,100m.

The Offset O_x= SQRT(R^2-X^2)-SQRT(R2-(L/2)2)

O_0= 26.79 (already computed)

O_{10}= SQRT(200^2=10^2)-SQRT(200^2-100^2)= 26.54 m

O_{20}= SQRT(200^2=20^2)-SQRT(200^2-100^2)= 25.79 m

O_{30}= SQRT(200^2=30^2)-SQRT(200^2-100^2)= 24.53 m

O_{40}= SQRT(200^2=40^2)-SQRT(200^2-100^2)= 22.75 m

O_{50}= SQRT(200^2=50^2)-SQRT(200^2-100^2)= 20.44 m

O_{60}= SQRT(200^2=60^2)-SQRT(200^2-100^2)= 17.58 m

O_{70}= SQRT(200^2=70^2)-SQRT(200^2-100^2)= 14.14 m

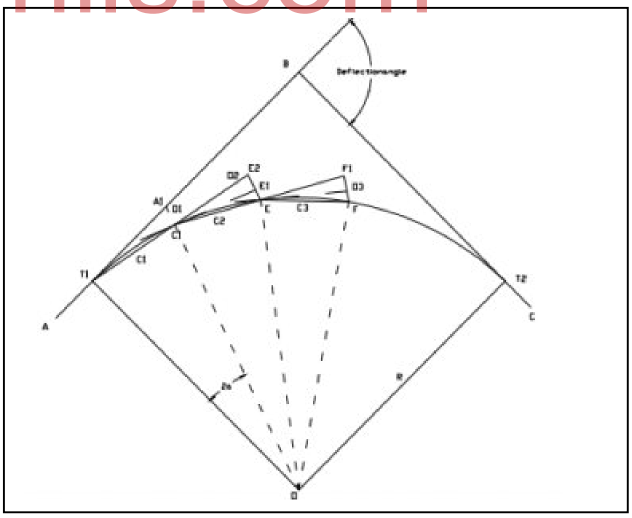
O_{80}= SQRT(200^2=80^2)-SQRT(200^2-100^2)= 10.10 m

O_{90}= SQRT(200^2=90^2)-SQRT(200^2-100^2)= 5.40 m

O_{100}= SQRT(200^2=100^2)-SQRT(200^2-100^2)= 0.00 m

4. Offset from tangent produced

Procedure for setting out of curve



1) Locate the tangent points T1 and T2 on the straights AB and CB.

2) Cut T1D1 equal to the length of the first sub chord (C1) already calculated along the tangentT1B.

3) With T1 as centre and T1D1 radius, swing the chain or tape such that the arc D1D= calculated offset O1, thus fixing the first point D on the curve.

4) Keep the chain along T1D and pull it straight in the forward direction of T1D until the length DE1 becomes equal to second C2 (i.e the length of normal chord).

5) With D as centre and DE1 as radius, swing the chain such that the arc E1E=calculated offset O2, thus fixing the second point E on the curve.

6) Continue the process repeating the point (d) and (e) until that end the curve is reached. The last point so fixed must coincide with the previously located points T2 (the last curve tangent point) if not,find out the closing error. If it is small (say within 2m) it should be distributed to

all the points by moving them sideways by an amount proportional to the square of their distances from the point T1, otherwise the whole curve should be set out again.

Example: 4

The chainage of the intersection point of two straight is 110 + 50 and the deflection angle is $45^0 20'$. A circular curve of 250m radius to set out to connect the two straight. Calculate the necessary data for setting out the curve by the method offset from the long chords produced. (Length of one chain = 20 m with 100 links and peg interval = 20m or 100 links)

Solution:

Given data:

Chainage of = 110+50 chains

Deflection angle = $45^o 20'$

Radius of curve R= 250 m

Tangent length= R tan (ø/2)

= 250 x tan ($45^o 20'/2$)

= 104.41 m

Length of arc l= ЛR ø/180^0

= Лx250x$45^o 20'/180^0$

=197.80 m

Chainage of T_1 = Chainage of I- T

= 110 x 20 +50 X0.2 - 104.41

= 2105.59 m

Chainage of T_2 = Chainage of T_1+l

=2105.59-197.80

=2303.39 m

Since the Chainage of T_1 is 2105.59 m the Chainage of first full station is calculated as below

2105.59 /20 = 105.2795 chains

Therefore Chainage of the next full chain station = 106 chains

= 106 X 20

=2120 m

`

Length of the first sub-chord = 2120-chainage of T_1

C_1 = 2120-2105.59

= 14.41 m

Since the Chainage of T_2 is 2303.39 m, the Chainage of last full station is calculated as below

2303.39/20=115.1695 chains

The Chainage of the last full chain station

= 115 Chains

=115 X 20

=2300 m

Length of the last sub-chord = Chainage of T_2-2300

0r C_n=2303.39-2300

= 3.39m

The number of normal chords = (2300-2120)/20=9

There fore Total number of chords= 1+9+1=11

First offset O_1 $= C_1^2/2R$

$= (14.41)^2/2X250$ **= 0.42 m**

O_2 = 20x(14.41+20)/2x250 = **1.38**

O_3 to O_{10}= C^2/R

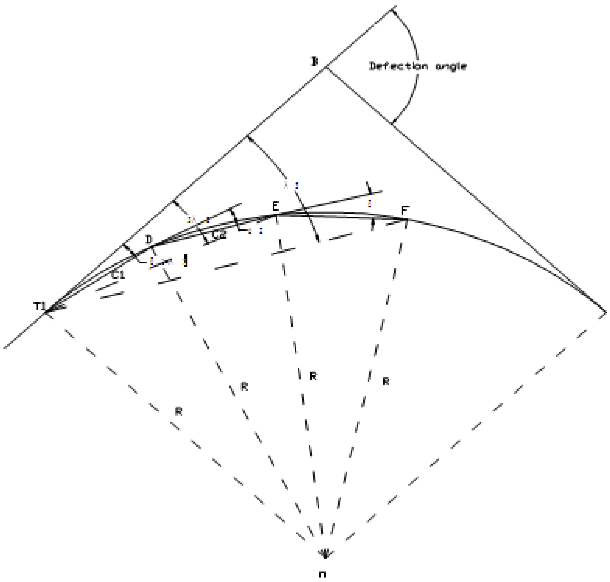
=20^2/250= 1.60 m

O_{11} $= C_n(C_n+C)/2R$

=3.39 X (3.39+20)/(2x250)= **0.16 m**

`

Rankine's method of deflection angles In this method the curve is set out by deflection angle and the chord lengths. The deflection roads angles are set out by a theodolite and a chain is used to mark off the chord lengths to obtain the respective points on the curve. Since only one theodolite is used to set out the deflection angles this method is known as single theodolite method and chain is also used along the theodolite, it can be also othrwiese known as chain and theodolite method.



Aim –: Setting out of simple circular curve by Rankine method of tangential angle.

Procedure for setting out of curve

1) Locate the tangent points T1 and T2 on the straights AB and CB.

2) Set up the theodolite at the beginning of the curve T1.

3) With the vernier A of the horizontal circle set to zero, direct the telescope of the ranging rod fixed at the point of intersection B and bisect it.

4) Unclamp the vernier plate and set the vernier A to the first tangential angle Δ1, the telescope being thus directed along T1D.

5) Measure along the line T1D, the length equal to first sub-chord (C1) thus fixing first point D on the curve.

6) Unclamp the vernier plate now and set the vernier A to the second total tangential angle Δ2 , the line of sight is now directed along T1E.

7) With the zero end of chain or tape at D1 and with a arrow held at distances of D1E=C2 (second chord or say normal chord),swing the chain about D1 until the line of sight bisects the arrow, thus fixing the second point Eon the curve.

8) Repeat the process until the last point T2 is reached.

Field Notes

The record, of various total tangential angles and angles to which the theodolite readings are to be set, is given in tabular from as under.

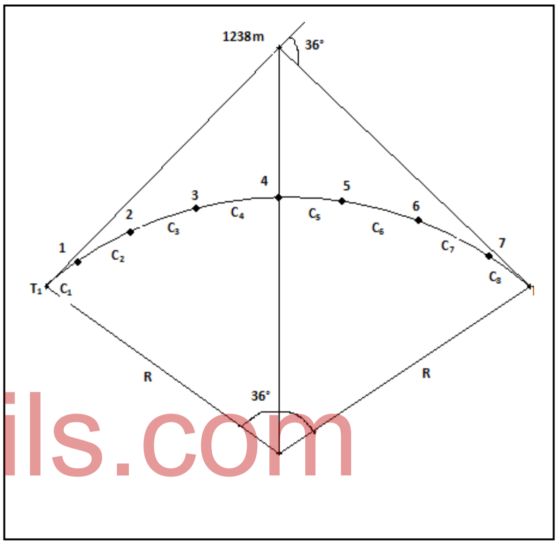
TABLE OF TANGENTIAL ANGLES

Point	Chainage in meters	Chord length in meters	Tangential angle (δ)			Total Tangential angle			Actual theodolite reading			Remarks
			0^0	'	"	0	'	"	0^0	'	"	

Example: 5

Calculate the necessary data for setting out a circular curve of 350m radius to connect the two tangents intersecting at the Chainage of 1238 m, the deflection angle being 36^0. Take the pege interval equal to 30 m.

Solution:



Chainage of I= 1238 m

Deflection angle of the curve Δ= 36°

Radius of the curve R= 350 m

Length of the normal chord C= 30 m

The point to be located on the curve are 1,2,3 etc.

We know that

Tangent length = T = R tan(Δ/2)

= 350 tan (36°/2)

= 113.72m

Length of arc = ЛRΔ/180°

= ЛX350X36^0/180^0

= 219.91 m

Chainage of T_1 = Chainage of I – T

= 1238 – 113.72 = 1124.28

37chains +4.28 $_m$

Chainage of the point 1 = 38 chains

Length of first sub-chords C_1 = 38 chains- (37 chains +4.28 m)

= 1 chain – 4.28 m = 25.72 m

`

Chainage of T_s = Chainage of T_1+ l

=1124.28+219.91

=1344.19 m =44 chains +14.19 m

Length of the last sub-chord C_n= 14.19 m

Total number of normal chords = 44-38 = 6

Check: Total number of chords n = 1+6+1 = 8

$=C_1+6xC+C_n= i$

=25.72+6C30+14.19= 219.91 m= l (Okay)

Computation of deflection angels:

$§_1$ = 1718.9 C_1/R

= 1718.9 x 25.72/350 = 126.71 minutes

= 2°06'19"

$§_{2 \text{ to } 7}$ = 1718.9C/R

=1718.9X30/350=147.33 minutes

=2°27'20"

$§_8$ = 17198.9C_n/R

=1718.9 x 14.19/350 = 69.69 minutes

=1°09'41"

We know that Total deflection angle $\Delta_n = \Delta_{n-1} + §_n$

Δ_1=	$\Delta_0 + §_1$		0+2°06'19"	**2°06'19"**
Δ_2=	$\Delta_1 + §_2$		2°06'19"+ 2°27'20"	**4°33'39"**
Δ_3=	$\Delta_2 + §_3$		7°00'59"+ 2°27'20"	**7°00'59"**
Δ_4=	$\Delta_3 + §_4$		7°00'59"+ 2°27'20"	**9°28'19"**
Δ_5=	$\Delta_4 + §_5$		9°28'19"+ 2°27'20"	**11°55'39"**
Δ_6=	$\Delta_5 + §_6$		11°55'39"+ 2°27'20"	**14°22'59"**
Δ_7=	$\Delta_6 + §_7$		14°22'59"+ 2°27'20"	**16°50'19"**
Δ_8=	$\Delta_7 + §_8$		16°50'19"+ 1°09'41"	**18°00'00"**
Check		$\Delta_n = \frac{1}{2}$ X Deflection angle		
		$= \frac{1}{2}$ X 36^0= 18^0 (Okay)		

`

Transition curve

Transition curve is curve is a curve of varying radius. It is inserted in between the straight and a circular curve or between the two branches of compound or reverse curve. It provides easy and gradual change over from straight to the circular curve.

It is also otherwise known as spiral or casement curve .The types of transition curve used are,

(i) Cubic parabola
(ii) Spiral and
(iii) Lemniscates

Vertical curve:

When two different gradients meet at a point along a road surface, they form a sharp point at the apex. Unless this apex point is rounded off to form a smooth curve, no vehicle can move along that portion of the road so for the smooth and safe running of vehicles, the meeting point of the gradients is rounded off to form a smooth curve in a vertical plane. This curve is known as a vertical curve.

Gradient

The gradient is expressed in two ways:

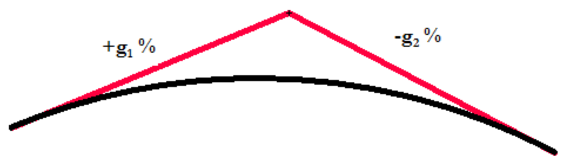
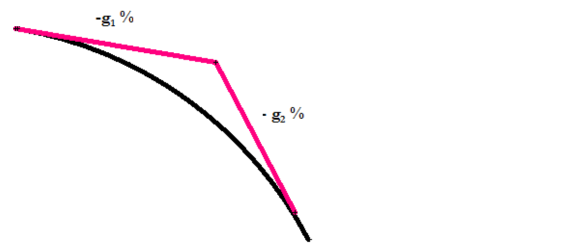
(a) As a percentage, e.g. 1%, 1.5%, etc.
(b) As 1 in n, Where n is the horizontal distance and 1 represents vertical distance
(c) E.g. 1 in 100, 1 in 200, etc.
The gradient may be 'rise' or 'fall'. An up gradient is known as 'rise' and is denoted by a positive sign.
A down gradient is known as 'fall' and is indicated by a negative sign.

Key Note

HORIZONTAL CURVES	VERTICAL CURVES
Curves provided in the horizontal plane to have the gradual change in direction	Curves provided in the vertical plane to obtain the gradual change in grade

Types of vertical curve:

(a) Summit curve

A summit curve where an up gradient is followed by down gradient	
A Summit curve where a down gradient is followed by another down gradient	

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(b) Sag Curve

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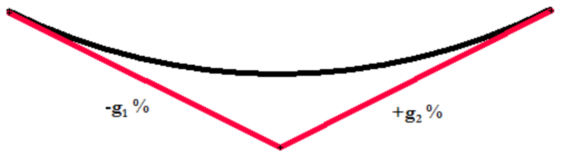
<tr><td>A sag curve where a down gradient is followed by an up gradient</td><td>
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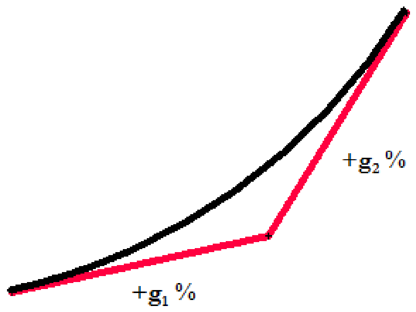
<tr><td>A sag curve where an up gradient is followed by another up gradient</td><td>
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Question bank

A. One-mark question

1. Draw neat sketch of simple curve
2. Mention the types of curve
3. Define back tangent

4. Define forward tangent
5. How curve are designated
6. What is meant by long chord
7. Define deflection angle
8. Define inter section angle
9. What is full chord
10. Define sub-chord
11. 4. Where the curves are employed?
12. 5. Mention the types of curves adopted?

B. Three marks questions

1. Differentiate between simple and compound curve
2. Differentiate between apex distance and mid –ordinate with sketch

3. Devices the relationship between the degree and radius of the curve.

C. Ten marks questions

1. Explain the types if curve with sketch
2. What are the various elements of a simple curve? Draw simple curve and show the various elements
3. Explain the following (i) Back tangent (ii) Forward tangent (iii) Point of curve (iv) Point of tangency (v) Length of curve
4. Describe the method of setting a circular curve by (i) Offset from long chord (ii) Offsets from tangent method
5. Describe the method of setting a circular curve by Rankine's deflection angle method
6. Two tangents meet at chainage 1022 m, the deflection angle being 36°. A circular curve radius 300m is to be introduced in between them. Calculate the (1) Tangent length (2) Length of circular curve (3) Chainage of tangent length (4) length of long chord (5) Apex distance
7. Two tangents intersect at an angle of 140° in chainage 1265m. If the radius of curve is 300m. Calculate and tabulate the necessary data for setting out a right hand curve by Rankine's deflection angles method. The peg interval may be20m

8. A simple circular curve of radius 300m is to be introduced between the two straight intersect at chain age (34 +24) . The deflection angle is 36°. Calculate the deflection angles Necessary to set out the curve taking a standard chord length of 30m . prepare the table of deflection angle, if the curve is a left handed curve.
9. Calculate various elements of a simple curve of radius 300m connecting the two straights interests an angle of 120°.

UNIT-V

TOTAL STATION AND GEOGRAPHICAL INFORMATION SYSTEM

5.1 TOTAL STATION

Introduction – Application of total station – Component parts of a Total Station – Accessories used – Summary of total station characteristics - Features of total station – Electronic display and data reading – Instrument preparation, Setting and Measurement (Distance, Angle, Bearing etc.) – Field procedure for co-ordinate measurement – Field procedure to run a traverse survey - Linking data files for various Applications.

5.1.1 Introduction

The development of the total station has made it possible to accurately gather enormous amounts of survey measurements quickly. In the last 20 years, total stations and data collectors have become common field equipment, and have largely replaced the traditional survey methods that utilized transits theodolites, and plane table. Digital theodolites and EDM instruments were perfected during the 1970s. In the early 1980's the surveying instrument manufacturers introduced the total station, by creating an electronic tacheometer that combined the digital theodolites and EDM devices. Directly storing direction and distance observations to a microprocessor helped eliminate many of the reading errors and writing errors that can occur with an optical theodolite Modern total stations can measure a distance to an accuracy of better than 5 millimeters plus 1 part per million, with some variation depending on the type of reflecting surface or prism used. Electronic angles can be resolved to about one-half arc second. In most land surveying situations, the normal crew size can be reduced to two persons when equipped with a standard total station.

Development of Total station

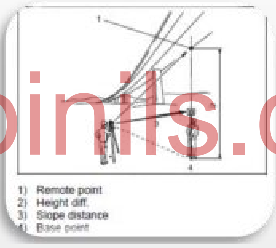
Total station instruments can record horizontal and vertical angles together with slope distance by electronic theodolite and EDM respectively. The microprocessor in Total station can perform various mathematical operations such as averaging multiple angle and distance measurements, calculating horizontal and vertical distances, X, Y, Z coordinates, distance between observed points and corrections for atmospheric and instrumental corrections. The data can be stored in inbuilt memory and it can transfer to a personal compute

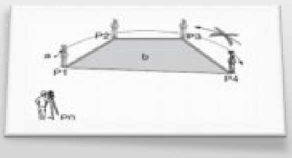
Total station

5.1.2 Application of total station:

Total Stations are being used for:

- Angle measurement
- Distance measurement
- Computes coordinates
- control surveys
- Contour and topographic mapping
- Setting out and construction work
- Traverse closure and adjustment
- Remote object elevation
- Distance between remote points
- Inversing
- Resections
- Setting out
- Area calculations, etc.
- Volume calculation
- Road design etc.



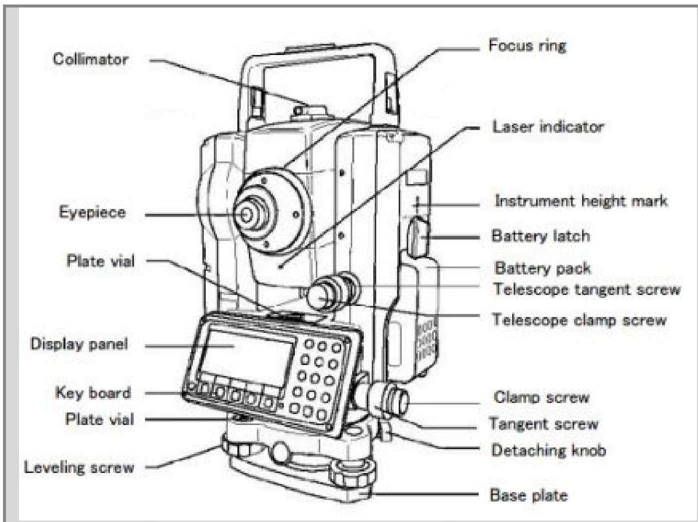


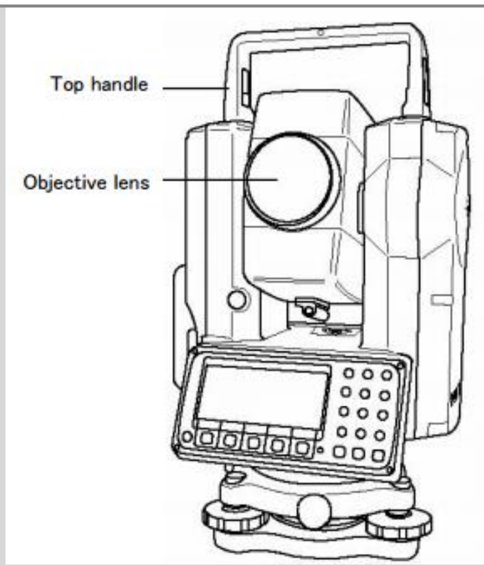
Areas and volume

What is a Total Station?

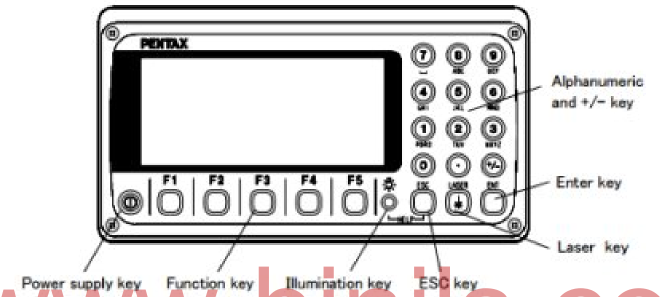
Total station is a surveying equipment combination of Electromagnetic Distance Measuring Instrument and electronic theodolite. It is also integrated with microprocessor, electronic data collector and storage system. The instrument can be used to measure horizontal and vertical angles as well as sloping distance of object to the instrument. Examples of Total Stations are LeicaTPS400series, TopconGPT3000series, Sokkia Set510,520 and PentaxR-325 etc.,

5.1.3 Component parts of a Total Station





Parts of the total station

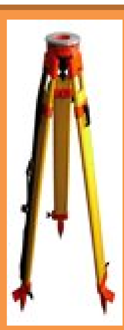


Display and keyboard

PART	**Function/Specification**
Collimator	Quick targeting
Handle	To lift the instrument
Telescopes	gives a bright, high-contrast erect image with magnification ≥30X
Focusing ring	Rotation “clockwise” makes it possible to focus on closer objects and “counterclockwise” will focus on further objects.
Clamp screw	Circle setting to angle measurements
Key board	To control the whole instrument with Alpha Numeric ,function and fixed keys either single / both sides
Display	LCE(Liquid Crystal Display)/ colour/ Touch screen
Three-pin tribrach	Mounting the main unit
Detaching knob	Main unit can attach and detach from the tribrach if necessary

5.1.4 Accessories used

Tripod: The tripod supports and holding the total station. These tripods are made of the finest hardwood which is carefully selected to guarantee the utmost in stability, durability and reliability.



Tripod

They have features like

* Plastic coated wooden legs ensure a long life
* Big, round tripod head for easy instrument set up
* Brass hinge pins for durability and stability
* Powder-coated hardware parts
* Replaceable steel points
* Snap Cap to protect the tripod head
* Carrying strap for easy transport
* Available with quick clamp, screw clamp or both.

1. Reflector with pole:

Prism: It is a corner cube prism kept in a prism holding assembly. It is available in single and triple combination

Prism pole: An extendable telescopic rods with centimeter graduation available up to the length of maximum 2.1m.The corner cube prism is attached to the top of it.

2. Power source : Rechargeable Li-ion battery Working duration at 25° C is about 7.5hours to 10 hours. Charging time at 25° C is about 2h

Prism pole | Single prism | Triple

Battery | Charger

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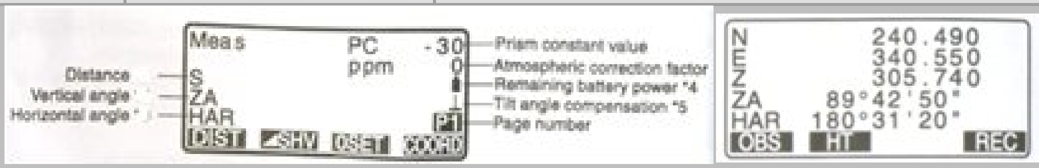
5.1.5 Summary of total station characteristics



S.No	Character	Unit/Description
1	Angle unit	Degree or gon
2	Distance unit	Feet or meter
3	Pressure unit	in Hg or mmHg
4	Temperature unit	°F or °C
5	Prism constant	-30mm or -40mm
6	HRA	Horizontal angle
7	ZA	Vertical angle
8	S	Slope distance
9	H	Horizontal distance
10	V	Vertical distance
11	PtID or PN	Point Identification or Point Number
12	E(X)	Easting
13	N(Y)	Northing
14	Z	Elevation
15	IH	Instrument height
16	PH or hr	Prism height /reflector height
17	Target	Reflector type
18	ESC	Return to the previous screen
19	ENT	Accept the selected, highlighted, choice or the displayed screen value

5.1.6 Features of total station

Total Station Components

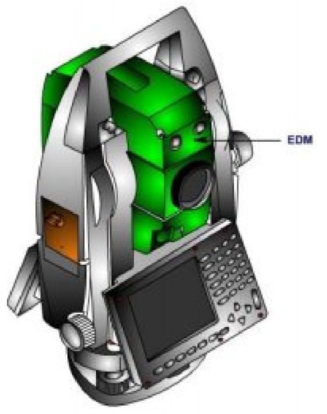


Figure Total Station

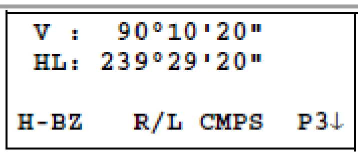
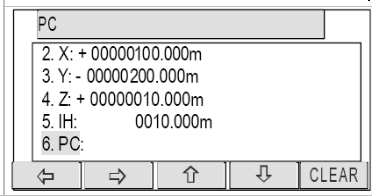
The two main components of a total station are a theodolite and an electronic distance meter (EDM). The former measures angles and the latter measures distance, as its name implies. The theodolite is one of the oldest surveying instruments, with its origins dating back a few centuries. Modern total stations are built around theodolites with EDMs integrated into the housing. The components function together as a telescope, with the operator bringing the opposite point into the telescope's crosshairs. The surveyor can then measure the angle and the distance between the two points. If the total station is also fitted with a data collector, the data collector reads out these measurements, enabling the surveyor to record them for drawing up the survey later.

It has telescopes of which gives a bright, high-contrast image. The course and fine focusing ensures that the target is seen sharp and clear. Pointing is fast and precise even in poor observing conditions. The Focusing knob rotation "clockwise" makes it possible to focus on closer objects and "counterclockwise" will focus on further objects.

TS has electronic clamp for circle setting to angle measurements. The whole instrument is controlled through the keyboard. The TS has two control panels, each with keyboard and LCDs displays. It can be used easily and quickly in both positions. The main operation require only as single Keystroke. Accepted keystrokes are acknowledged by a beep.

The Total station is then mounted in the three-pin tribrach and main unit can attach and detach from the tribrach if necessary using detaching knob. Features include dual-axis compensation to ensure consistently accurate measurements. Some total stations provide out of level warnings to the operator. Either optical or laser plummets are used for final centering over a point.

5.1.7 Electronic display and data reading

Electronic display	
Electronic display unit is capable of displaying various values when respective keys are pressed. The system is capable of displaying horizontal distance, vertical distance, horizontal and vertical angles, difference in elevations of two observed points and all the three coordinates of the observed points.	 

Data reading:

The conventional method of recording surveys is overtaken by development in computer mapping and survey instrumentation which made electronic data recording and transfer essential.

The following are some methods of recording data electronically.

3. **Data recorders:**

These are dedicated to a particular instrument and can store and process surveying observations. These are also referred to as electronic field books. They use solid-state technology enabling them to store large amounts of data in a device of the size of a pocket calculator.

2. Field computers:

These are hand-held computers adapted to survey data collection. Comparing with a data logger, they offer a more flexible approach to data collection since they can be programmed for many forms of data entry.

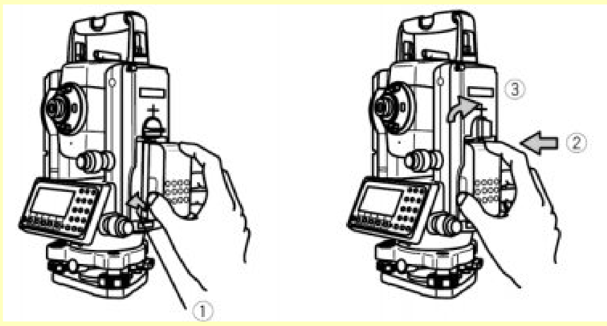
3. **Recording modules:**

These are also called memory cards which take the form of plug-in cards onto which data is magnetically encoded by a total station. Data is transmitted to the memory card using a non-contact magnetic coupling system which eliminates the need to attach sockets or pins to the card.

4. **Internal memories**

A total station can be fitted with an internal memory capable of storing 900 to 10,000 points. This enables data to be collected without the need for a memory card or data recorder.

5.1.8 INSTRUMENT PREPARATION

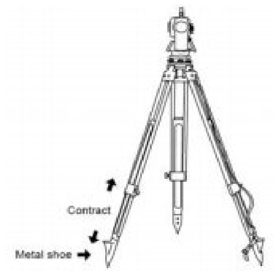
Attaching the battery	
❖ Align the guide grooves on the battery pack with the guide grooves on the instrument and push the top of the battery pack into place. ❖ Turn the lock lever clockwise to fix.	

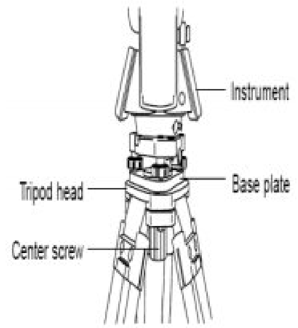
PREPARATIONFORSURVEYING

Centering and Leveling of the Instrument

Setting up the instrument and the tripod

- Adjust the tripod legs so that a height suitable for observation is obtained when the instrument is set on the tripod.
- Hang the plumb bob on the hook of the tripod, and coarse center over the station on the ground. At this time, set the tripod and fix the metal shoes firmly into the ground so that the tripod head is as level as possible, and the plumb bob coincides with the station on the ground.
- If the tripod head is mis-leveled by the action of fixing the metal shoes into the ground, correct the level by extending or retracting each leg of the tripod.
- Tripod is adjusted according to the following points by extending or contracting the legs so that the bubble of the Circular vial goes to the center of the circle.
- Shorten the leg at the side of the bubble or extend the leg opposite of the bubble to position the bubble the center of the vial circle.
- All three legs are extended or contracted until the bubble is in the center. During this process, the foot is not placed on the tripod leg point and the position of the tripod points do not change

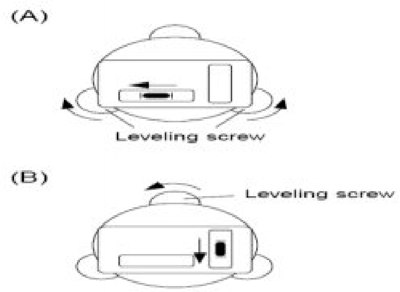


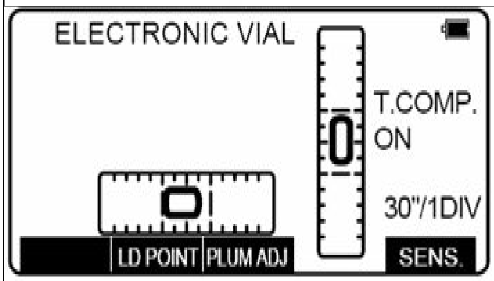


Leveling With Electronic vial

[Electronic vial screen]

- Turn two Leveling screws arbitrarily chosen in an opposite direction mutually and put the vial of the horizontal Electronic vial in the center.(Figure A)
- Put the bubble of the lengthwise Electronic vial in the center by operating the Leveling screw of one remainder. (Figure B)

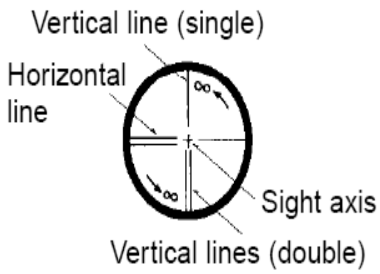




Eyepiece Adjustment

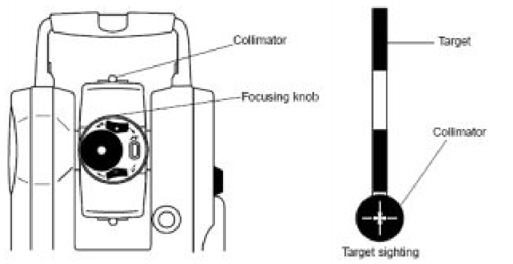
The eyepiece adjustment is performed before target sighting.

- Remove the telescope lens cover
- Point the telescope at a bright object, and rotate the eyepiece ring full counter-clockwise.
- Look through the eyepiece, and rotate the eyepiece ring clockwise until the reticle appears as its maximum sharpness.



Target sighting by Manual focus

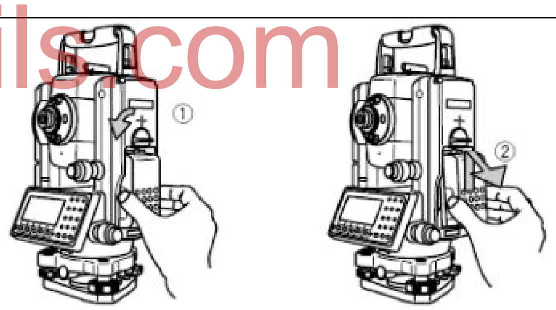
- The Focusing knob rotation “clockwise” makes it possible to focus on closer objects and “counterclockwise” will focus on further objects.



REMOVING THE BATTERY

- ❖ Turn the lock lever anticlockwise and remove the Battery.
- ❖ Lift up the battery pack and remove it from the instrument.

Be absolutely sure to turn the power off when removing the battery as removing the battery while the power is still on may result in damage to the instrument.



5.1.9 Setting and Measurement (Distance –Angle-Bearing, etc.)

(i) Determination of distance

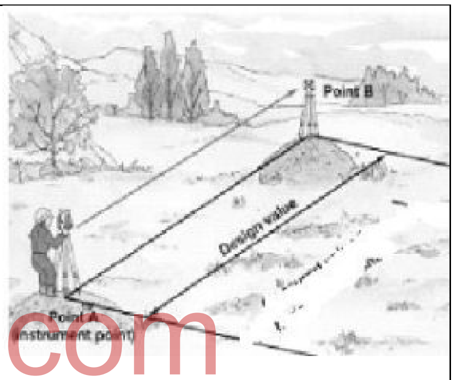
Objective: To find distance between station A to point B

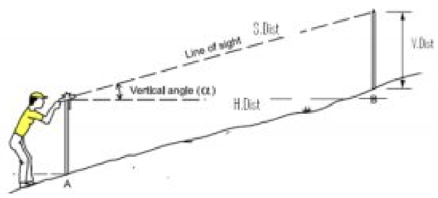
Instrument Required:

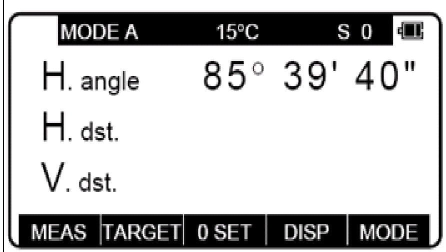
- Total station with tripod
- Data collector – optional
- Charged batteries
- 1 Adjustable poles
- 1 Target holders
- 1 Reflector

Procedure:

1. Set and level the total station exactly over the given station at "A"
2. Attach the battery in battery pack then press power on key
3. Press the enter button to view measure mode screen
4. Hold the prism pole attached with prism over the point B
5. Bisect the prism exactly by using both telescope clamp and lower clamp Screw
6. Edit the Measured Instrument height and target height
7. Press [MEAS]/(DIST) function key to activate the EDM
8. Record the distance between station A and B appeared in the display screen
9. Power off the machine







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(ii) Measurement of horizontal angle/bearing

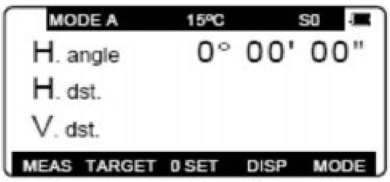
Objective: To measure a horizontal angle between the two lines OA and OB by general method

Instrument required:

- Total station with tripod
- Data collector – optional
- Charged batteries

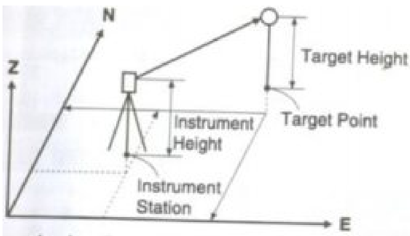
Procedure:

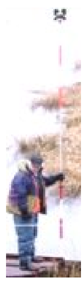
1. Set and level the total station over the given instrument station "O" and level it
2. Attach the battery in battery pack then press power on key
3. Bisect the point "A" exactly by using lower and upper clamp screw and it tangent
4. Set the horizontal angle 0º 0' 0" by pressing twice [RST] or [0 set] key by pressing continuously twice
5. Turn the telescope and bisect the point B exactly by loosening lower and upper clamp screw and it tangent
6. Record the H. angle between the line AOB displayed in the LCD
7. Power off the machine



5.1.10 Field procedure for co-ordinate measurement

Theory: By performing coordinate measurements it is possible to find the 3- dimensional coordinates of the target based on station point coordinates, instrument height target and azimuth angles of the back sight station which are entered in advance.





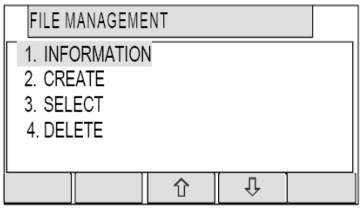
Instruement required:

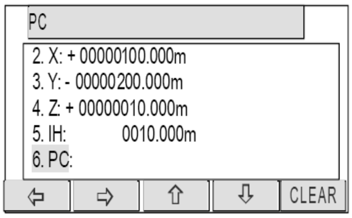
- ❖ Total station with tripod
- ❖ Data collector – optional
- ❖ Charged batteries

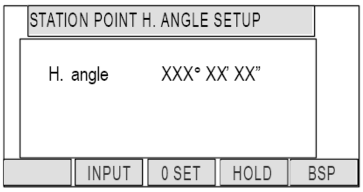
- ❖ 2 Adjustable poles
- ❖ 2 Target holders
- ❖ 2 Reflector
- ❖ 2 two – way radios – Optional

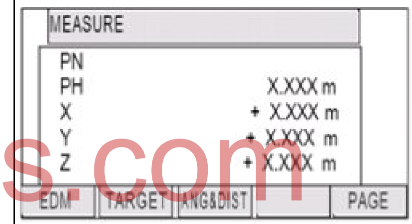
Procedure

1. Set up and level instrument over station "O"
2. Turn on total station
3. Create a file or open existing file
4. Select Coordinate menu to view station point setup
5. Record Occupied Station Name, number and instrument height
6. Input the Coordinate (X, Y, Z) for Occupied Station point
7. Bring the telescope toward magnetic meridian, and set the back sight angle 0'00'00" (Hz).
8. Set reflector height
9. Measure the foresight station by keeping prism over the point
10. Record the displayed coordinate with specific name and number of that point
11. Continue the above two steps to collect coordinates of the remaining points
12. After completion of recording all the fore sight the data, Power off the machine



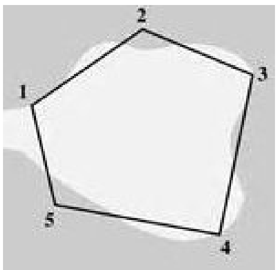






5.1.11 Field procedure to run a traverse survey

AIM: To run a closed traverse by connecting given points 1,2,3,4 and 5 using total station



Closed traverse 12345

Instruement required:

- ❖ Total station with tripod
- ❖ Data collector – optional
- ❖ Charged batteries
- ❖ 2 Adjustable poles
- ❖ 2 Target holders
- ❖ 2 Reflector
- ❖ 2 two – way radios – Optional

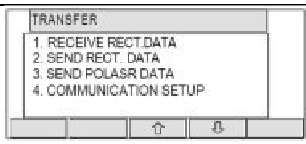
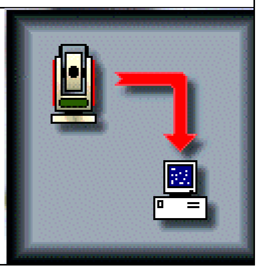
Procedure

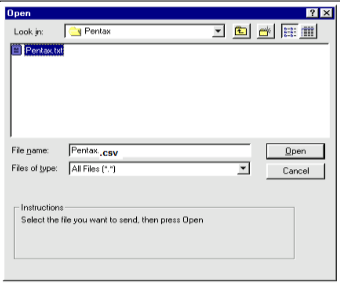
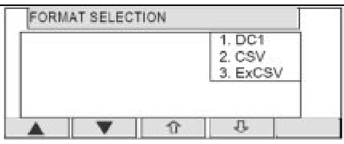
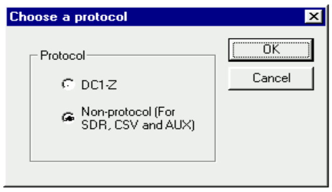
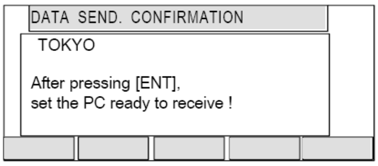
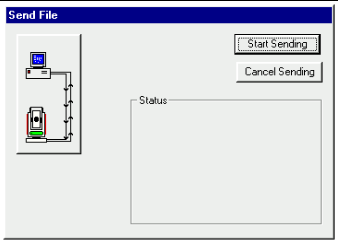
1. Set up and level instrument over station “1”
2. Turn on total station
3. Create a file or open existing file
4. Select Coordinate menu to view station point setup
5. Record Occupied Station Name, number and instrument height
6. Input the known Coordinate for the Occupied Station point
7. Bring the telescope toward magnetic meridian, and set the back sight angle 0'00’00” (Hz).
8. Edit the reflector height
9. Measure the foresight station by keeping prism over 2
10. Record the displayed coordinate with name and number of that point
11. Shift the instrument to set and level instrument over station “2”
12. Input the observed Coordinate of point 2 for Occupied Station point
13. Input the current instrument height
14. Input the known coordinate of point 1 for the back sight
15. Keep the prism over the point 1 and take the back sight
16. Measure the foresight station by keeping prism over 3
17. Record the displayed coordinate with name and number of that point
18. Continue the above seven steps over the remain corner pints (3, 4 and 5) to collect coordinates of those points
19. Complete the traverse after observing coordinates of 1 from station 5
20. After completion Power off the machine

Check for the accuracy: The observed coordinate of point 1 should be equal to known coordinate of point 1

5.1.12 Linking data files for various applications:

Link software manages survey data from total station survey. The raw data gathered on a variety of total station is stored, organized, edited, reduced and analyzed. The data can be imported from a wide variety of file formats.

Sending total station data to PC	
A. Total station setting for sending the file	B. PC setting through Data link software Receiving a file
Select TRANSFER menu 	Select receive from the pull down File menu/Or, press the receive button on the main screen 

Select the 2. SEND RECT. DATA by pressing the down arrow key , and press [ENT] to view the FORMAT SELECTION screen. 	A selection dialog box will pop up, and then you can select/create the file you want to send in csv format 
Select CSV format and press [ENT] to view the DATA SEND. CONFIRMATION screen 	Choose non-protocol. 
Press [ENT], and set the PC to be ready to receive. 	The Send File dialog box will pop up. Prepare the device to start receiving the selected file 

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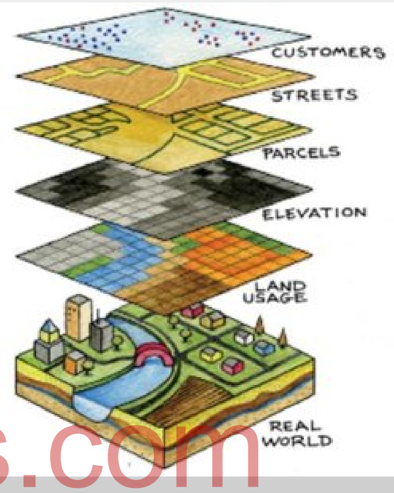
Introduction – Geographical information – Development of GIS – Components of GIS – Steps in GIS mapping - Ordinary mapping to GIS – Comparison of GIS with CAD and other system – Fields of Applications : Natural resources, Agriculture, Soil, Water resources, Wasteland management and Social resources – Cadastral survey and Cadastral records – Land Information System(LIS).

5.2.1 Introduction

A GIS (geographic or geospatial information system) is a modern extension of traditional cartography with one fundamental similarity lies in the fact that both a cartographic document and. The two differences are that there is the GIS contain examples of a base map to which additional data can be added further no limit to the amount of additional data that can be added to a GIS map and secondly the GIS uses analysis and statistics to present data in support of particular arguments which a cartographic map cannot do. Now days Geographic Information Systems are used by the Government, businesses, educators/scientists, utility services, resource managers, environmental protection agencies and more. All of these users are able to utilize GIS in their own unique way by building project specific database



GIS cake layer artwork drawn by Scott Mealy

5.2.2 Geographical Information system

Geographic Information Systems is a computer-based tool that analyzes, stores, manipulates and visualizes geographic information on a map.

The Geographical information system is consist of three subsystems such as computer hardware, software and spatial data. It is used for capturing, storing, displaying, updating and analyzing all forms of referenced data. A Geographical information system combines computer drawn maps with a database management system which bring cost Savings from Greater Efficiency GIS is also the go-to technology for making better decisions about location and managing geographic with sustainability. GIS-based

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maps and visualizations greatly assist in understanding situations that improves communication between different teams, departments, disciplines, professional fields, organizations, and the public.

5.2.3 Development of GIS

The Phase development

There have been four distinct phases in the development of Geographic Information Systems. Phase one, between the early 1960s and the mid-1970s saw a new discipline being dominated by a few key individuals who were to shape the direction of future research and development. The second phase, from the mid-1970s to early 1980s saw the adoption of technologies by national agencies that led to a focus on the development of best practice. Phase three, between 1982 until the late 1980s saw the development and exploitation of the commercial market place surrounding GIS whilst the final phase since the late 1980s has seen a focus on ways of improving the usability of technology by making facilities more user centric.

Roger Tomlinson – the father of GIS. In the 1960s when he initiated, planned and directed the development of the Canadian Geographic System (CGIS)

Decade wise development

The first GIS, Canada Geographic Information System was developed in mid-1960s to identify the nation's land resources and their existing, and potential uses.

In the late 1960s, US Bureau of the Census created the DIME program (Dual Independent Map Encoding) for all US streets to support automatic referencing and aggregation of census data.

In late 1970s, Harvard University's Laboratory for Computer Graphics and Spatial Analysis developed a general-purpose GIS (ODYSSEY GIS).

The first automated cartography developments occurred in the 1960s, and by the late 1970s most major cartographic agencies were already partly computerized.

GIS began to take off in the early 1980s, when the price of computing hardware had fallen to a level that could sustain a significant software industry and cost-effective applications.

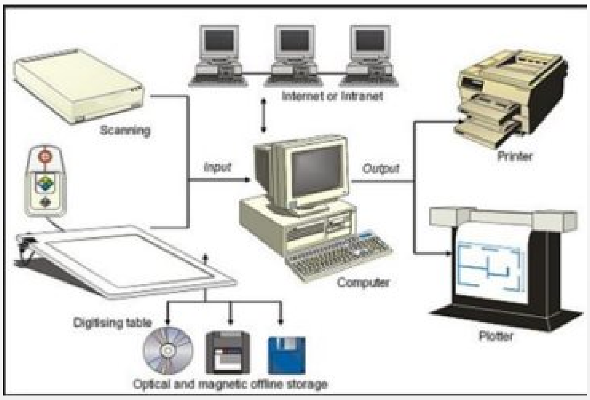
Esri is now the world's leading experts in GIS software development and it has played a key role in the history of GIS.GIS Developments in India starts from the year 2000 onwards in the analyses

5.2.4 Component of GIS

A GIS can be divided into five components: Hardware, Software, Data, People, and methods. All of these components need to be in balance for the system to be successful. No one part can run without the other.

1. Hardware

Hardware consists of the technical equipment needed to run a GIS including a computer system with enough power to run the software, enough memory to store large amounts of data, and input and output devices such as scanners, digitizers, GPS data loggers, media disks, and printers . Hardware ranges from powerful servers to mobile phones.



GIS-Hardware

2. Software

There are many different GIS software packages available today. From ArcGIS, QGIS, GRASS GIS, SuperGIS, SAGA GIS to JUMP GIS…etc. All packages must be capable of data input, storage, management, transformation, analysis, and output, but the appearance, methods, resources, and ease of use of the various systems may be very different. Today's software packages are capable of allowing both graphical and descriptive data to be stored in a single database, known as the object-relational model. Before this innovation, the geo-relational model was used. In this model, graphical and descriptive data sets were handled separately. The modern packages usually come with a set of tools that can be customized to the users

Open source software

1.QGIS – Formerly Quantum GIS.

2 gVSIG.

3 Whitebox GAT.

4 SAGA GIS.

5 GRASS GIS.

6 MapWindow.

7 ILWIS.

8 GeoDa.

3. **Data:** Data is the most important component of a GIS. Geographic data and related tabular data can be collected in house, compiled to custom specifications and requirements, or

purchased from a commercial data provider. A GIS can integrate spatial data with other existing data resources, often stored in a DBMS. The integration of spatial and tabular data stored in a DBMS is a key functionality afforded by GIS.

4. **People**

People The people are the component who actually makes the GIS work. They include a plethora of positions including GIS managers, database administrators, application specialists, systems analysts, and programmers. They are responsible for maintenance of the geographic database and provide technical support. People also need to be educated to make decisions on what type of system to use. People associated with a GIS can be categorized into: viewers, general users, and GIS specialists.

Job level in GIS
GIS Executive Management (e.g. GIO)
GIS Management or Project Management
GIS Developer
GIS Analyst (no programing required)
GIS Specialist
GIS Technician
GIS Intern

5. **Methods:** Method include how the data will be retrieved, input into the system, stored, managed, transformed, analyzed, and finally presented in a final output. The procedures are the steps taken to answer the question needs to be resolved. The ability of a GIS to perform spatial analysis and answer these questions is what differentiates this type of system from any other information systems.

5.2.5 Steps in mapping: These are the key steps involved in producing a GIS map

1. **Base map(Raster topo sheet)Preparation**
2. **Shape file creation**
3. **Georeferencing**
4. **Digitization of map**
5. **Attribute data entry**
6. **Set layer properties**
7. **Symbology**
8. **Analysis**
9. **Report**

5.2.6 Ordinary mapping to GIS

The computer based Geographic Information Technology (GIT) is now widely used for computer-assisted management and analysis of data concerning geographically related feature

The following is a comparison of digital and manual mapping with respect to activity

Activities:	Ordinary mapping	GIS mapping
STORAGE	Different scales on different standards, voluminous and bulky	GIS Database , standardized and integrated, compact memory capacity
RETRIEVAL	Manual check	Quick retrieval by computer
UPDATING	Expensive and time consuming	Systematically done
OVERLAY	Time and energy consuming	Very fast integration of complex, multiple spatial and non spatial data sets
SPATIAL ANALYSIS	Complicated	Easy
DISPLAY	Expensive	Cheap and fast

5.2.7 Comparison of GIS with CAD and other system

CAD	GIS
CAD is a graphics program	GIS is a database program
Design and drawing	Mapping and analyzing geographic features
it's the lines that are important, i.e. the drawing is the important, i.e. the drawing is the information	the lines are just a representation of the data behind it
Static and concrete	Dynamic
CAD modeling things in the real work	GIS models the world itself
The CAD objects such as lines, circle , are , text don't know each other	GIS object know about each other
CAD doesn't have topology	GIS has topology
CAD are usually memory based hence CAD files are typically smaller	GIS systems are usually disk-based can cover model larger area such as regional, state or even world

Topology: Bring the objects (lines, polylines, points etc. together in to logical groups to form real world models

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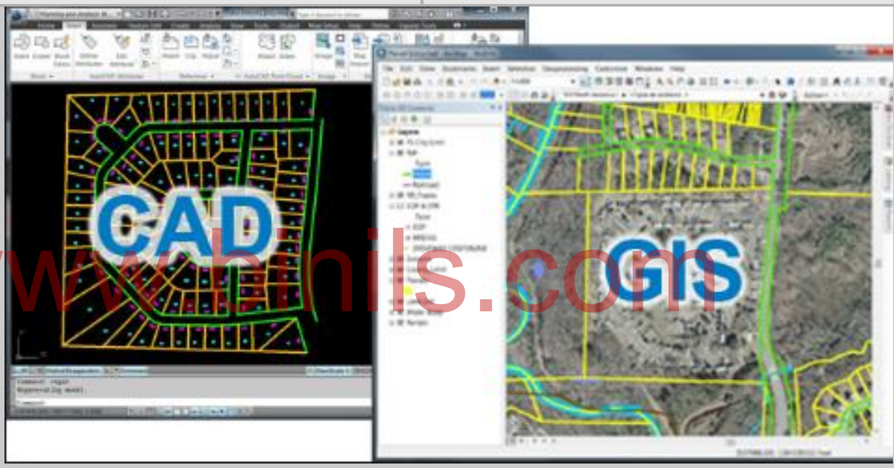
Main difference:

	CAD		GIS	
File format		AutoCAD DWG, Microtstation DGN		Esri shape file, Esri Geodatabase, GML
Mathematical description	Single complex object in 3D(e.g free from cured surface) with high accuracy		Large number of objects in common embedding	
Coordinate system	2D and 3D orthogonal world		Many different coordinate system model the spherical(ellipsoid or geoids) world	
Coverage	Smaller area		Large area (the whole earth)	
Representation	Mainly3D		Mainly 2D	
Time scale	Project basics		Very long period of data collection, and data maintenance(almost an endless lifecycle)	
Tools				

	Advantages	Disadvantages
GIS	•Strong attribute managing ability •Powerful in spatial analyzing	• Relatively week in graphic visualizing and editing
CAD	•Powerful in graphic modeling and editing	•Week attribute managing ability •Lack of spatial analyzing ability

GIS Vs. CAD

Relationship of GIS and other main

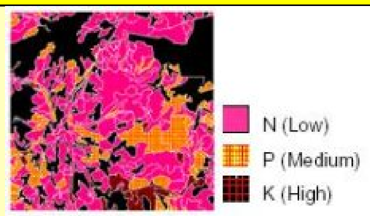
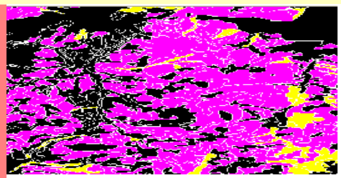
5.2.8 Field Application

Natural resources—Biologists, botanists, ecologists, environmental regulators, hydrologists, planners, petroleum engineers, foresters, and farmers — rely on the analytical power of GIS for help in making critical decisions to manage the earth's resources.

Oil and gas exploration, hydrology harnessing, forest resources, and mining operations require sound assessment using GIS to steer growth into areas that can support it while preventing contamination of rivers or destruction of resources.

Agriculture: GIS application in agriculture has been playing an increasingly important role in crop production throughout the world by helping farmers in increasing production, reducing costs, and managing their land resources more efficiently

Within the study area, several maps are prepared using characteristics of soil. The rang of maps formed in different layers are

<u>Analysis of Fertilizer Consumption:</u> To determine the amount of fertilizer need to put land under a certain crop. The amounts of nitrogen (N), phosphorus (P2O5)) and potassium (K2O) fertilizers required are shown.	 Soil chemical characteristics map
<u>Soil properties:</u> Map showing great soil groups with different soil properties combinations such as Bare Rocks, Outcrops &Debris Overflow Mantle Coastal Dunes Sand Dunes Marsh & Swamps Lands Covered by Permanents Snow Canopy	 Soil physical characteristics map

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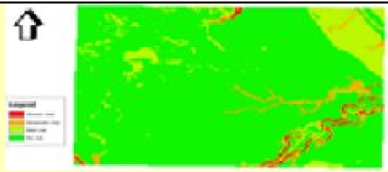
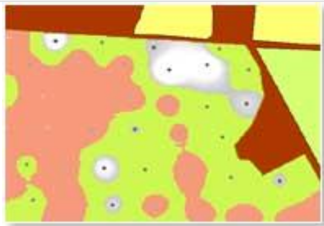
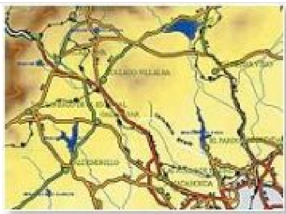
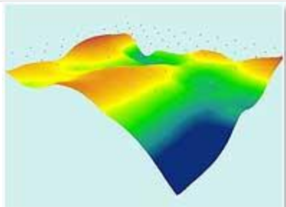
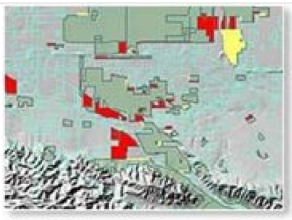
<u>An Erosion Map:</u> showing classified types of erosion of it nature such as Slight, Moderate, Sever and Very sever	 . Soil erosion map

Soil: Soil GIS mapping provides resource information about an area. It helps in understanding soil suitability for various land use activities. It is essential for preventing environmental deterioration associated with misuse of land. GIS Helps to identify soil types in an area and to delineate soil boundaries. It is used for the identification and classification of soil. Soil map is widely used by the farmers in developed countries to retain soil nutrients and earn maximum yield.

Production: Application of phosphorus in a field	
Agribusiness Inputs: Locating existing and potential customers on a map	
Agribusiness Outputs: Tracing efficient transportation route	
Research: Can take preventative measures against erosion	
Government: Can use for rural areas development	

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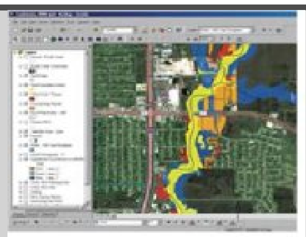
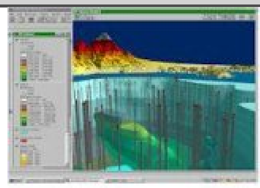
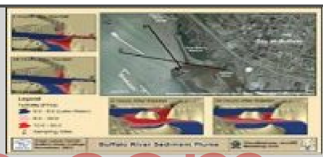
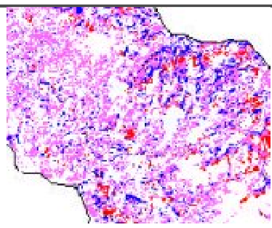
Water resources

GIS is a powerful tool for developing solutions for water resources such as assessing water quality and managing water resources on a local or regional scale.	 Chennai flood
Flood Management: Effective floodplain management for reducing flood damage	
Groundwater: 2D advection–dispersion modelling of groundwater flow and constituents in groundwater	
Water Quality: To monitor water quality changes within a water body	

Wasteland management: Landsat Thematic Mapper and Indian satellite (LISS-II) data were used for detailed mapping (1:50 000 scale). And showing the percentage frequency of wasteland in each state and the relative frequency of categories of wasteland for the districts mapped.

Characters of waste land

- ✓ Gullied land
- ✓ Steep slope land
- ✓ Stone waste Up land with scrub
- ✓ Stone waste Up land with out scrub



satellite imaginary

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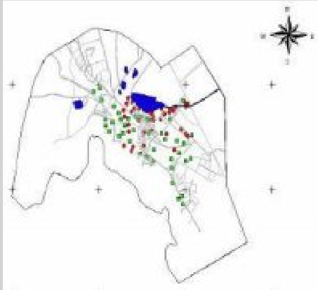
Social resources

GIS modelling with social data such as

- ✓ education,
- ✓ housing patterns,
- ✓ education and
- ✓ health

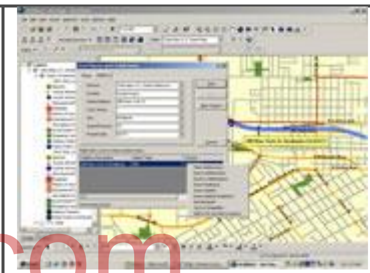
Study on public infrastructure faculties such as

- ❑ water supplies,
- ❑ waste disposal



Cadastral survey and Cadastral records

Accurate, cadastral maps define legal repositories of land ownership, value, and location, allowing individuals and businesses to raise capital based on property values. In turn, the capital can be used to purchase other property, start businesses, and pay for higher education,etc.,

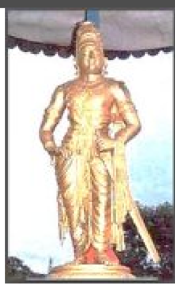


5.2.9 Land Information System(LIS)

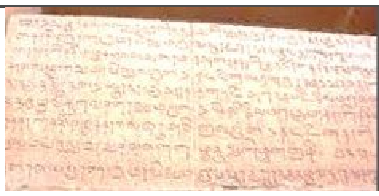
GIS based land acquisition management system will provide complete information about the land. It would help in assessment, payments for private land with owner details, tracking of land allotments and possessions identification and timely resolution of land acquisition related issues.

History of Indian LIS

In India, the land surveys for revenue purposes were made about 1000 years ago during the time of Raja-Raja Cholian the Great, of Chola Empire in Tamil Nadu.



The record of this survey is found inscribed on the North Wall of Raj Rajeshwari Temple in Tanjavur.



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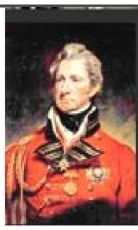
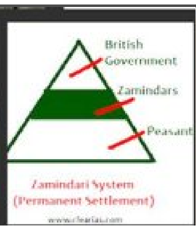
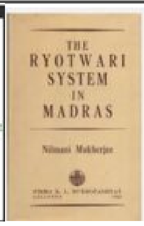
The British, in India started Ryot wari settlement surveys in the Salem District of Ex-Madras Presidency during 1793-1798.



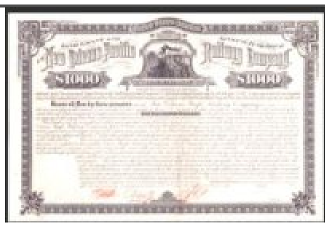




Methods of presentation of the Land information

Photography /aerial or terrestrial.

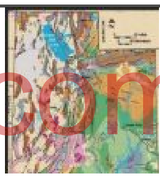
Written records



Computers: Maps may be drawn/scanned with information stored in digital form and then can be retrieved.

Group of Land Information

First: The geological information like shape, size, land forms, minerals and soil.



Second: The economic information like land use, irrigation, crops etc.

Third: The legal rights, registration, and taxation etc.

Class Exercise

A. Fill in the blanks.

1. Total station is also called as ---
2. Total station combined both -----------------------------and --------------------------device
3. Collimator in the total station is used for --
4. Total station is controlled with -----------------------------
5. ----------is the prism constant
6. Circle setting to angle measurements in the total station is made with the help of ----
7. The tripod ---------------------------- and--------------------- the total station.
8. Generally the Prism constant---------------- or -----------------
9. ZA stands for -----------------------
10. Telescopes in the total station of gives a --------------, ------------------------- image
11. Identify the soft key which is used for setting horizontal angle 0^{o} is---------------------
12. GIS is stands for --
13. ------------------------------------is the father of GIS
14. --------------is the file type is used in GIS
15. LIS stands for ---------------------------------

Question bank

B. Answer the following questions in one or two sentence/ One marks questions

1. What is total station
2. Mention any two important parts of a total station
3. Abbreviate the term LCD
4. Name the accessories used in total station surveying
5. Abbreviate the term GIS
6. What is GIS
7. What are the component of GIS
8. Mention any two name GIS software
9. What are the advantages of GIS
10. What is LIS

C. Answer the following questions in short /Three marks questions

1. Mention any three application of total station
2. Write short notes on display unit of a total station
3. Write any three features of tripod used in total station surveying
4. Write short notes on reflector with pole
5. Write short notes on features of total station
6. What are the basics steps involved in linking data files for various application
7. Write short notes on development of GIS
8. Write short notes on hardware components of GIS
9. Mention the steps In GIS mapping in the sequential order
10. Mention the advantages and disadvantages of GIS and CAD
11. Write short notes on application if GIS in Cadastral survey and Cadastral records
12. Write short notes on Land Information System(LIS)

D. Answer the following questions in details/Ten marks questions

1. Draw the neat sketch of total station any mention it salient components
2. Enumerate the various characteristics of total station
3. Draw the neat sketch electronic display of total station a mention the salient keys on it
4. Describe the temporary adjustment of total station before taking observation
5. Describe the step by step procedure for measurement of distance and angle using total station
6. Explain the field procedure for co-ordinate measurement using total station
7. Explain the field procedure to run a traverse survey using total station
8. Explain the various components of GIS
9. Differentiate the between Ordinary mapping to GIS
10. Compare the GIS with CAD and other system
11. Explain the field application GIS in a) Natural resources b)Agriculture and c) Soil
12. Explain the field application GIS in Water resources ,Wasteland management and Social resources

Reference Book

1. Plane surveying by A.M Chandra New Age International Publishers
2. Surveying and Levelling by Rangwalla Charotar Publishing House
3. Surveying Vol 1 by S K Duggal Tata McGraw –Hill Publishing Company Limited
4. Surveying and Levelling by N N Basak Tata McGraw –Hill Publishing Company Limited
5. Surveying Vol-I by Dr. Punmia
6. Advanced Surveying Total station, GIS and Remote sensing by Satheesh Gopi , R.SSathikumar and N. Madhu Pearson
7. Geographic Information computer by Narayan Panigrahi Universities press