MODULE 1

INTRODUCTION TO SURVEYING

Surveying is the art of making measurements of objects on, above or beneath the ground to show their relative positions on paper. The relative position required is either horizontal or vertical.

APPLICATIONS OF SURVEYING

Some of the important applications of surveying are listed below:

1. Astronomical survey helps in the study of astronomical movements of planets and for calculating local standard times.

2. Maps prepared for countries, states and districts, etc. avoid disputes.

3. Plans prepared record the property boundaries of private, public and government which help in avoiding unnecessary controversies.

4. Topographical maps showing natural features like rivers, streams, hills, forests help in planning irrigation projects and flood control measures.

5. Road maps help travelers and tourists to plan their programmers.

6. Locality plan help in identifying location of houses and offices in the area

7. Maps and plans help in planning and estimating various transportation projects like roads, bridges, railways and airports.

8. For planning and executing water supply and sanitary projects one has to go for surveying first.

9. Marine and hydrographic surveys help in planning navigation routes and harbours.

10. For making final payments in large projects surveying is to be carried out

11. Military surveys help in strategic planning

12. For exploring mineral wealth mine surveys are required.

13. Geological surveys are necessary for determining different strata in the earth's crust so that proper location is found for reservoirs.

14. Archaeological surveys are required for unearthing relics of antiquity.

PRIMARY DIVISIONS IN SURVEYING

The survey in which earth's curvature is considered is called geodetic surveying and the survey in which earth's curvature is neglected is called Plane surveying.

CLASSIFICATION OF SURVEYING

Surveying may be classified based on the following three points:

1. Natural of the field of survey

2. Objects of survey

3. Instrument used

4. The methods employed

Classification Based on Nature of the Field of Survey

On this basis field of survey may be classified as land survey. Marine or hydraulic survey and astronomical survey.

Land survey: It involves measurement of various objects on land. This type of survey may be further classified as given below:

i.   Topographic surveys: They consist of measurement of various points to plot natural features such as rivers, streams, lakes, hill and forests as well as man – made features like roads, railways, towns, villages and canals.

ii. Cadastral survey: These surveys are for marking boundaries of municipalities, states, etc. the surveys made to mark properties of individual also come under this category.

iii. City survey: The surveys made in connection with the construction of streets, water supply and sewage lines fall under this category.

Marine of Hydrographic Surveys: The survey conducted to find depth of water at various points in bodies of water like sea, river and lakes fall under this category of surveying. Finding depth of water at specified points is known as soundings.

Astronomical Surveys: Observations made to heavenly bodies like sun and stars to locate absolute position of points on the earth and for the purpose of calculating local times is known as astronomical survey.

Classification Based on Object of Surveying

On the basis of objective of surveying, the classification can be as engineering survey. Military survey, mines survey, geological survey and archaeological survey.

1. **Engineering survey:** The objective of this type of surveying is to collect data for designing roads, railways, irrigation, water supply and sewage disposal projects. These surveys may be further subdivided into:

   a. Reconnaissance survey for determining feasibility ad estimation of the scheme.

   b. Preliminary survey for collecting more information to estimate the cost o the project selected, and

   c. Location survey to set the work on the ground.

2. **Military Survey:** This survey is meant for working out points of strategic importance.

3. **Mine survey:** This is used for exploring mineral wealth.

4. **Geological survey**: this survey is for finding different strata in the earth's crust.

5. **Archaeological survey**: this survey is for unearthing relics of antiquity.

Based on the instruments used, surveying may be classified into the following:

   1. Chain Survey

   2. Compass Survey

   3. Plane Table Survey

   4. Theodolite Survey

   5. Tacheometric Survey

   6. Modern Survey using electronic equipment like distance metres and total stations.

7. Photographic and Aerial Survey.

Classification Based on the Methods Employed

Based on the methods employed, surveying may be classified as triangulation and traversing.

1. Triangulation: In this method control points are established through a network of triangles

2. Traversing: In this scheme of control points consist of a series of connected points established through linear and angular measurements. If last line meets the starting point it is called as closed traverse. If it does not meet, it is known as open traverse.

MEASUREMENTS

Linear measurements are horizontal or vertical only. Here angular measurements are also involved. Commonly used linear units in surveying are kilometre, metre and millimetres. For measurement of angles sexagesimal system is used. In this 1 circumference = 360 degrees

SCALES

It is not possible and also not desirable to make maps to full scale. All distances are reduced by fixed proportion and drawings are made. The scale of a map or the drawing is the fixed proportion which every distance on the map bears to he corresponding distance on the ground. Thus, if 1 mm on the paper represents 1m on the ground, then the scale is 1 mm = 1 m (or 1 cm = 10m or 1: 1000.

To make scale independent of units it is preferable to use representative factor, which is defined as the ratio of distance of one unit on paper to one unit on ground. Thus, 1mm = 1m is equivalent to RF=1/1000.

Plain Scale: On a plain scale it is possible to read two dimensions directly such as unit and tenths.

Diagonal Scale: In plain scales only units and tenths could be shown whereas in diagonal scales it is possible to show units, tenths and hundredths. Units and tenths are shown as in plain scale. To show hundredths, principle of similar triangles is used

PRINCIPLES OF SURVEYING

To get accurate results one should follow the two basic principles explained below:

1. **Work from whole to part**

In surveying large areas, a system of control points is identified and they are located with high precision. Then secondary control points are located using less precise methods. With respect the secondary control point's details of the localized areas are measured and plotted. This is called working from whole t part. This principle in surveying helps in localizing the errors. If the surveying is carried out by adding localized areas, errors accumulate.

2. **Fixing positions of new control points**

For fixing new control points with respect to already fixed points, at least two independent processes should be followed. IF A and B are two already located control points and with respect to them new control point C is to be located, apart from the minimum two measurements required, one more reading should be taken. Fixing of check lines and tie lines will also serve this purpose.

SURVEY OF INDIA AND TOPOLOGICAL MAPS

The survey of India is the oldest scientific department of Government of India. It was established in 1767 by the East India Company which was ruling India at that time. It works

under the Department of Science and technology. It is assigned the role of a principal mapping agency of the country. The survey of India ensures that the countries domain is explored and mapped suitably and provides base maps for expeditions and integrated development.

Bit by bit of Indian terrain was completed y pains taking efforts of batches of surveyors appointed by East India Company. Efforts of batches lead by Lambton and Sir George Everest are noteworthy. The topological maps prepared by the survey of India are continuously updated adding more features and more precision by using better equipment and mapping techniques. The maps prepared meet the needs of defense forces, planners and the scientists in the field of geosciences, land and resource management.

The survey of India had five directorates in 1950. Presently the number has grown t eighteen.

The topographical maps show details of natural features like roads, railways, towns villages and canals. They also show contour lines and position of Great Trigonometric survey benchmarks. One can purchase these topographic maps from the survey of Indian by contacting survey or Generals office, PB No 37, Dehra Dun – 248001

Numbering of Topo Maps of India

The entire area covered by India is divided into A $4^0 * 4^0$ longitude and latitude and each grid is numbered as shown in Fig.1. Each grid is further divided in 4 * 4 grid of size $1^0 * 1^0$ longitude and latitude and they are numbered as shown in Fig 2.

The scale used for $4^0 * 4^0$ grid map is 1:25000 and the scale used for $1^0 * 1^0$ grid maps is 1:50,000 the $1^0 * 1^0$ longitudinal nad lateral grids are further divided in 15' * 15' grids and are numbered. These maps are available in 1:50,000 to 1:25000 scales. A map corresponding to 55th A of 6th grid is referred to as NH 55 A – 6, where NH refers to Northern Hemisphere
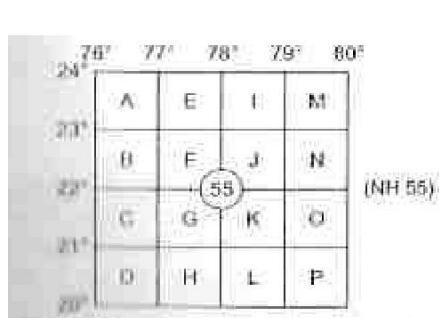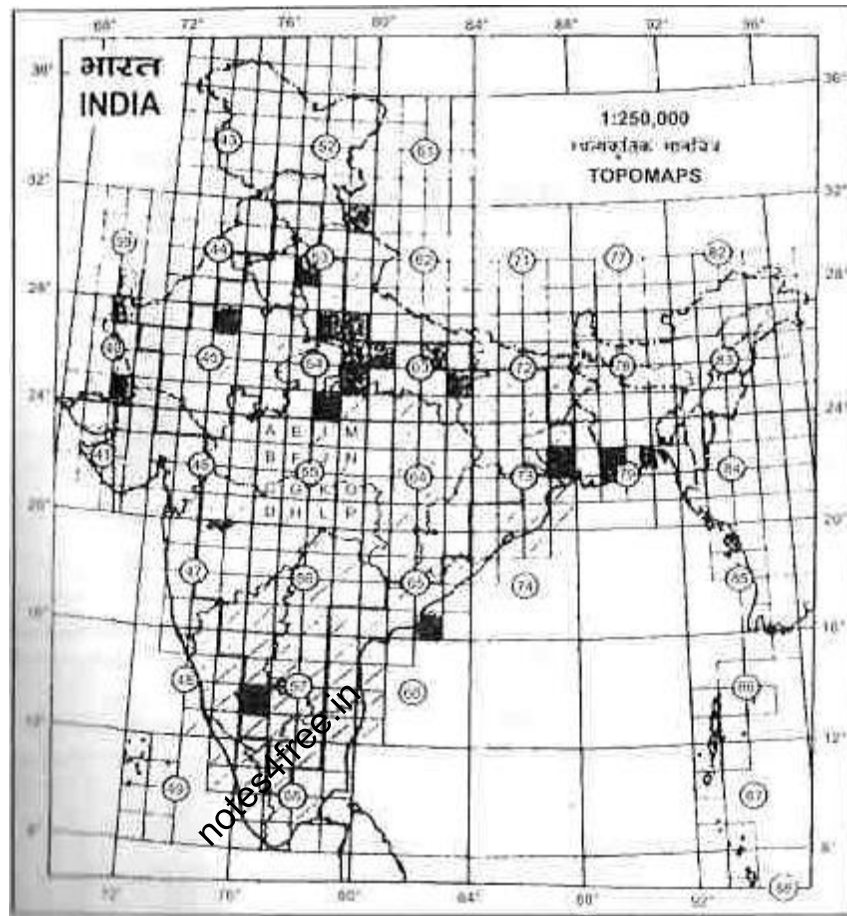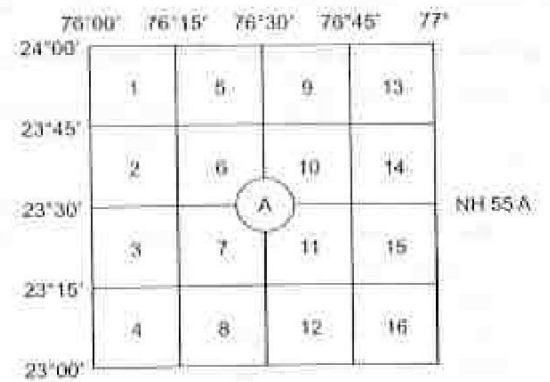
Fig 1 - Grid Topomap

Fig 2 – Grid Topomap

PHASES OF WORKS IN SURVEYING

Survey work has the following phases:

1. Planning

2. Care and Adjustment of Instruments

3. Field work, and

4. Office work

ERRORS IN SURVEYING

TYPES OF ERRORS:

The errors which creep in surveying may be classified into the following three:

1. Mistakes

2. Systematic errors

3. Accidental errors

Mistakes: Mistakes are the errors due to carelessness of the observer. They may be due to wrong reading or recording of the observations. These errors are very large and can be easily detected by the following field procedures:

a) Carefully targeting objects before taking reading

b) Taking multiple scale readings

c) Recorded loudly announcing the readings so that reader hears what he records.

d) Taking additional readings for checking.

Systematic errors: The errors which follow a well – defined pattern are classified as systematic errors. They can be determined by mathematical expressions. They are regarded as positive, if they make result too great and as negative if they make result too small. Examples of such errors are use of a tape which is shorter than the actual as per marking or using a steel tape at a temperature different from calibrated temperature. If tape is short, makes each measured length longer, hence contributes posit6ive error. FI the actual length of the tape is determined actual measured length can be calculated.This type of errors is called cumulative errors, since each measurement adds to the error in the same sense.

Accidental errors: There are errors in measurements which cannot be prevented, even with sufficient care. These errors may be positive or negative their magnitude may vary from reading to reading for example taking a reading with a survey instrument Human eye has a limitation of distinguishing between two close readings. Marking the end of a chain length is another common example of accidental error.

The thickness of marking and its exact position contribute to accidental errors. These errors ae not deterministic they are probabilistic hence they cannot be estimated using standard functional relations. However, using laws of probability they may be accounted satisfactorily.

SOURCES OF ERRORS

Errors may arise from the following sources:

1. Instrumental errors
2. Natural errors
3. Human limitations
4. Carelessness

Instrumental errors: Instruments used for linear measurements may not be having true length due to manufacturing defects and instruments may not show true horizontal and vertical angles due to manufacturing defects or out of adjustments.There are limitations on the scales used which contribute to instrumental errors.

Natural errors: Errors will creep in because of the natural phenomena like variation in temperature humidity refraction, curvature of the earth and magnetic declination. They are to be properly accounted to arrive at exact values.

Human limitations: Human eye cannot distinguish between two points closer than 0.25 mm. when ends of a chain/tape line is marked, the thickness of line contributes to error, when next length is measured.

Carelessness: These errors are purely due to the mistakes. They are quite large. They can be avoided by following good surveying practice by taking precautions and check readings.

MOST PROBABLE VALUE OF ACCIDENTAL ERROR

Though accidental errors are unpredictable, the following features of these errors are observed:

a) Positive and negative errors will occur with equal frequency

b) Small errors occur more frequently

c) Very large errors do not occur.

This type of error distribution is called normal distribution. Gives two such distributions. In both frequency of occurrence of error is high when error is very little, positive and negative errors occur with equal frequency and very large errors occur rarely.

MEASUREMENT OF HORIZONTAL DISTANCES

APPROXIMATE METHODS OF DISTANCE MEASUREMENTS

These methods are used in reconnaissance surveys or to detect major mistakes. They give better results on smooth roads; error can be within I per cent. These approximate methods of direct measurements are listed below:

1. Pacing

2. Measurement with passometer

3. Measurement with pedometer

4. Measurement with odometer

5. Measurement with speedometer

PACING: The surveyor walks along the line to be measured and counts number of steps. Then the distance measured is equal to no. of steps * average length of a step. Average length of a step can be found by walking along a known length. A normal man takes a step of length 0.75m.

PASSOMETER: A passometer is a watch – like instrument which should be carried vertically in the shirt pocket or tied to a leg. Mechanism of the instrument gets operated by the motion of the body and records number of paces. Thus, the problem of counting paces is eliminated.

PEDOMETER: It is a instrument similar to passometer, but it records the distances instead of paces. In this before walking zero setting is made and length of pace is set depending upon the person.

ODOMETER: It is an instrument which is attached to the wheel of a cycle or other vehicle. It records number of revolutions made by the wheel. Knowing the circumference of the wheel, the distance travelled may be found.

SPEEDOMETER: Odometer may be calibrated to give distance directly, if it is used for a particular vehicle. This is called speedometer.

TAPES

Tapes are used for measuring lines and offsets and are classified depending on the materials used as:

1. Cloth or linen tape

2. Metallic tape

3. Steel tape and

4. Invar tape.

Cloth or linen tape: 12 to 15 mm wide cloth or linen is varnished to resist moisture and graduations are marked. They are provided with brass handle at the ends. End to end length of brass handles is the total length of tape. They are available in the length of 10 m, 20 m, 25 m and 30 m, these tapes are light and flexible and hence easy to handle. However because of the following disadvantages. They are not popular is use:

1. Due to moisture or dampness they shrink

2. Extend due to stretching

3. Not strong

4. Likely to twist and tangle

Metallic tape: These are made up of varnished strip of waterproof linen interwoven with small wires of brass, copper or bronze. They are provided with handle at the end. About 100 m lengths to tapes are provided with leather or suitable strong plastic materials. Tapes of length 10 m, 20 m, 30 m and 50 m are available in a case of leather or corrosion resistant metal fitted with a winding device. On one side of tape markings are made to indicate distance from the end of handle. Red and black coloured markings are used for indicating full metres and its fractions in centimeters.

Steel tape: Steel tape consists of 6 to 10 mm wide strip with metal ring at free end and wound in well sewn lealher or a corrosion resistant metal case. A suitable winding device is provided. The tapes are marked legibly on one side only indicating 5 mm, centimeters, decimeters and metres clearly. The end 10 cm length is marked with millimeters also. The tapes are available in 1 m, 2 m , 10 m, 2 0 m, 30 m, and 50 m lengths.

Steel tapes are superior to a metallic tape as for as accuracy is concerned, however, they are delicate. Care should be taken to wipe the tape clean before winding. They should be oiled regularly to prevent corrosion.

Invar tape: It is made up of an alloy of nickel (36%) and steel, which has very low coefficient of thermal expansion. The width of the tape is 6 mm. it is available in 30 m, 50 m and 100 m lengths.

It is the most accurate tape but is expensive. It is delicate and hence should be handled with care. It undergoes change in length due to continuous use, which is known, as creep of the material. Hence, it is necessary to ascertain its true length, if it is old. This tape is used for base line measurement in surveying.

ACCESSORIES REQUIRED FOR HORIZONTAL MEASUREMENTS.

1. ARROWS: When the length of the line to be measured is more than chain length, there is need to mark end of a chain length,. Arrows are used for this purpose. They are made of 4 mm diameter tempered steel wire with one end sharpened and other end bent into a loop.

2. PEGS: To mark the station points wooden pegs are used they are made of hard wood of 25 mm *

25 mm section. 150 mm long with a tapered. When driven in ground they project to about 40 mm.

3. RANGING RODS: For ranging intermediate points in measuring 2 to 3 m long rods are used. They are made of hard wood and are provided with an iron shoe at one end.

The rods are usually circular in section with 30 mm diameter. They are painted with 200 mm colour bands of red and white or with black and white. Sometimes they are provided with black, red and white in succession. They are easily visible up to a distance of 200 m. if distance is more they are provided with 200 mm. square multicolored flags at their top. Since they are painted with alternate colours of band 200 mm, they may be used for rough measurements of short distances also.

4. RANGING POLES: Ranging poles are similar to ranging rods except that they are longer. They are 4 m to 8 m long and their diameter varies from 60 mm to 100 mm. they are made up of hard wood or steel. They are fixed in the ground by making 0.5 m holes and then packed to keep the pole vertical. They are provided with larger flags at their top.

5.OFFSET RODS: These rods are also similar to ranging rods, 3 m long. They are made up of hardwood and are provided with an iron shoe at one end. A hook or a notch is provided at other end. Apart from two narrow slits at right angle to each other provided at height of the eye. The hook helps to pull chain through bushes. The slits help in aligning offset lines which are to be at right angles to the main line. The coloured bands on the rod are useful for measuring offsets of short length.

6. LATHS: Laths are 0.5 m to 1.0 m long sticks of soft wood. They are sharpened at one end. They are provided with white or light colours. They are used as intermediate points while ranging long lines or while crossing depressions.

7. WHITES: Whites are the pieces of sharpened thick sticks cut from the nearest in the field. One end of stick is sharpened and the other end is split. White papers are inserted in the split. The whites are used for the same purpose as laths.

8. PLUMB BOB: In measuring horizontal distances along sloping ground plumb bobs are required to transfer the points to ground. They are also used to check the verticality of ranging poles.

9.LINE RANGER: It is an optical instrument used for locating a point on a line. It consists of two isosceles prisms placed one over the other and fixed in an instrument with handle. The diagonals of the prisms are silvered so as to reflect the rays. Referring to Fig (a) AB is a line and it is intended to locate point C on it. The surveyor holds the instrument in hand stands near point selected as the desired point by observation. If the position of the observer is not exactly on the line AB, ranging rods at A and B appear separated as shown in Fig (b) the surveyor moves to and fro at right angles to the line AB till the images of ranging rods at A and B appear in a single line as shown in Fig(c). It happens only when the optical square is exactly online AB. Thus, the desired point is located. It needs only one person for ranging. The line ranger should be tested occasionally for its accuracy. For this a point should be located between the two test points. Then line ranger is held in this position and tested. If the images of the two ranging rods do not appear in the same line, one of the prisms is adjusted by operating the screw till the two images appear in the same vertical lines.
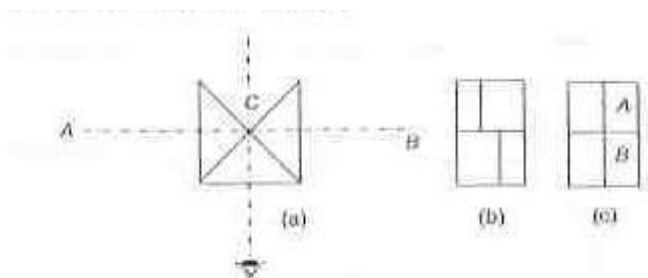
RANGING A SURVEY LINE

When survey line is longer than a chain length, it is necessary to align intermediate points on survey line. The process of locating intermediate points on survey line is known as ranging. The methods of ranging are classified as direct ranging and indirect ranging.

Direct ranging: This is possible. If the first and last points on the survey line are intervisible. Fig. shows the end points A, B in a survey line which is intervisible. Now it is necessary to locate point C on line AB, which is slightly less than a chain length from A. It needs two persons. At points A and B ranging rods are erected. The assistant of survey positions himself as close to line AB as possible at a distance slightly less than a chain length and hold a ranging rod. The survey or positions himself approximately 2 m behind A and sights ranging rods at A and B. He directs the assistant to move to the left or right of line AB till he finds the ranging rods at A,B and C in a line. The surveyor should always observe at lower portion of the ranging rods. The signals used in instructing the assistant at C while ranging.

(a) Plan view

(b) Sectional view
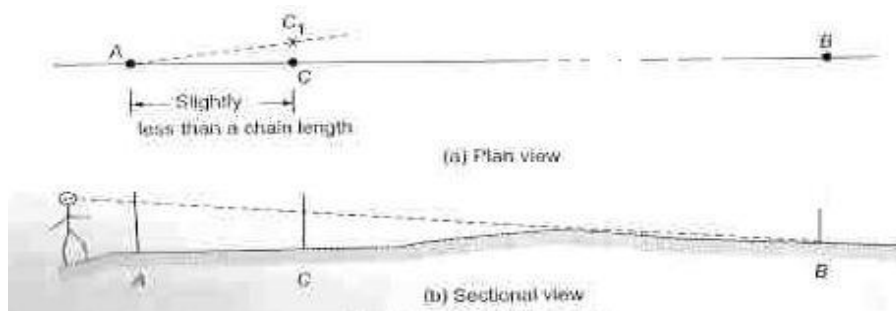
Indirect ranging: If the two end points of the line to be measured are not intervisible, the surveyor has to go for indirect ranging. This is also called reciprocal ranging. The invisibility of points may be due to unevenness of the ground or due to long distance Fig (a) shows cross – section of the ground which is a typical case of invisibility of point B of the line from point A. Fig (b) shows the plan .M and N are the two points to be fixed or AB such that both points are visible from A as well as B. It needs four people to fix points M and N one person near each point A, B, M and N.
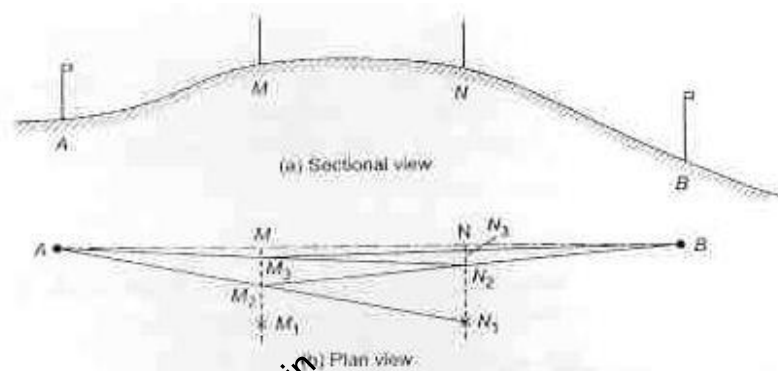
The persons at M and N position themselves near M and N say at M1 and N1. First person at A directs the person at M to come to M2 so that AM2N1 are in a line. Then person at B directs the person at N1 to move to N2 so that BN1M2 are in a line. In the next cycle again person at A directs the person to M to move to M3 such that AM3N2 are in a line which is followed by directing person at N2 to move to N3 by person at B. the process continues till AM NB

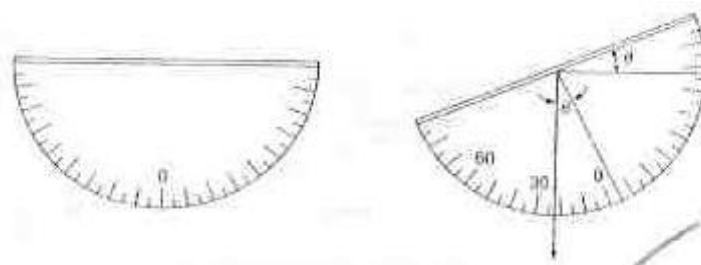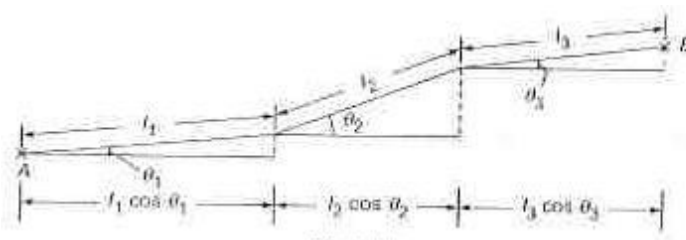**MEASUREMENT OF DISTANCES ON SLOPING GROUND**

In surveying horizontal distances are required. If the ground is sloping there are two methods to get horizontal distances:

1. Direct method

2. Indirect method.

Direct method: This method is known as method of stepping also, since the line is measured in smaller step length. Let AB be the length of line to be measured on a sloping ground the surveyor holds the tape firmly at A and the leader goes with a convenient length l1 of tape say, 5 m, 10 m, 15 m, and a ranging rod in hand. After ranging, the leader holds the chain horizontally. He may be guided by the surveyor or others in the party for horizontality of the tape. After stretching the tape, with the help of a plumb bob or by dropping a pebble, the leader transfers the end of the tape to the ground and marks. The length of te tape selected is such that the drop is never more than the eyesight of the leader. The length l1 is noted and they move to measure next step length. The two step lengths need not be the same. The procedure continues till the total length is measured. It is preferable to measure down the slope rather than up the slope, since the surveyor can hold the tape firmly, if the measurements are down the hill. In this method tape is preferred over chain since it is light and hence can be stretched horizontally, keeping sag at minimum.

Indirect method: If the slope of the ground is gentle these methods may be employed. In these methods linear measurement is along the sloping ground and it involves angular measurement also. The following three methods are in common use:
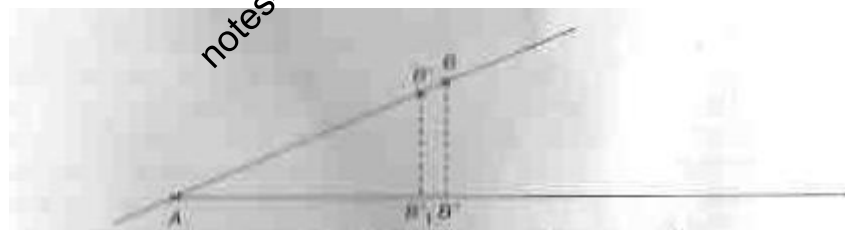
a) First method: Total length to be divide into each segment having particular slope. **D=Σlcosθ**

b) Second method: The difference in level 'h' is measured by knowing the sloping ground length 'l' and the equivalent horizontal length L can be calculated

$$L = \sqrt{l^2 - h^2}$$
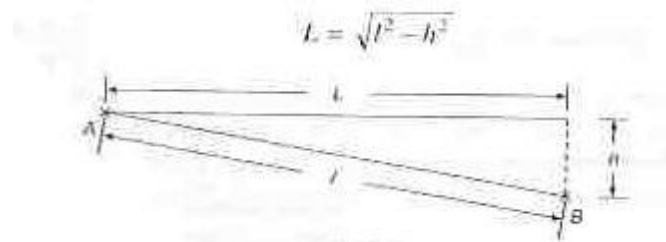
c) Third method: This method is useful when intermediate points on a line are to be used for taking offsets.

PRECISE MEASUREMENT / BASE LINE MEASUREMENT

A base line is an important line in the skeleton of triangulation used for preparing maps. In preparing a map normally this is the first line to be drawn over which the other lines are drawn to form triangular skeleton. Then with respect to the secondary lines other details are filled up. The base line is to be measured more precisely to minimize the errors in surveying.

For the measurement of base line steel tapes are used and the care is taken to check the length of tape frequently; force applied in stretching tape is measured; horizontality of the line is ensured and temperature is recorded so that necessary corrections can be applied. The instruments used by various persons may differ slightly, but basic method of baseline

measurement is as given below.

Always three standard tapes are used for measurement and the other two for checking the true length of the tape used. The tape is placed over rear and forward stakes which are provided with zinc strips at their top. Straining devices are provided with spring balances to measure the force applied on the tape while measuring. Intermittent stakes are used to support the tape so that sag is reduced. The elevations of top of all stakes are adjusted so that they are at the same level. Six thermometers are used for measuring the temperature and two for checking the thermometers used.

TAPE CORRECTIONS

The following five corrections may be calculated for the measured length of chain or tape:

1. Correction for absolute length
2. Correction for slope
3. Correction for temperature
4. Correction for pull, and
5. Correction for sag

CORRECTION FOR ABSOLUTE LENGTH

Let,   l = designated length of tape

               la = absolute length of the

tape Then correction per chain length

$$c = la - l$$

Hence, if the total length measured is L, the
               correction is $Ca = L\ c/l$

If absolute length of tape la is greater, correction is +ve and if negative, the correction is also negative. Thus correct length L' is given by

$$L'=L+Ca$$

If A is the measured area with incorrect tape, the correct area is given by

$$A'=A(1+2c)$$

CORRECTION FOR SLOPE

If length measured 'L' and the difference in the levels of first and last point 'h' are given then correction for slope is,

$$Csl=h^2/2L$$

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If θ and L are given, **Csl=L(1-cosθ)** This correction is always subtractive.

CORRECTION FOR TEMPERATURE

Let   α- Coefficient of thermal expansion of the material of tape

        Tm – Mean temperature during measurement

        To - Temperature at which tape is standardized, and

        L – Measured length

Then temperature correction Ct is given by **Ct=Lα(Tm-To)**

CORRECTION FOR PULL

Let,   E – Young's modulus of the material of tape

A – Cross – sectional area of the tape

P – Pull applied during measurement

P0 - Standard pull, and

L – Measured length of chain

Then, the correction for pull Cp is given by **Cp=(P-P0)L/AE**

The above expression takes care of signs of the correction also.

CORRECTION OF SAG

While taking reading, if the tape is suspended between two supports, the tape sags under its own weight as shown in Fig. 3.18. The shape of tape is a catenary. Hence, measured length is more than the actual length. Hence, this correction is subtractive. This correction is given by

$$Cs=1/24(W/P)L$$

Where, W – the weight of the tape per span length

P – the pull applied during the measurement

L – Measured length.

If pull is larger than standard pull, the correction is +ve and , correction for sag is always negative. The pull for which these two corrections neutralize each other is called Normal tension.

PROBLEMS

Example 1

A distance of 2000 m was measured by a 30 m chain. After the measurement, the chain was found to be 10 cm longer. It was found to be 15 cm longer after another 500 m was measured. If the length of the chain was correct before the measurement, determine the exact length of the whole measurement.

Solution : For first 2000m length:

Average correction per chain length= (0+10)/2= 0.05

Correction for measured length

Ca = L c/l= 2000*0.05/30= 3.33m

True length = 2000+3.33 = 2003.33 m

For the next 500 m length:

Average correction =(10+15)/2 =0.125m

Correction for measured length = 500*0.125/30 =2.08m

True length = 500 + 2.08 =502.08 m

Exact length of the whole line = 2003.33+502.08=2505.41 m

Example 2

The length of a survey line when measured with a chain of20 m, nominal length was found to be 841.5m when compared with a standard it was found to be 0.1 m too long. Compute the correct length of the line.

Soltuion: Correction for chain length = 0.1 m

Measured length L = 841.5

Nominal length of chain = 20 m

Ca=841.5*0.1/20 =4.21

Actual length of line =841.5+4.21=845.71m

SOLVED QUESTION AND ANSWERS

1 **a) Distinguish between the following** (June-july 2011, Dec2011)

i) Plane surveying: curvature of earth is not taken into account small areas.

Geoditic survey: curvature of earth is taken into account large areas.

ii) Precision: Consistency with repetition

Accuracy: nearness to true value

iii) Systematic error: Reason for error known and correction can be computed. + or –

Random error: reason not known error will be + as well as – ve – probability method.

iv) Instrumental error: Instrument not in adjustment

Personal error: error in observations.

2. Discuss the classification of surveying (Dec-2012)

1. Engineering survey: The objective of this type of surveying is to collect data for designing roads, railways, irrigation, water supply and sewage disposal projects. These surveys may be

further subdivided into:

a. Reconnaissance survey for determining feasibility ad estimation of the scheme.

b. Preliminary survey for collecting more information to estimate the cost o the project

selected, and

c. Location survey to set the work on the ground.

2. Military Survey: This survey is meant for working out points of strategic importance.

3. Mine survey: This is used for exploring mineral wealth.

4. Geological survey: this survey is for finding different strata in the earth's crust.

5. Archaeological survey: this survey is for unearthing relics of antiquity.

Based on the instruments used, surveying may be classified into the following:

1. Chain Survey

2. Compass Survey

3. Plane Table Survey

4. Theodolite Survey

5. Tacheometric Survey

6. Modern Survey using electronic equipment like distance metres and total stations.

7. Photographic and Aerial Survey.

3. **Explain briefly how the maps are numbered by survey of India.**(june-july 2011

&Dec2011)

The entire area covered by India is divided into A 40 * 40 longitude and latitude and each grid is

numbered as shown in Fig.1. Each grid is further divided in 4 * 4 grid of size 10 *10 longitude

and

latitude and they are numbered as shown in Fig 2.

The scale used for 40 * 40 grid map is 1:25000 and the scale used for 10 *10 grid maps is

1:50,000

the 10 *10 longitudinal nad lateral grids are further divided in 15' * 15' grids and are numbered.

These maps are available in 1:50,000 to 1:25000 scales. A map corresponding to 55th A of 6th

grid is

referred to as NH 55 A – 6, where NH refers to Northern Hemisphere

1.Explain the principles of surveying (Dec-2012 ,,June-july 2011)

To get accurate results one should follow the two basic principles explained below:

1. **Work from whole to part**

In surveying large areas, a system of control points is identified and they are located with high precision. Then secondary control points are located using less precise methods. With respect the secondary control point's details of the localized areas are measured and plotted. This is called working from whole t part. This principle in surveying helps in localizing the errors. If the surveying is carried out by adding localized areas, errors accumulate.

2. **Fixing positions of new control points**

For fixing new control points with respect to already fixed points, at least two independent processes should be followed. IF A and B are two already located control points and with respect to them new control point C is to be located, apart from the minimum two measurements required, one more reading should be taken. Fixing of check lines and tie lines will also serve this purpose.

Problems (Dec-2012.June-July2011)

1. The distance between two points measured along a slope is 800 m. Find the distance between the points if,

i) The difference in level between the points is 60 m.

ii) The angle of slope between the points is 10° (06 Marks)

L = distance measured along slope = 800 m

H = difference in level between two points= 60 m

$l2 - h2 = (800)2 - (60)2$

$D = 787.84m$

Q= angle up slope = 100

L = distance measured = 800 m along slope

Horizontal distance = D = l cos q

= 800 cos 10'

2. Explain the basic principle of EDM devices.(June-July 2011)

Positions are a fundamental element of geographic data. Sets of positions form features, . Positions are produced by acts of measurement, which are susceptible to human, environmental, and instrument errors. Measurement errors cannot be eliminated, but systematic errors can be estimated, and compensated for. Land surveyors use specialized instruments to measure angles and distances, from which they calculate horizontal and vertical positions. The Global Positioning System (and to a potentially greater extent, the emerging Global Navigation Satellite System) enables both surveyors and ordinary citizens to determine positions by measuring distances to three or more Earth-orbiting satellites. As you've read in this chapter (and may known from personal experience), GPS technology now rivals electro-optical positioning devices (i.e., "total stations" that combine optical

angle measurement and electronic distance measurement instruments) in both cost and performance. This raises the question, "If survey-grade GPS receivers can produce point data with sub-centimeter accuracy, why are electro-optical positioning devices still so widely used?" In surveying horizontal distances are required. If the ground is sloping there are two methods to get

horizontal distances:

1. Direct method

2. Indirect method.

Direct method: This method is known as method of stepping also, since the line is measured in smaller step length. Let AB be the length of line to be measured on a sloping ground the surveyor holds the tape firmly at A and the leader goes with a convenient length I of tape say, 5 m, 10 m, 15 m, and a ranging rod in hand. After ranging, the leader holds the chain horizontally. He may be guided by the surveyor or others in the party for horizontality of the tape. After stretching the tape, with the help of a plumb bob or by dropping a pebble, the leader transfers the end of the tape to the ground and marks. The length of te tape selected is such that the drop is never more than the

eyesight of the leader. The length I is noted and they move to measure next step length. The two step lengths need not be the same. The procedure continues till the total length is measured. It is preferable to measure down the slope rather than up the slope, since the surveyor can hold the tape firmly, if the measurements are down the hill. In this method tape is preferred over chain since it is light and hence can be stretched horizontally, keeping sag at minimum.

`Indirect method`: If the slope of the ground is gentle these methods may be employed. In these methods linear measurement is along the sloping ground and it involves angular measurement also.

The following three methods are in common use:

a) First method: Total length to be divide into each segment having particular slope.

$D = \Sigma l \cos \theta$

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MODULE 2

MEASUREMENT OF DIRECTIONS AND ANGLES

COMPASS SURVEY

Compass survey:

Compass survey is used to survey an area in which network of lines starts from a point, goes around the area and ends at the same point. This is called closed traverse.

If the road project or canal project starts surveying goes along many interconnected lines and ends at some other point called open traverse.

The direction of a survey line may be defined by

1)Horizontal angle between the line and adjacent to it or

2)The angle between a reference line called meridian and the survey line. The reference line is called meridian and the angle between the line and the meridian is called bearing.

The direction of a survey line can either be established with relation to each other or with relation to any meridian.The first will give angle between two lines. The second will give the bearing of the line.

The common instruments used for direction measurements are prismatic and surveyor's compass.

The common instruments used for angle measurements are theodolite and sextant.

COMPASS:

A compass consist of

i) A magnetic Niddle

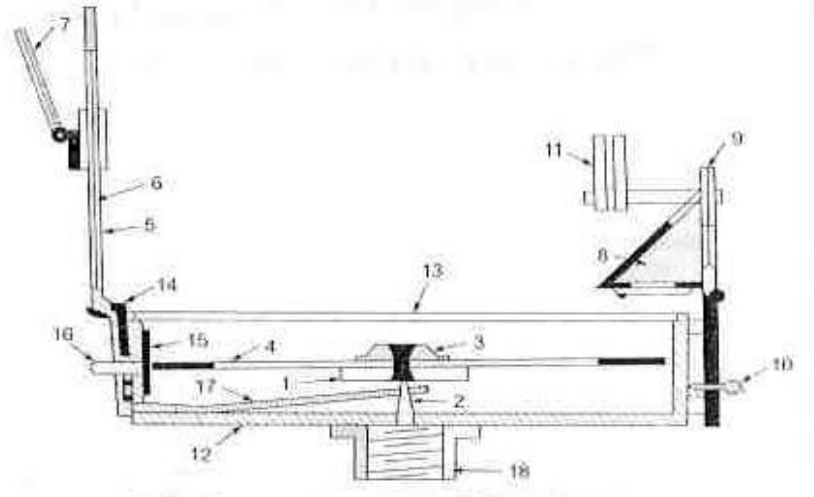ii) A graduated circle

iii) The line of site

iv) Box house the above

The two forms of compass that all used commonly for angle measurement

1. Prismatic compass

2. Surveyor's compass

1**. Prismatic Compass:**

Parts of Prismatic compass:

1. Box

2. Needle

3. Graduated circle

4. Object vane

5. Eye vane

6. Prism

7. Prism cap

8. Glass cover

9. Lifting pin

10. Lifting lever

11. Break pin

12. Spring break

13. Mirror

14. Pivot

15. light spring

16. Agate cap

17. Focusing stud

18. Dark sun Glasses.

Details of instrument

1)Accuracy of a magnetic compass depends upon how much freely the needle is supported on pivot. The top of the pointed pivot is protected with agte cap.

2)An aluminum graduated disc is fixed to the top of the needle. The graduation are from $0 - 360^0$ in clockwise direction when read from top.The north direction is treated as 0^0 east as 90^0 south as 180^0 and west as 270^0.The graduation are written inverted because they are sighted through a prism

3)The line of sight consists of object unit & reading unit.

4)Object unit consist of a slit metal frame hinged to the box. The slit carries centrally horse

hair or fine wire.

5) The metal frame is provided with a hinged mirror which can be placed upward or downward on the frame.It can be adjusted so that the reflection of the objects too high & too low can be sighted.

6)Diametrically opposite to this unit, a reading unit is provided.

7)It consists of reflecting prism with a sighting eye vane.

8)The prism magnifies the readings on the graduation disc below it for the purpose of focusing the prism can be rised or lowered on the frame carrying it by means of stud.

9) Dark sun glasses provided near prism can be interposed in the line of sight if the object to be sighted are luminary.

10)The bottom of the box which is about 85mm supports the pivot needle firmly at its centre.

11)The object vane and prism are supported on the sides of the box

12)The box is provided with a glass lid which protects the graduation disc at the same time permits the reading of graduation from the top.

13)When object vane is folded on the glass top is presses a lifting pin which lift the needle of the pivot.It prevents undue wear of the pivot point.

14)While taking reading if graduated disc vibrates it can be dampned by means of light spring fitted inside the box.

15)The box may be closed in metal lid when the compass is not in use. The box is provided with the support to fit it on to a tripod.

(a) (b) Markings on disc
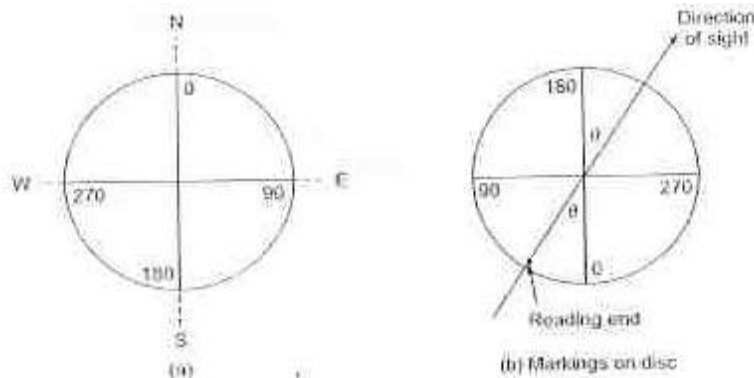
2. **The surveyor's compass:**

The graduated ring is directly attached to the box & not with needle.
The edge bar needle freely rests over the pivot thus the graduated card or ring is not oriented in the magnetic meridian.When the line of sight is in magnetic meridian the north & south ends of the needle will be over the o ^{0} graduations of the graduated card. The card is graduated in quadrant system having o^0 at N & S ends.And 90^0 at east & west ends.Let us take the case of a line AB which is in north – east quadrant in order to sight the point B.The box will have to be rotated about the vertical axis, in doing so the pointer of the needle remains fixed in position.

Difference between prismatic compass & surveyor's compass.

Prismatic compass	Surveyor's compass
The graduation circle is fixed to broad needle.It does not rotate with line of sight.	The graduation circle is fixed to the box and rotates with line of sight
There is a prism at viewing end.	No prism.Only slit
The graduations are in WCB system.	The graduation are in Q.B system.
The graduations are marked inverted.	The graduations are marked directly.
Magnetic needle do not act as index.	Magnetic needleacts as index.
Tripod mayor may not be provided, the instrument can be used even by holding suitably in hand	The instrument can't be used without tripod.

Temporary adjustments:

1.Centering

2.Levelling and

3.Focusing the prism

True meridian and Magnetic meridian:

The points of intersection of earth's axis with the surface of earth are known as geographical north & south poles.At any point on earth's surface the line passing through the point and north & south pole of the earth is called true meridian.

The angle made by a line with true meridian is called the true bearing of the line. The north & south pole of the earth are established by astronomical observations.

Whole circle bearing and quadrantal bearing system.

In whole circle bearing (WCB) the bearing of line at any point is measured w.r.t magnetic meridian. It's value may vary from $0^0 - 360^0$. 0^0 is magnetic north & the bearing increases in clockwise direction. This type of bearing system is used in prismatic compass.

In quadrantal bearing system (QB) : the bearing are read from north or from south. Towards east or west.The angle measured w.r.t magnetic meridian is designated with letter N or S in the beginning to indicate whether it's from North or from south.The letters E or W indicates whether bearing read is to the east or west respectively.

Reduced bearing (RB): This system is also known as reduced bearing system.

Magnetic dip and Magnetic declination

A balanced needle after magnetisation will dip towards north in northern hemisphere in southern hemisphere.If it is taken to the pole of earth it will take vertical position.The vertical angle between the horizontal at the point and direction shown by perfectly balanced needle is

known as dip.

All important surveys are plotted with reference to true meridian since the direction of magnetic meridian at a place changes with time.The horizontal amgle made between the two meridians such as magnetic and true meridian is known as magnetic declination.

The following are four types of declination:

1) Secular variation

2) Annual variation

3) Daily variation

4) Irregular variation.

Determination of true bearing

True bearing = magnetic bearing (+ or -) declination.

Problems

1) The magnetic bearing of a line is $48^0 24'$ calculate the true bearing if the magnetic declination is $5^0 38'$ east.

Solution: Declination = $+5^0.38'$

$\qquad$ True bearing = $48^0.24' + 5^0.38' = 54^0.02'$

Errors in compass survey

The errors may be classified as

a. Instrumental errors

b. Personal errors

c. Errors due to natural causes.

1. **Instrumental errors:**

They are those which arise due to the fault adjustments in instruments.

   1. The needle not being perfectly straight

   2. Sluggish needle.

   e) Pivot bent

   f) Improper balancing weight

   g) Blunt pivot point

2. **Personal errors:**

   a. Inaccurate levelling

   b. Inaccurate centering

   c. Inaccurate bisection

3. Natural errors:

      a.   Variation in declination

      b.   Local attraction due to forces around

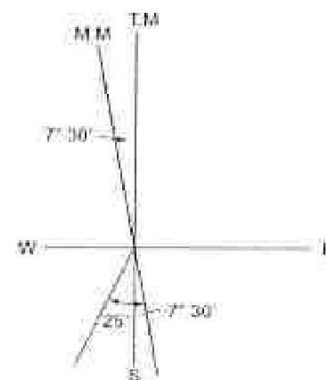   c.   Irregular variations due to storms

Problems:

1) The magnetic bearing is $S25^0$ 0'W

At that time of observation if magnetic declination is $7^030'$ west

Find the true bearing of the line.

Solution:

True bearing= 25^0 0' - $7^030'$ =$S17^030'$ W

2) Find the true bearing of line if it's magnetic bearing is S 30^0 W declination is 8^0 west.

Solution:

True bearing, TB = MB + MD

$$= 30^0 - 8^0 = 22^0 0'$$

3) In the map line AB was drawn to a MB of 138^0 30' when M.D of $4^0 30'$ east. To what M.B the line should be set. Known magnetic declination is 8^0 .30' west.

Solution: Magnetic bearing $= 138^0$ 30'$+4^0$ 30'$+8^0$ 30'

M.D $= 151^0$ 30'

Fore bearing:

For line AB bearing from A to B is called forward bearing for the same line bearing taken from B to A is called back bearing of line AB.

(a)   (b)
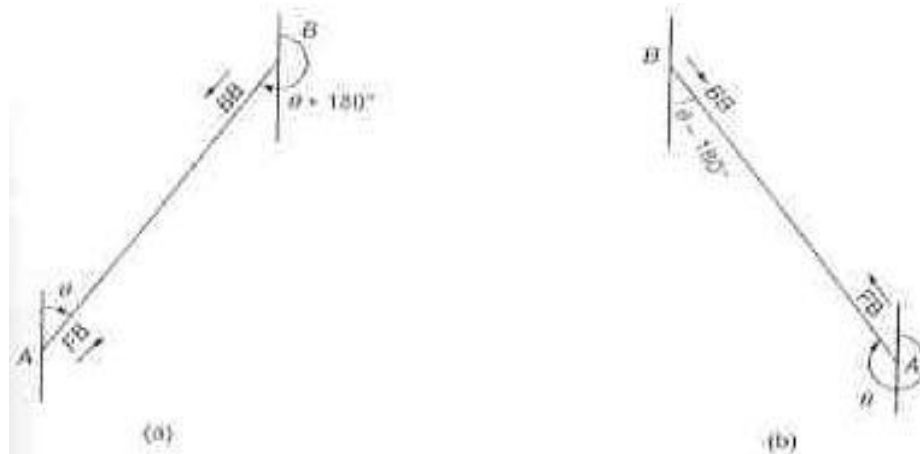
Fore bearing and back bearing difference will be 180^0. Hence in whole circle bearing BB = FB(+ or -) 180^0 use sign if FB is less than 180^0 and – sign if FB is more than 180^0

Convert the following quadrant into whole circle bearing and find their back bearing

Sl No	Q B	WCB (whole circle)	BB (Back bean 180th)
1	N 68^0 E	68 (68)	248
2	S 33^0 E	147(180 – 93)	327
3	N 28^0 w	332 (360 – 28)	152
4	N 43 w	223 (180 + 43)	43

LOCAL ATTRACTION

A magnetic meridian at a place is established by a magnetic needle which is uninfluenced by other attracting forces. However, sometimes, the magnetic needle may be attracted and prevented from indicating the true magnetic meridian when it is in proximity to certain magnetic substances. Local attraction is a term used to denote any influence, such as the above, which prevents the needle from pointing to the magnetic north in a given locality. Some of the sources of local attraction are : magnetite in the ground, wire carrying electric current, steel structures, railroad rails, underground iron pipes, keys, steel – bowed spectacles, metal buttons, axes, chains, steel tapes etc., which may be lying on the ground nearby.

Detection of local attraction.

The local attraction at a particular place can be detected by observing the fore and back bearings of each line and finding its difference. If the difference between fore and back bearing is 180^0, it may be taken that both the stations are free from local attraction, provided there are no observational and instrumental errors. If the difference is other than 180^0, the fore bearing should be measured again to find out whether the discrepancy is due to avoidable attraction from the articles on person, chains, tapes etc. it the difference still remains, the local attraction exists at one or both the stations.

Strictly speaking, the term local attraction does not include avoidable attraction due to things about the person or to other sources not connected with the place where the needle is read. Elimination of local attraction. If there is local attraction at a station. All the bearings measured at that place will be incorrect and the amount of error will be equal in all the bearings. There are two methods for eliminating the effects of local attraction.

First method.

In this method, the bearings of the lines are calculated on the basis of the bearing of that line which has a difference of 180^0 in its fore and back bearings. It is. However, assumed that there are no observational and other instrumental errors. The amount and direction of error due to local attraction at each of the affected station is found. If, however, there is no such line in which the two bearings differ by 180^0, the corrections should be made from the mean value of the bearing of that line in which there is least discrepancy between the back sight and fore sight readings.

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

Second method.

This is more a general method and is based on the fact that though the bearings measured at a station may be incorrect due to local attraction, the included angel calculated from the bearings will be correct since the amount of error is the same for all the bearings measured at the station. The included angles between the lines are calculated at all the stations. If the traverse is a close one, the sum of the internal included angles must be right angles. If there is any discrepancy in this, observational and instrumental errors also exist. Such error is distributed equally to all the angles. Proceeding now with the line, the bearings of which differ by 180^0, the bearings of all other lines are calculated.

Special case:

Special case f local attraction may arise when we find no line which has a difference of 180^0 in its fore and back bearings. In that case select the line in which the difference in its fore and back bearings is closest to 180^0. The mean value of the bearing of that line is found by applying half the correction to both the fore and back bearings of that line, thus obtaining the modified fore and back bearings of that line differing exactly by 180^0. Proceeding with the modified bearings of that line, corrected bearings of other lines are found.

Problem: The following bearings were observed while traversing with a compass.

Line	F.B	B.B	Line	F.B	B.B
AB	45^0 45'	226^0 10'	CD	29^0 45'	209^0 10'
BC	96^0 55'	277^0 5'	DE	324^0 48'	144^0 48'

Mention which stations were affected by local attraction and determine the corrected bearings.

Solution:

On examining the observed bearings of the lines, it will be noticed that difference between back and fore bearings of the line DE is exactly 180^0. Hence both stations D and E are free from local attraction and all other bearings measured at these stations are also correct. Thus, the observed bearing of DC is correct. The correct bearing of CD will, therefore, be 209^0 10' -180^0 =29^0 10' while the observed bearing is 29^0 45'. The error at C is therefore + 35' and a correction - 35' must be applied to all the bearings measured at C. the correct bearings of CB thus becomes 277^0 5'-35' = 276^0 30' and that of BC as 276^0 30'-180^0 = 96^0 30'. The observed bearing of BC is 96^0 55'. Hence the error at B is + 25' and a correction of – 25' must be applied to all the bearings measured at B. the correct bearing of BA thus becomes 226^0 10'-25'=225^0 45', and that of AB as 225^0 45'-180^0 = 45^0 45' which is the same as the observed one. Station A is, therefore, free from local attraction.

The results may be tabulated as under:

Line	Observed bearing	Correction	Corrected bearing	Remarks

AB	$45^0\ 45$'	0 at A	$45^0\ 45$'	
BA	$226^0\ 10$'	-25' at B	$225^0\ 45$'	Station B and C
BC	$96^0\ 55$'	-25' at B	$96^0\ 30$'	
CB	$277^0\ 5$'	-35' at C	$276^0\ 30$'	
CD	$29^0\ 45$'	-35' at C	$29^0\ 10$'	are affected by
DC	$209^0\ 10$'	0 to D	$209^0\ 10$'	Local attraction.
DE	$324^0\ 48$'	0 to D	$324^0\ 48$'	
ED	$144^0\ 48$'	0 to E	$144^0\ 48$'	

Problem – 1

The following bearings were observed with a compass. Calculate the interior angles.

Line	Fore bearing
AB	$60^0\ 30$'
BC	$122^0\ 0$'
CD	$46^0\ 0$'
DE	$205^0\ 30$'
EA	$300^0\ 0$'

Solution:

Included angle = Bearing of previous line – Bearing of next

A = Bearing of AE – Bearing of AB

$= (300^0 - 180^0\ ) - 60^0\ 30^1$

$= 120^0 - 60^0\ 30^1$

$A = 59^0.\ 30^1$

B = Bearing of BA – Bearing of BC

$= (60^0 \ 30^1 - 180^0) - 122^0$

$= 240^0 \ . \ 30^1 - 122^0 =$

$118^0.30$' C = Bearing of CB –

Bearing of CD

$= (122^0 + 180^0) - 40^0$

$= 302^0 - 46^0$

$C = 256^0$

D = Bearing of DC – Bearing of DE

$= (46^0 + 180^0) - 205^0 \ 30^1$

$= 226^0 - 205^0 30^1$

$D = 20^0 30^1$

E = Bearing of ED – Bearing of EA

$= (205^0.30^1 - 180^0) - 300^0 +368$

$= 25^0 \ 30^1 - 660^0$

$= 85^0 30^1$

SUM = A +B + C +D +E

$= 59^0 \ 30^1 + 118^0 \ 30^1 + 256^0 + 20^0 \ 30^1 + 85^0 \ 30^1$

Sum $= 540^0 \ 0^1$

Check

$= (2n - 4) \ 90^0$

$= (2*5-4) \ 90^0$

$= (10-4) \ 90^0$

$= 6*90^0$

$= 540^0 \ 0^1$

Problem-2

The following interior angles were measured a with a box sextant in a closed traverse.The bearing of the line AB was measured as $60^0 00^1$. With prismatic compass. Calculate the bearings of all other line

If   A = $140^0 \ 10^1$

   B = $990^0 \ 8^1$

   C = $60^0 \ 22^1$

   D = $69^0 \ 20^1$

Bearing of AD = Bearing of BA + 140^0 10^1-180^0

$$= (180^0+60^0) +140^0 \; 10^1 - 180^0$$

$$= 200^0 \; 10^1 \; - \; 20^0 \; 10^1 = AD$$

Bearing of DC = Bearing of AD+ $69^0 \; 20^1$-180^0

$$= 200^0+10^0 +690^0 \; 20^1 - 180^0$$

$$= 89^0 \; 30^1$$

Bearing of CD = $269^0 \; 30^1$

Bearing of CB = Bearing of DC+ $60^0 \; 22^1$-180^0

$$= 89^030^1+60^0 \; 22^1 + 180^0 \; = \; 329^0 \; 52^1$$

Bearing of BC = $149^0 \; 52^1$

 BC= Bearing of CB+ $90^0 \; 8^1$-180^0

$$= 329^0 \; 52^1+90^0 \; 8^1 - 180^0$$

$$= 420^0 \; - 180^0$$

$$= 240^0$$

Bearing of AB = 60^0 (Check)

Problem-3

Determine bearing of side of regular pentagon of sides. If the bearing of AB is 80^0.

Solution:

Back bearing of AB = $80^0 + 180^0 = 200^0$

Bearing of BC= $260^0 - 180^0 = 152^0$

Back bearing of CB = $152^0 + 180^0 = 332^0$

Bearing of CD= $332^0 - 108^0 = 224^0$

Back bearing of DC = $224^0 - 180^0 = 44^0$

Bearing of DE= $44^0 - 108^0 + 360^0 = 296^0$

Back bearing of ED = 116^0

Bearing of EA= $166^0 - 108^0 = 8^0$

Back bearing of AE = $B^0 + 180 = 188$

Check

       Bearing of AB = $188^0 - 108 = 80^0$

THEODOLITE SURVEY

Theodolite and types

Theodolite is the most precise survey instrument used commonly by engineers for measuring horizontal and vertical angles accurately

Theodolites are broadly classified into two as

1.Transit

2.Non-transit

1.Transit theodolite: A theodolite in which if the telescope can be revolved through a complete resolute about its horizontal axis in the vertical plane is called as a transit theodolite.

2.Non transit theodolite: This kind of theodolites are plain or 'Y'theodolites,in which the telescope cannot be transited.

Theodolites are also classified into two as

1.Vernier theodolites

2.Micrometer theodolites, based on the system used to observe the reading.

1.Vernier   theodolite: verniers are used to measure accurately the horizontal and vertical angles.A 20" verinier theodolite is usually used.

2.Micrometer theodolite : An optical system or a micrometer is used to read the angles in this case.The precision can be as high as 1"

* Fundamental Axis and part of transit theodolite
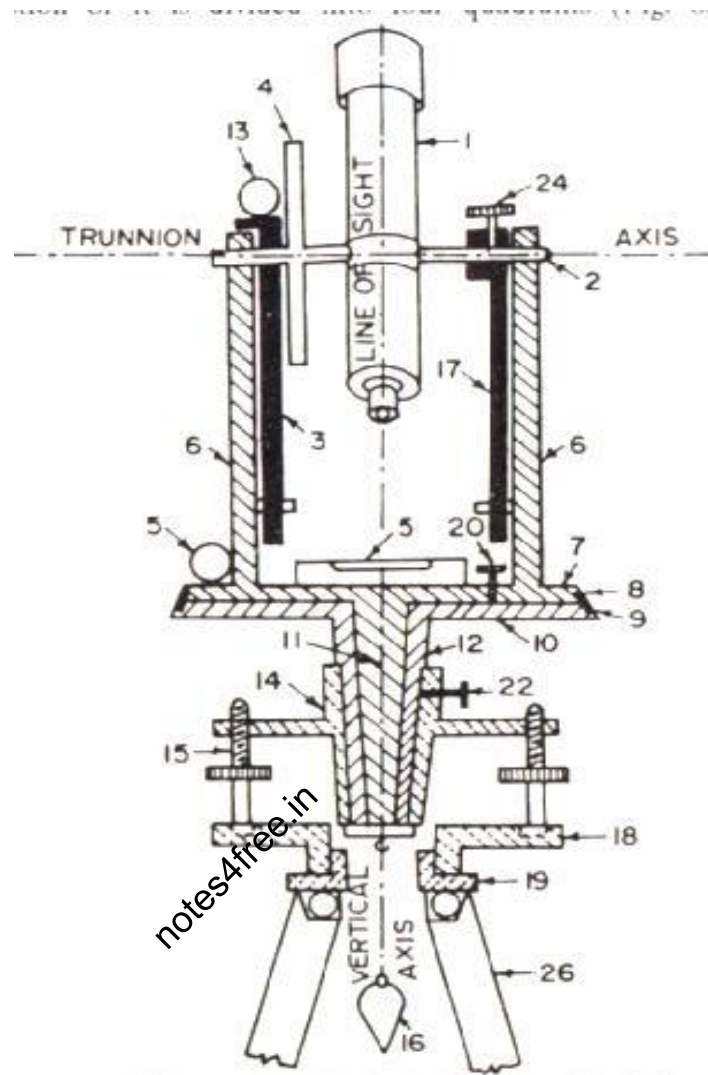
Parts of theodolite

1.Telescope: The telescope of the theodolite is mounted on a spindle known as "Trunnion axis". In most of the transit theodolite an internal focusing telescope is used. It consists of object glass, a diaphragm and an eye-piece. The main functions of the telescope is to provide line of sight.

2.The vertical circle: The vertical circle is rigidly connected to the transverse axis of the telescope and moves as the telescope is raised or depressed. It is graduated in degrees with graduations at 20'. Graduation in each quadrant is numbered from 0' to 90' in the opposite directions from the two zeros placed at the horizontal diameter of the circle.

3.The index frame or T-frame or Vernier frame: It consists of a vertical portion called dipping arm and a horizontal portion called an index arm. The 2 verniers of the vertical circle are fixed

to the two ends of the index arm. The index arm can be rotated slightly for adjustment purpose, with the help of clip screw.

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

5.Levelling head: It consists of 2 parts namely

a)Tribrach- It is the upper triangular plate which carries 3 levelling screws at the

   three ends of the triangle.

  b)Trivet or the lower plate (foot plate) used three grooves to accommodate the 3

     levelling screws.

The leveling head has 3 main functions namely

1.To support the main part of the instrument

2.To attach the theodolite to the tripod

3.To provide a mean for leveling.

6.The two spindles : Inner spindle is conical and fits into the outer spindle which is hollow. Inner spindle is also called upper axis and outer spindle is called lower axis.

7.The lower plate (scale plate): It carries the circular scale which is graduated from 0-360'.It is attached to the outer spindle which turns in a bearing within the tribrach of the leveling head.It is fixed using lower clamping screws lower tangent screws enable slow motion of the outer spindle.

8.Upper plate(vernier plate): It is attached to the inner axis and carries 2 verniers with magnifiers at two extremities diametrically opposite.Upper damping screw and a corresponding tangent screw are used for moving upper plate.

9.The plate levels : The upper plate carries one or 2 plate levels which can be centred with the help of foot screws.

10.Accessories:

     a)Tripod : with 3solid legs

     b)Plumb bob : for centering

     c)Compass : tubular or trough

     d)Striding level : for yesting the horizontality of the transit axis or trunnion axis.

Fundamental lines

These are basically 2 planes and 5 lines in a theodolite .The planes are horizontal plane with the horizontal circle and vernier; and vertical plane with vertical circle and vernier.

The fundamental lines are

1.Vertical axis

2.Horizontal axis

3.Line of collimation (line of sight)

4.Axis of plate level

5.Axis of altitude level

6.Axis of striding level,if provided

Definitions and Terms

1. centering: Setting the theodolite exactly over an instrument station so that its vertical axis lies immediately above the station point is called centering

2. The vertical axis : It is the axis about which the instrument can be rotated in a horizontal plane.

3.The horizontal axis: It is the trunnion axis about which the telescope

4.Line of sight or line of collimation: It is the imaginary line passing through the intersection of the cross hairs (vertical and horizontal) and the optical center of the object glass and its continuation

5.Axis of level tube : It is also called as bubble line,it is the straight tangential line to the longitudinal curve of the level tube at its centre

6.Axis of the altitude level tube: It is the axis of the level tube in altitude spirit level

7.Transiting: It is the process of turning the telescope vertical plane through 180' about the trunnion axis. This process is also known as plunging or reversing.

8.Swinging the telescope: It is the process of turning the telescope in horizontal plane.If the telescope is rotated in clock wise direction , it is known as right swing and other wise left swing.

9.Face right observation: If the vertical circle is to the left of the observer, then the observation is ca;;ed as face left

10.Face right observation: If vertical circle is to the right of the observer,then the observation called as face right.

10.Telescope normal and telescope inverted: If the telescope is in such a way that the face is left and bubble is up,then it is said to be in normal position or direct.If the face is right and bubble is down then the telescope is said to bein inverted position or reversed position.vertical circle to the right of the observer,if originally to the left and vice versa.it is done by first revolving the telescope through 180' in a vertical plane and then rotating it through 180' in the horizontal plane,ie first transiting and then swinging the telescope.

Temporary adjustments of a transit theodolite.

The temporary adjustments of atransit theodolite is done by 3 important operations.

1. Setting up: The instrument have to be setted up properly on the station point.the tripod stand should be approximately leveled before fixing the instrument.this is achieved with the help of moving the legs of the tripod.there is a small spirit level on the tripod head for the leveling of tripod.centering of the instrument over the station mark is achieved by a plumb bob or by using optical plummet.

2. Levelling up: After centering and approximate leveling ,accurate leveling is to be carried out with the help of the foot screws and using the plate level tube.in this step the vertical axis of the instrument is made truly vertical.Levelling the instrument depends on the number of foot screws available.

For a screw head,the procedure for leveling is as fallows:

a)Turn the upper plate until the longitudinal axis of the plate level is paralle to the line joining any two foot screws(let it be A and B)
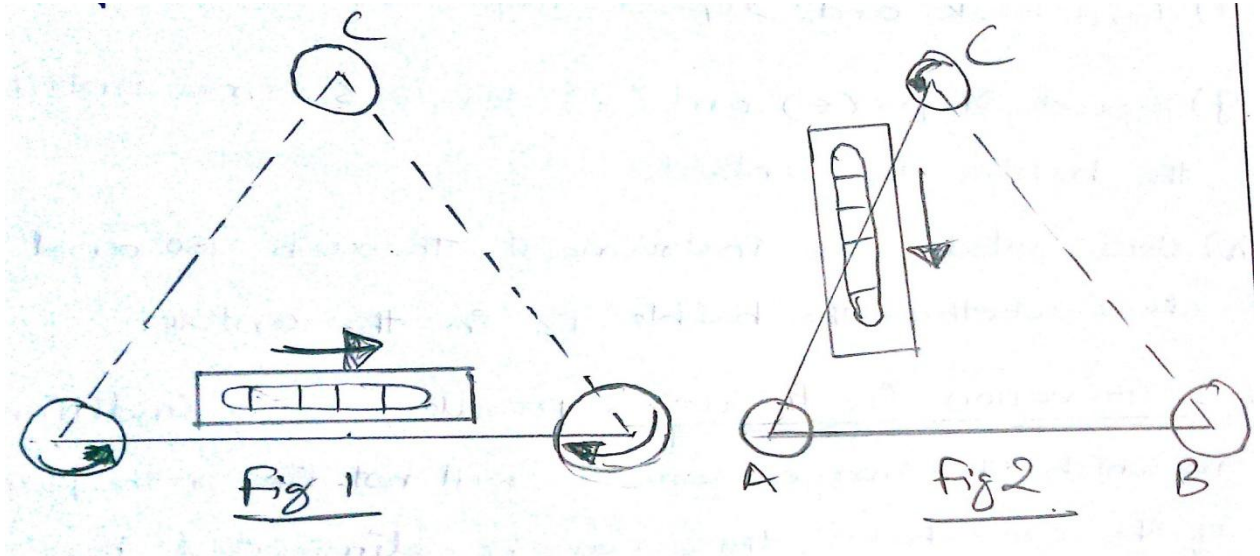
b)hold the 2 foot screws A and B between the thumb and the fore fingers of each hand and turn them uniformly so that the thumb move either towards each other until the bubble is central.Bubble moves in the direction of the left foot screw.

c)Turn the upper plate through 90' until the axis of the level passes over the position of the third leveling screw C

d)Turn this leveling screw until the bubble is central

e)Return the upper plate to original position (fig1) and repeat step(b)

f)Turn back and repeat step (c)

g)Repeat steps (e) and (f) for 2-3 times until the bubble is central.

h)Now rotate the instrument through 180' and check whether the bubble is in the centre.

3. Ellimination Of Parallax: Parallax is a condition in which the image is formed will not lie on the plane of the cross hair,this can be eliminated by focusing the eye-piece and the objective.

For focusing the eye-piece ,hold a white paper infront of the objective and move eye-piece in or out, until the cross-hairs are distinctly visible.objective lense focused by rotating the focusing screw,until the image appears clear and sharp.
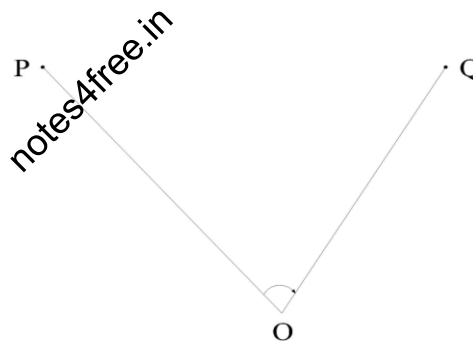
Measurement Of Horizontal Angles

Theodolites are majorly used to measure horizontal and vertical angles.Horizontal angles are usually ,easured by using any of these methods.

1.Ordinary method

2.Method of repetition

3.Method of reiteration

1.Ordinary Method

FIG

To measure an angle POQ,THE FOLLOWING PROCEDURE IS USED.

1.Set up the instrument at 0 , Set it up,level it accurately and perform the temporary adjustments

2.Release the upper clamp screw and lower clamp screw.Turn the upper and lower plates such that the vernier A reads 'zero' (0) and the vernier circle is to the left of the observer.Clamp both the plates and bring the vernier A to zero to coincide with the main scale zero using the upper tangent screw.Check the reading on vernier A,it should read 180'

3.Loosen the lower clamp and rotate the telescope to view point P.Clamp lower plate and using lower slow motiomn screw sight P exactly.Check the readings on both th vernier to see that it had not changed.

4.unclamp the upper clamp and rotate the instrument clock-wise until point Q is bisected tighten the clamp and using tangent screw bisect Q accurately.

5.Reading is observed from verners A and B .Reading of A vernier gives angle POQ and B vernier gives 180'+POQ

Read degres,minutes and seconds from the vernier scale by observing which line on the vernier scale is having correct coincidence with the reading in the main scale.

In a 20' transit theodolite ,the least count is 20" or the minimum reading which can be measured from the scale is 20".The reading coinciding with the vernier-zero is considered to be the main scale reading .If there is no exact coincidence for the vernier zero line ,then the reading to the immediate left of the vernier scale,on the main scale should be considered.This reading should be added with the vernier reading for the total value.

Reading onmain scale=128' 40'

Reading on vernier scale=3' 00"

Therefore total reading =128'40'+3'00"

                =128'43'00"

In B scale ,the degree reading is not required ,where as the minutes reading from the main scale is noted and add with vernier reading and this will give the B scale reading.

6.Enter the readings in a field book of tabular format

Tabular Column

7.Change the face by transiting and repeat the same process.

8.The mean of the 2 vernier reading gives the angle on face right

10.Average horizontal angle is calculated from the mean horizontal angle of face left and face right values.

Repetition Method

This method is used for very accurate work.In this method,the same angle is added several times mechanically and the total angle is divided by no of repetitions to obtain the correct value of angle.there are 2 methods by which this method can be conducted

To measure an angle POQ by the method of repetition,the following procedure is adopted

1.Obtain the first reading of the angle following the procedure outlined in the previous method.Read and record the value.

2.Loosen lower clamp,and turn the the telescope clockwise to sight P again and bisect properly using lower tangent scew.check the vernier and see that the readings are not changed.

3.Unclamp the upper clamp and turn the instrument clockwise and sight Q again

4.Repeat the process for 3 times

5.consider the average horizontal angle for face left by dividing the final reading by three

6.change face and make 3 more repetitions find the average angle.

7. Total average angle is obtained by adding up the results of 2 faces and then dividing by 2

For high precision surveys, repetition method can be conducted in two ways

a)the angle is measured respectively for six times, keeping the telescope normal (face left) and then calculating the average.

b)In another way ,angle is measured clockwise by first 3 with clockwise with face left and last 3 with telescope inverted.Then in anticlockwise also 3 face left and face right observations are taken.

Elimination of errors by method of repetition

The following errors are eliminated by adopting method of repetition

a)Errors due to eccentricity of verniers and centres by measuring both vernier readings.

b)Errors due to line of collimation not being perpendicular to the horizontal axis of the telescope.

c)Errors due to horizontal axis of telescope not being perpendicular to the vertical axis.

d)Erroe due to the line of collimation not coinciding with the axis of the telescope

These 3 errors can be eliminated by changing their face of the theodolite.

e)Errors due to inaccurate graduations this can be eliminated by taking 2 vernier readings

f)Error due to inaccurate bisection of the object this eliminated by taking repeated readings.

Reiteration Method

This method is also known as direction method or method of series several angles are measured successivelu and finally the horizon is closed.

To measure a series of angles AOB,BOC,COD etc by reiteration,this procedure is fallowed
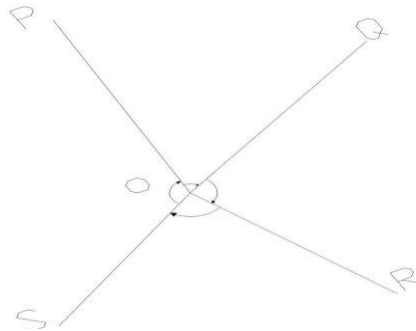
1.Set the instrument at O, level it and centre it.

2.Measure the angle AOB in the same way as already explained.

3.similarly bisect the successive ranging rods C,D etc and keep onobserving the readings.Each included angle is obtained by taking the difference of 2 consecutive readings.

Angle BOC=angleAOC – angle AOB

4.Finally close the horizon by sighting A.The reading in the vernier should be zero (360).If not ,note down the reading and distribute evenly to all angles.

Repeat the same steps in other face

The sets of reading are usually taken first in clockwise direction and then after changing the face in anticlockwise direction.

MEASUREMENT OF VERTICAL ANGLES

A vertical angle is an angle between the included line of sight and horizontal.the instrument has to be leveled with respect to the altitude bubble for measuring vertical angles

1.Level the instrument with reference to plate level

2.keep the altitude bubble tube parallel to 2 foot screws and bring the bubble central.rotate telescope 90' and adjust the bubble using the 3rd foot screw.repeat the procedure till the bubble is central.

3.loose the vertical clamp screw,rotate the telescope in vertical plane .to sight the object use tangent screw for correct bisections.

4.read vernier C and D.mean gives correct vertical angle.

5.change the face and continue the procedure .

If the vertical angle is measured above the horizontal line,it is called angle of elevation or in other case as angle of depression.

Uses Of Theodolite

Theodolite is not only used for measuring horizontal angles and vertical angles.but it is also used for the following:

1.To measure a magnetic bearing of a line

2.To measure direct angles

3.To measure deflection angles

4.To prolong a straight line

5.To run a straight line between 2 points

6.To locate the intersection points of 2 straight line

7.To lay off a horizontal angle etc.

PROLONGING A STRAIGHT LINE

1.When The Instrument Is In Adjustment

   I.   Method A: Set the instrument at A and sight B accurately.Establish point C in the line of sight shift the instrument to B, sight C and establish point D.The process is continued till the last point.

  II.   Method B :Set the instrument at B and take a back sight on A.Clamp all the screws and then plunge the telescope,if the instrument is in good adjustment point C will be established.Similarly shift the instrument to C,back sight B ,plunge the telescope and establish D,continue the procedure till the end .

2. When instrument is in poor adjustment (not in adjustment)

If the instrumrnt is not in adjustment ,then instead of B,C,D some other points B',C',D' etc will be established.

In such a case,set the instrument at B,take a back sight to A.plunge the telescope and establish point C1,change the face and take back sight on A.Plunge the telescope to establish C2 at the same distance. 'C' wil be in midway between C1 andC2.shift the instrument to 'c' and repeat the process. The process is repeated till the end point.This method is also called as Double sighting.

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ΣD= total error in departure

Σl= length of the
perimeter l= length of
any side

CL=ΣL*(l/Σl) and CD=ΣD*(l/Σl)

2) Transit Method: It is employed when angular measurements are more precise than linear measurements.

The Transit rule is: **Correction to latitude (or departure) of any side =**

Total error in latitude (or departure)* latitude L(or departure D) of that line
                                      Arithmetic sum of latitude LT(or departure
                                      DT)

CL=ΣL*(L/LT)   and   CD=ΣD*(D/DT)

3) Graphical Method: Bowditch's rule may be applied graphically without doing theoritical calculation. It is not necessary to calculate latitudes and departures. However before plotting the traverse directly from the field notes the angles or bearings may be adjusted to satisfy geometric conditions of the traverse.

Problem: Calculate the latitudes, departure and closing error for the traverse using bowditch's rule.

Line	Length	R.B	Latitude	Departure
AB	89.31	N 45^0 10' E	62.97	63.34
BC	219.76	N 72^0 05' E	67.61	209.1

CD	151.18	S18⁰ 08' E	-143.67	47.05
DE	159.1	S48⁰ 43' W	-104.97	-119.56
EA	232.26	N 59⁰ 18' W	118.58	-199.71
		Sum	0.52	0.52

Closing error, $e = \sqrt{(0.52)^2 + (0.22)^2} = 0.565$ m

$$\Theta = \tan^{-1} (0.22/0.52) = 22^0 \ 55' \ 56''$$

Toatal correction for latitude = -0.52 , Total correction for departure = -0.22

Σl = Perimeter of traverse = 89.31+119.76+151.18+159.1+232.26 = 851.61

Correction for latitude, CL=$\Sigma L*(l/\Sigma l)$ = 0.52* l/851.61 = -6.106*10⁻⁴ l

Correction for departure, CD=$\Sigma D*(l/\Sigma l)$ = 0.2* l/851.61 = -23485*10⁻⁴ l

Line	Latitude			Departure		
	Latitude	Correction	Corr. Lat	Departure	Correction	Corr. Dept
AB	62.97	-0.05	62.92	63.34	-0.02	63.32
BC	67.61	-0.13	67.48	209.1	-0.06	209.04
CD	-143.67	-0.09	-143.76	47.05	-0.04	47.01
DE	-104.97	-0.1	-105.07	-119.56	-0.04	-119.6
EA	118.58	-0.15	118.43	-199.71	-0.06	-199.77
	Sum	-0.52	0		-0.22	0

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TACHEOMETRY

Basic principle

Tacheometry is a branch of angular surveying in which the horizontal and vertical distances of points are obtained by Instrumental observations this method is rapid and accurate.

The common principle in all tacheometric survey is that the horizontal distance b/n an instrumental station and a point as well as the elevation point,relatively to the instrument can be determined from the angle subtended at the instrument by a known distance at point and vertical angle from instrument to the point.

Uses of tacheometric survey

1.it is rapid in rough and difficult terrain where ordinary leveling is tedious,chaining is inaccurate,diffcult and slow.

2.used when obstacles such as steep and broken ground,deep ravines and streches of water are met with.

3.used to prepare contor maps requiring both the horizontal as well as vertical control.

4.used in hydrographic survey,location surveys,road surveys, railway and reservoir surveys.

5.used for checking more precise instruments.

Types of tacheometric survey

There are 3 types-

1. Stadia method

2. Tangential method

3. Measurment by means of special instruments.

Stadia method is further classified into two:

A. Fixed hair method b. Movable hair method

Stadia method:

Instruments employed in stadia method are

1.tacheometer:

It is a transit theodalite having a stadia telescope with 2 horizontal hairscalled stadia hairsin addition to regular cross hairs.

2.stadia rod:

It is a rod with 5cm to 15cm width and 3 to 4 cm long. A leveling staff also can be used as a stadia rod.

Fixed hair method:

In this method stadia hair interval is fixed when a staff is sigthed through the telescope, a certain length of staff(staff intercept) is intercepted by the stadia lines and from this values the distance from the instrument to the staff station may be determined.

Priciple of stadia method(tacheometric eqn for horizontal line of sight)

Let o be the optical centre of the object glass

A,b&c –the bottom, top and the central axis at diaphram

A,b&c –the points on the staff cut by the three lines

Ab=i=interval b/n stadia lines

Ab=s=staff intercept

F=focal length of the object glass

U=the horizontal distance from the optical centre to the staff

V=the horizontal distance from the optical centre to the image of the staff.

U&v are the conjugate focal distance

D=the horizontal distance from o to the vertical axis of the tacheometer.

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

From similar triangles aob and aob

$$I/s=v/u$$

$$V=iu/s……1$$

From the formulae of lenses $1/f=1/u+1/v……..2$

$$1/f=1/u+1/(iu/s)=1/u+s/iu$$

$$1/f=1/u+s/iu=(1+s)/iu$$

$$1/f=(i+s)/iu$$

$$Iu=(i+s)*f$$

$$U=(i+s)*f/i=(i/i+s/i)*f$$

$$U=(1+s/i)*f=f+f(s/i)$$

But $d=u+d$

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

$$D=(f/i)*s+(f+d)$$

or

$$D=ks+c$$

This eqn is known as the distance eqn or the tacheometric eqn

The quantites $(f/i)\&(f+d)$ are the tacheometric constants

$(f/i)=k$ it is called as multiplying constant

$(f+d)=c$, addittion constant

The value of (f/i) or k actually 100.

Determeination of tacheometric constants(field measurments)

First method

- Sight any far object and focus it properly

- Measure the distance along the top of the telescope b/n the object glass and the plane of the cross hairs with a rule.

- Measure the distance d

- Measure several lengths d1,d2…along ab from instrument position a and obtain the staff intercept s1,s2,s3…….at each of the length

- Add f & d to find c=f+d

- Knowing c determine the several radius of f/i or k from eqn d=ks+c

- Mean of the several values give the required values of the multiple constants(f/i)

Second method:

- Measure a line accurately oa about 300 long on a farely level ground and fix pegs at the interval 30m

- Set up the instrument at o and obtain the staff in tercept by taking the stadia readings on the staff held vertically on each of the pegs

- Substitute the values of d and s in eqn d=ks+c from the member of eqn formed by the substituting values d and s

$$D1=ks1+c \qquad d2=ks+c$$

$$D1*s2-d2*s1/(s2-s1)$$

Distance and elevation formulae when staff held vertical :

Tacheometric eqn for horizontal line of sight:
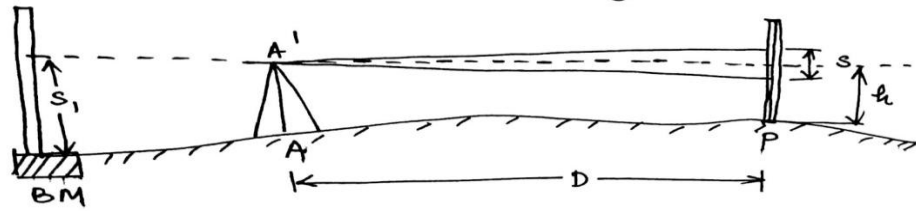
$$D=ks+c, \qquad k=100 \& c=0$$

$$\text{Then } d=100*s$$

$$\text{Rl of p=rl of bm } +s1-h$$

Tacheometric eqn for inclined line of sight

$$L=ks'+c$$

$$\text{Cos(alpha)}=d/l$$

$$D=l\cos(\text{alpha})$$

$$\text{Sin (alpha)}=v/l$$

$$V=l\sin(\text{alpha})$$
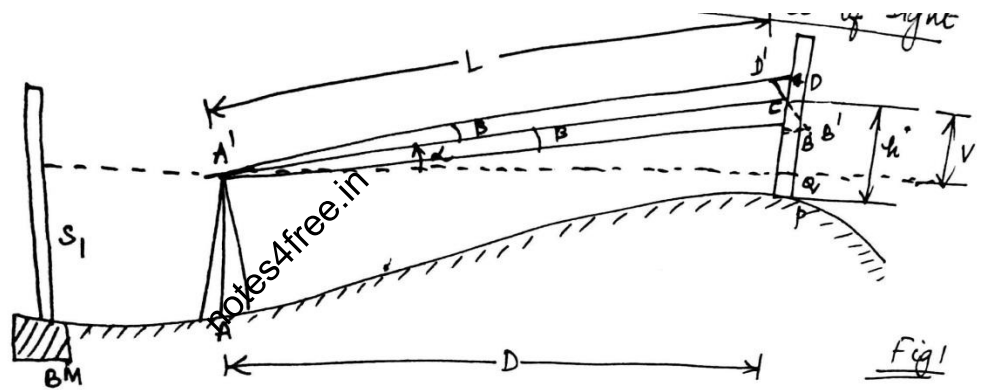
Let d&c are the 3 points on the staff cut by the upper middle and the lower cross hairs ,db is stadia reading=s

From fig2 bc=cb=s/2

$$D = l\cos(\alpha) \dots\dots 1$$

$$V = l\sin(\alpha) \dots\dots 2$$

$$L = ks' + c \dots\dots 3$$

Form fig4 $\cos(\alpha) = s'/s$

or

$$s' = s\cos(\alpha)$$

For angle of elevation,

Rl of p=rl of bm +s1+v-h

For angle of depression

Rlof p=rlof bm+s1-v-h

H is the middle hair reading or actual hair reading

Tacheometric eqn for the line of sight inclined and the staff held normally in line of sight:
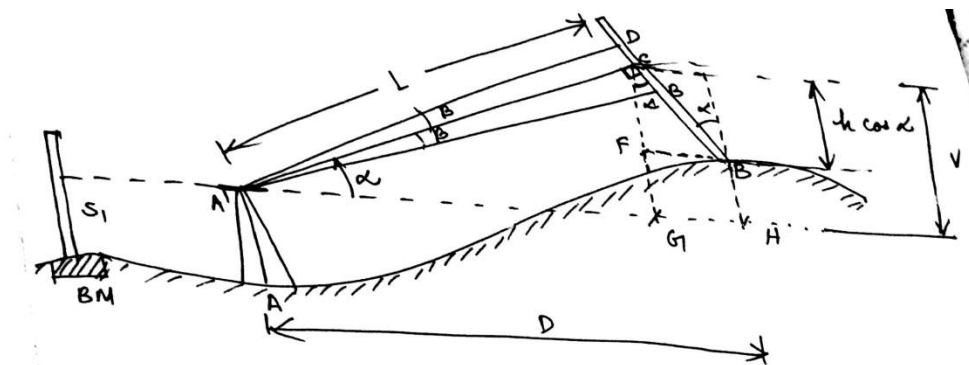
In this case the line of sight is perpendicular to the staff

Axial hair reading h is inclined from triangle cfb

$$cf = h\cos(\alpha) \dots\dots 1$$

$$D = delta' *g + gh \dots\dots 2$$

$$A'g = l\cos(\alpha) \ldots \ldots 3\backslash$$

$$Cg = l\sin(\alpha) \ldots \ldots 4$$

$$Fb = h\sin(\alpha) \ldots \ldots 5$$

$$D = l\sin(\alpha) + h\sin(\alpha) \ldots \ldots 6$$

Here l=ks+c

Tacheometric eqn for inclined line of sight:

$$D = (ks+c)\cos(\alpha) + h\sin(\alpha)$$

$$D = ks\cos(\alpha) + \cos(\alpha) + h\sin(\alpha) \ldots \ldots 7$$

If c=0

$$D = ks\cos(\alpha) + h\sin(\alpha) \ldots \ldots 8$$

$$V = ks\sin(\alpha) + c\sin(\alpha) \ldots \ldots \ldots 9$$

If c=0

$$V = ks\sin(\alpha) \ldots 10$$

For the angle of elevation

$$Rlof\ b = rlof\ bm + s1 + v - h\cos(\alpha) \ldots \ldots 11$$

For angle of depression

$$Rlofb = rlof\ bm + s1 - v - h\cos(\alpha) \ldots \ldots 12$$

Moving hair method

In this the instruments used are a theodalite equipped with a diapharm which has stadia hairs which can be moved by a separate sliding frame by micrometer screw with a large graduated head.

Distance through which the stadia wires are moved is given by the sum of the readings.eventhough stadia interval is variable,staff intercept remains constant.

The horizontal distance, d is given by the formula,d=ks/n+(f+d)

Where, n=sum of micrometer readings.

Tangential method

When telescope is not fitted with stadia diapharm,this method is used. The horizontal and vertical distances from the staff stations from the instruments may be computed from observations taken to 2 vanes or targets on the staff at known distance (s) apart usually,

1. **When both are angles of elevation**

$$D= \quad s/(\tan \text{alpha2-tan alpha1})$$

$$V=d\tan(\text{alpha2})$$
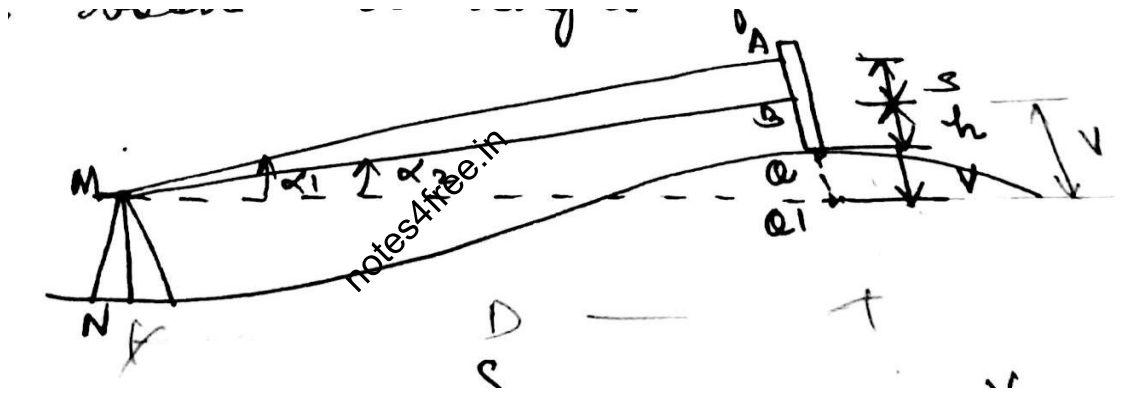
$$\text{Rlof } q=\text{rlof } bm+s1+v-h$$

2.   When both are angles of depression

$$D= \quad s/(\tan \text{alpha1-tan alpha2})$$

$$V=d\tan(\text{alpha2})$$

$$\text{Rlof } q=\text{rlof } bm+s1-(v+h)$$

Problems

1. Two distances of 20&100 are accurately measured and the intercepts on the staff b/n the enter stadia meter were 0.196m @ the former distance and 0.996 @ the lateral. Calculate the tacheometric constants.

Solun:

$$D1=20m \qquad d2=100m$$

$$S=0.196m \qquad s2=0.996m$$

$$d=ks+c$$

$$d1=ks1+c$$

$$d2=ks2+c$$

$$20=k*0.196+c\ldots\ldots.1$$

$$-100=-k*0.996+c\ldots\ldots 2$$

$$-80=-0.8k$$

$$k=80/0.8 \quad = \quad 100.$$

$$k*0.196+c=20.$$

$$c=20-k*0.196 = 20-19.6 = 0.4$$

$$c=0.4$$

MODULE 4

LEVELLING

Levelling is the art of determining the elevation of given points above or below a datum line or establishing in given points of required height above or below the datum line. It evolves measurement in vertical plane.

Definition of basic term's used in leveling:

Level surface: Any surface parallel to the mean spheroid of the earth is called level surface and the line drawn on level surface is known as level line.

Horizontal surface: Any surface tangential to level surface at a given point is called - Horizontal surface at point. Hence horizontal line is at right angles to plumb line.

Vertical surface: It is the line connecting the point & centre of earth.Vertical & horizontal line is normal to each other.

Datum: The point or the surface with respect to which levels of other points or planes are calculated is called – Datum or surface.

Mean sea level (MSL): Mean sea level is the average height of sea of all stages of tides.Any particular place is derived by averaging over a long period of 19 years. In India the mean's sea level used is that at Karachi (Pakistan).In all important survey this is taken as datum.

Reduced level: Levels of various points are taken as heights above the datum surface are known as Reduced level.

Bench mark: Bench mark is a relatively premanent point of reference whose Elevation w.r.t some assumed datum is known.There are four types of bench mark

1. G.T.S (Great trigonometry survey)

2. Permanent bench mark

3. Arbitrary bench mark.

4. Temporary bench mark.

LEVELLING INSTRUMENTS:

A level is an instrument giving horizontal line of sight & magnifying the reading far away from it. It consist of following 4 parts.

1. Telescope to provide line of sight.

2. Level tube to make the line of sight horizontal.

3. The leveling head to bring the bubble in its centre of run.

4. A tripod to support instrument

TYPES OF LEVELS:

1. Dumpy level
2. wye level
3. Cooke's – Reversible level
4. Tilting level
5. Auto level
6. Cushing's level

Working principle of auto & dumpy level
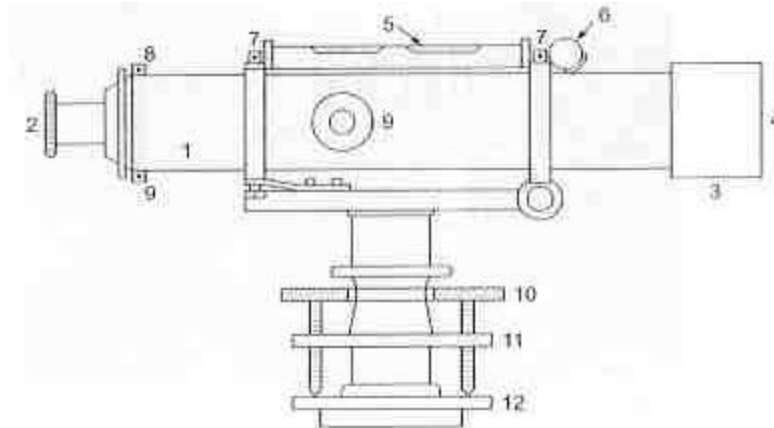
Parts of figure

1. Telescope

2. Eye piece

3. Shade

4. Objective end

5. Longitudinal bubble

6. Focusing screw

7. Foot screws

8. Upper parallel plate

9. Diaphragm adjusting screws

10. Bubble tube adjusting screw

11. Transverse bubble tube

12. Foot plate.

The dumpy level originally designed by – Gravatt consists of a telescope tube firmly secured in two collars fixed by adjusting screws to the stage carried by the vertical spindle.

The modern form of dumpy level has the telescope tube & the vertical spindle cast as one piece & a long bubble tube is attached to the top of the telescope.This form is known as – solid dumpy.

Leveling head generally consists of two parallel plates with either three- foot screws or four – foot screws. The upper plate is known as – tribrach

Lower part is known as – trivet which can be screwed on to a tripod.

The advantages of the dumpy level over the wye level

1. Simple construction with fewer movable parts

2. Fewer adjustments to be made

3. Longer life.

Levelling staff: A levelling staff is a straight rectangular rod having graduations.The foot of the staff representing 0 reading.During levelling staff is held vertical at the point and from level horizontal sight is taken.

Leveling staff may be divided into 2 groups
1. Self – reading
2. Target staff.

Parts of telescope
1. Objectives
2. Eye piece
3. Diaphragm
4. Focusing device

Fundamental axis of a level:

1. **Vertical axis:** It is the centre line of axis of notation of the level.

2. **Axis of level – tube:** It is an imaginary line tangential to the longitudinal curve of the tube at its middle point.It is horizontal when the bubble is central.

3. **Axis of telescope**: It is the line joining the optical centre of the object glass & the centre of eye piece.

4. **Line of collimation or line of sight:** It is the line joining the intersection of cross hairs & optical centre of the object glass.

Temporary staff adjustment of a level

1. Setting up
2. Leveling up
3. Focusing

Setting up: It is to set the tripod stand to a convenient height by bringing bubble to the centre of run through the movement of tripod legs radially.

Levelling up: To make the vertical axis truly vertical the levelling is made with the help of foot screws.

1. Loosen the clamp and turn the instrument until bubble axis is parallel to line joining any two screws.

2. Turn the two screws inward or outward equally till bubble is centered.

3. Turn the telescope through 90 degrees so that it lies over the third screw.

Focusing: For quantitative measurements it is essential that the image should always be formed in the fixed plane in the telescope where the cross – hairs are situated

The operation of forming or bringing the clear image of the object in the pane of cross hairs is known – as – focusing

Complete focusing involves two steps

1. Focusing the eye – piece

2. Focusing the objective

Telescope in which the focusing is done by the external movement of either objective or eye – piece is known as – External focusing telescope.

Telescope in which the focusing is done by the internally with a negative less is known as – internal focusing telescope

Sensitiveness of a bubble tube: When the difference in elevation between any two points is determined from a single set up by back sighting on one point and fore sighting on the other. The error is due to non parallelism. When the bubble is not in the centre of run and sensitivity is lost, due to the error of curvature and refraction which is eleminated if lengths of 2 sides are made equal.

Error due to Curvature: The horizontal line of sight does not remain straight but it slightly bends towards having concavity towards earth surface due to refraction.

$$CC = d^2/2R$$

Error due to Refraction: As the line of sight is curved downwards towards the earth surface reading gets decreased. To make the objects appear higher than they really are, this correction is applied to staff readings, $CR = 0.01121d^2$ where d is in Km.

TERMS USED IN LEVELLING:

1. **Station:** Station is the point where leveling staff is held & not the point where level is kept.

2**. Height of instrument:** For any set up of the level the height of instrument is the elevation of the plane of sight respect to assumed datum. This also known as – plane of collimation.

3. **Back sight:** It is sight taken on a level staff held at a point of known elevation with an intension of determining plane of collimation or sight.

4**. Intermediate sight (I.S):** Sight taken on after taking back sights before taking last sight from an instrument station is known as – intermediate sight.

The sight is also known as +ve sight (add)

5**. Fore sight (F.S): T**his is the last reading – taken from instrument just before shifting the instrument.This is also – ve sight.

6. **Change point (C.P):** This is a point on which both fore sight & back sight are taken.

7. **Reduced level:** Reduced level of a point is the level of the point with respect to assumed datum.

TYPES OF LEVELLING

1. Simple levelling

2. Differential levelling

3. Fly levelling

4. Profile levelling

5. Crossectioning

6. Reciprocal levelling

Simple levelling: It is the difference in levels of two near by points. It is obtained by simple levelling

Differential levelling: When the distance between two points is very large it may not be possible to tak ethe readings from single setting of instruments. Each shifting facilitated by taking CP.

Fly levelling: It is to carry out levelling with respect to temporary bench mark in convenient direction taking number of CP

Crossectioning: In many engineering projects to calculate earth work involved not only LS is involved but CS of ground is taken in regular intervals.

Reciprocal levelling: When it is not possible to balance FS and BS due to non-parallelism of line of collimation and axis of bubble tube and also due curvature and refraction this is used.

H=[(ha-hb)+(h'a-h'b)]/2

Reduction Of Levelling and Contouring

BOOKING & REDUCING THE LEVELS

Height of instrument method:

Sl No.	Station	BS	IS	FS	HI	RL	Remark
1	A	2.45			102.450	100.000	BM
2	E1		0.86			101.590	
3	Cp1	1.43		2.14	101.740	100.310	

4	E2		0.76			100.980	
5	Cp2	1.38		2.18	100.940	99.560	
6	B			1.54		99.400	

1. HI = RL + BS = 100+2.45 = 102.450 (HI)

2. RL = HI- IS = 102.450 – 0.86 = 101.590 (RL)

3. RL = HI – FS = 102.450 – 2.14 = 100.310 (RL)

4. HI = RL +BS = 100.310 + 1.43 = 101.740

5. RL = HI – IS = 101.740 – 0.76=100.980

6. RL = HI – FS = 101.740 – 2.18 = 99.560

7. HI = RL + BS = 99.560 + 1.38 = 100.940

8. RL = HI – FS 100.940 – 5.54 = 99400

Rise & fall method:

Station	BS0	IS	FS	Rise	Fall	RL	Remark
A	2.45					100.00	
E1		0.86		1.59		101.590	
Cp1	1.43		2.14		1.280	100.310	
E2		0.76		0.670		100.980	
Cp2	1.38		2.18		1.420	99.560	
B			1.54		0.160	99.400	

In this method difference in staff reading at a point with previous reading is found.If the present reading is less than the previous reading it indicates – rise.If it is more it is fall in the level of presenting point.If the reduced level of 1st point is known using rise & fall values of consecutive readings, the reduced level of all pointcan be calculated one after the other.

Note: 1. Previous reading - Present reading is +ve then it's –
Rise 2. Previous reading – present reading is - -ve then
it's – fall

Step 1:

From A to E1 Difference = 2.45 - 0.86

Rise = 1.59 (rise)

E1 to Cp1 = 0.86 – 2.14

Fall = -1.28

E2 – CP2 = 0.76 –
2.18 = -1.42 - fall
Cp2 – B   = 1.38 -
1.54

=   -0.16   -
fall Previous Reading
= 2.45 Present = 0.86

= 2.45 - 0.86
= 1.56

ΣBS – ΣFS=Σrise - Σfall=last RL - first RL

5.260-5.860=2.26-2.86=99.400-100

-0.6 = -0.6 = -0.6

Problem -1

1) The following was observed successively with a levelling instrument.The instrument was shifted after fifth & level readings are.

1) 0.585

2) 1.010

3)1.735

4) 3.295

5) 3.775

6) 0.350

7) 1.300

8) 1.795

9) 2.075

10) 3.375

11) 3.895

12) 1.785

13) 1.635

14) 1.605

Draw up page of level book & determine the RL – of various p a is RL of first point is 136.440m.

Solution: Height of Instrument method:

Station	BS	IS	FS	HI	RL
A	0.585			137.025	136.440
		1.010			136.015
		1.735			135.290
		3.295			133.730
Cp1	0.350		3.775	133.600	133.250
		1.300			132.300

		1.795			131.805
		2.675			131.525
		3.375			130.225
Cp2	1.785		3.895	131.440	
		1.635			130.805
			1.605		129.835

ΣBS=2.72

ΣFS=9.275

first RL=136.440

last RL=129.835

ΣBS-ΣFS= last RL-first RL

6.605=6.605

Problem 2:

RISE & FALL METHOD

Station	BS	IS	FS	Rise	Fall	RL
A	0.585					136.440
		1.010			0.425	136.015
		1.735			0.725	135.290
		3.295			1.560	133.730
	0.350		3.775		0.480	133.250
		1.300			0.950	132.300
		1.795			0.495	131.805
		2.575			0.780	131.025
		3.375			0.800	130.225
	1.735		3.895		0.520	129.705

Check: ΣBS-ΣFS=Σrise-Σfall = last RL-first RL

2.670-9.275=1.100-7.705=129.835-136.440

-6.605 = -6.605 = -6.605 (fall)

3) Staff reading were absorb successive between the instrument was been move after second fifth & eighth reading

1) 0.675

2) 1.230

3) 0.750

4) 2.565

5) 2.225

6) 1.935

7) 1.835

8) 3.220

9) 3.115

10) 2.875

The first staff reading taken with on a bench mark of RL 100m enter the reading in a level

book calculate RL of all the point also apply the asthmatic check use – height of Instrument method.

Case: Height of Instrument

Station	BS	IS	FS	HI	RL	Remark
A	0.675			100.675	100.00	
	0.750		1.230	100.195	99.445	Cp1
					97.630	
		2.565			99.970	
	1.935		2.225	99.905	98.070	
		1.835			96.685	
	3.115		3.220	99.8		

4) Following staff reading were taken with a level instrument having after 3rd, 6th, 8th reading.

1) 2.225

2) 1.625

3) 0.985

4) 2.095

5) 2.795

6) 1.265

7) 0.605

8) 1.980

9) 1.045

10) 2.685

Enter the above reading in a page of level book. Calculate the RL of the all the point 1st reading bench mark of RL 100 use the rise & fall method.

Station	BS	IS	FS	Rise	Fall	RL
A	2.225					100.00
		1.625		0.600		100.00
	2.095		0.985	0.640		101.240
		2.795			0.700	100.54
	0.605		1.205	1.530		102.070
	1.045		1.980		1.375	100.695
			2.685		1.64	99.055

Check: ΣBS-ΣFS=Σrise-Σfall = last RL-first RL

   -0.945 = -0.945 = -0.945 (fall)

PROFILE LEVELLING:

This type of leveling is known as – longitudinal section.

The reduced levels of various points at regular intervals are found along a line or a set of lines. Then the engineers draw the sectional view of the ground to get the profile. This type of leveling is commonly employed in deciding railways, highways, canal, sewage line routes.

After getting reduced level of various points along the line, profile of the ground is plotted on a drawing sheet. Normally vertical scale is much larger than the horizontal scale to clearly view the profile. Then when the engineers decide the formation level of the proposed project

The decision is mainly based on balancing, cutting & filling so that the transport of earth is minimum.

However the proposed gradient of formation level should not be more than as permitted. After deciding the formation level & the gradient the difference between two consecutive points is known. If RL of first point is known RL of other points are calculated.

Problem 1:

The following consecutive reading are taken in the level and a (4m) levelling staff along continuously sloping line. AB at a common interval of 20m.

1) 0.385

2) 1.030
3) 1.925
4) 2.825
5) 0.625
6) 2.005
7) 3.110

The RL of the first point is 200m.

Enter the readings in a level book & calculate the RL of each point by Rise & fall method & calculate the gradient joining first & last point.

Solution:

Station	Distance	BS	IS	FS	Rise	Fall	RL
A	20	0.385	1.030			0.645	199.35J
	40		1.925			1.545	198.46
	60	0.625		2.825		0.900	1975
	80		2.005			1.38	196.1
	100			3.110		1.105	195.075

Check: ΣBS-ΣFS=Σrise-Σfall = last RL-first

RL -4.925 = -4.925 = -4.925 (fall)

Problem 2: Following consecutive readings are taken with a level continuously on sloping ground.

1) 0.600

2) 1.235

3) 1.860

4) 2.575

5) 0.235

6) 0.915

7) 1.935

8) 2.870

9) 0.565

10) 1.825

11) 2.725

The reduced level of 1st point was 192.125 Enter the level book reading

Calculate the reduced level of point .Apply check, find the gradient of line joining 1st &

last point.Use height of instrument method.

Station	Distance	BS	IS	FS	Rise	Fall	RL
A	0	0.600			192	725	192.125
	20		1.235				191.490
	40		1.860				190.865
	60	0.235		2.575	190	385	190.150
							189.470
	80		0.915				188.450
	100		1.935				187.515
	120	0.565		2.870		188.080	187.512
	140		1.825				186.255

Check: ΣBS-ΣFS = last RL-first RL -6.77= -6.77 (fall)

Gradient= -6.77/160 =-0.042

MODULE 5

AREAS AND VOLUMES

Areas

The degree of accuracy of the computed area depends on

1. The accuracy of the field measurements

2. The accuracy of plotting when the calculations are made from a plan.

3. The method adopted for the computations.

Area is usually expressed in

1. Hectare (ha) = $(100m)^4 = 10^4$ m²

2. 1 arc (a) = $(10m)^2 = 10^2$m²

3. 1 square kilometre (km)² = $(1000m)^2 = 10^6$m²

Areas from field Measurnments

a) Area consisting of regular boundary

1)Area of triangle = $\sqrt{s(s-a)(s-b)(s-c)}$

$\qquad\qquad\qquad = ½ bh \qquad\qquad\qquad S = \dfrac{a+b+c}{2}$

$\qquad\qquad\qquad = ½ ab \, Sin \, C$

$=\dfrac{a}{Sin\,A} = \dfrac{b}{Sin\,B} = \dfrac{c}{Sin\,C}$

$b = \dfrac{C\,.\,Sin\,B}{}$

Sin C

2) Area of a trapezoidal $= \frac{1}{2}$ (sum of parallel sides) * altitude

$$= \frac{1}{2}(a + b)*h$$

Area from co-ordinates

a) Co-ordinates (Latitudes and Departures)

 Area of closed traverse ABCDE,

A = Area of (A a e E) + Area of (E e d D) + Area of (D d c C) +

Area of (B b c C).

$A = \frac{1}{2}(y_1 + y_5)(x_5 - x_1) + (y_5 + y_4)(x_4 - x_5) + (y_3 + y_4)(x_3 - x_4) - (y_2 + y_1)(x_2 - x_1) - (y_3 + y_2)(x_3 - x_2) \rightarrow$ ①

since, Area of a trapezoidal A a e E is given as $\frac{1}{2}(aA + eE) * ae$

But $aA = (y_1 - 0)$ $eE = (y_5 - 0)$ $ae = (x_5 - x_1)$

Area of AaeE $= \frac{1}{2}[(y_1 - 0) + (y_5 - 0)](x_5 - x_1)$

$$= \frac{1}{2}[(y_1 + y_5)(x_5 - x_1)] \rightarrow ②$$

Similarly,

Area of EedD $= \frac{1}{2}[(y_5 + y_4)(x_4 - x_5)] \rightarrow ③$

Area of DdcC $= \frac{1}{2}[(y_3 + y_4)(x_3 - x_4)] \rightarrow ④$

Area of AabB $= \frac{1}{2}[(y_2 + y_1)(x_2 - x_1)] \rightarrow ⑤$

Area of BbcC $= \frac{1}{2}[(y_3 + y_2)(x_3 - x_2)] \rightarrow ⑥$

① $\rightarrow A = 1/2[y_1x_5 - x_1y_5 + y_5x_4 - y_4x_5 - y_4x_3 + y_2x_1 - y_1x_2 + y_3x_2 - y_2x_3]$

$A = \frac{1}{2}[x_1(y_2 - y_5) + x_2(y_5 - y_1) + x_3(y_4 - y_2) + x_4(y_5 - y_3) + x_5(y_1 - y_4)]$

If a traverse have n sides , then

$A = \frac{1}{2}[x_1(y_2 - y_n) + x_2(y_3 - y_1) + \ldots + x_{n-1}(y_n - y_n) + x_n(y_1 - y_{n-1})$

$A = \frac{1}{2}[x_1 y_2 - y_1 x_2 + x_2 y_3 - y_2 x_3 + \ldots x_n y_1 - y_n x_1]$

Calculations of area from cross staff Survey

In cross – staff survey, area of a plot can be calculated by the direct use of field notes. In this method, the chain line run through the centre of the area so that offset to boundaries are taken in order of their chainages. Cross staff is used for setting out perpendicular offsets. For accurate work, an optical square or prism can be used. The plot is divided into right angled triangles and trapezoidal and area of each figure is calculated separately.

Area of a right angle triangle $= \frac{1}{2} * b * h$

Area of a trapezoidal $= \frac{1}{2}(a+b) * h$

$= (O_0 + O_2)\dfrac{2d}{2} + O_1 . \dfrac{4d}{3} - O_0 . \dfrac{2d}{3} - O_2 . \dfrac{2d}{3}$

$= O_0 . \dfrac{2d}{2} + O_2 . \dfrac{2d}{2} + O_1 . \dfrac{4d}{3} - O_0 . \dfrac{2d}{3} - O_2 . \dfrac{2d}{3}$

$= O_0 (\dfrac{2d}{2} + (- \dfrac{2d}{3}) + O_1 . \dfrac{4d}{3} + O_2(\dfrac{2d}{2} – \dfrac{2d}{3})$

$= O_0 \dfrac{(6d – 4d)}{6} + O_1 . \dfrac{4d}{3} + O_2 \dfrac{(6d – 4d)}{6}$

$= O_0 .\dfrac{2d}{6} + O_1 . \dfrac{4d}{3} + O_2 . \dfrac{2d}{6}$

$= O_0 . \dfrac{d}{3} + O_1 . \dfrac{4d}{3} + O_2 . \dfrac{d}{3}$

$= \dfrac{d}{3} [O_0 + 4O_1 + O_2]$

similarly for the next 2 divisions ,

$= \dfrac{d}{3} [O_2 + 4O_3 + O2]$

The total area $= \dfrac{d}{3} [O_0 + 4O_1 + O_2 + O_2 + 4O_3 + O_4 + \ldots \ldots]$

$d[O_0 + On + 4(O_1 + O_3 + O_5 + \ldots) + 2(O_2 + O_4 + O_6)$.

Planimeter

There are 2 types of planimeter

a) Amsler polar palnimeter

b) Rolling planimeter

Amsler polar Planimeter is being commonly used to determine the areas of figures plotted to scale, with irregular boundaries.

It consists of 2 bars PQ and QR hinged at 'Q '. A tracing point 'P' is grined around the boundary of the area to be measured. The QR boundary of the area to be measured . The QR terminate at R which is a stationary point and is fixed on the papers by a needle point and weight.

The displacement of the tracing point is measured by the wheel W fixed in a plan perpendicular to PQ. While PQ moves, the wheel partly rotates and partly shades on the paper. The total normal displacements is measured by the rotation, the axial component of the motion causes slip and does not affect the measurement.

The wheel W is geared to a dial which records the members of revolutions made by the wheel. One revolution of the wheel is equal to one divisions of the dial which has 10 equal divisions on its periphery. The wheel is graduated on its

periphery with 100 divisions, subdivide into tenth by a vernier . Thus each reading consists of four digits. The units being read on the dial, the thousands on the vernier dial can be utilized to know the number of the index . The distance between P and Q is adjustable and on the tracing arm opposite the index by means of the clamp and tangent screw.

Use of Planimeter

The method for using the planimeter as follows :-

1) To obtain the area in the desired unit, adjust the tracing arm.

2) Fix the needle point on the paper outside the area to be measured in such a manner that the tracing point can reach all parts of the boundary of the area.

3) Select any arbitrary point on the boundary and move the tracing point from this point all around the boundary in a clockwise direction till the tracing point reaches the starting point.

The area of the plan is calculated from the following formula :

$$A = M (R_F - R_1 \pm 10N + C)$$

M = multiplying constant whose value is marked on the tracing arm

R_F and R_1 = Initial and final readings

N = No of compute revolutions of the dial

[+ = when in clockwise direction]

[- = when in anticlockwise direction]

C = Constant of the instrument

$C = 0$, for the needle point outside area

Zero circle is a circle round the circumference of which if the tracing arm is moved, the wheel W does not rotate at all , and , no change in reading.

A_0 is the area of zero circle, which is equal to $C * M$.

Digital   Plannimeter

It is an area – curvature which plots the apex of the polygon figures without the
need to trace straight lines. Very small areas and curved contours are also
measured. This it is a computer + digitizer. It consist of

 i) A control panel = which offers a large number number of facilities for
calculation of area and length,

ii) a display screen – which displays numbers, upto eight fingures, measuring units,
memory and functions.

iii) a high frictions roller – for greater precisions.

iv) Trace arm :- Which travels up and down on the surface to be measured.

v) Trace arm facing lever :-It release the trace arm and power gets switched on
when it is lifted.

vi) Trace lens :- a large eccentric lens for wider view.

vii) Trace point :- for putting on top of line to be traced.

viii) LED indicator :- Shows continuous made tracing.

ix) Measuring mode shifting switch :- Point mode and continuous mode can be
attered.

x) Start Switch :- To start the operalers.

The working of a digital planimeter is easy. Point mode is used to measure
straight lines and continuous mode for curved lines. For switching on the power,
trace arm fixing lever is lifted. Required units is set with numbers key and
horizontal and vertical scale ratios are set. Centre mark of the trace lens is set
at the initial point and start switch is prssed. For a straight line measurement ,
the trace mark is set at the other apex of the straight line and start switch is
again depressed. For curved lines other mode is pressed. To find area enclosed by
the figure, ENDA/L key, pressed and area, length etc in given units pushed down
to switch off the power.

This planimeter has some special features like

a) Simultaneous length and area measurement.

b) Metric and British units can be measured.

c) Wide measuring range

e) High resolutions

f) High precisions.

Capacity Contours

The contour plan can be used to calculate the capacity of a reservoirs. The area enclosed by the contours can be measured using planimeter. The volume of water in between 2 contours will be equal to average areas of two contours multiplied by contours interval. Total volume can thus be calculated.

If A_1 , A_2 , A_3 . . . . An are the area of successive contours,

h is the contours interval and

V is the capacity of reservoirs

then by trapezoidal formula ,

$$V = h \left(\frac{A_1 + An}{2} + A_2 + A_3 + \dots An-1 \right)$$

By prismoidal Formula,

$$V = \frac{h}{3} \left(A_1 + 4A_2 + 2A_3 + 4A_4 + \dots + 2A_{n-2} + 4A_{n-1} + An \right)$$

When n is an odd members

$$V = \frac{h}{3} \left(A_1 + 4(A_2 + A_4 + \dots) + 2(A_3 + A_5 + \dots) + An \right)$$

Volumes

1) From cross – sections (for calculating volume of catch work)

a) Average end area (trapezoidal) formula

$$V = d \left(\frac{A_1 + A_n}{2} + A_2 + A_3 + \dots A_{n-1} \right)$$

b) Volume by prismoidal formula

$$V = \frac{d}{3} [(A_1 + A_n) + 4(A_2 + A_4 + \dots) + 2(A_3 + A_5 + \dots)$$

From spot levels (for finding volume of length earth work)

$$V = A \frac{(\sum h_1 + 2\sum h_2 + 3 \sum h_3 + 4 \sum h_4)}{4}$$

A = Area of horizontal base of prism.

h_1 , h_2 are the depth of corners of truncated prism.

From contours (for calculating the reservoir capacity)

i) $V = h \left[\frac{A_1 + A_n}{2} + A_2 + A_3 + \dots A_{n-1} \right)$

when A_1 , A_2 etc are whole area under each contour line.

h = contour interval.

CONTOURING

While introducing surveying, it was mentioned that showing natural and manmade features on land in a plan is topographic surveying. Instead of showing the features only in their

plan view if their positions in elevation are also shown, it will enhance the value of topographic map. The various methods tried to show the relative vertical positions of features in a plan are shading, spot heights, hatching and contour lines, of all these methods commonly used method is by drawing contour line in the plan.

A contour line is a imaginary line which connects points of equal elevations. Such lines are drawn on the plan of the area. Since the water in a still lake is a level surface, its periphery represents a contour line Fig, shows a lake with water surface at a level of 110 m. Its periphery in the plan represents a contour line RL 110 m. if water level goes down by 5 m, the periphery of water shows, contour line of RL 105 m. when periphery of water surface in the pond for various levels are down, it becomes contour map of ground level of the lake.

While conducting surveying, if levels of various points on the ground are also taken, it is possible to draw ground features in the plan as well as draw the contour lines of different elevations. The field and office work involved in drawing contour lines is called contouring.

CONTOUR INTERVALS

The vertical distance between two consecutive contour lines is called contour interval.

The horizontal distance between two points on any two consecutive contours is called horizontal equivalent. Obviously, contour interval divided by horizontal equivalent will give slope of the ground along the lien joining those two points. If the contour interval is small, the undulation of ground is known better. At the same time smaller the contour interval, cost of the survey project is larger. Choice of contour interval for a plan depends upon the following:

i) Nature of the ground

ii) Purpose and extent of map

iii) Scale of map and

iv) Time and funds available
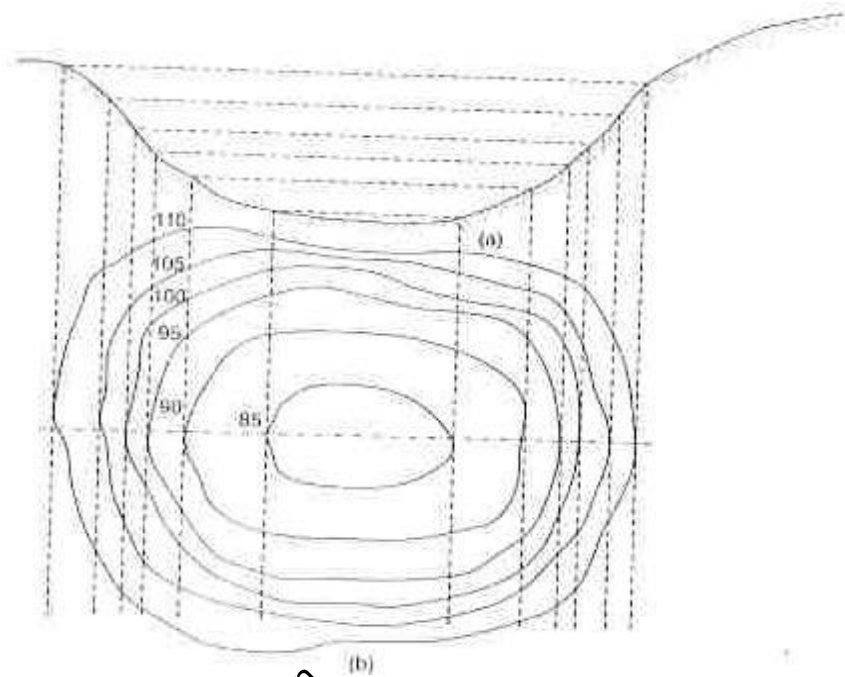
i) **Nature of ground:** If the ground is flat, contour interval selected is small. If the ground is undulating large contour interval is selected, if not done so, the contour lines come too close to each other.

ii) **Purpose and extent of Map:** If survey is intended for detailed earth work calculation small contour interval is preferred. In such case the extent of survey is generally small. For example, in developing building sites. In case of location surveys for roads, railways, sewage lines and for reservoirs contour interval selected is large. In such cases generally the extent of survey is also large.

iii) **Scale of map:** Contour interval selected is inversely proportional to the scale of map. Smaller the scale, larger is the contour interval and larger the scale, smaller the contour interval.

iv) **Time and funds available:** If contour interval is small more time is required in the field work and office work. Hence, requirement of funds is more. If there is limitation of time and fund lager contour interval may be selected.

Considering the above points contour intervals suggested for different purposes are shown in table 1 while table 2 shows suggested scales for different nature of ground and scale.

Table 1: Contour intervals for survey of different purposes

Purpose of survey	Scale	Contour intervals (mm)
Building sites	1:1000 or less	0.2 to 0.5
Town planning, reservoir, etc.	1:5000 to 1:10000	0.5 to 0.2
Location surveys	1:5000 to 1:20000	1 to 3

CHARACTERISTICS OF CONTOURS

The Contours have the following characteristics:

1. Contour lines must close, not necessarily in the limits of the plan
2. Widely spaced contour indicates flat surface
3. Closely spaced contour indicates steep ground
4. Equally spaced contour indicates uniform slope
5. Irregular contours indicate uneven surface
6. Approximately concentric closed contours with decreasing values towards centre indicate pond.

7. Concentric closed contours with increasing values towards centre, indicate hills
8. Contour lines with V – shape with convexity towards higher ground indicate valley
9. Contour lines with U – shape with convexity towards lower ground indicate ridge.
10. Contour lines generally do not meet or intersect each other.
11. If contour lines are meeting each other in some portion, it shows existence of vertical cliff or wall in that portion.
12. If contour lines cross each other, it shows existence of overhanging cliff or a cave.
13. Contours do not have sharp turnings.
14. The direction fo the steepest slope at a point on the contour is at right angles to the contour.

METHODS OF CONTOURING

Contouring consists of finding elevations of various points in the area surveyed. At the same time the horizontal positions of those points should also be found. Thus, it needs vertical control and horizontal control in the work. For vertical control levels, theodolite or clinometers may be used while for horizontal controls chain, compass, plane table or theodolite are used Bases on the instruments used, there can be different methods of surveying.

However, broadly speaking there are two methods of surveying: i) Direct methods, ii) Indirect methods.

Direct method involves finding vertical and horizontal controls of the points which lie on the selected contour line. In indirect method, the levels are taken at some selected points, their levels are reduced and the horizontal controls also carried out. After locating these points in plan, reduced levels are marked and contour lines are interpolated between selected points.

DIRECT METHOD OF CONTOURING

Since in this method points on a selected contour are traced first and then horizontal controls are established this method is also known as tracing out contours. This method is slow, tedious but accurate. It is suitable for small areas.

For vertical control leveling instrument is commonly used. An instrument station is established at a commanding point in the area by taking fly level from a nearby bench mark. Height of the instrument is calculated and the staff reading required for a particular contour is found. For example, if the height of the instrument is 90.8 m, the staff reading for 90 m contour is 0.8m for 89 m it is 1.8 m, for 88 m contour it is 2.8m and for 87m contour the staff reading is 3.8m the instrument man asks staff an to move up o down in the area till the required staff reading is found. For horizontal control for that point is usually exercised with plane table survey. Then staff man is directed to another point on the same contour. After locating few points, plane table person draws eh contour line. Simultaneously 2 – 4 contour lines are traced in the area levelling

instrument can command. Then instrument station is shifted by taking change point. Shifting of leveling and plane table need not be simultaneous. For getting speed in levlling, sometime hand level or on Abney level are used. In this method, after locating a first point on a contour lien say 90 m contour lien the surveyor stands on that point with hand level suspended at a convenient height. For convenient reading the height may b e 1.5 m then a pole with marking at 0.5 m, 1.5 m and 2.5 m may be held at various points in the area to locate contours of 91 m, 90 m, 89 m. for every point selected horizontal control is exercised and plotted. This method is fast but it is at the cost of accuracy.

INDIRECT CONTOURING

As stated earlier in this method points are selected first and their levels are found. For selecting points any one of the following methods may be used:

i) Method of squares

ii) Cross – section method or

iii) Radial line method.

METHOD OF SQUARES

This method is suitable, if the area to be surveyed is not very large and undulation of the ground is not much. In this method the area is divided into a number of squares and all grid points are marked.

 Commonly used size of squares varies from 5 m * 5 m to 20 m * 20 m . By leveling reduced levels of all grid points are obtained. The grid of squares is plotted, reduced levels of all grid points marked and contour lines interpolated.
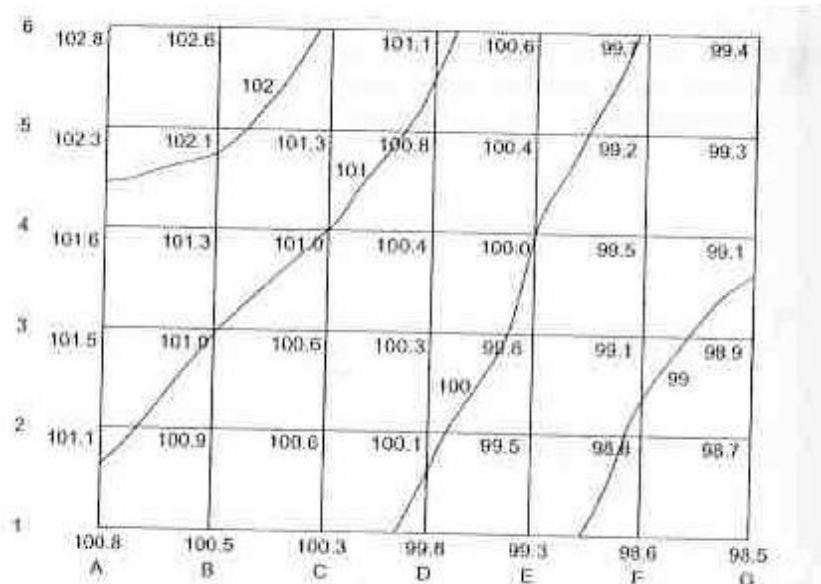
CROSS – SECTION METHOD

In this method also a selected lien cross – sectional readings are taken at regular interval. By usual leveling procedure reduced levels of all selected points on cross sections are found. They are marked on drawing sheets and then contours are interpolated.The spacing of cross – section depends upon the nature of the ground, scale of the map and the contour interval. It varies from 20 m to 100 m. the cross sections may be at closer intervals whenever abrupt changes in levels take place. It may be noted that cross – sectional points always need not be at 90^0 to main line. They may be at different angles also but that angle should be carefully noted down in the field book. This method is suitable for road and railway projects.

RADIAL LINE METHOD

In this method from a selected point radial lines at known intervals are taken. Level readings are taken on every ray at regular interval reduced levels are found and contour maps plotted. Instead of using level and tape, both vertical and horizontal controls can be exercised with tacheometry so that larger area can be easily covered in single setting. This method is ideally suited for hilly areas.
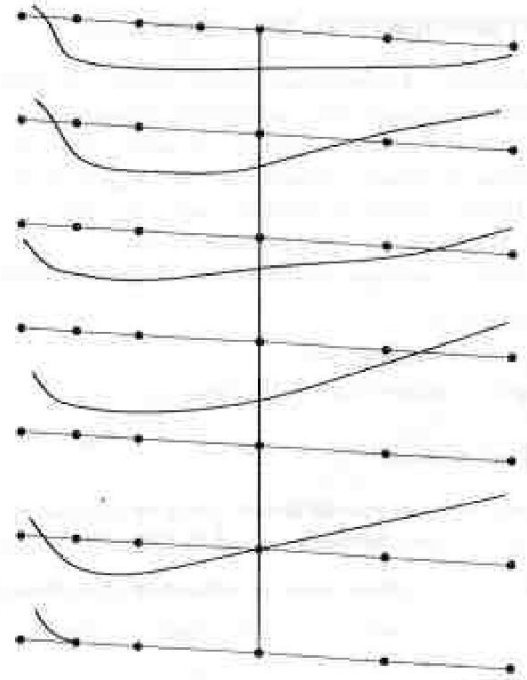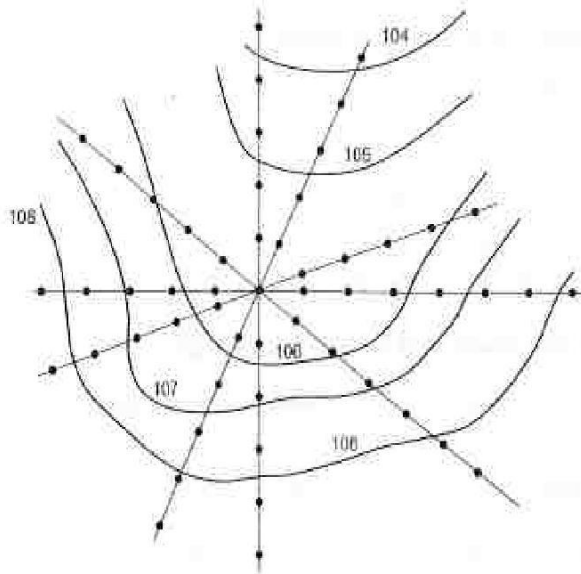
INTERPOLATION OF CONTOURS

After finding RL of many points on the ground and plotted the position of those points. Points on contour lines are identified assuming uniform slope between any two neighbouring points is uniform. In other words, the points on contour lines are interpolated linearly between the two neighbouring points. For example, in Fig 100th contour lies between points D3 and E2 assuming ground slopes uniformly form 100.3 it 99.8 between these two points contour point is located for this purpose any one of the following three methods may be used.

i) Estimation

ii) Arithmetic calculation

iii) Mechanical or Graphical method.

Estimation: By eye judgment or estimation the point on contour is located between the two points. For example, between D3 and E3 where elevations are 100.3 and 99.8 m, the contour point is estimated at a distance . From E3. Similarly the point on DL E2 where RLs are 100.1

and 99.5 the point should be at a distance This method is rough and is used for small – scale works. However, it is very fast.

Arithmetic calculation: In this method, instead of estimating the position of points on contour, arithmetic calculations are made for locating the points on contour.

Mechanical or Graphical method: Any one of the following two methods are used for linearly interpolating contour points using tracing sheet.

Method 1: On a tracing sheet several parallel lines are drawn at regular interval. Every fifth or tenth line is made dark for easy counting. If RL of A is 98. 4 m and that of B is 100.2 m assume bottom most dark line represents 98 m RL and every parallel line is at 0.2 m intervals. Then hold a point on second parallel line on A. Rotate tracing sheet so that 100.2th parallel line passes through point B. then intersection of dark lines on AB represents the points on 99 m and 100 m contours similarly. Contour points along any line connecting two – level points can be obtained and contour lines interpolated and pricked. This method maintains the accuracy of arithmetic calculations, at the same time is fast also.
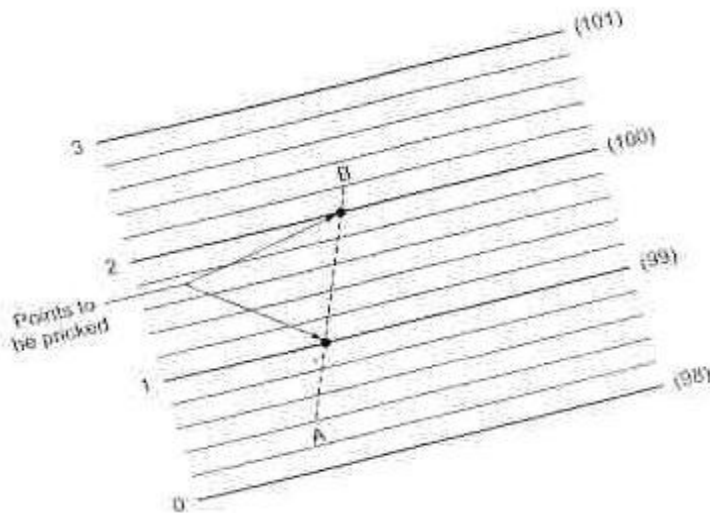
Method II : In this method a line PQ is drawn on a tracing sheet from the mid – point of PQ say R a perpendicular line RO is drawn. 'O' is selected at any convenient distance. PQ is divided into a number of equal parts, say 20 parts. Then the radial lines from 'O' to these equally spaced points are drawn.A number of guide lines 1-1, 2-2, etc. are drawn parallel to PQ. To interpolate between two points A and B on drawing sheet, tracing sheet is held with its guide lines parallel to AB. OQ is assigned a contour lien point just below that of RL of A. Of dark lines are at every 5 ray interval, and contours are required at every 1 m interval, the interval between two consecutive rays is 0.2 m. Appropriate ray is made to appear on A and tracing sheet is rotated till the ray corresponding to B coincides with B. Then the contour points on AB correspond to the dark lines intersection with AB. These points are produced and the contour points on lien AB are obtained. Thus, in this case also exact interpolation is made mechanically.
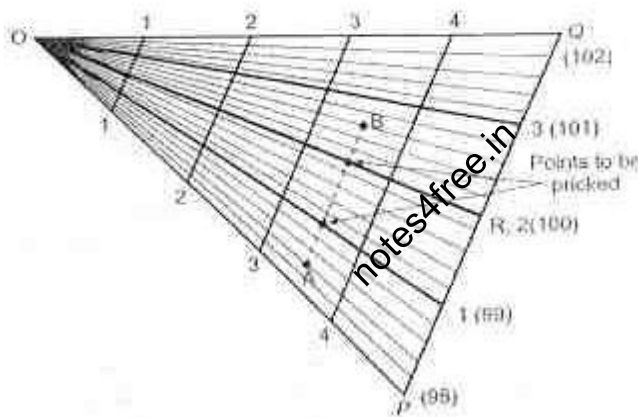
DRAWING CONTOURS

After locating contour points between a network of guide points, smooth contour lines are drawn connecting corresponding points. For drawing contour lines French curves should be used. A surveyor should not lose the sight of characteristics of the contours. Brown colour is preferred to draw the contours so that they can be easily distinguished from other features; Every fifth contour is made thicker for easy readability. On every contour line its elevation is written either above, below or by breaking the line. If map size is large, it is written at the ends also. However, in writing these elevations uniformly should be maintained.

CONTOUR GRADIENT

A contour gradient is a line having uniform slope on the ground. Method of plotting contour gradient on a plan and identifying it on the ground are discussed below.

i) **Contour gradient on a map:** The contour lines are at 1 m interval and the map is to a scale of 1:500. Since slope is assumed uniform between two contour lines, the length of gradient line between two contour lines should be equivalent to 20 m on the ground, it should be 20/500 m on paper, 40 mm from starting point a draw an arc of radius 40 mm to interest next contour line at b. from b this procedure is repeated to get point c line joining a,b,c… is the desired gradient line.

ii) **Contour gradient on ground:** For setting contour gradient on ground level a clinometers may be used.If a clinometers is used, it is set at the required slope. A person stands near point A, suspends the sloping clinometers at a convenient height to view. The looks through clinometers, and directs a person holding ranging pole, which is tied with a target at the same height as the height of instrument from the ground point A. Tape is used to maintain the required distance from A. after getting next point B, the clinometers is shifted to point B and the staff man moves to next probable point. The procedure is continued till the last point is established. The method is fast but any small angular error gets magnified.

USES OF CONTOUR MAPS

Contour maps are extremely useful for various civil engineering works as explained below:

i) **Preliminary selection of project sites**:

Characteristics of contour lines give considerable information about nature of ground. Sitting in the office studying contour lines, a civil engineer decides various possible sites for his project.

ii) **Drawing of sections**:

From contour plan, it is possible to study profile of the ground along any line, which is normally required for earthwork calculation along a formation ground. Intersection of line AB with contour lines are projected on the x – axis. Along the y – axis the corresponding heights as found from contour lines are mar4ked and then the profile of the ground is obtained.
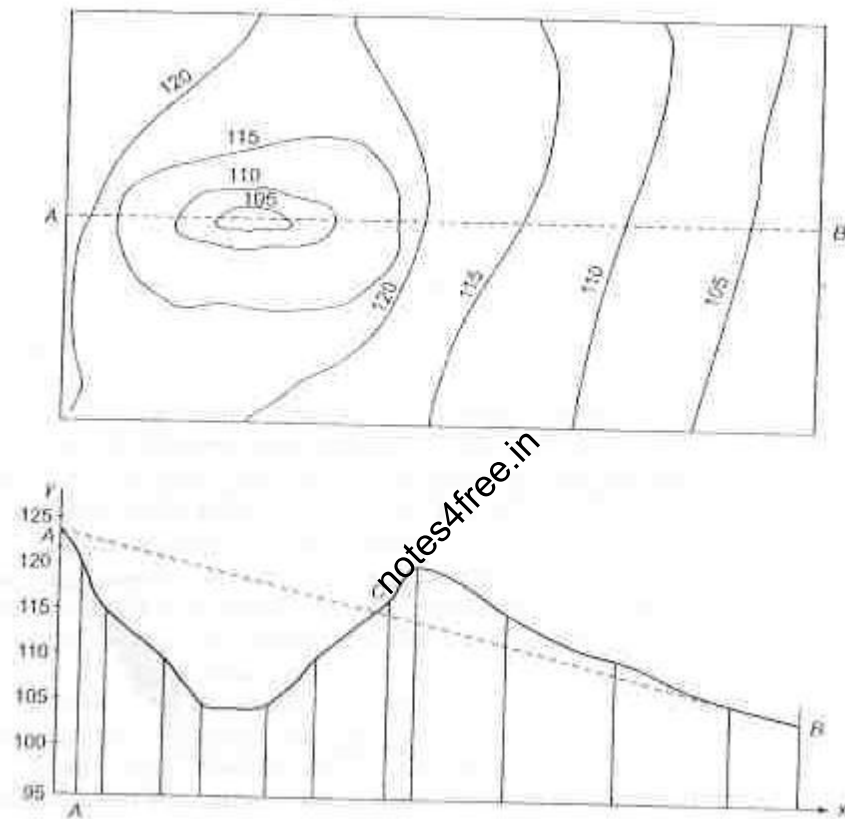
iii) **Determination of intervisibility:**

If intervisibility of any two points is to be checked, using contour, profile of the ground along the line joining those two points can be drawn. Then the line joining those two points is drawn. If the ground portion is above this portion, the two stations are not intervisible.

iv) **Location of routes:**

The routes of railway, road, canal or sewer lines can be decided with the help of contour maps. After deciding the gradient of the route, it can be set on the map as explained.

v) **Determining catchment Area;**

The area on which fallen rainwater drains into river at a particular point is called catchment area of the river at that point. This area can be determined from contour plans. The catchment area is also known as drainage area. First the line that separates the catchment basin from the rest of area is drawn. This is called watershed time. It normally follows ridge line.Then the area within watershed line is measured. This area is extremely useful in studying flood level and quantity of water flow in the river.
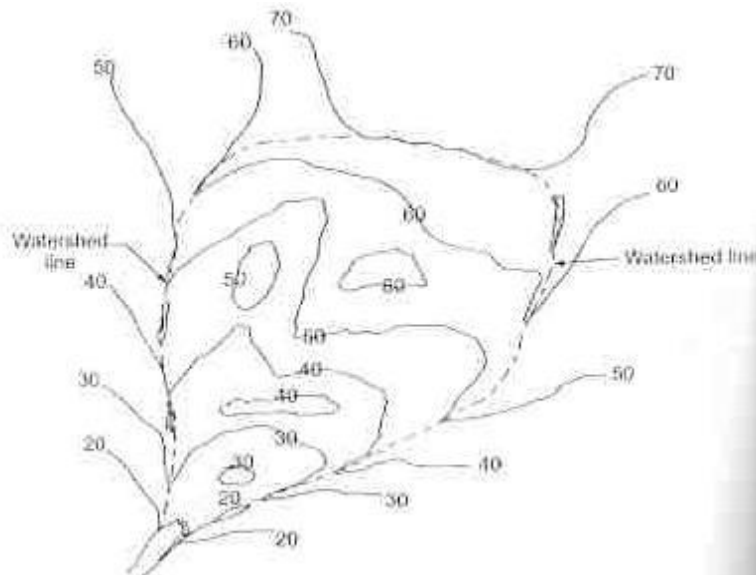
vi) **Calculation of reservoir capacity:**

The submerged area and the capacity of a proposed reservoir by building bund or dam can be found by using contour maps. After determining the height of the dam its full reservoir level is known. Then area between any two contour lines and the dam line is measured by using plan meter thus if A1,A2, ……An are the areas within contours and h is the contour interval reservoir capacity is given.

SOLVED QUESTION AND ANSWERS

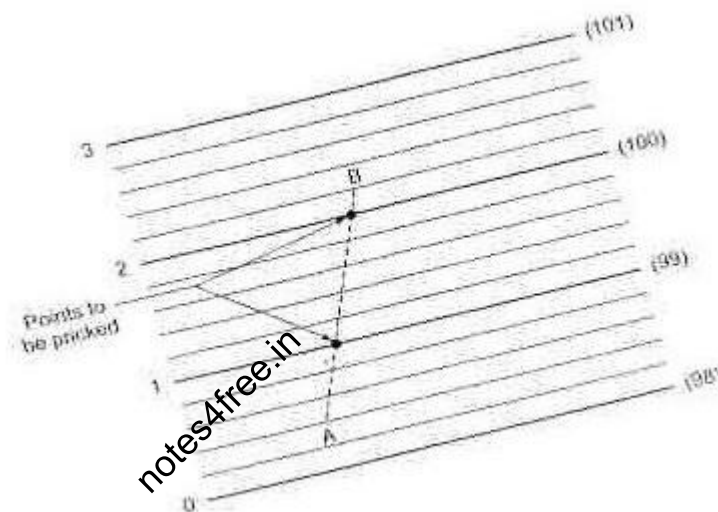## 1. INTERPOLATION OF CONTOURS (Dec11, june-july11)

After finding RL of many points on the ground and plotted the position of those points. Points on contour lines are identified assuming uniform slope between any two neighbouring points is uniform. In other words, the points on contour lines are interpolated linearly between the two neighbouring points. For example, in Fig 100th contour lies between points D and E assuming ground slopes uniformly form 100.3 it 99.8 between these two points contour point is located for this purpose any one of the following three methods may be used.

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For example, between D and E where elevations are 100.3 and 99.8 m, the contour point is

estimated at a distance . From E . Similarly the point on D E where RLs are 100.1 and 99.5 the point should be at a distance This method is rough and is used for small – scale works. However, it is very fast. Arithmetic calculation: In this method, instead of estimating the position of points on contour, arithmetic calculations are made for locating the points on contour. Mechanical or Graphical method: Any one of the following two methods are used for linearly interpolating contour points using tracing sheet.

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2. How do you trace a contour gradient of 1 in 50 on a map having contour interval 2.0 m. (Dec11, june-july11)

i) Contour gradient on a map: The contour lines are at 20 m interval and the map is to a scale of 1:500. Since slope is assumed uniform between two contour lines, the length of gradient line between two contour lines should be equivalent to 50 m on the ground, it should be 50/500 m on paper, 40 mm from starting point a draw an arc of radius 40 mm to interest next contour line at b. from b this procedure is repeated to get point c line joining a,b,c… is the desired gradient line.

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