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MINE SURVEYING

TAPE CORRECTIONS

Following corrections are applied to the measured distances

Correction for Standardisation

The correction is required if the true length (actual length, on site length) of the tape is not equal to the nominal length (factory length, designated length).

Correction per tape length (C) = l' – l

where l is nominal(designated, factory length) length of the tape and l' is actual length(on site) of tape.

The correction is positive when the actual length (l') is greater than the nominal length (l) and vice versa.

Total correction in measured distance

$$C_a = \frac{(l' - l)}{l} \times L = \frac{C \times L}{l}$$

Correction for Slope

Correction $C_g = D - L = -L\,(1 - \cos\theta) = -2L \sin^2\theta/2$

where D = horizontal equivalent, L = slope distance and θ is angle of slope.

Alternatively $C_g = \sqrt{(L^2 - h^2)} - L$

where h is difference in elevation of the end points.

Correction for Pull

The correction for pull (C_p) is given by

$$C_p = \frac{(P - P_0)L}{AE}$$

where P is pull applied during measurement (N), P_0 is standard pull (N), L is measured length, A is cross sectional area of the tape and E is Young's modulus of elasticity (for steel E = 2 x 10^5 N/mm^2 or 2 x 10^5 MPa).

If the pull applied at the tape is greater than the standard pull, the actual length (on site) of the tape is greater than the nominal length (factory length), and a *positive* correction is required.

Correction for Temperature

The temperature correction is given by

$$C_t = \alpha(T - T_0)L$$

where α is coefficient of linear expansion, T is mean temperature of the tape (^{0}C) and T_0 is standard temperature (^{0}C). The sign of C_t is directly given by above equation.

For steel tapes, $\alpha = 1.15 \times 10^{-15}$.

Correction for Sag

Correction for sag is given by

$$C_s = -\frac{l_1(wl_1)^2}{24P^2}$$

where w is weight of tape per unit length (N/m), P is applied pull (N) and l_1 is length of the tape suspended between the supports (m).

The above equation can also be written as

$$C_s = \frac{l_1 W_1^2}{24P^2}$$

where W_1 = total weight of the tape between supports.

The sag correction is *always* negative.

If the tape is suspended between the supports is pulled by a large force, there is a decrease in the sag and an increase in the length of the tape because of tension applied. Normal tension (P_n) is the theoretical pull at which the pull correction is numerically equal to the sag correction.

$$\frac{(P_n - P_0)l_1}{AE} = \frac{l_1 W_1^2}{24P_n^2}$$

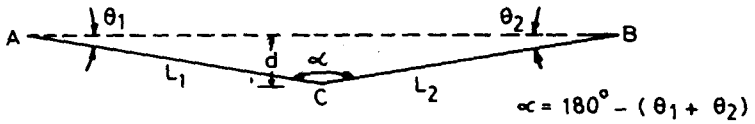
or

$$P_n = \frac{0.204 W_1 \sqrt{AE}}{\sqrt{p_n - P_0}}$$

This equation can be solved by trial and error.

Correction for Misalignment

If the survey line is not accurately ranged out, the error due to misalignment occurs. The measured distance is always greater than the correct distance, and hence the error is positive and the *correction is negative*.



The correction due to misalignment is given by

$$C_m = (L_1 \cos\theta_1 + L_2 \cos\theta_2) - (L_1 - L_2)$$

or $$C_m = \sqrt{L_1^2 + L_2^2 - 2L_1L_2\cos\alpha} - (L_1 + L_2)$$

or $$C_m = \left[\sqrt{L_1^2 - d^2} + \sqrt{L_2^2 - d^2}\right] - (L_1 + L_2)$$

Correction for Local Scale Factor

A short traverse on a plane surface can be easily represented on a plan with grid lines without any apparent distortion, but in the case of larger areas the curvature of the spherical surface of earth greatly affect the actual shape of the map. Objects on the curved surface of earth can not be represented on a flat sheet of paper in the same relative positions which they occupied on the globe. A slight difference between the grid distances occurs because the meridians converge and the grid lines are drawn parallel to one central meridian. Therefore, in precise surveys of larger areas the distances computed from coordinate will require a correction by a factor known as "local scale factor". The value of the local scale factor will vary with geographical position and depending upon the departure east or west from the central meridian.

Example

What is normal pull ? Show that for a chain of 3 mm^2 area and 0.48 Kg weight (E = $2x10^6$ Kg/cm^2) Standardised at 8 Kg tension, the normal pull is 12 Kg.

Solution

The pull which when applied to a tape supported in air over two ends equalizes the correction due to pull and the correction due to sag is known as normal pull.

The correction for pull = $c_1 = (P - P_0)/AE$ (+ ve)

The correction for sag $c_2 = LW^2/24P^2$ (-ve)

equating numerically the two equations , we get

$$(P-P_0)/AE = LW^2/24P^2$$

$$P = (0.204W\sqrt{AE} / \sqrt{(P-P_0)}$$

The value of normal pull `P' is determined by trial and error with the help of above equation.

Area $A = 3\ mm^2 = 0.03\ cm^2$

$E = 2 \times 10^{6}$ Kg/cm^2

W - 0.48 Kg, P0 = 8 Kg

to prove P = 12 Kg

$P = 0.204 \times 0.48 \sqrt{(0.03 \times 2 \times 10^6)} / \sqrt{(12 - 8)}$

$= 11.993 = 12$ Kg Hence Proved.

Example

An mining land was measured with an incorrect 30 m chain and a plan was drawn. From these measurements the area on the plan was measured and calculated and was found to be 16.25 km^2. Find to correct area of the mining land, if the length of the chain was 30.06 m.

Solution

True area = $(L/30)^2 \times 16.25$ km^2 = $(30.06/30)^2 \times 16.25$ km^2 = 16.315 km^2

Example

A steel tape of nominal length 30 m was used to measure a line AB by suspending it between supports. If the measured length was 29.861 m when the slope angle was $3^0 45'$, and the mean temperature and tension applied were respectively 10^0C and 100 N, determine the corrected horizontal length.

The standard length of the tape was 30.004 m at 20^0C and 44.5 N tension. The tape weighted 0.16 N/m and had a cross-sectional area of 2 mm^2. E = 2 x 10^5 N/mm^2. α = 1.12 x 10^{-5} per ^{0}C.

Solution

Slope correction

$= - L(1 - \cos\theta)$

$= - 29.861 (1 - \cos 3^0 45')$

$= - 0.064$ m

Standardisation correction

$$= L \times \left(\frac{l' - l}{l} \right)$$

$$= 29.861 \left(\frac{30.004 - 30.000}{30.000} \right) = +0.004 \text{ m}$$

Temperature correction

$$= \alpha(T - T_0)L$$

$$= 1.12x10^{-5}(10 - 20)\,x29.861 = -0.003\,m$$

Pull correction

$$= \frac{(P - P_0)L}{AE}$$

$$= \frac{(100 - 44.5)\,x29.861}{2x2x10^5} = +0.004\,m$$

Sag correction

$$= \frac{w^2 l_1^3}{24P^2}\cos^2\theta$$

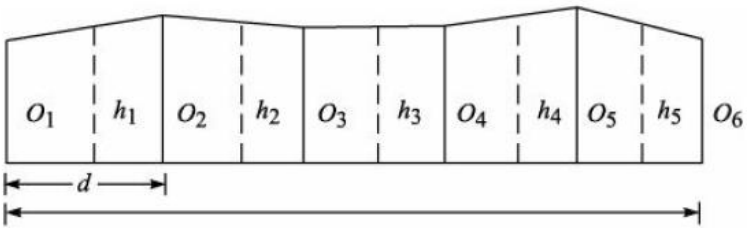
$$= \frac{-(0.16)^2\,x\,(29.861)^3\,x\cos^2 3^0 45'}{24x(100)^2} = -0.003\,m$$

Total correction = - 0.064 + 0.004 - 0.003 + 0.004 - 0.003 = - 0.062

Correct horizontal distance = 29.861 – 0.062 = 29.799 m

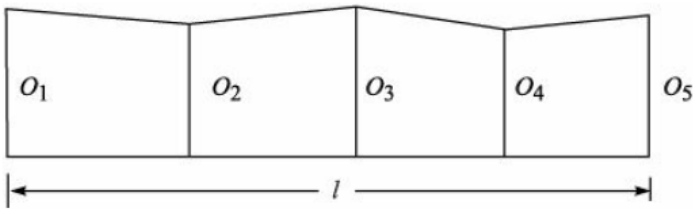
COMPUTATION OF AREA

Mid-ordinate Rule



Area = Common distance × sum of mid-ordinates

Average-ordinate Rule



Area = (sum of ordinates/no. of ordinates) x length of base line

Example

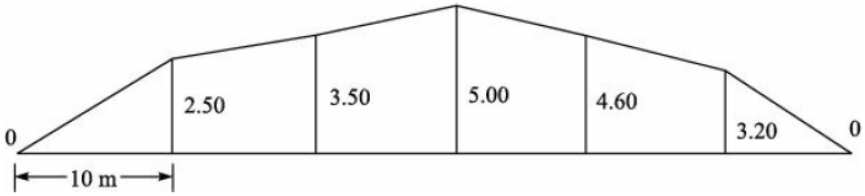
The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m:

0, 2.50, 3.50, 5.00, 4.60, 3.20, 0 m

compute the area between the chain line, the irregular boundary line and the end offsets by (i) the mid-ordinate rule (ii) average ordinate rule.

Solution

Figure



Mid ordinate rule

$h_1 = (0 + 2.50)/2 = 1.25$ m; $h_2 = (2.50 + 3.50)/2 = 3.00$ m;

$h_3 = (3.50 + 5.00)/2 = 4.25$ m; $h_4 = (5.00 + 4.60)/2 = 4.80$ m

$h_5 = (4.60 + 3.20)/2 = 3.90$ m; $h_6 = (3.20 + 0)/2 = 1.60$ m

Required area = $10(1.25 + 3.00 + 4.25 + 4.80 + 3.90 + 1.60) = 10 \times 18.80 = 188$ m^2

Average ordinate rule

Here, d = 10 m and n = 6 (no. of divisions)

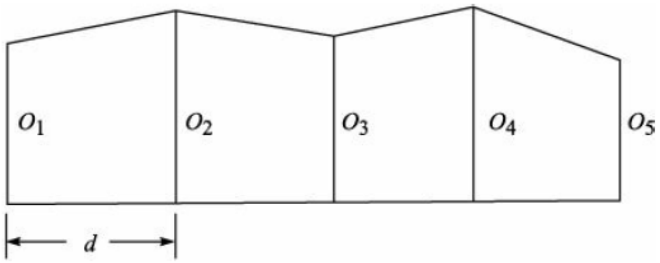
Base length = $10 \times 6 = 60$ m

Number of ordinates = 7

$$\text{Required area} = 60\,x\left(\frac{0+2.50+3.50+5.00+4.60+3.20+0}{7}\right) = 161.14 \text{ m}^2$$

Trapezoidal Rule

While applying the trapezoidal rule, boundaries between the ends of ordinates are assumed to be straight. Thus the areas enclosed between the base line and the irregular boundary line are considered as trapezoids.



Total area = (d/2)(1st ordinate + last ordinate + 2 x sum of other ordinates)

Simpson's rule

Total area = (d/3)(1st ordinate + last ordinate + 4 x sum of even ordinates + 2 x sum of remaining odd ordinates)

Limitation This rule is applicable only when the number divisions is even, i.e. the number of ordinates is odd.

☞ Sometimes one, or both, of the end ordinates may be zero. However, they must be taken into account while applying these rules.

Example

The following offsets were taken at 15 m intervals from a survey line to an irregular boundary line:

3.50, 4.30, 6.75, 5.25, 7.50, 8.80, 7.90, 6.40, 4.40, 3.25 m

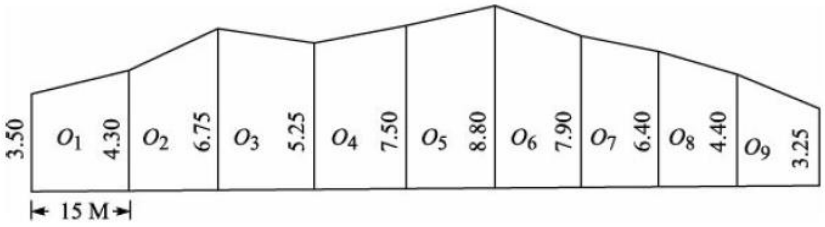
Calculate the area enclosed between the survey line, the irregular boundary line, and the first and last offsets, by

(a) The trapezoidal rule

(b) Simpson's rule

Solution

Figure



Trapezoidal Rule

Required area

=(15/2) {3.50 + 3.25 + 2(4.30 + 6.75 + 5.25 + 7.50

+ 8.80 + 7.90 + 6.40 + 4.40)} = (15/2){6.75 + 102.60} = 820.125 m^2

Simpson's Rule

If this rule is to be applied, the number of ordinates must be odd. But here the number of ordinate is even (ten).

So, Simpson's rule is applied from O_1 to O_9 and the area between O_9 and O_{10} is found out by the trapezoidal rule.

A_1 = (15/3){3.50 + 4.40 + 4(4.30 + 5.25 + 8.80 + 6.40) + 2 (6.75 + 7.50 + 7.90)}

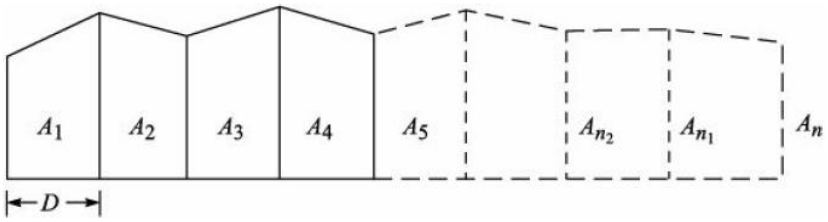
= (15/3) {7.90 + 99.00 + 44.30} = 756.00 m^2

A_2 = (15/2) {4.40 + 3.25} = 57.38 m^2

Total area = A_1 + A_2 = 756.00 + 57.38 = 813.38 m^2

COMPUTATION OF VOLUME

Calculation of Volume by trapezoidal rule



Volume = (Common distance/2) x {area of first section + area of last section

+ 2 (sum of area of other sections)}

Calculation of Volume by Prismoidal rule

V = (common distance/2) {area of first section + area of last section + 4(sum of areas of even sections) + 2 (sum of areas of odd sections)}

☞ *The prismoidal formula is applicable when there are an odd number of sections. If the number of sections is even, the end strip is treated separately and the area is calculated according to the trapezoidal rule. The volume of the remaining strips is calculated in the usual manner by the prismoidal formula. Then both the results are added to obtain the total volume.*

Example

The areas enclosed by the contours in a lake are as follows:

Contour (m)	*270*	*275*	*280*	*285*	*290*
Area (m^2)	*2,050*	*8,400*	*16,300*	*24,600*	*31,500*

Calculate the volume of water between the contours 270 m and 290 m by (a) the trapezoidal formula, and (b) the prismoidal formula.

Solution

Volume according to trapezoidal formula

$$V = (5/2) \times \{2{,}050 + 31{,}500 + 2(8{,}400 + 16{,}300 + 24{,}600)\} = 330{,}375 \text{ m}^3$$

Volume by prismoidal formula

$$V = \{2{,}050 + 31{,}500 + 4(8{,}400 + 24{,}600) + 2(16{,}300)\} = 330{,}250 \text{ m}^3$$

Example

An embankment of 10 m width and side slopes 1½ : 1 is required to be made on a ground which is level in a direction transverse to the centre line. The central heights at 40 m intervals are as follows:

0.90, 1.25, 2.15, 2.50, 1.85, 1.35, and 0.85

Calculate the volume of earth work according to the trapezoidal formula.

Solution

Area, $\Delta = (b + sh) \times h$

$\Delta_1 = (10 + 1.5 \times 0.90) \times 0.90 = 10.22 \text{ m}^2$

$\Delta_2 = (10 + 1.5 \times 1.25) \times 1.25 = 14.84 \text{ m}^2$

$\Delta_3 = (10 + 1.5 \times 2.15) \times 2.15 = 28.43 \text{ m}^2$

$\Delta_4 = (10 + 1.5 \times 2.50) \times 2.50 = 34.38 \text{ m}^2$

$\Delta_5 = (10 + 1.5 \times 1.85) \times 1.85 = 23.63 \text{ m}^2$

$\Delta_6 = (10 + 1.5 \times 1.35) \times 1.35 = 16.23 \text{ m}^2$

$\Delta_7 = (10 + 1.5 \times 0.85) \times 0.85 = 9.58 \text{ m}^2$

Volume according to trapezoidal formula:

$$V = (40/2) \times \{10.22 + 9.58 + 2(14.84 + 28.43 + 34.38 + 23.63 + 16.23)\}$$

$$= 20\{19.80 + 235.02\} = 5{,}096.4 \text{ m}^3$$

DISTORTED OR SHRUNK SCALES

Due to change in climatic conditions, the plans and maps generally get distorted. If no graphical scale is drawn on the plan, correct scale of the distorted plan (or map), may be calculated by the following method:

Step 1

Measure a distance between any two well defined points on the plan and calculate its corresponding ground distance from the scale i.e.,

1 cm = x metres. Let it be l metres.

Step 2

Measure the horizontal distance between the same points on the ground by chaining. Calculate the distance on plan with the scale. Let it be y cm.

Step 3

Calculate the shrinkage ratio or shrinkage factor which is equal to shrunk length / the actual length.

Step 4

Shrunk scale of plan = Shrinkage factor × Original scale.

Example

The area of a plot on a map is found, by planimeter, to be 10.22 cm^2. The scale of the map was 1:25000, but at present it is shrunk such that a line originally 5 cm on the map is now 4.8 cm. What is the correct field area in hectares ?

Solution

The area of the plot measured by the planimeter on the shrunk map = 10.22 cm^2 (Given)

The ratio of shrinkage = 4.8/5.0 = 0.96

The ratio of shrinkage of the area = $(0.96)^2$ = 0.9216

The area of the plot on unshrunk map on 1:25,000 scale = 10.22/0.9216 = 11.08941 cm^2

Area of 1 cm^2 on scale 1:25000 = 250 × 250 = 62500 m^2

∴ Area of 11.08941 cm on scale 1 : 25000 = 62500 × 11.08941

= 693088.12 m^2

= 69.3088 hectares.

Example

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

Solution

According to the given conditions,

19.5 cm on the map was originally 20 cm.

Therefore, 1 cm on the map was originally (20/19.5) cm, and

1 cm^2 on the map was originally ($20^2/19.5^2$) cm^2

125.50 cm^2 was originally ($20^2/19.5^2$) x 125.50 = 132.0184 cm^2

Scale of map was 1 cm = 40 m

1 cm^2 = 1,600 m^2

Area on the ground = 1,600 × 132.0184 = 211,229.44 m^2

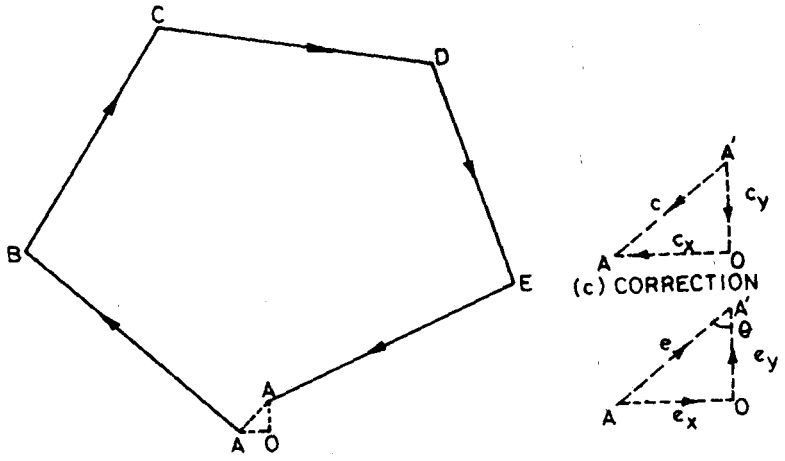
Since the chain was 0.05 m too long,

True area = ($20.05^2/20^2$) × 211,229.44 = 212,286.90 m^2 = 21.2286 hectares

(1 hectare = 10,000 m^2)

CLOSING ERROR IN A TRAVERSE

If a closed traverse is plotted according to the field (procedure) measurements and if the end of traverse will not coincide exactly at the starting point. This discrepancy is due to the errors in the field observations i.e. magnetic bearings and linear distances. Such error is known as closing error.



The x and y components of the closing errors can be determined as

$$e_x = \sum D$$

$$e_y = \sum L$$

However in the case of link traverse

$$e_x = X' - X$$

and $$e_y = Y' - Y$$

where X' and Y' are computed coordinates of the final control point, and X and Y are the corresponding known coordinates.

The total closing error is given by

$$e = \sqrt{(e_x)^2 + (e_y)^2}$$

The direction of the closing error AA' is given by

$$\tan\theta = \frac{e_x}{e_y}$$

The signs of e_x and e_y will define the quadrant in which the closing error lies.

The signs of corrections c_x and c_y will be opposite to those of errors.

$$c_x = -e_x$$

and $$c_y = -e_y$$

Surveyor's Compass (Miner's Dial)

The surveyor's compass (see comparison table)) is an instrument for determining the horizontal direction of a line with reference to the direction of a magnetic needle.

The needle is balanced at its center on a "pivot" so that it swings freely in a horizontal plane. The pivot is at the center of a horizontal circle which is graduated to degrees and half-degrees, and numbered from two opposite zero-points each way to 90°.

The zero-points are marked with the letters N and S, and the 90° points are marked E and W.

The circle is covered with a glass plate to protect the needle and the graduations.

A screw is provided for raising the needle from the pivot by means of a lever. The needle always should be raised when the compass is lifted or carried, to prevent dulling the pivot-points; a dull pivot-point is a source of error.

Both the circle and the pivot are secured to a brass frame, on which are two vertical sights so placed that the plane through them also passes through the two zero-points at the circle. This frame rests on a tripod and is fastened to it by means of a ball-and-socket joint. On the frame are two spirit levels at right angles to each other, which afford a means of levelling the instrument. This ball-and-socket joint is connected with the frame by means of a spindle which allows the compass-head to be revolved in a horizontal plane, and to be clamped in any position.

The magnetic needle possesses the property of pointing in a fixed direction, namely, the "**Magnetic Meridian**". The horizontal angle between the direction of this meridian and of any other line may be determined by means of the graduated circle, and this angle is called the "**Magnetic Bearing**" of the line, or simply its "Bearing".

If the bearings of two lines are known, the angle between them may be computed. Bearings are counted from 0° to 90°, the 0° being either at the N or the S point and the 90° either at the E or the W point. The quadrant in which a bearing falls is designated by the letters N.E., S.E., S.W., or N.W. For example, a line makes an angle of 20° with the meridian and is in the southeast quadrant, its bearing is written S 20°E.

Sometimes the bearing is reckoned in a similar manner from the "true bearing". In general, this will not be the same as the magnetic bearing. True bearings are often called "**azimuths**", and are commonly measured from the south point right-handed (clockwise) to 360°; a line running due west has an azimuth of 180°; a line due east an azimuth of 270°. Sometimes, however, the azimuth is measured from the north as, for instance, when observing the azimuth of the Pole-Star.

Method of Taking a Magnetic Bearing

The surveyor's compass is set up (and levelled) at same point on the line whose bearing is desired. The needle is let down onto the pivot, and the compass sights pointed approximately along the line.

While looking through the two sights the surveyor turns the compass-box so that they point exactly at a lining pole or other object marking a point on the line.

The glass should be tapped lightly over the end of the needle to be sure that the latter is free to move. If it appears to cling to the glass this may be due to the glass being electrified, which condition can be removed at once by placing the moistened finger on the glass.

The position of the end of the needle is then read on the circle and recorded. Bearings are usually read to the nearest quarter of a degree although it is possible to estimate somewhat closer.

Since the needle stands still and the box turns under it, the letters E and W on the box must be reversed from their natural position so that the direct reading of the needle will give not only the angle but also the proper quadrant.

When the north point of the compass-box is toward the point whose bearing is desired, read the north end of the needle. When the south point of the box is toward the point, read the south end of the needle.

If a bearing of the line is taken looking in the opposite direction, it is called the "**reverse bearing**". Reverse bearings should be taken to check for error.

BEARING OF LINES

The bearing of a line is the horizontal angle which the line makes with some reference direction or meridian. The reference direction employed in surveying may be (I) a true meridian (ii) a

magnetic meridian or (iii) an arbitrary or assumed meridian. Magnetic meridian is used in plane surveys.

True Meridian

The true meridian passing through a point is the line in which the earth's surface is interested by a plane through the north and south poles and the given point.

Magnetic Meridian

The magnetic meridian of a place is the direction indicated there by a freely floating and properly balanced magnetic needle, uninfluenced by local attractive forces. Magnetic meridian does not coincide with true meridian except in certain localities, and the horizontal angle between the two directions is termed as Magnetic Declination of the needle.

Arbitrary Meridian

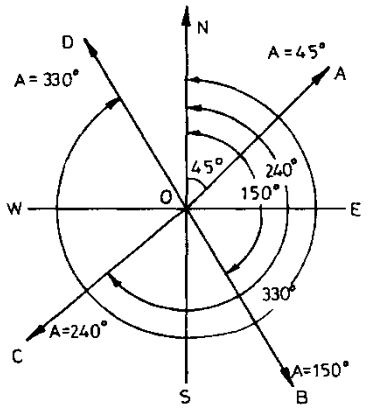
For small surveys, especially in mapped country, any convenient direction may be assumed as a meridian. This artificial meridian is usually the direction from a survey station either to same well defined and permanent point or to an adjoining station.

Designation of Bearings

The bearings are designated by the following two systems:

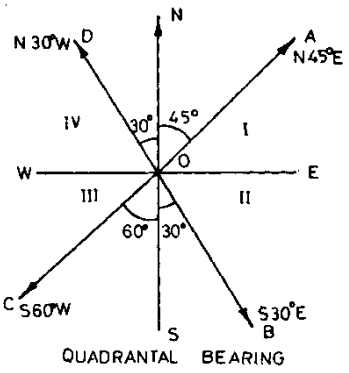
1. Whole circle system,
2. The quadrantal system

Whole Circle System. The bearing of lines, in this system, are measured from the *north* point towards the line in a *clockwise* direction. This is termed as the whole circle bearing of the line and expressed usually as W.C.B.



In this system, the whole circle bearing of a line may have any value from 0° to 360°. The letters N, S, E and W are not at all used to designed the bearing.

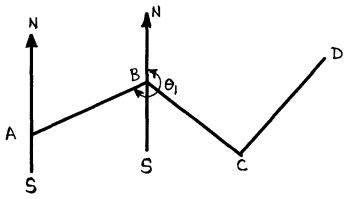
Quadrantal System. The bearing of a line is measured *clockwise or anticlockwise* from the *north and south* point whichever is nearer to the line towards the east or west. The plane around the station is divided into four quadrants.



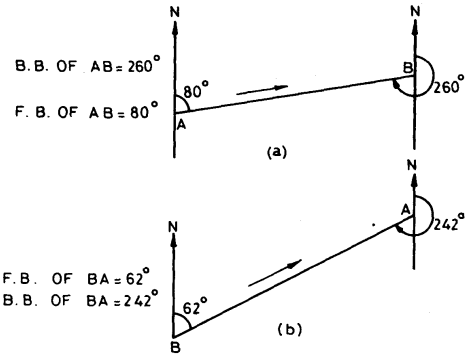
The first quadrant is denoted by NE, second by SE, third by SW and fourth by NW. The quadrantal bearing never exceeds 90°. The bearings obtained in surveyor's compass are the quadrantal bearings.

Fore and back Bearing

The bearings of a survey line in the direction of the survey is called "fore bearing" whereas its bearing in an opposite direction is known as its "back bearing".



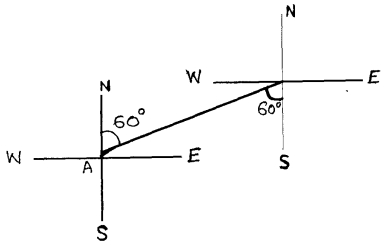
In given figure, the bearings of the line AB from the point A to the point B is termed as the fore bearing or simply the bearings of the line AB. The bearing from B to A is termed as back bearing of the line AB or the fore bearing of line BA. from the figure it is clear that the fore bearing of the line AB is θ and the back bearing of AB is θ_1 .



In the *whole circle system*, the back bearing of a line may be obtained by the fore bearing from the following relationship :

$$\text{Back bearing} = \text{fore bearing} \pm 180^0$$

Plus sign may be used when the fore bearing is less than 180^0 and minus sign may be used when the fore bearing is more than 180^0.



Following figures show a few examples.

In the *quadrantal system*, the fore and back bearing of a line will be numerically same but will be in the diametrically opposite quadrants, i.e. if the fore bearing of a line is $N60^0E$ then its back bearing will be $S60^0W$. This is illustrated in Figure.

Therefore, the back bearing of a line may be obtained by simply interchanging N for S, or S for N, and E for W, or W for E.

Reduced Bearings (or Quadrantal Bearings)

For finding the values of the trigonometrical functions of whole circle bearings exceeding 90^0, reference has to be made to the tables to the corresponding angles which are less than 90^0 and which possess the same numerical values of the functions. This angle is called the **reduced bearing**. The following table may be referred to obtain the reduced bearings from the whole circle bearings.

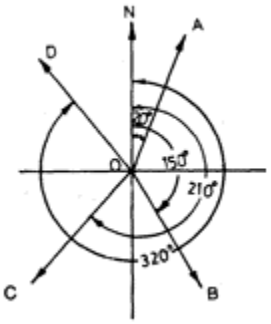
No.	W.C.B.	Corresponding R.B.	Quadrant
1	0^0 and 90^0	W.C.B.	NE
2	90^0 and 180^0	180^0-W.C.B.	SE
3	180^0 and 270^0	W.C.B. - 180^0	SW
4	270^0 and 360^0	360^0 - W.C.B.	NW

Example

Convert the following whole circle bearings to the quadrantal bearings: (i) 20^0; (ii) 150^0 (iii)210^0; and (iv) 320^0.

Solution

The lines OA, OB, OC and OD show the given directions.



The required quadrantal bearings are:

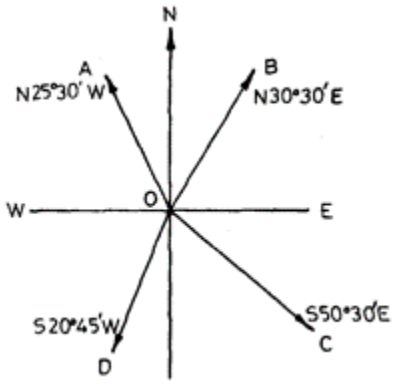
Line OA	N 20^0 E
Line OB	S 30^0 E
Line OC	S 30^0 W
Line OD	N 40^0W

Example

Convert the following quadrantal bearings to the whole circle bearings: (i) N 25^0 $30^{'}$ W; (ii) N 30^0 30' E; (iii) S 20^0 45' W; (iv) S 50^0 30' E

Solution

The lines OA, OB, OD and OC show the given lines.



The whole circle bearing can be written directly.

Line OA	$360^0 - 25^0$ 30' = 334^0 30'
Line OB	30^0 30'
Line OD	$180^0 + 20^0$ 45' = 200^0 45'
Line OC	$180^0 - 50^0$ 30 = 129^0 30'

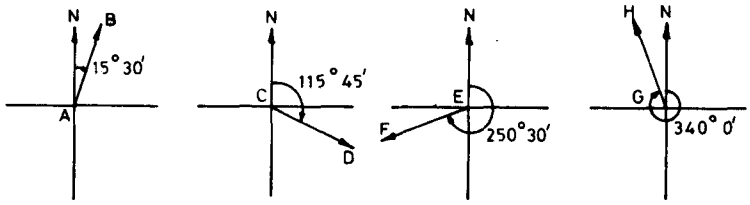
Example

The fore bearings of the four lines AB, CD, EF and GH are, respectively, as under: (i) 15^0 30' (ii) 115^0 45' (iii) 250^0 30' (iv) 340^0 0'

Determine the back bearings.

Solution

Given figures show the four lines.



The back bearings of lines AB and CD are obtained by adding 180^0 as the fore bearings are less than 180^0.

Line AB $15^0\,30' + 180^0 = 195^0\,30'$

Line CD $115^0\,45^0 + 180^0 = 295^0\,45'$

The back bearing of lines EF and GH are obtained by subtracting 180^0 as the fore bearings are greater than 180^0.

Line EF $250^0\,30' - 180^0 = 70^0\,30'$

Line GH $340^0 - 180^0 = 160^0\,0'$

The reader should verify the back bearings obtained by reversing the directions of arrows.

CALCULATIONS OF BEARINGS FROM INCLUDED ANGLES

In a traverse, sometimes the included angles are measured directly with a theodolite, If the bearing of anyone line (generally, the first line) is also measured in the field, the bearings of all other lines can be calculated from the observed bearing of one line and the included angles.

When two lines meet at a point, they form an *interior* angle and an *exterior* angle, An interior angle is the one which is on the inner side of the traverse, It is generally the smaller of the two angles formed, but sometimes it can be the larger of the two angles, A sketch should be drawn to determine whether the interior angle is smaller than the exterior angle or not.

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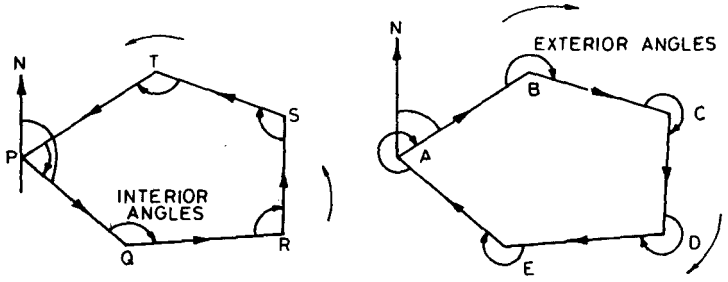


(a) Counter clockwise traverse **(b) Clockwise traverse**

The clockwise angles will be the interior angles if the traverse is run in the counter-clockwise direction (see figure "a").

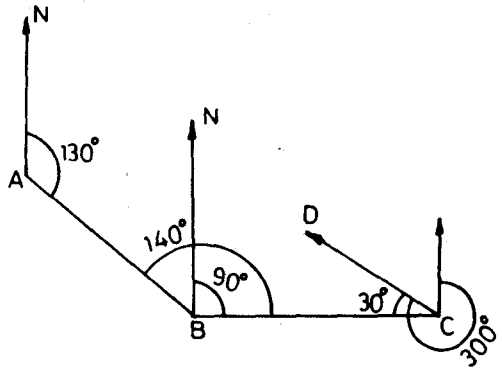
However, the clockwise angles will be exterior angles if the traverse is run in the clockwise direction (see figure "b").

In the case of a traverse in which the included angles have been measured, the bearing of a line is determined by applying the following rule:

Bearing of any line = Bearing of the preceding line + included angle.

If the sum of the terms on R.H.S. is greater than 360^0, deduct 360^0,

For example, in following figure, the bearing of the line BA is 310^0, and the included angle is 140^0.



Bearing of line BC= Bearing of BA + included angle

Bearing of line BC = $310^0 + 140^0 = 450^0$

Deduct 360^0 as the sum is greater than 360^0.

Therefore, bearing of line BC = $450^0 - 360^0 = 90^0$.

Similarly, the bearing of line CD can be found.

Bearing of line CD = bearing of CB + included angle = $270^0 + 30^0 = 300^0$

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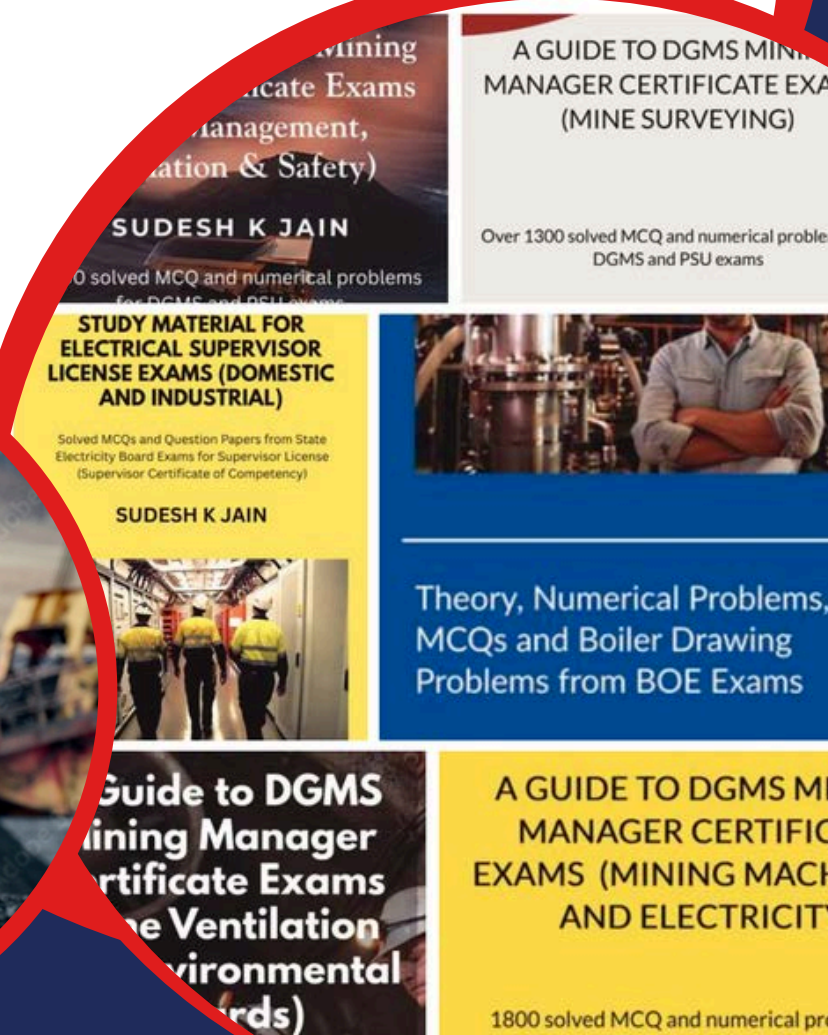


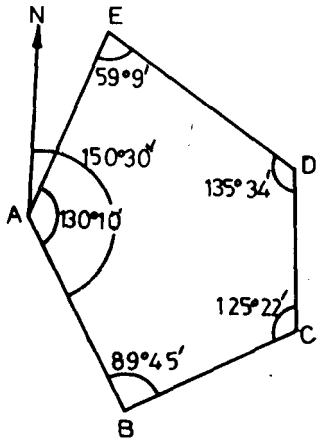
Example

In a closed traverse ABCDE, the bearings of the line AB was measured as $150^0 30'$. The included angles were measured as under: $\angle A = 130^0 10'$, $\angle B = 89^0 45'$, $\angle C = 125^0 22'$, $\angle D = 135^0 34'$, $\angle E = 59^0 9'$.

Calculate the bearings of all other lines.

Solution

See following figure.



Bearing of BC = Bearing of A + angle ABC = $(150^0 30' + 180^0) + 89^0 45' = 420^0 15' = 60^0 15'$

Bearing of CD = Bearing of CB + angle BCD = $(60^0 15' + 180^0) + 125^0 22' = 365^0 37' = 5^0 37'$

Bearing of DE = Bearing of DC + angle CDE = $(5^0 37' + 180^0) + 135^0 34' = 321^0 11'$

Bearing of EA = Bearing of ED + angle DEA = $(321^0 11' - 180^0) + 59^0 9' = 200^0 20'$

Check: *For checking the calculations, it is advisable to calculate the bearing of the first line from bearings of the last line.*

Bearing of AB = Bearing of AE + angle EAB = $(200^0 20' - 180^0) + 130^0 10' = 150^0 30'$ Hence O.K.

LOCAL ATTRACTION

The magnetic needle does not record the direction of magnetic meridian when it is under the influence of attractive bodies in its surroundings such as steel structures, electric cables carrying current etc. Such a disturbing influence is called ***local attraction.*** Local attraction causes deflection of compass needle.

Detection of Local Attraction

Unless the local attraction at a place is found out and corrected, the bearings taken from a compass can not be regarded as correct. To test for local attraction, it is necessary to observe the bearing of each line from both its ends. If the fore bearing and back bearing of a line differ by 180^0, it may be taken for granted that no local attraction exists at either station, provided the compass is free from the instrumental error. If the back and fore bearings of a line do not differ by 180^0 then the deviation may be due to observational error or the local attraction. The reading should be again taken and verified. If they do not agree then the local attraction may be at one or both the stations. The observed bearings of the lines may be corrected for local attraction by finding out the amount of error and its nature and applying the same to affected bearings of sides.

Rule: *If at a station, observed bearing of a line is more than that of its correct one, then the error at the station is +ve and the correction is –ve and vice versa.*

Example

A closed compass traverse ABCDE was run and the observed bearings of the lines are as follows :

Line	***Observed bearing***
AE	$319^0 00'$
AB	$72^0 45'$
BA	$252^0 00'$
BC	$349^0 00'$
CB	$167^0 15'$
CD	$298^0 30'$
DC	$118^0 30'$
DE	$229^0 00'$
ED	$48^0 00'$
EA	$135^0 30'$

Correct the bearings for local attraction.

Solution

By inspecting the values of the fore and back bearings of the lines, it is clear that the observed fore and back bearings of the line CD(i.e. bearings of CD and DC) exactly differ by 180^0.

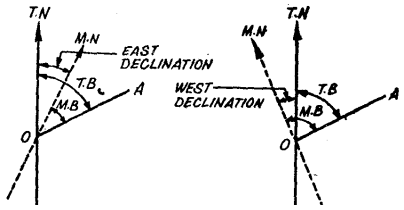
Therefore, both the stations C and D are free from local attractions and therefore the bearings observed from these stations should be correct bearings. Therefore, the bearings of the line DE should be correct bearing.

Therefore, bearing of ED(back bearing of DE) = $229^0 - 180^0 = 49^0$. But the observed bearing of the line ED = 48^0, i.e. the station E is affected by local attraction. To bring the observed, incorrect bearing of the lines at the station E, to its corrected value 1^0 should be added. Therefore, the error at the station E is said to be $+1^0$, i.e. to correct the values of bearings observed at the station E, $+1^0$, should be added to the observed bearings, therefore, the correct bearings of lines ED and EA are 49^0 and $136^0 30'$ respectively. From the correct bearing of the line EA, the station A may be corrected as follows : Bearing of AE(back bearing of EA) = $136^0 30'$(bearing of EA) + $180^0 = 316^0 30'$. But the observed bearing of the line AE is 319^0. Therefore, the station A is affected by local attraction. To correct the bearing of AE, observed from station A, $2^0 30'$ should be deducted. Therefore, the correction for local attraction = $-2^0 30'$. The correct bearings of AE and AB are, therefore, $316^0 30'$ and $70^0 15'$ respectively. Therefore the correct bearing of BA(back bearing of AB) = $70^0 15'$(bearing of AB) + $180^0 = 250^0 15'$. But the observed bearing of BA is 252^0. Therefore, the station B is affected by local attraction. The correction for local attraction is $1^0 45'$. Therefore, the correct bearings of BA and BC are $250^0 15'$ and $347^0 15'$ respectively.

The readings may be tabulated as under:

Line	Observed bearing	Correction	Corrected bearing
AE	$319^0 00'$	$-2^0 30'$	$316^0 30'$
AB	$72^0 45'$	$-2^0 30'$	$70^0 15'$
BA	$252^0 00'$	$-1^0 45'$	$250^0 15'$
BC	$349^0 00'$	$-1^0 45'$	$347^0 15'$
CB	$167^0 15'$	$0^0 00'$	$167^0 15'$
CD	$298^0 30'$	$0^0 00'$	$298^0 30'$
DC	$118^0 30'$	$0^0 00'$	$118^0 30'$
DE	$229^0 00'$	$0^0 00'$	$249^0 00'$
ED	$48^0 00'$	$+1^0 00'$	$49^0 00'$
EA	$135^0 30'$	$+ 1^0 00'$	$136^0 30'$

MAGNETIC DECLINATION



The magnetic meridian at a place does not coincide with the true meridian at that place except in few places. The horizontal angle which the magnetic meridian makes with the true or geographical meridian is knows as the *magnetic declination*.

When the north end of the needle points to the east of the true meridian, the declination is said to be east(n^0 E); when the north end of the needle points to the west, of the true meridian, the declination is said to be west(n^0 W).

Determination of True Bearing.

True bearing of a line = magnetic bearing of the line ± declination

Use + sign when declination is *east*, and minus sign when it is *west.*

This rule is applicable to W.C.B.

Determination of Magnetic bearing

Magnetic bearing of a line = true bearing of the line minus/plus magnetic declination.

Use the minus sign when the declination is *east*, and plus sign when it is *west.*

This rule is applicable to W.C.B.

Example

The magnetic bearing of a line is 197^0. Find its true bearing if the magnetic declination is 3^0 W.

Solution

Since the magnetic meridian is deflected towards west of the true meridian, true bearing of the line will be magnetic bearing – declination = 197 – 3 = 194^0.

Example

If the magnetic bearing of a line is N 37^0 W and the magnetic declination is 2^0 E, find true bearing.

Solution

True bearing = magnetic bearing – declination = N (37 – 2) W = N 35^0 W

Note: In this problem it is advisable to draw a figure indicating given magnetic bearing and declination and then find true bearing. Because here, above given rules will not be applicable as these rules are applicable to W.C.B.

Levelling is the art of finding the relative heights and depths of the objects on the surface of the earth. Levelling is of prime importance to an engineer for the purposes of planning, designing and executing the various engineering project such as roads, railways, canals, dams, water supply and sanitary schemes etc.

The principle of levelling lies in furnishing a horizontal line of sight and finding the vertical distances of the points above or below this line. This is done with the help of a level and a levelling staff respectively.

THE LEVEL

Level consists essentially of a telescope and a level tube to provide a horizontal line of sight, both fitted above the levelling head. It is supported on a tripod-stand when in use.

The different types of level is use are:

- Dumpy level
- Wye level
- Cooke's Reversible level
- Cushing's level
- Tilting level or India Office Pattern (I.O.P).

TERMS USED IN LEVELLING

Datum Surface or line

It is an imaginary level surface or line from which vertical distance of the points above or below this line are measured. In India the datum adopted for GTS (Great Trigonometrical Survey) of India Dept. is mean sea level at Karachi.

Reduced Level (R.L.)

Reduced level of any point is the height or depth of that point above or below any datum.

Line of Collimation

It is an imaginary straight line joining the intersection of the cross hairs at diaphragm to the optical centre of the object glass and its continuation. It is also called line of sight.

Back Sight (B.S.)

It is a staff reading taken on a point of known R.L. as on a bench mark or a change point. It is the first staff reading taken after the level is set up and levelled. It is considered to be positive.

Fore Sight (F.S.)

It is a staff reading taken on a point whose R.L. is to be determined. It is the last staff reading taken before shifting of the level to other position. It is considered to be negative.

Intermediate Sight (I.S.)

It is any reading other than foresight and back sight taken on a point of unknown R.L. from the same setup of the level.

Height of Instrument (H.I.)

It is the R.L. of the line of collimation when the instrument is correctly levelled.

Change point (C.P.) or Turning Point (T.P.)

It is an intermediate staff station on which two sights F.S. and B.S. are taken and is used for the purpose of the changing of the instrument.

Bench Mark

It is a fixed reference point of known elevation.

Permanent Bench Marks

The permanent bench marks are established at a closer interval between widely spaced GTS bench marks. These bench marks are established by Survey of India or by P.W.D.

Temporary Bench Marks

These are the bench marks established temporarily whenever required. These are generally the points at which a day's work is closed and from next day's work is started.

REDUCTION OF LEVELS

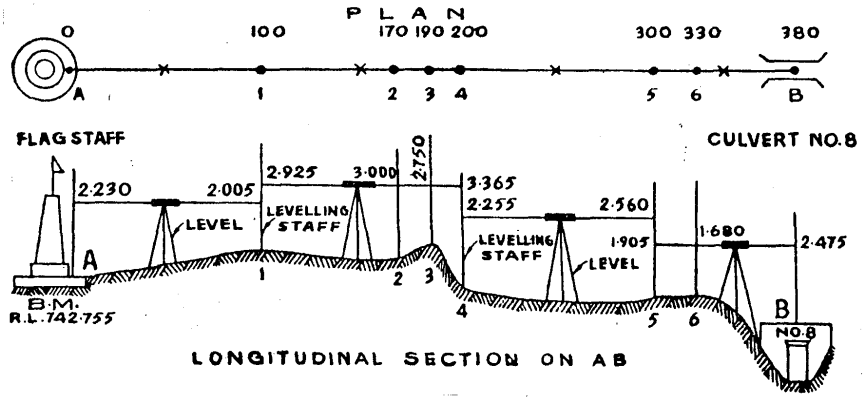
There are two methods of working out the reduced level of the points from the staff readings taken I the field:

- Height of Instrument of Collimation Method
- Rise and Fall Method

Height of Instrument Method

It consists in finding the elevation of the plane of collimation i.e. (H.I.) for every setting of the instrument and then obtaining the reduced levels of the points with reference to the respective plane of collimation.

To start with, the H.I. is calculated by adding the back sight of the R.L. of the starting point. The reduced levels of the intermediate point and the first change point are then obtained by subtracting the respective readings from the H.I. When the instrument is shifted, a new plane of collimation is set up and the height of which is calculated by adding back sight reading to the R.L. of the first change point.



The reduced levels of the successive points and the second change point are found out by subtracting their staff readings from this new H.I. The process is repeated until all the R.Ls are worked out, and then the arithmetical check is applied. This method may more be understood by the following example. Entries are shown in following table.

Stn	Distance in metres	Magnetic Bearings		Readings			Height of Instrument (H.I.)	R.L.	Rem
		Fore	Back	B.S.	I.S.	F.S.			
A	0	$250^0 15'$		2.230			744.985	742.755	Flag staff
1	100			2.925		2.005	745.905	742.980	C.P.
2	170				3.000		DO	742.905	
3	190				2.750		DO	743.155	
4	200			2.255		3.365	744.795	742.540	C.P.
5	300			1.905		2.560	744.140	742.235	C.P.
6	330				1.680		DO	742.460	
B	380		$70^0 15'$			2.475	DO	741.665	Culvert 8
				9.315		10.450			

Arithmetical Check: $\sum \text{B.S.} - \sum \text{F.S.} = 9.315 - 10.405 = -1.090$

Also, $\text{Last R.L.} - \text{First R.L.} = 741.665 - 742.755 = -1.090$

Both values are equal hence O.K.

Rise and Fall Method

In this method, the difference between consecutive points is calculated by comparing each point after the first with that immediately preceding it. The difference of their staff readings indicates rise or fall according as any staff reading is smaller or greater than that at the preceding point. The R.L. of each point is then found by adding rise or subtracting fall to or from the R.L. of the preceding point.

This method is made clear by finding R.Ls of the points in given figure(above).

Stn	Dist. (m)	Magnetic Bearings		Readings			Rise(+)	Fall (-)	R.L.	Rem
		Fore	Back	B.S.	I.S.	F.S.				
A	0	$250^0 15'$		2.230					742.755	Flag staff
1	100			2.925		2.005	0.225		742.980	C.P.
2	170				3.000			0.075	742.905	
3	190				2.750		0.250		743.155	
4	200			2.255		3.365		0.615	742.540	C.P.
5	300			1.905		2.560		0.305	742.235	C.P.
6	330				1.680		0.225		742.460	
B	380		$70^0 15'$			2.475		0.795	741.665	Culvert 8
Total				9.315		10.450	0.700	1.790		

Arithmetical Check: $\sum \text{B.S.} - \sum \text{F.S.} = 9.315 - 10.405 = -1.090$

Also, $\text{Last R.L.} - \text{First R.L.} = 741.665 - 742.755 = -1.090$

And $\sum \text{Rise} - \sum \text{Fall} = 0.700 - 1.790 = -1.090$

Example

A line of levels was run from a bench mark no. 1 of R.L. 765.733 to a bench mark no.2 of R.L. 767.538. The sum of the back sights was 30.815 and that of the fore sights was 28.975. What was the closing error of the levelling work ?

Solution

R.L. of B.M. No.2 as calculated from the observed reading

= R.L. of B.M. No.1 + $\sum B.S. - \sum F.S.$

= 765.733 + 30.815 - 28.975

= 767.573

But R.L. of B.M. No.2 = 767.538 (given)

∴ Closing error = 767.573 - 767.538 = + 0.035 m

Example

The following readings were taken with a level and 4 m staff. Draw up a level book page and reduce the levels by the height of instrument method.

0.578 B.M.(= 58.250 m), 0.933, 1.768, 2.450, (2.005 and 0.567) C.P., 1.888, 1.181, (3.679 and 0.612) C.P., 0.705, 1.810.

Solution

The first reading being on a B.M., is a back sight. As the fifth station is a change point, 2.005 is fore sight reading and 0.567 is back sight reading. All the readings between the first and fifth readings are intermediate sight-readings. Similarly, the eighth station being a change point, 3.679 is fore sight reading, 0.612 is back sight reading, and 1.888, 1.181 are intermediate sight readings. The last reading 1.810 is fore sight and 0.705 is intermediate sight-readings. All the readings have been entered in their respective columns in the following table and the levels have been reduced by height of instrument method. In the following computations, the values of B.S., I.S., H.I., etc., for a particular station have been indicated by its number or name.

Section-1:

$H.I._1 = h_1 + B.S._1 = 58.250 + 0.578 = 58.828$ m

$h_2 = H.I._1 - I.S._2 = 58.828 - 0.933 = 57.895$ m

$h_3 = H.I._1 - I.S._3 = 58.828 - 1.768 = 57.060$ m

$h_4 = H.I._1 - I.S._4 = 58.828 - 2.450 = 56.378$ m

$h_5 = H.I._1 - F.S._5 = 58.828 - 2.005 = 56.823$ m

Section-2:

$H.I._5 = h_5 + B.S._5 = 56.823 + 0.567 = 57.390$ m

$h_6 = H.I._2 - I.S._6 = 57.390 - 1.888 = 55.502$ m

$h_7 = H.I._2 - I.S._7 = 57.390 - 1.181 = 56.209$ m

$h_8 = H.I._2 - F.S._8 = 57.390 - 3.679 = 53.711$ m

Section-3:

$H.I._8 = h_8 + B.S._8 = 53.711 + 0.612 = 54.323$ m

$h_9 = H.I._8 – I.S._9 = 54.323 – 0.705 = 53.618$ m

$h_{10} = H.I._8 – F.S._{10} = 54.323 – 1.810 = 52.513$ m

Additional check for H.I. method:

Σ [H.I. . (No. of I.S.s + 1)] – Σ I.S. – Σ F.S. = Σ R.L. – First R.L.

[58.828 . 4 + 57.390 - 3 + 54.323 . 2] – 8.925 – 7.494 = 557.959 - 58.250 = 499.709 (O.K.)

Station	B.S.	I.S.	F.S.	H.I.	R.L.	Remarks
1	0.578			58.828		B.M. = 58.250 m
2		0.933			57.895	
3		1.768			57.060	
4		2.450			56.378	
5	0.567		2.005	57.390	56.823	C.P.
6		1.888			55.502	
7		1.181			56.209	
8	0.612		3.679	54.323	53.711	C.P.
9		0.705			53.618	
10			1.810		52.513	
Σ	1.757	8.925	7.494		557.956	
Check: 1.757 – 7.494 = 52.513 – 58.250 = - 5.737 (O.K.)						

Example

The following series of readings of back sight and fore sights was taken in a Fly-levelling. The first reading was taken on a point of R.L. 100.000 m. Draw a page of levelling-field book and enter readings on it. Find R.Ls. of all the points. Apply check.

1.235, 1.396, 2.345, 1.986, 2.148, 0.735, 0.325, 2.568, 1.465, 2.435, 1.356, 0.768, 1.985, 2.655.

Solution

In flying levelling the 1st, 3rd, 5th, etc. and the last but one readings are the back sights and 2nd, 4th, 6th etc. and the last readings are the fore sights. The reduced levels of the points are calculated by rise and fall method and tabulated in table.

Stn.	Readings			RISE	FALL	R.L.	REMARKS
	B.S.	I.S.	F.S				
1	1.235					100.000	B.M. of R.L. 100.000
2	2.345		1.396		0.161	99.839	C.P.
3	2.148		1.986	0.359		100.198	C.P.
4	0.325		0.735	1.413		101.611	C.P.

5	1.465		2.568		2.243	99.368	C.P.
6	1.356		2.435		0.970	98.398	C.P.
7	1.985		0.768	0.588		98.986	C.P.
8			2.655		0.670	98.316	End point
Total	**10.859**		**12.543**	**2.360**	**4.044**		

Arithmetical check:

$\sum B.S. - \sum F.S.$	$\sum Rise - \sum Fall$	Last R.L. - 1st R.L.
= 10.859 - 12.543	= 2.360 - 4.044	= 98.316 - 100.000
= - 1.684	= - 1.684	= - 1.684 O.K.

CLASSIFICATION OF DIRECT LEVELLING

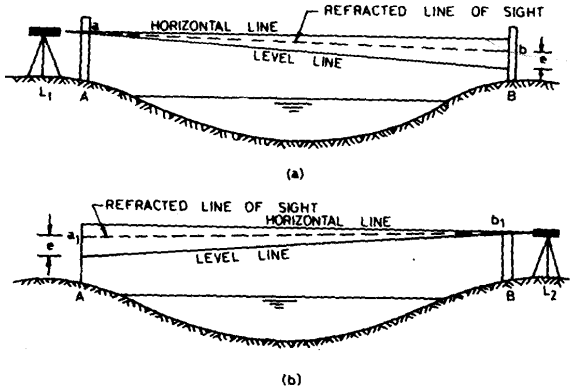
The different type of levelling are :

- Differential levelling
- Check levelling
- Precise levelling
- Reciprocal levelling
- Longitudinal levelling
- Cross-levelling
- Levelling for giving levels for works and establishing contours.
- Barometric levelling
- Hypsometry
- Trigonometrical levelling

RECIPROCAL LEVELLING

It is a method of levelling adopted to determine the difference of levels between two points when it is not possible to set up the level midway between them as in the case of a river, a deep valley etc. This method also eliminates;

- the error due to curvature and refraction, and
- the error due to line of collimation not being parallel to the bubble line which may otherwise occur due to non-equality of back and fore sight distances.



The operation involves two sets of observations giving two incorrect differences of levels, the mean of which is the true difference. Let A and B be two points [Fig.(a) and (b)] on opposite banks of a wide river.

Then to find the true difference of level between A and B, proceed as follows:-

1. Set up and level the instrument very near to A over L_1 [Fig.(a)]
2. Read the staff held at A and B and let the staff readings be a and b respectively.
3. Transfer the instrument to B and set it up and level very near to B or over L_2 [Fig.(b)].
4. Observe the readings on the staff held at A and B and denote them as (a_1) and (b_1) respectively.

Now $h = \frac{(a_1 - b_1) + (a - b)}{2}$, where h is correct difference of levels.

- **Elimination of Error Due to Change in Refraction**. The error can be eliminated by using two instruments and taking simultaneously observations from L_1 to L_2. The apparent differences of levels h' and h" are determined from the instrument at L_1 and L_2 respectively. The correct difference of levels will be h = (h' + h"/2).
- **Elimination of Collimation Error.** The total error is given by equation

$$e = \frac{(a_1 - b_1) - (a - b)}{2}$$

The breakup of this error is

$$e = e_l + e_c - e_r$$

where e_l is collimation error, assumed positive, when the line of collimation is inclined upwards, e_c is error due to curvature = $-0.0785d^2$ and e_r = error due to refraction = $0.0112d^2$.

The collimation error e_l is given by

$$e_l = e - 0.0785d^2 + 0.11d^2 = e - 0.0673d^2$$

Example

The following notes refer to the reciprocal levels taken from one level:

Inst. Stn	***Staff readings on***		***Rem***
	A	***B***	
A	*1.029*	*1.634*	*Distance between A and B = 800 m*
B	*0.943*	*1.542*	*R.L. of A = 421.543*

Find (i) the true R.L. of B (ii) combined correction for curvature and refraction (iii) the error in collimation adjustment of the instrument.

Solution

(i) Instrument at A

Incorrect difference of level between A and B = 1.634 – 1.029 = 0.605 m

Instrument at B

Incorrect difference of level between A and B = 1.542 – 0.943 = 0.599 m

True difference of level between A and B = mean of the two incorrect differences = (0.605 + 0.599)/2 = 0.602 m (fall from A to B)

R.L. of A = 421.543

Subtract fall = 0.602

We get R.L. of B = 420.941

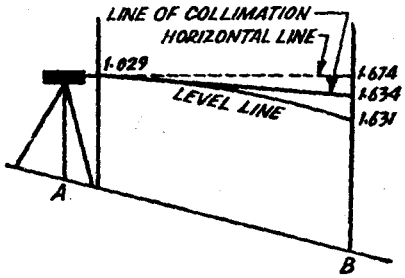
(ii) The combined correction for curvature and refraction = $0.0673d^2$ = 0.0673 $(800/1000)^2$ = 0.043 m

(iii) Instrument at A

Reading at A = 1.029

fall from A to B = 0.602

The required reading touching the level line = 1.029 + 0.602 = 1.631 (see figure below which shows correct readings when the instrument is at A)



Since the combined effect of curvature and refraction is to increase the staff reading, therefore, the observed staff reading at B touching the horizontal line should be = 1.631 + the combined effect of the curvature and refraction i.e. 1.631 + 0.043 = 1.674.

But the actual observed reading at B i.e. the reading touching the line of collimation = 1.634, which is less than 1.674.

$\therefore$ the line of collimation is inclined downward and the error due to this

= 1.674 – 1.634 = 0.040 m

SUBSIDENCE MEASUREMENTS

The term subsidence means the lowering of the surface: When an underground opening is established due to extraction of a coal seam or ore body, the original equilibrium of strata is disturbed with resultant stress concentrations. It causes effects on the surface where particles suffer vertical and horizontal displacement, creating subsidence basin or trough (surface depression) which flattens out to sides until it is level with the existing ground. The area of the surface affected above a goaf depends on the angle of draw, which is the angle between a vertical line from the edge of the goaf and a line extended to a point at which subsidence tails out to zero. The angle of draw varies with depth, nature and inclination of the strata, and other geological features.

On the basis of subsidence survey carried put for the horizontal and vertical surface movements, the five parameters of subsidence may be ascertained. These are:

- Vertical subsidence.
- Differential change in the ground slope.
- Change in the surface curvature.
- Horizontal displacement of different surface points.
- Horizontal strain

In order to maintain records of the amount of subsidence it requires periodic measurements of surface movement which consist of-

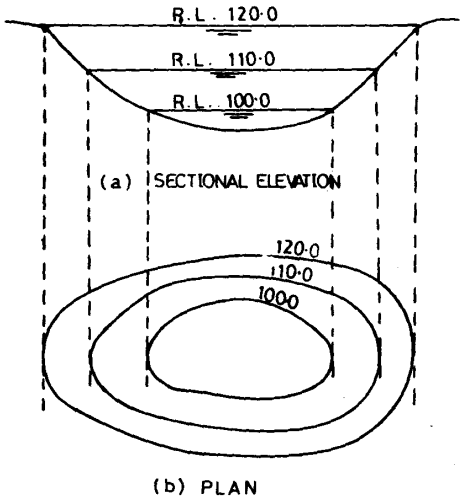
- Vertical movement, and

- Horizontal movement

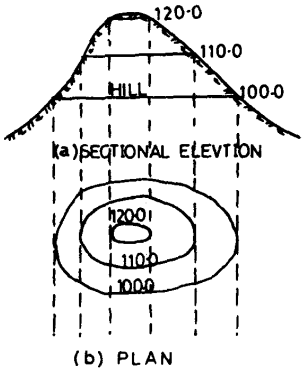
CONTOURING

A contour line may be defined as the line of intersection of a level surface with the surface of the ground. The process of tracing contour lines on the surface of the earth is called contouring and the maps upon which these lines are drawn are called contour maps.

Following figure shows a pond with water level of 120.00 m. Figure (b) shows the plan of water surface. The plan shows contours of RL 120.0, 110.0 and 100.0.



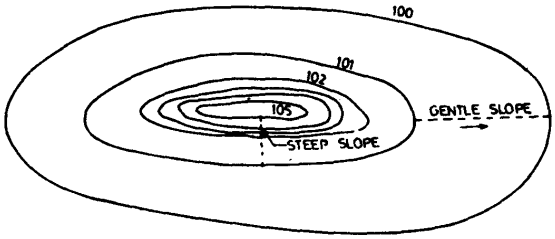
Following figure shows contours of a hill.



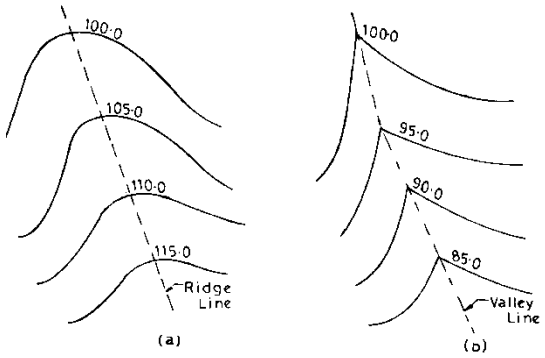
The constant vertical distance between two consecutive contours is called contour interval and the horizontal distance between any two adjacent contours is termed as horizontal equivalent. The horizontal equivalent depends upon slope of ground. The contour interval depends on nature of ground, scale of map, etc.

Characteristics of Contours

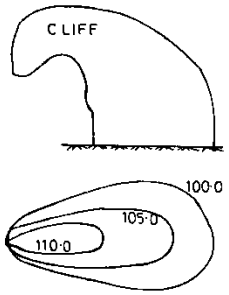
- All points in a contour line have same elevation.
- Flat ground is indicated where the contours are widely separated and steep ground where they run close together.



- A uniform slope is indicated when the contour lines are uniformly spaced and a plane surface when they are straight, parallel and equally spaced.
- A series of closed contour lines on the map represent a hill, if the higher values are inside.



- A series of closed contours on the map indicate a depression, if the higher values are outside.
- Contour lines across ridge or valley lines at right angles: If the higher values are inside the bend or loop in the contour, it indicates a *ridge*. And if the higher values are outside the bend, it represents a *valley*.
- Contour lines can not end anywhere but close on themselves either within or outside the limits of the map.
- Contour lines cannot merge or cross one another on map except in the case of an *over hanging cliff*.



Contour interval is taken 0.2 to 0.5 m for large scale maps of building sites, flat country etc. For reservoir and town planning, it is taken as 0.5m to 2 m. For location survey its value is 2 to 3 m.

TRACING OF OUTCROP ON CONTOUR PLAN

Outcrop of a coal seam may be traced on a contour plan when any seam outcrops at a point and the depths of the same seam from the surface at different points are known.

Example

Attached is a contour plan of an area wherein a coal seam outcrops at A. Two broeholes drilled at B and C encounters the same seam at the depth of 120 m and 180 m respectively. Trace the outcrop of the coal seam in the plan enclosed. Explain the manner in which the outcrop is traced.

Solution

Datum level of seam at B = 240 - 120 = 120 m

Datum level of seam at C = 270 - 180 = 90 m

Horizontal distance AB measured = 408 m

Horizontal distance AC measured = 372 m

Fall from A to B along coal seam = 180 - 120 = 60 m

Fall from A to C along coal seam = 180 - 90 = 90 m

Gradient of coal seam along AB = 80 in 408 = 1 in 6.8

Gradient of coal seam along AC = 90 in 372 = 1 in 4.1

AB and AC are joined, 6.8 units along AB and 4.1 units along AC from A are measured. The line joining these two points is the strike line of the coal seam. Direction of full dip of the coal seam, at right angle to the strike line, is shown by the line with arrow head and the amount of dip appears to be about 1 in 4.

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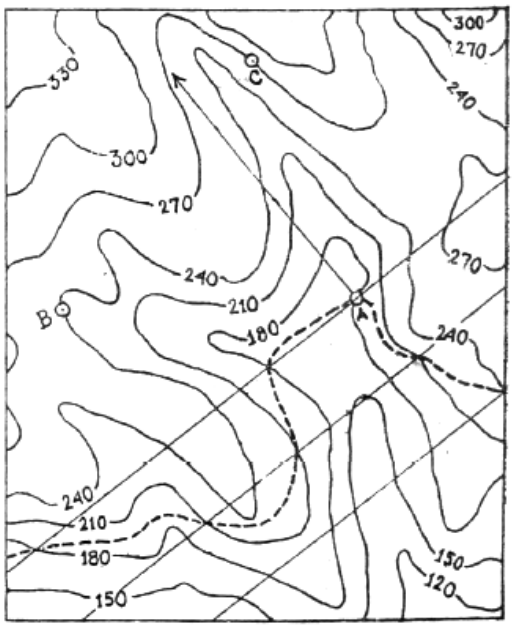


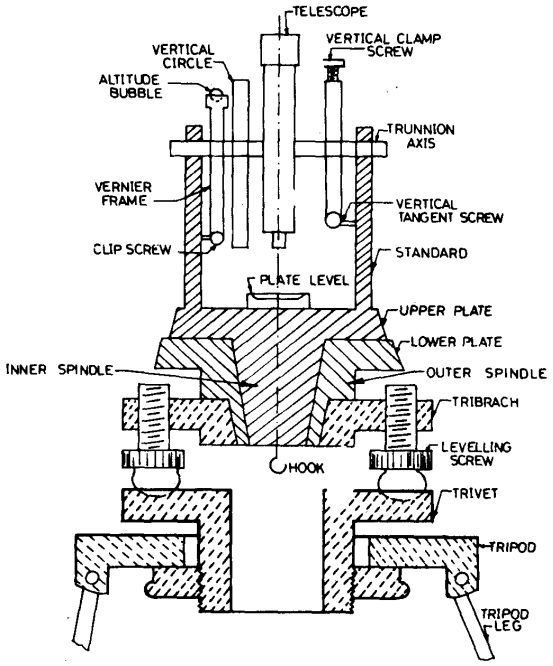
The scale of the map is 1 inch = 120 m and the contour lines are 30m vertically apart. A strike line is drawn at right angles to the full dip and passing through A across the map. This is 180 m stratum contour line. Line across the map parallel to this line of strike are drawn 1" (2.5 cm) apart giving 120 m surface distance on the map and stratum contours 30 m apart vertically as the seam dips at 1 in 4. To the rise of the seam the next line from the 180 m line is the 210 m stratum line because it represents the plane of the seam at 30 m higher level than the point of outcrop A. On the dip side the stratum contours are 150m. 120 m, etc. but their surface contours are on the dip side and, therefore, can be no outcrop. Where a stratum contour intersects a corresponding surface contour line, the point is marked. The seam outcrops there. These points are joined by a scattered line which is the outcrop of the seam.

THEODOLITE SURVEYING

The theodolite is most accurate instrument used mainly for measuring horizontal and vertical angles. Theodolite are classified as

- Transit
- Non-transit.

It is called to be transit when its telescope can be transited i.e. revolved through a complete revolution about its horizontal axis in the vertical plane, whereas in non-transit type, the telescope cannot be transited.

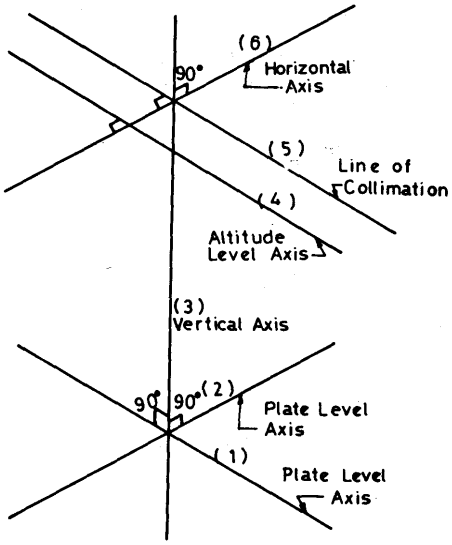


FUNDAMENTAL LINES

There are 5 fundamental lines (or axes) of a theodolite.

These are

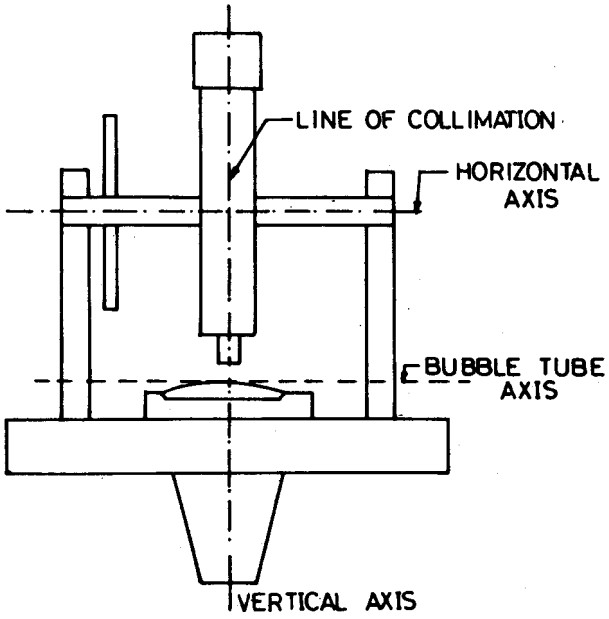
1. Vertical axis
2. Horizontal axis
3. Line of collimation
4. Altitude level axis
5. Plate level axis



When the theodolite is in proper adjustment, the following four conditions between fundamental lines are satisfied.

1. The axes of the plate level is perpendicular to the vertical axis.
2. The horizontal axis is perpendicular to the vertical axis.
3. The line of collimation is at right angles to the horizontal axis.
4. The axis of the altitude level is parallel to the line of collimation when it is horizontal, and the vertical circle reads zero.

Following figure shows various axes and lines in theodolite.



ADJUSTMENTS OF A THEODOLITE

The adjustments of a theodolite are of two kinds :

- Temporary Adjustments.
- Permanent Adjustments

Temporary Adjustments

The adjustments which are required to be made at every instrument station before making observations, are known as temporary adjustments.

The temporary adjustments of a theodolite include the following:

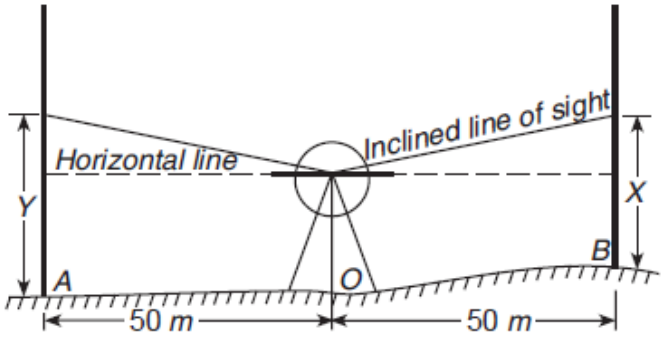
- Setting up and centering the theodolite over the station.
- Levelling of the theodolite.
- Elimination of the parallax.

Permanent Adjustments

The permanent adjustments of a transit theodolite are :

- **Adjustment of the horizontal plate level** With this adjustment the axis of the plate levels, is made perpendicular to the vertical axis of the theodolite.
- **Adjustment of the horizontal axis** With this adjustment the horizontal axis (or trunnion axis) is made perpendicular to the vertical axis.

- **Adjustment of the telescope** The object of this adjustment is that the line of collimation should remain horizontal when the bubble of the level tube fitted on telescope is brought at the centre of its run. (See figure)



MEASUREMENT OF HORIZONTAL ANGLES

These are three mean of measuring horizontal angles:

1. Ordinary Method
2. Repetition Method
3. Reiteration Method

ADJUSTMENT OF A CLOSED TRAVERSE

It includes :

1. Adjustment of Angular Error
2. Adjustment of Bearings
3. Adjustment of Closing Error

Adjustment of Angular Error

Angular Error. The theoretic sum of the interior angles of a traverse should equal (2N - 4) right angles, and that of the exterior angles should equal (2N + 4) right angles, where N is the number of sides of a closed traverse. The difference between the theoretic sum and the sum of the measured angles in a closed traverse is called the angular error of closure. It should not exceed the least count X $\sqrt{N}$

Adjustment. When all angles are measured with equal care, this error is distributed equally among all the angles.

Adjustment of Bearings

Closing Error in Bearings. If traversing is done by taking bearings of the lines, the closing error in beating may be determined by comparing the back and fore bearings of the last line of

the closed traverse as observed at the first and last stations of the traverse respectively. When the traverse ends on a line of known bearings, the error in bearing may be determined by finding the difference between its observed bearing and known bearing.

Adjustment. If e is the closing error in bearing, and N is the number of the sides of the traverse, then the correction applied to the bearings of the sides are :

Correction to the first bearing = $\frac{e}{N}$

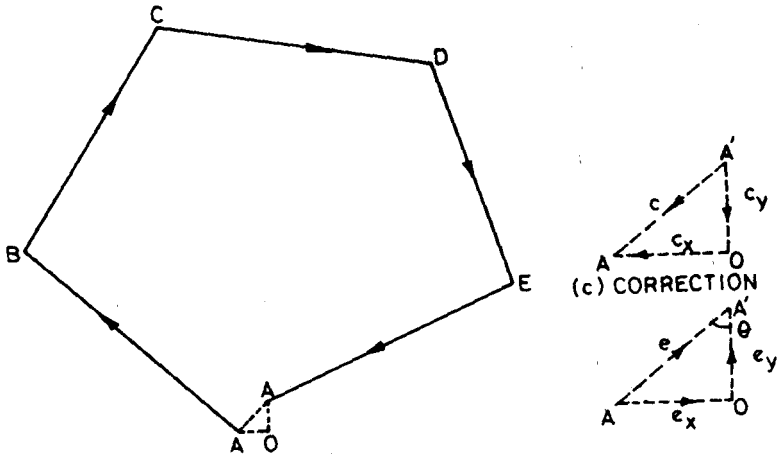
" " second " = $\frac{2e}{N}$

" ' third " = $\frac{3e}{N}$

and so on to the last bearing = $\frac{N.e}{N} = e$

Adjustment of Closing Error

If a closed traverse is plotted according to the field (procedure) measurements and if the end of traverse will not coincide exactly at the starting point. This discrepancy is due to the errors in the field observations i.e. magnetic bearings and linear distances. Such error is known as closing error.



The x and y components of the closing errors can be determined as

$e_x = \sum D$

$e_y = \sum L$

However in the case of link traverse

$e_x = X' - X$

and $e_y = Y' - Y$

where X' and Y' are computed coordinates of the final control point, and X and Y are the corresponding known coordinates.

The total closing error is given by

$$e = \sqrt{(e_x)^2 + (e_y)^2}$$

The direction of the closing error AA' is given by

$$\tan\theta = \frac{e_x}{e_y}$$

The signs of e_x and e_y will define the quadrant in which the closing error lies.

The signs of corrections c_x and c_y will be opposite to those of errors.

$c_x = -e_x$

and $c_y = -e_y$

Example

The following are the lengths and bearings of the sides of a closed traverse ABCD :

Line	***Length In Meters***	***Bearing***
AB	*78.2*	*$140^O12'$*
BC	*198.0*	*$36^O24'$*
CD	*37.8*	*$338^O48'$*
DA	*?*	*?*

Calculate the length and bearing of line DA.

Example

The bearings of tow inaccessible stations A and B taken from station C were 225^0 0' and $153^026'$ respectively. The coordinates of A and B were as under:

Station	*Easting*	*Northing*
A	*300*	*200*
B	*400*	*150*

Calculate the independent coordinates of C.

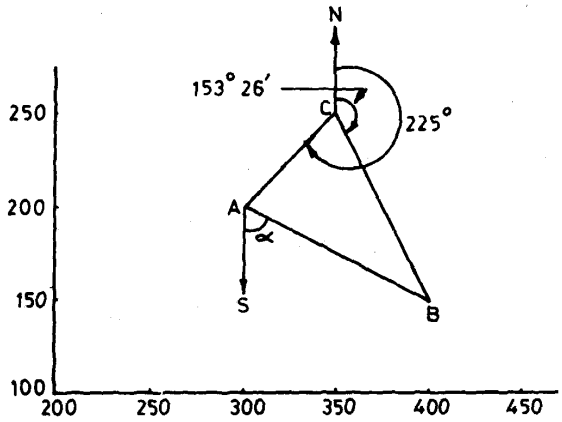
Solution

Let us draw a figure.

$$\text{Length AB} = \sqrt{(400-300)^2 + (200-150)^2} = 111.80 \text{ m}$$

$$\text{Tan } \alpha = \frac{100}{50} = 2.0$$

$\alpha = 63^0\ 26'$



The W.C.B. bearing of the line AB is $180^0 - 63^026' = 116^034'$

In triangle ACB,

$\angle CAB = 116^034' - 45^0 = 71^034'$

$\angle ACB = 225^0 - 153^026' = 71^034'$

$\angle ABC = 180^0 - (71^0\ 34' + 71^034') = 36^0\ 52'$

By sine law $\dfrac{111.80}{\sin 71^034'} = \dfrac{BC}{\sin 71^034'} = \dfrac{CA}{\sin 36^052'}$

CA = 70.70 m

Latitude of CA = $70.70 \cos 225^0 = -50$ m

Departure of CA = $70.70 \sin 225^0 = -50$ m

Independent coordinates of C,

Easting = 300 + 50 = 350 m

Northing = 200 + 50 = 250 m

CORRELATION SURVEY

Correlation, in surveying, is the method of surveying by which the surface survey and underground survey are connected to the same base.

METHODS

The methods employed for correlation surveys are controlled to a large extent by the conditions at the mine. The different methods are

- Direct traversing when workings are reached by adits or drifts.
- Shaft plumbing methods when access is by means of vertical shafts- (a) One wire in each of two shafts. (b) Two or more wires in a single shaft by (i) Coplaning or exact alignment method; (ii) Approximate alignment or method of Weisbach triangles; (iii) Methods of Weiss quadrilateral.
- Optical method or direct transference of azimuth down a shaft by a transit theodolite.
- Magnetic methods (a) By tubular compass (b) By magneto theodolite.
- Gyro-Theodolite method.

PURPOSES OF CORRELATION SURVEYING

The various purposes of correlation of surface and underground surveys are

- To determine with high degree of accuracy the relative positions of the underground roadways and faces may be correctly laid down on the working plan.
- To fix the boundary of the mine for limiting the extent of underground working.
- To fix the positions of railways, roads and important buildings or structures on mine
- plans so that statutory restrictions imposed may be strictly followed during workings under them.
- To mark the river, lake, tank, pond, etc. on mine plans so as to take adequate precautions against inundation.
- To select suitable site for proposed pits, drifts, or boreholes to connect with underground workings.

COPLANING METHOD

The coplaning of direct alignment method consists of setting up theodolites in the plane of the two plumb lines in one shaft at the surface and underground and thus transferring the bearing of surface survey directly belowground.

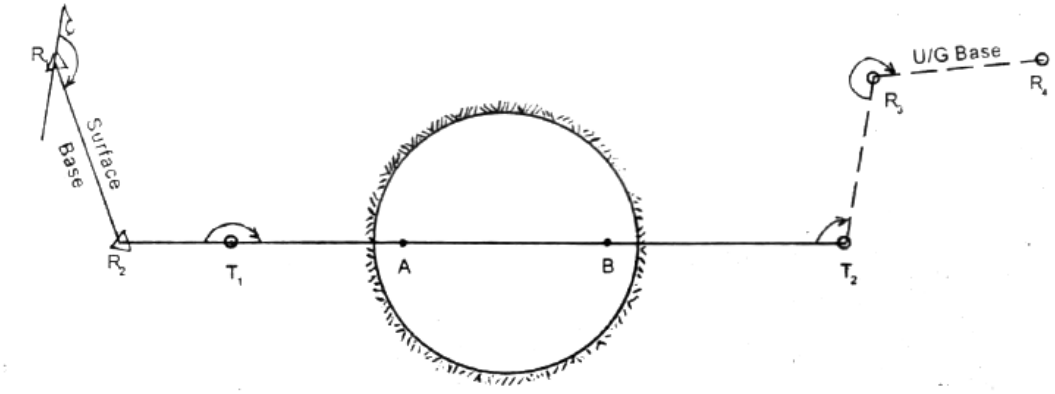
Instrument Required

- Two hard drawn piano wire each 0.1 cm diameter having breaking strength of 150 kg and of sufficient strength to cover the shaft.
- Two small drums fitted with brake or ratchet.
- Two wooden plummets weighing 2 kg. each.
- Two symmetrically shaped lead plumb bobs weighing 25 kg. each and fitted with vanes to dampen the twisting motion when immersed in water.
- Two small pulleys, tin washers, buckets of water.
- A theodolite preferably microptic type with telescopic legs.

- Steel tape, thermometer, spring balance.
- Scale for determining mean position of wires.
- Field book, pencil, oiled paper, safety lamps, pegs, nails, paints, etc.
- Telephone or other means for communication of messages from surface to underground and vice versa.

Procedure

The plumb wires are lowered down the shaft through small pulleys on the headgear as far apart as possible with small wooden plummets attached to the wires. When the wires have reached the bottom of the shaft light tin washers are passed down the wires from surface to check that the wires hang free from obstruction or kink. The smaller plummets are then replaced by heavier plumb-bobs made of lead to eliminate possibility of magnetic attraction.



The plumb-bobs are immersed in buckets of water to cut down the oscillation of the wire. It is necessary to raise the plumb-bobs from time to time to allow for the stretch of the wires. As the wires are affected by the ventilation current, the fan should be stopped during and at least one hour before commencing the survey operations.

From the surface traverse, two stations R_1 and R_2 are fixed as the surface base. The theodolite is set at station T_1 marked on the surface pit bank near the shaft judged by naked eyes in alignment with the two wires. The station T_2 should be as close to the nearest plumb wire as good focussing will allow so that both wires can easily be seen by change of focus.

The wires are distinguished by the fact that the near wire is thickest. The instrument with the vernier at zero is manoeuvred by means of its sliding head until it is in exact line with the wires.

The angle AT_1R_2 is measured. Then the theodolite is set up at station R_2 and the angle $R_1R_2T_1$ is measured. Distances R_2T_1, T_1A and AB are measured very precisely. The azimuth of the plumb-plane AB and the co-ordinates of A and B with respect to surface base station R_2R_1 are calculated.

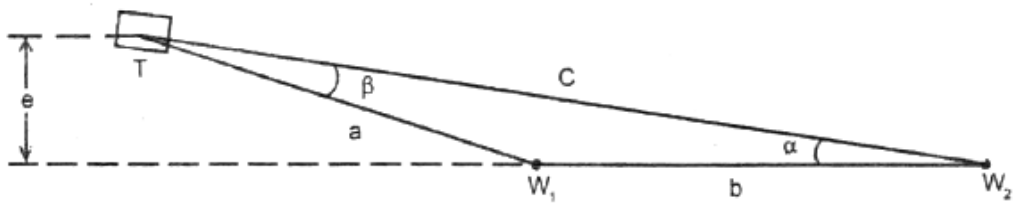
Similarly the theodolite is set at station T_2 belowground in exact alignment with the plumb plane AB and the angles BT_2R_3 and $T_2R_3R_4$ are measured. Distance BT_2, T_2R_3 and R_3R_4 are also measured.

A number of roof stations are fixed on the underground base line R_3R_4 for subsequent underground surveys. The azimuth of the underground survey lines and the co-ordinates of the underground stations are calculated from the azimuth of the plumb-plane AB and the coordinates of the plumb wires A and B which have already been established at the surface.

WEISBACH TRIANGLE

When transferring ground level azimuths underground by suspensions of two wires down a shaft it is difficult to set up the theodolite exactly in line with them. In addition, one wire being nearer obscures the wire beyond, leading to an inaccurate bisection.

In Weisbach's method the theodolite is set up slightly out of line forming a small triangle with the two wires. This triangle is known as **Weisbach's Triangle** and the azimuth of the line joining the two wires is found by solution of the triangle so formed.



α is obtained from the triangle W_1TW_1 by using sine rule.

$$Sin\alpha = \frac{aSin\beta}{b}$$

and $$e = c.Sin\alpha$$

In a Weisbach triangle, the angle W_2TW_1 called the Weisbach angle β.

WEISBACH TRIANGLE METHOD OF CORRELATION

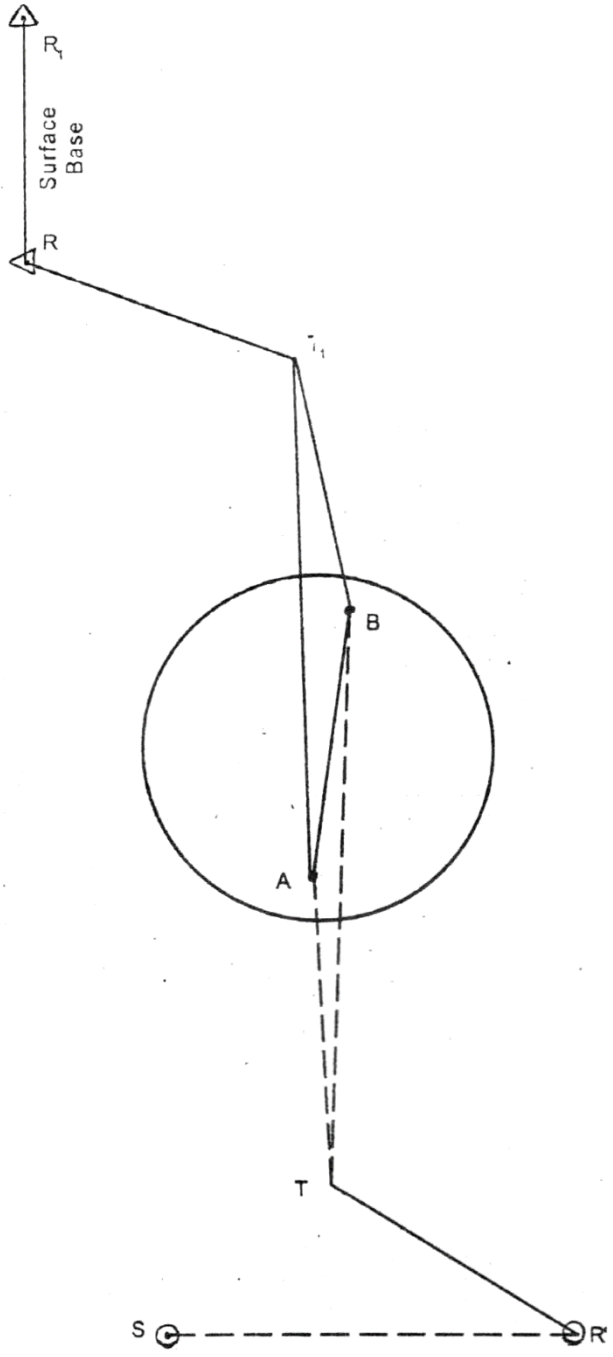
Out of the different methods, the Weisbach method of correlation is usually preferred when **only one shaft is available**.

The method consists of suspending two plumb wires which form a plumb plane and azimuth of the same is determined at the surface and taken underground at the shaft inset and thereafter transferred to the underground base or reference line.

Two piano wires A and B are lowered down the shaft by means of biconical wooden plummets weighing 2 kg. each. When the wires have reached the bottom of the shaft tin washers are passed down by the wires from the surface to see that they are free of obstruction. The smaller

plummets are then replaced by symmetrically shaped finned plumb bobs made of lead weighing about 25 kg. When the wires hang vertically each plumb bob is immersed in a barrel of water to cut down oscillation

From the surface traverse, stations R and R_1 fixed as surface base RR_1 whose azimuth is very carefully determined.



The theodolite is set at T_1 at surface as closed to the near wire B as focussing will allow and almost in line with the plumb-plane AB produced and connecting to the surface base. The small angle BT_1A should be only few minutes of arc. The triangle ABT_1 is known as the "Weisbach triangle".

To measure the small angle BT, A, the theodolite is set up at T1 and all the temporary adjustments are made. The telescope is directed to wire B and then the angle BT.R is observed in the usual manner, taking at least three rounds of readings on each face of the instrument. The mean value is the correct value of the angle. In a smilar manner, the value of angle AT1 R is obtaied and the difference between the two angles thus obtained is the correct value of the angle BTA

The measurements of the sides of the triangle ADT1 are made with a steel tape. At least six readings of each side should be taken, the tape being held quite level and fully supported, the average value being used as the correct one.

The theodolite is next set up at R and the angle R_1RT, is observed and the distances RR_1 and RT_1 are also carefully measured.

To calculate the azimuth of the plumb-plane AB, the small angle T_1AB must be calculated. The triangle T_1AB

$$\frac{BT_1}{SinA} = \frac{AB}{SinT_1}$$

or $$SinA = \frac{BT_1 SinT_1}{AB}$$

The sines of very small angles are proportional to the angle themselves and therefore

$$\text{A (in seconds)} = \frac{BT_1 \, x \, (T_1 \, in \, \text{sec} \, onds)}{AB}$$

The azimuth of the plumb-plane having obtained at the surface, a series of similar observations are taken belowground with the theodolite at T and so having calculated the angle TBA at the shaft bottom, the azimuths of AT and BT can be calculated. The angles ART' and BTR' have also been measured and so the azimuth of the line TR" can be calculated. Finally be measuring the angle SR'T the azimuth of the underground base line RS can be deduced.

Example

The following are the details of observation made in connection with correlation by Weisbach triangle method. A and B are two plumblines suspended from the pit top. D and E are the stations in underground traverse survey which is required to be connected with the surface

survey bearing of AB, as found in the surface survey is $40^0 40'00''$ and the length of AB = 2.286 metres.

The observation obtained in underground are

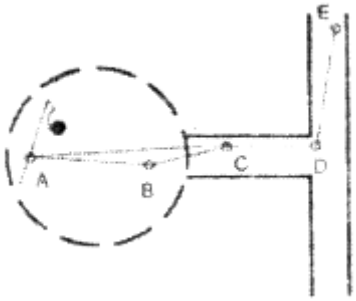
AB - 2.286 m	*$\angle BCD = 181^0 0000''$*
BC - 2.621 m	*$\angle CDE = 96^0 00'$*
AC = 4.907 m	*CD = 18.345*
DE = 30.480 m	

Weisbach angle $\angle ACB = 0^0 1'40''$

Find the bearing of the underground draft DE.

Solution

Rough plot of survey



Apply Sine rule

$$Sin\,BAC = \frac{BC}{AB}.Sin\,ACB$$

As angles are very small

$$\text{Angle BAC} = \frac{BC}{AB}\,x\,Angle\,ACB = \frac{2.621}{2.286}\,x\,100\,\text{sec}\,onds$$

= 114.65 seconds = 1'54"

Bearings

The bearing of AC The bearing of CA

= (40°40’ - 1 '54") = 40°38'06" = 40°38'06" + 180° = 220°38'06"

Since the angle ACB = 1'40"

The bearing of CB Now the angle BCD

The bearing of CD = 220°38'06" - 1'40" = 220°36'26" = 181°00'00"

Bearing of BC = 40°36’26" = (40°36’26" + 181°)- 180° = 41°36'26"

Again the angle CDE = 96°00'00"

∴ The bearing of DE = (41°36‘26" + 96°) + 180° = 317°36’26"

Hence the bearing of the underground draft DE = 317°36'26".

CORRELATION BY TWO SHAFTS

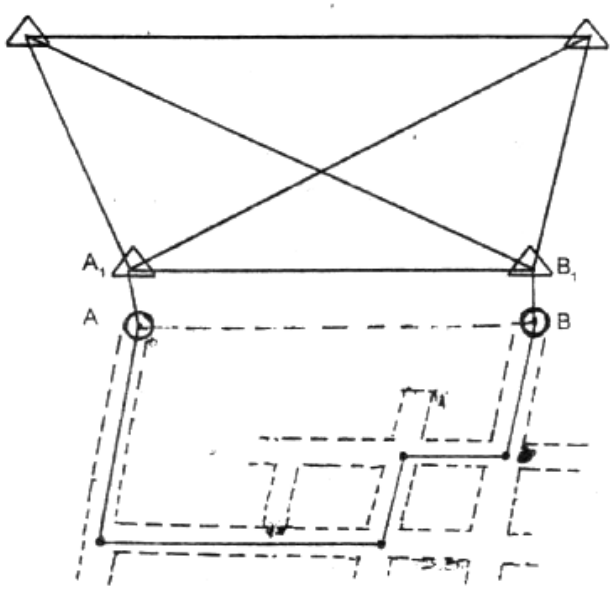
When two shafts are available for surveying purposes, one of the most suitable methods of correlation surface and underground surveys is to hang a plumb-line in both shafts.

The plumb-line form the terminal stations of the surface line form the terminal stations of the surface and underground surveys, and consequently when the surveys have been completed their relative positions may be determined by calculation or by orientation of the plane of underground workings upon the plane of the surface, when these are plotted to the same scale.

The main steps in the process of connecting the surveys by these methods are as follows:

- To make a triangulation survey at the surface to form an accurate frame work for the detailed survey of the surface.
- To suspend a wire In each shaft.
- To locate the positions of the wires accurately at the surface by connecting them to the surface triangulation so that the length and bearing of the line joining the two wires and the co-ordinates of the wire stations may be computed.
- To make an accurate underground traverse between the two wires.

Let A and B represent the plumb lines in the two shafts. New triangulation stations A_1 and B_1 near the mouth of each shaft, are established which are connected to the main triangulation scheme and also to the plumb-lines by measuring the angles and distances precisely. The coordinates of plumb-points A and B and also the length and bearing of AB are then calculated.



A traverse is made underground from wire to wire along the roadway connecting the two shafts using an assumed or false directional base. The co-ordinates of second wire B relative to the first wire A are again calculated from the underground traverse, and the length and also the bearing of the plumb-plane AB joining the two wires computed.

The bearing of the plumb plane as calculated from surface co-ordinates of the wires, is taken as its true value and compared with the bearing of the plumb-plane calculated from the coordinates of underground traverse. The length of AB as computed from underground traverse should agree closely with that obtained at the surface, but the bearing will differ owing to the assumption of a zero base direction. The difference in bearings is the error in direction of the assumed underground meridian, and the whole of the underground traverse has to be swung through this angle to bring the surface and underground plumb-planes in coincidence.

Angle through which the traverse had to be swung is applied as a correction to the bearing of each traverse line and the whole of the co-ordinates recalculated from the adjusted bearings.

If the amount of swing is small (less than 1 degree), the assumed co-ordinates can be adjusted to true value be applying the correction formula, and the work of recalculating the whole of the traverse avoided.

New latitude of B_1 = Latitude of B ± Departure of B x (swing in seconds/206265)

New Departure of B_1 = Departure of B ± Latitude of B x (swing in seconds/206265)

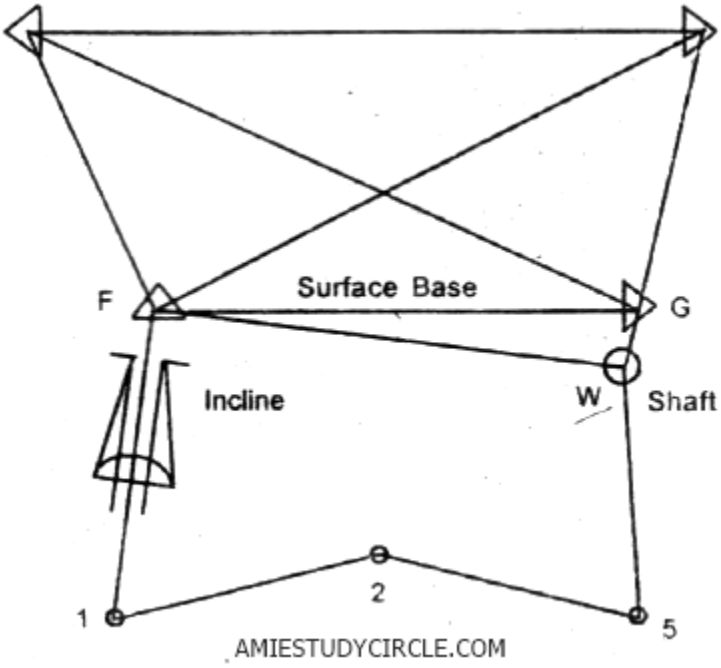
CORRELATION THROUGH A SHAFT AND A DRIFT

An inclined shaft or a drift offers the most favourable condition under which it is possible to carry out the correlation of the surface and underground surveys.

The survey is commenced from one of the lines in the surface triangulation or from a course of draft in the traverse of the surface.

One of the lines in the surface survey has been marked out in close proximity to the inclined entrance to the mine, the theodolite is erected and accurately centred over one of the line stations F and having adjusted the verniers on the horizontal limb of the instrument to the bearing of the line, the telescope is directed to the other station, and then the vertical axis of the instrument is clamped in that position.

A traverse survey is then carried out through the drift and the connecting roads extending from the bottom of that drift to the plumb-line W which is suspended in the shaft. The survey is also carried out from the base-line to the plumb-line over the surface.



When the surveys are plotted on the plan surface will be shown in its true position relative to the workings of the mine.

Example

In making a correlation of underground and surface surveys at mine by means of plumb lines A and E hanging in separate shafts, following data re obtained from an underground traverse made with the theodolite:

Line	*Assumed Azimuth*	*Distance, m*
AB	150^0	*303*
BC	90^0	*150*

CD	*44^0*	*363.5*
DE	*300^0*	*63.15*

If the true azimuth of the line AE at the surface is $333^0$31', calculate the true azimuths of lines in the underground traverse as accurately as possible.

Solution

Making coordinate table

The following table shows the calculated co-ordinates of the various points in the traverse, from which the bearing and distance of the line connecting the two shafts are obtained.

Line	Azimuth	Q.B.	Distance (m) L	Latitude Lcosθ		Departure LSinθ		Sin
AB	150^0	S30^0E	303					
					262.40	151.50	-	B
BC	90^0	Due East	150					
				-	-	150.0	-	C
CD	45^0	N45^0E	363.5					
				257.03	-	257.03	-	D
DE	300^0	N60^0W	63.15					
				31.575	-		54.689	E
			Total	+288.605	-262.40	+558.53	+54.689	
				-262.400		-54.689		
			Total coordinate of E	+26.205		+503.841		

Bearing and Azimuth of line AE

Natural tangent of bearing line AE = 503.841/26.205 = 19.227

∴ Bearing of line AE = N $87^0$1' E

Azimuth of AE = $87^0$1'

True Azimuth of other lines

Difference between surface and underground line connecting the two shafts

= (360° - 333°31') + 87°01' = 113°30'

True azimuth of line AB = 150° - 113°30' = 36°30'

True azimuth of line BC = 90° - 113°30' + 360° = $336^0$30'

True azimuth of line CD = 45° - 113°30' + 360° = 291 °30'

True azimuth of line DE = 300° - 113°30' = 186°30'

Example

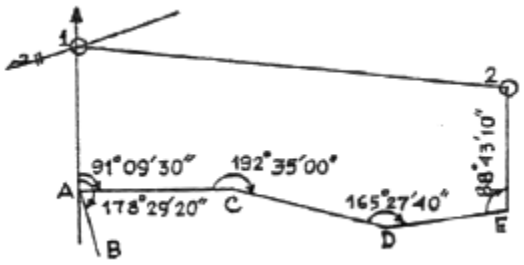
Underground and surface surveys at a colliery are to be connected by reference to single wires suspended in shafts 1 and 2, respectively. Azimuth of the line between the wires 1 and 2 obtained from surface survey is $202^0$14'20". Undernoted are clockwise angles and horizontal distances of an underground survey made from wire 1 to 2:

Angle		*Horizontal distance, m*
1 AC	*$91^0$09'30"*	*1A - 45*
ACD	*$192^0$35'00"*	*AC - 61.8*
CDE	*$165^0$27'40"*	*CD - 55*
DE2	*$88^0$43'10"*	*DE - 38.8*
		E2 - 46.4

During the survey, an underground baseline AB was set out and the values of angle 1AB was found to be $178^0$29'20". Calculate the azimuth of the line AB in relation to that between the shaft wires at the surface.

Solution

Rough sketch



Making coordinate table

Assuming the line A1 of underground traverse is the meridian, the co-ordinates of the underground traverse stations are calculated as follows:

Line	**Angle**	**Azimuth (θ)**	**Distance L (m)**	**Latitude LCosθ**	**Departure LSinθ**
1A		180^0	45		
A				-45	
AC	1AC = $91^0$09'30"	$91^0$09'30"	61.8		
C				-1.2493	+61.7874
CD	ACD = $192^0$35'00"	$103^0$44'30"	55		

D				-13.0649	+53.4257
DE	CDE = $165^0$27'40"	$89^0$12'10"	38.8		
E				+0.5398	+38.7962
E2	DE2 = $88^0$12'10"	$357^0$55'20"	46.4		
2				+46.3695	-1.6823
			Total coordinate of stn 2	-12.4049	+152.327

Azimuth of line AB

Traverse bearing of line 1-2 = $\tan^{-1}\frac{152.327}{-12.4049} = S85^0 20'39" E$

$\therefore$ Azimuth = $94^0$39'21"

Actual bearing of line 1-2 = $202^0$14'20"

Difference = $202^0$14'20" - $94^0$39'21" = $107^0$34'59"

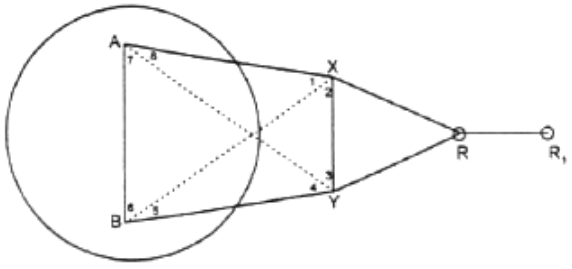
Hence Azimuth of the line AB in relation to shaft wires 1-2 at surface

= 178°29'20 + 107°34’59" = 286°04’19"

WEISS QUADRILATERAL METHOD

If the layout of the shaft, either at the surface or underground, precludes exact or approximate alignment, the method of Weiss quadrilateral provides a possible alternative.

The Weiss quadrilateral is formed by the two wires and the two instrument stations. Theodolites are set up at station X and Y and the angles 1, 2 and YXR are measured at X and the angles 3, 4 and XYR are measured carefully by constant repetitions. The lengths of the sides of the quadrilateral and the diagonals are measured as a check on the angles although all the calculations involve the angular measurements.



The angles at R, YRX, XRR, and YRR, are measures Sufficient informations have now been obtained to calculate the azimuth of the plumb-plane at the surface or the azimuth of RR_1 below ground.

To calculate the angles at A and B

5 = 180° - (2+ 3+ 4)

$8 = 180° - (1 + 2 + 3)$

It is known that the product of the sines of the left-hand angles equals the product of the sines of the right-hand angles in any polygon.

$$\therefore \quad \frac{Sin\,7}{Sin\,6} = \frac{Sin\,2.Sin\,4.Sin\,8}{Sin\,1.Sin\,3.Sin\,5}$$

Since all the angles on the right hand side are known then Sin 7/Sin 6 has a value say x But & + 6 = 2 + 3 which in known value say θ.

∴ 7 + 6 = θ and hence 7 = θ - 6

Since Sin 7/Sin 6 = x

Then Sin 7 = xSin 6

and Sin(θ - 6) = xSin 6

Expanding Sinθcosθ - cosθSin 6 = xSin 6

Sinθcos6 = xsin6 + cosθsin6 = sin6(x + cosθ)

Therefore $$\frac{Cos\,6}{Sin\,6} = \frac{x + \cos\theta}{Sin\theta}$$

or $$Cot\,6 = \frac{x + \cos\theta}{Sin\theta}$$

Angle 6 can thus be calculated and angle 7 found by subtraction. Thus the azimuths of the line are determined and eventually the co-ordinates of stations. Observations may be made from both sides of the shaft if possible. Several plumb-planes may also be used.

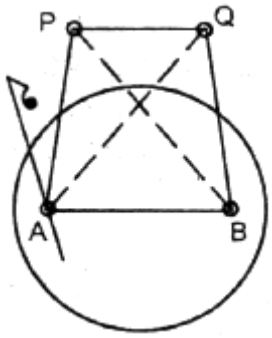
Example

The Weiss quadrilateral angles are measured at two points A and B with the object of determining the azimuth of PQ; Azimuth of AB = $89^0 42'$; ∠PAQ = $39^0 54'$; ∠QAB = $42^0 19'$; ∠PBQ = $41^0 08'$; ∠ABP = $44^0 24'$; Length of AB = 30 m. Find the bearing of PQ.

Solution

Rough sketch

In the given data, the length of AB = 30 m, and the bearing of AB = 89°42'. The angle PAQ, BAQ, PBQ and ABP are also given.



Angles

Angle ABQ = (44°24' + 41°08') = 85°32'

Angle PAB = (39°54' + 42°19') = 82°13'

Angle AQB = 180° - (85°32’ + 42°19') = 52°09'

Angle APB =180° - (82°13' + 44°24’) = 53°23‘

Sine rule

Now from the sine rule in the triangle ABQ:

$$BQ = \frac{SinQAB}{SinAQB} x\, AB = \frac{Sin42^0 19'}{Sin52^0 09'} x\, 30 = 25.5779\, m$$

From the triangle APB:

$$PB = \frac{SinPAB}{SinAPB} xAB = \frac{Sin82^0 13'}{Sin53^0 23'} x30 = 37.0321 m$$

Calculations for Angle BQP

In the triangle PQB:

$$\tan\frac{Q-P}{2} = \frac{PB-QB}{PB+QB} x\, Cot\frac{B}{2} = \frac{37.0321-25.5779}{37.0321+25.5779} x\, Cot\frac{41^0 08'}{2} = 0.2094$$

$\therefore$ $\frac{Q-P}{2} = \tan^{-1}(0.2094) = 11^0 49'50"$

$\therefore$ Q - P = 2 x $11^0$49'50" = $23^0$39'40"

Again Q + P = 180^0 - $41^0$08' = $138^0$52'

Solving Q = 81°15’50"

Given the bearing of AB = 89°42',

$\therefore$ Angle ABQ = 85°32'

Bearing of the reference line PQ

The bearing of BQ = (89"42' + 85°32') + 180° = 355°14'

But the angle BQP = 81°15'50"

∴ The bearing of QP = (355°14" + 81 °15'50") - 180° = 256°29'50"

∴ The bearing of PQ = (256°29’50" - 180°) = 76°29’50"

Hence the bearing of the reference line PQ = $76^0$29'50"

PHOTOGRAPHIC SURVEY

Photographic surveying or photogrammetry or photometry is the science of making measurements on photographs. It relies on photographs taken from stations on the ground. Since photographs do not provide exact plans of the terrain to be surveyed, they -must be processed in accordance with the objects surveyed and their images. The discipline dealing with the methods of processing photographs to prepare precise plans of the field and locate the objects being surveyed spatially is photogrammetry.

Principle of Photogrammetry

The principle of photogrammetric survey in its simplest form is very similar to that of the plane survey. The only difference is that the most of the work which in plane table survey is executed in the field, is done in office. The principal point of each photograph is used as a fixed station in plane table surveys and rays are drawn to get points of intersections very similar to those used in plane table.

Photogrammetry is particularly suitable for topographical or engineering surveys and also for those projects demanding higher accuracy.

AERIAL PHOTOGRAMMETRY

Aerial Photographs

These photographs are taken from camera stations in the air with the axis of the camera vertical or nearly vertical.

According to the direction of the camera axis at the time of exposure, aerial photographs are further divided into the following main classes:

- Vertical photographs.
- Oblique photographs.

Vertical photographs

These photographs are taken from the air with the axis of the camera vertical or nearly vertical. A truly vertical photograph closely resembles a map. These are utilised for the compilation of topographical and engineering surveys on various scales.

Oblique photographs

These photographs are taken from air with the axis of the camera intentionally tilted from the vertical. An oblique photograph covers larger area of the ground but clarity of details diminishes towards the far end of the photograph. Depending upon the angle of obliquity, oblique photographs may be further into two categories.

- **Low oblique photographs**. An oblique photographs which does not show the horizon, is known as low oblique photograph. Such photographs are generally used to compile reconnaissance maps of inaccessible areas.
- **High oblique photograph**. An oblique photograph which is sufficiently tilted to show the horizon, is known as high oblique photograph. Such photographs were previously used for the extension of planimetric and height control in areas having scantly ground control. With the introduction of radial line assumptions theory, oblique photographs are no longer used.

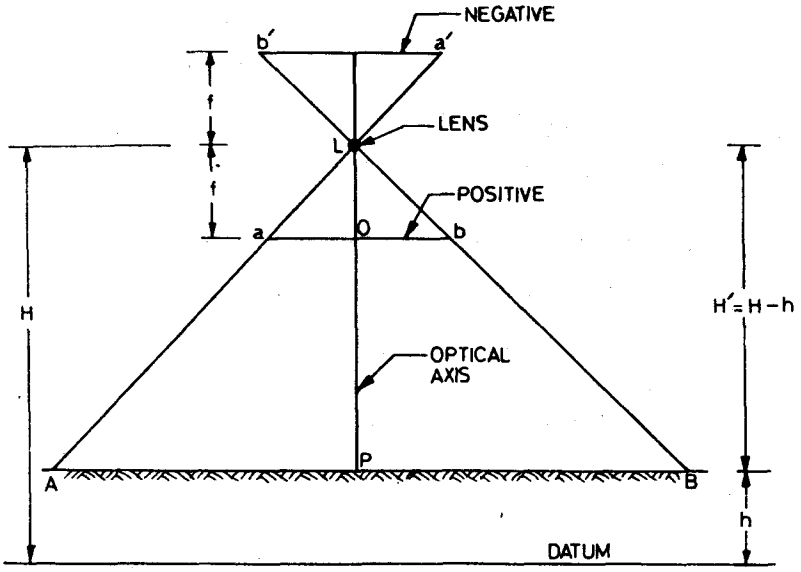
Scale of Vertical Photograph

Scale of photograph = photo distance/ground distance

In photoscale, the scale varies with the variation in the elevation of terrain because a photograph is the perspective projection. In the case of a photograph, the terrain area which has a higher elevation is closer to the camera and, therefore, appears larger than the terrain area lying at a lower elevation.

Flat Terrain

Following figure shows a section taken from a vertical photograph.

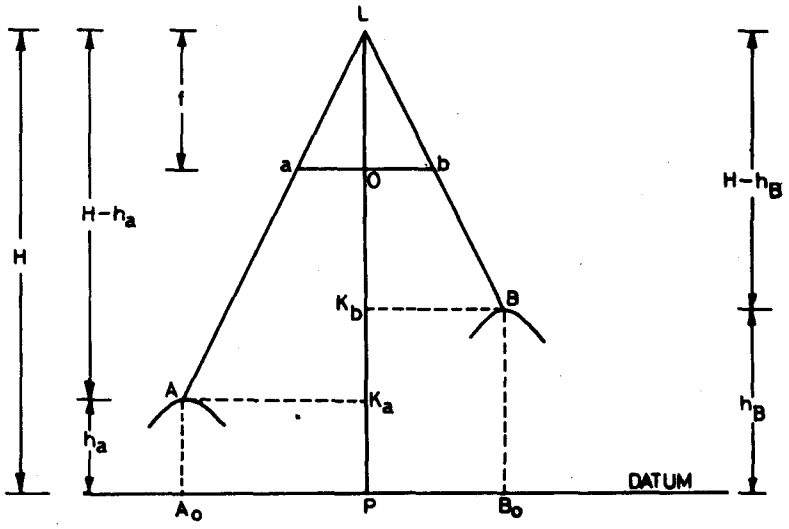


The elevation of the lens above the datum is called the flying height (H). The elevations of A and B are h above the datum. The datum is usually taken at Mean Sea Level (M.S.L.). In this figure, the ground is assumed to be flat at an elevation of h. Let P be the datum principal point exactly below the exposure station L.

$$S = f/H' = f/H - h$$

Variable Elevation Terrain

Following figure shows vertical photograph of the terrain in which the elevation is not constant. As the elevation varies, the object distance (H') also varies. Therefore scale also varies. The scale increases with an increasing terrain elevation (h).



Let the elevations of the two points A and B be respectively h_a and h_B above the datum. These points are imaged on the positive as a and b. The principal point P of the vertical photograph is also shown in figure.

The scale of the vertical photograph at the point A is given by

$$S_A = \frac{f}{H - h_a}$$

Similarly, the scale of the vertical photograph at a point B is given by

$$S_B = \frac{f}{H - h_B}$$

Thus the scale at B is different from that at A.

Datum scale (S_d)

Datum scale of a vertical photograph is the scale that would be effective over the entire photograph if all the ground points were projected vertically downwards on the datum (MSL) before being photographed. From above figure

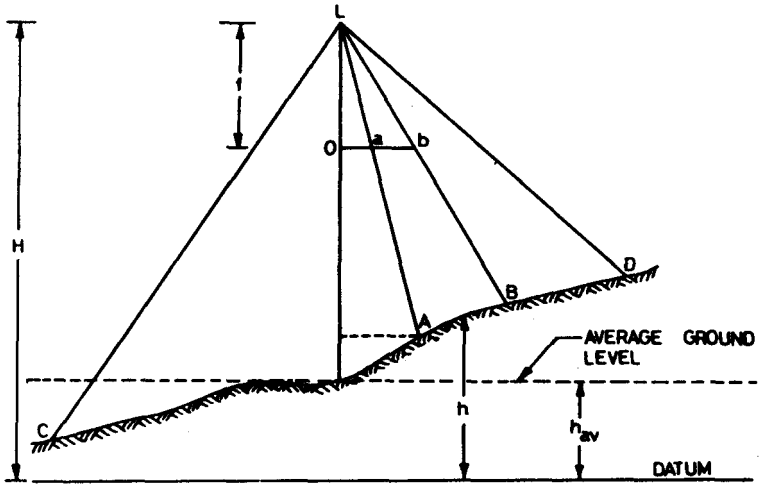
$$S_d = f/H$$

Average Scale (S_{av})

Average scale is the scale at the average elevation of the terrain covered by a vertical photograph.

$$S_{av} = \frac{f}{H - h_{av}}$$

where h_{av} is the average elevation as shown in following figure.



Example

A vertical photograph is taken over a flat terrain from an altitude of 1800 m above datum (M.S.L.). If the elevation of the ground is 300 m, determine the scale of the photograph. The focal length of the camera is 152 mm.

Solution

We know $S = \frac{f}{H-h} = \frac{0.152}{1800-300} = 1.0133 \times 10^{-4}$ = 1 in 9868

Example

The elevations of three points A, B, C above the datum are respectively 1500 m, 1200 m and 1000 m. If the flying height above the datum is 3000 m, determine the maximum scale, minimum scale and average scale. The focal length of the camera is 150 mm.

Solution

We know S = f/H – h

Scale at A, $S_A = S_A = \frac{0.150}{3000-1500} = 1 \times 10^{-4}$ or 1 in 10,000

Similarly S_B = 1 in 12000

and S_C = 1 in 13,333

Average elevation

$$= \frac{1}{3}(1500+1200+1000) = 1233.33\,\text{m}$$

Average scale

$$S_{av} = \frac{f}{H-h_{av}} = \frac{0.150}{3000-1233.33} = 8.491 \times 10^{-5} \text{ or } 1 \text{ in } 11778$$

REQUIRED NUMBER OF PHOTOGRAPHS

Method 1: Length and width of area is given

L_1 = length of ground area

W_1 = width of ground area

Then $N_1 = \dfrac{L_1}{(1-p_l)\left(\dfrac{1}{S}\right)l} + 1$

where N_1 is number of photographs in each strip.

Number of flight lines are

$$N_2 = \frac{W}{(1-p_w)\left(\frac{1}{S}\right)w} + 1$$

Thus total number of photographs for entire area

$$N = N_1 x N_2$$

Method 2: Area of ground is given

$$L = (1-p_l)\frac{1}{S}l$$

$$W = (1-p_w)\frac{1}{S}w$$

where p_l is end lap between successive flight strips and p_w is the side lap between flight strips.

Now ground area

$$a = LW$$

$\therefore$ Number of photographs required to cover entire area is

$$N = \frac{A}{a}$$

Example

Determine the number of photographs required to cover an area 25 km x 20 km if the scale is 1 in 10,000 and the format is 230 x 230 mm. Take longitudinal lap as 60% and the side lap as 30%.

Solution

$$N = \left[\frac{L_1}{(1-p_1)\frac{1}{S}} + 1\right]\left[\frac{W}{(1-p_w)\frac{w}{S}} + 1\right]$$

$$= \left[\frac{25x10^3}{(1-0.60)(0.230x10^4)}+1\right]\left[\frac{20x10^3}{(1-0.30)(0.230x10^4)}+1\right]$$

$$= 406$$

Example

Determine the number of photographs required to cover an area of 500 sq km if the scale of the photographs is 1 in 10,000 and the photograph format is 230 x 230 mm. Take the longitudinal overlap as 60% and the side overlap as 30%.

Solution

$$a = (1-p_l)(1-p_w)(lw)\frac{1}{S^2}$$

$$= (1-0.60)(1-0.30)(0.230x0.230)(10,000)^2$$

$$= 1481200 \text{ m}^2 = 1.4812 \text{ km}^2$$

$$\therefore \quad N = \frac{A}{a} = \frac{500}{1.4812} = 338$$

GLOBAL POSITIONING SYSTEM (GPS)

The global positioning system (GPS) consists of a network of 24 satellites in roughly 12 hour-orbits, each carrying atomic clocks on board. The orbital radius of satellites is roughly twice the earth diameter. The orbits are nearly circular, with eccentricity of less than 1%. Orbital inclination its the earth's equator is roughly 55 degrees. The satellites have orbital speeds of about 3.9 km/s in a frame centered on the earth. Normally, satellites occupy one of six equally spaced orbital planes. Four satellites occupy each plane spread at roughly 90 degree intervals around the earth in that plane. The precise orbital periods of the satellites are close to 11 hours and 58 minutes so that ground tracks of the satellites repeat day after day.

The satellite clocks are fully synchronized with the ground atomic clocks. Knowing the instant a signal is sent from a satellite and the time delay for that signal in reaching a ground receiver, the accurate distance between the satellite and the ground receiver can be determined.

By using four satellites to triangulate with these determined distances, the position of the unknown location can be determined with good precision.

ASTRONOMICAL SURVEYING

It deals with

- Science deals heavenly bodies such as Sun,

- Moon planets, Polaris i.e. North Star.
- Find absolute position of any point.
- Angular direction of the Sun, Star relative to
- Horizon.
- Celestial sphere.
- Motion of Celestial sphere.
- Earth and its Motion

Purposes of Field Astronomy

- Determine absolute location/ position and direction of any line on the surface of the Earth.
- Determine absolute location of any points/object → astronomical observation to celestial bodies such as Moon, Sun, Star and planets.
- Used to find Angular position of Stars.
- Determination of the position of Points on the Earth
- Determination of Orientation.

DEFINITION OF ASTRONOMICAL TERMS

The celestial sphere

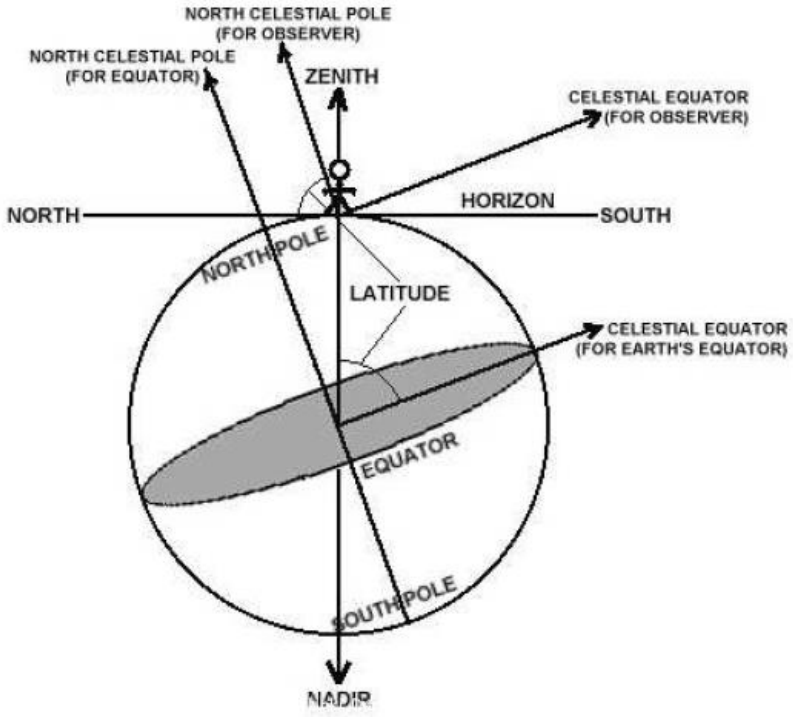
- The Imaginary sphere on which the stars appear to lie or to be studded is known as the Celestial sphere.
- Radius of celestial sphere may be of any value.
- The center of the Earth may be taken as the center of celestial sphere.

The Zenith and Nadir

- The Zenith (Z) is the point on the upper portion of the celestial sphere marked by plumb line, above the observer.
- The point on the celestial sphere immediately above the observer's station.

The Nadir (Z')

- Is the point on the lower portion of the celestial sphere marked by plumb line, below the observer.
- The point on the celestial sphere vertically below the observer's station.



Celestial Horizon/True Horizon/Geocentric Horizon

- It is the great circle traced upon the celestial sphere by the plane which is perpendicular to the Zenith-Nadir line and which passes through the center of the earth.
- Great circle is a section of a sphere when the cutting plane passes through the centre of the sphere.

Terrestrial Poles

- The Terrestrial Poles are the two points in which the Earth's axis of rotation meets the earth's sphere.
- Terrestrial poles are the points where the Earth's surface and Earth axis of rotation meets.

Terrestrial Equator

- The Terrestrial Equator is the great circle of the Earth, the plane of which is at right angles to the axis of rotation.
- The two poles are equidistant from it.

Celestial Poles

If the Earth's axis of rotation is produced indefinitely , it will meet the celestial sphere in two points called the North and South celestial poles

Celestial Equator

It is the Great circle of the celestial sphere in which it is intersected by the plane of celestial equator.

Sensible Horizon

- The circle in which a plane passing through the Earth's surface and point of observations and tangential to the earth's surface or (normal to the Zenith nadir line) intersects with celestial sphere is called as the sensible Horizon.
- The line of sight of an accurately levelled telescope lies in this plane.

Visible Horizon

- The circle of contact with the Earth surface of the visible rays passing through the point of observation is called as visible horizon.
- Visible Horizon is a small circle of Earth.

Vertical Circle

- A vertical circle of the celestial sphere is a Great circle passing through the Zenith (Z) and Nadir (Z').
- All the vertical circle cut the celestial Horizon at Right Angles.

Observer's Meridian

- The Meridian of any Particular point is that circle which passes through the Zenith (Z)and Nadir (Z') and Poles P and P1 of the point as well as through the poles.
- It is thus a vertical circle.

Azimuth (A)

The azimuth of a heavenly body is the angle between the observer's meridian and the vertical circle passing through the heavenly body

Equinoctial Points

- The points of the intersection of the ecliptic with the equator are called the equinoctial points.
- The declination of the sun is zero at the equinoctial points.
- The Vernal Equinox or the first point of Aries is the point in which sun's declination changes from South to North, and marks commencement of spring.
- It is a fixed point on celestial sphere.

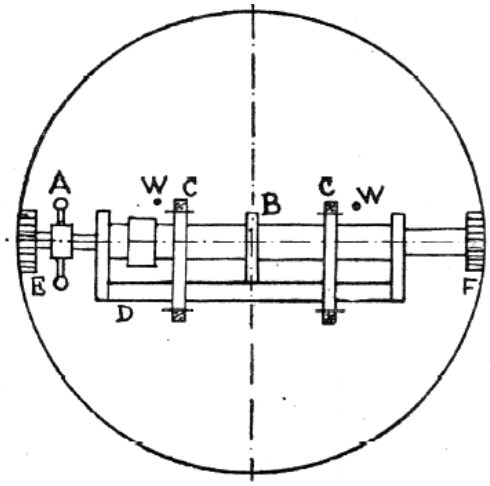
- The Autumnal Equinox or the first point of Libra is the point in which Sun's declination changes from North to South, and marks the commencement of autumn.
- Both the Equinoctial points are six month apart in time

Ecliptic: (Road of The Sun)

- The Ecliptic is the Great circle of the heavens which the sun appears to describe on the celestial sphere with Earth as a centre in the course of the year.
- The plane of the ecliptic is inclined to the plane of the equator at an angle of about 23° 27' .

Solstices

- Solstices are the points at which the North and South declination of the Sun is maximum.
- The point C at which North declination of the Sun is maximum is called Summer solstice.
- The point C' at which South declination of the Sun is maximum is known as winter solstices.



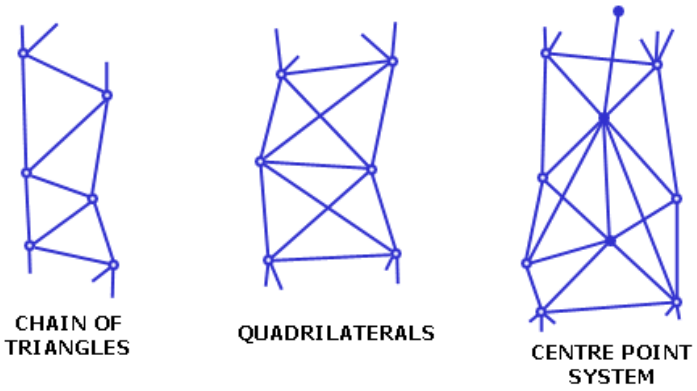
A: Screw jack adjustment; B: Sight vane; C: Thumb Screw; D: Graduated rod

E: Gapping (Swivel end); F: Capping (fast end); W - W = Wires

The upper frame carries a graduated wooden rod which can be adjusted for position by means of thumb screws. The rod is fitted with the scale at each end, and a sighting vane of its centre.

TRIANGULATION

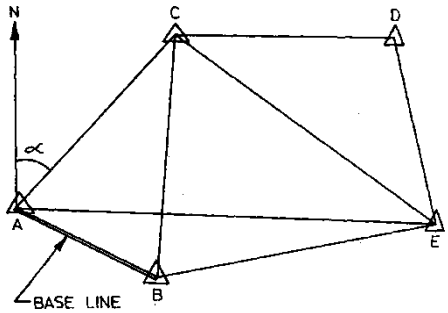
Triangulation consists of a series of connected triangles which adjoin or overlap each other, angles being measured from determined fixed stations. Triangulation reduces the number of measures that need to be taped and for this reason is often a preferred method of survey. A known base-line measurement is required. Three examples of triangulation systems are shown below.



A single chain of triangles is a rapid and economical system for covering a narrow strip of land. A chain of quadrilaterals is more accurate with checks being made by various combinations of angles and sides as the survey proceeds. Larger areas use a central point arrangement. A point to note is that all angles should be **more than 20°**. Angles less than 20° are not considered valid for fixing position. They introduce inaccuracies. This is much the same in navigation where a fix by two bearings requires an angle of intersection of approximately 90°, and for three bearings approximately 60°. Angles less than 30° are not acceptable.

The best shape of a triangle is an isosceles triangle whose base angles are $54^0 14'$ each. However, from practical considerations, it has been found that an equilateral triangle is the most suitable and may be taken as the most well conditioned triangle.

Following shows a triangulation network consisting of 5 triangulation stations A, B. C. D and E.



The length of the base line AB is measured accurately. (The base line is usually shown by a thick line or by two parallel lines). The measured base line provides the scale of the network because all sides are proportional to this line. Because this line provides the scale of network, it is called the base line. All the horizontal angles at each station are also measured accurately with a theodolite. The horizontal angles and the base line measurements are used to compute the horizontal positions of the triangulation stations.

To define the survey datum of the triangulation network, the position of at least one point and the direction of at least one line must be known. Generally, one of the stations in a large triangulation network is a GTS station which is a part of the national geodetic network whose position has already been established by the Survey of India.

CLASSIFICATION OF TRIANGULATION

On the basis of accuracy and purpose, triangulation systems are generally classified as under:

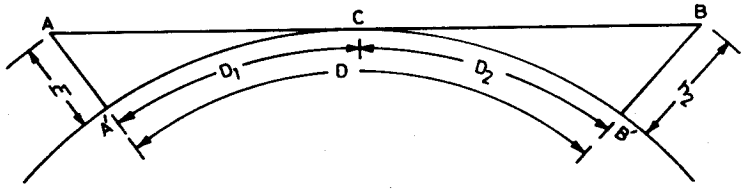
1. Primary triangulation or first-order triangulation.
2. Secondary triangulation or second-order triangulation.
3. Tertiary triangulation or third-order triangulation.

DETERMINATION OF ELEVATIONS OF STATIONS

If the elevation of station A is known, the elevation of another station B can be determined so that it shall be visible from station A.

No obstruction due to intervening ground

See following figure.



Now $h_1 = 0.06735 D_1^2$

and $h_2 = 0.06735 D_2^2$

Level of station B = datum + h_2

where h_1 is known elevation of station A above datum (m) and h_2 is required elevation of station B above datum (m).

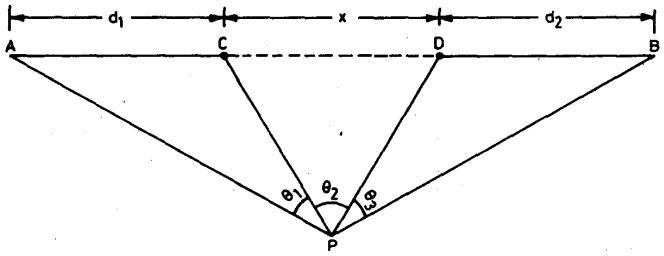
Height of scaffold = (datum + h_2) – R.L. of ground at B

The actual height of signal is usually kept 2 to 3 m greater than the calculated height.

COMPUTATION OF A PORTION OF A BASE LINE

Sometimes a portion of a straight line can not be directly measured in the field due to an intervening obstruction, such as stream. In such a case, the length of that portion of the base line is found by computation.

Following figure shows a straight base line AB in which the portion CD can not be measured in the field. A point P is selected on one side of the base line. The angles θ_1, θ_2, and θ_3 are measured with theodolite. Let the distance CD be x. Let angles BAP and ABP be respectively α and β.



Now $$(d_1 + x)(d_2 + x) = d_1 d_2 . \frac{\sin(\theta_2 + \theta_3)\sin(\theta_1 + \theta_2)}{\sin\theta_1 . \sin\theta_3}$$

This equation can be solved for x if all other parameters have been measured in the field.

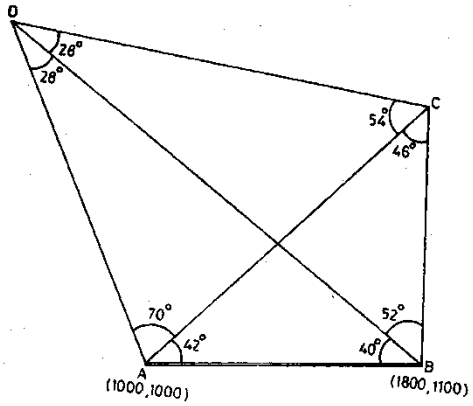
COMPUTATIONS IN TRIANGULATION

Computations in a plane triangulation system are usually done in the following sequence.

1. Adjustments of observed angles.
2. Computation of lengths.
3. Computation of azimuths and the latitudes and departures of all the sides,
4. Computation of independent coordinates

Example

Given figure shows a triangulation system. The coordinates of the ends of the base line are (1000, 1000m) and (1800, 1100 m). Determine the coordinates of C and D.



Solution

The strongest route is through the triangle ABC and ACD.

From the triangle ABC,

$$\frac{AB}{\sin 46^0} = \frac{BC}{\sin 42^0} = \frac{AC}{\sin 92^0}$$

Lengths of the line

$$AB = \sqrt{(1800-1000)^2 + (1100-1000)^2} = 806.226 \text{ m}$$

$$\therefore \qquad BC = \frac{AB \text{ x } \sin 42^0}{\sin 46^0} = \frac{806.226 \text{ x } 0.6691306}{0.7193398} = 749.952 \text{ m}$$

$$AC = \frac{806.226 \text{ x } \sin 92^0}{0.7193398} = 1120.103 \text{ m}$$

Now in the triangle ACD,

$$\frac{AC}{\sin 56^0} = \frac{CD}{\sin 70^0} = \frac{AD}{\sin 54^0}$$

or $$CD = \frac{AC \text{x} \sin 70^0}{\sin 56^0} = \frac{1120.103 \text{x} 0.9396926}{0.8290376} = 1269.608 \text{ m}$$

$$AD = \frac{1120.103 \text{x} \sin 54^0}{0.8290376} = 1093.053 \text{ m}$$

Azimuth and latitudes and departures of line

Line AB, $$\tan\theta = \frac{1800-1000}{1100-1000}$$

or $$\theta = 82^0\ 52'\ 30''$$

Azimuth = $82^0\ 52'\ 30''$

Latitude = 806.226 cos $82^0\ 52'\ 30''$ = 100.00 m

Departure = 806.226 sin $82^0\ 52'\ 30''$ = 800.00 m

Line AC, Azimuth = $82^0\ 52'\ 30'' - 42^0 = 40^0\ 52'\ 30''$

Latitude = 846.954 m

Departure = 733.008 m

Line BC, Azimuth = $180^0 + 82^0\ 52'\ 30'' + 92^0 = 354^0\ 52'\ 30''$

Latitude = 746.954 m

Departure = -66.992 m

Line CD, Azimuth = $(354^0\ 52'\ 30'' - 180^0) + 100 = 274^0\ 52'\ 30''$

Latitude = 107.894 m

Departure = -1265.015 m

Line AD, Azimuth = $82^0\ 52'\ 30'' + (360^0 - 112^0) = 330^0\ 52'\ 30''$

Latitude = 954.848 m

Departure = -532.007 m

Independent coordinates of C and D

Point C,

X – coordinate = 1000 + Departure of AC

= 1000 + 733.008 = 1733.008 m

Y – coordinate = 1000 + latitude of AC

= 1000 + 846.954 = 1846.954

Point D,

X – coordinate = 1733.008 + Departure of CD

= 1733.008 + (-1265.015) = 467.993 m

Y – coordinate = 1846.954 + latitude of CD

= 1846.954 + 107.894 = 1954.848 m