PART I

TOPOGRAPHICAL SURVEYING FOR GEOLOGICAL WORK

INTRODUCTION

Besides routine geological mapping, on standard topographical maps, on the usual scale of 1 inch=1 mile, the geologist in India is often required to undertake various other types of work which call for a fair knowledge of surveying methods. Instruments used in surveying, their construction and detailed descriptions have been given in several standard text books on the subject. For carrying out his normal work, the field geologist requires only a few practical hints as to the handling and the care of these instruments. Thus the purpose here is to show how the survey instruments are employed in carrying out surveys for geological work.

Simple methods of surveying, such as tape and compass survey, chain survey, and plane table survey have been given their due importance. Sufficient emphasis on the use of the Brunton Compass has also been laid. In the methods of survey, often used by the geologist, for mapping various types of mineral deposits, the Brunton Compass is indispensible. It is also handy for underground work, such as stope surveys and detailed geological mapping of mine workings, and in the examination of excavations for large scale projects.

Considerable improvements have been affected to surveying instruments within the last two decades. The introduction of the microptic instruments, such as the microptic telescopic alidade, and the microptic theodolite have gone a long way in relieving the burden of the surveyor. These instruments are sufficiently robust and can be used with considerable advantage in the survey of mineral deposits. The construction and use of such instruments has been indicated under the relevant chapters. The later developments in this line, are the introduction of microptic stadia instruments known as self reducing tacheometers. These instruments have further contributed to easing the task of the field geologist. But at the present time these instruments are rather expensive and may be beyond the pocket of the private practising geologist. These instruments are obtainable from manufactures in the USSR and W. Germany etc., but it is expected that similar instruments of indigenous manufacture will be more easily available in the near future.

Since World War II application of air photography to surveying has increased manyfold; and air survey has become a routine for surveying virgin terrain, since air photos, especially verticals provide additional information, besides data for the construction of maps. Air photos are used in soil conservation, agriculture, irrigation projects, engineering projects, forestry etc., and often contain a fund of geological information. Thus the subject of photogeology has come into existence.

Photogrammetry, which includes, preparation of maps from air photos, together with measurement of heights from photos, by stereoscopic methods, and contouring, is an essential requisite for the photogeologist. An endeavour has been made to compress all essential information required for preparing simple maps from air photos, both verticals (stereo-pairs) as well as from obliques.

PREPARATION FOR SURVEY WORK

1. Assembling of instruments, accessories and surveyor's kit, required for the particular method to be adopted, is primary.

2. Reconnaissance or preliminary examination of the area by traverses is carried out, so as to obtain a thorough knowledege of the lay out, with a view to decide what details are to be included, depending on the method proposed to be adopted and the amount of time available. A pair of binoculars is useful for purposes of reconnoitering.

3. Fixing of stations—The stations should be so located such that there is no obstruction to direct visibility between any two adjacent (at least) stations, unless, such an obstruction is unavoidable.

4. Referencing stations—All important stations should be located with reference to three permanent objects such as masonry work or large trees etc., and a sketch plan showing distances and direction from such permanent objects should be made out in the field.

5. The sketch plan of the whole area should be prepared-showing the stations and other salient features. The north line and approximate scale should also be indicated.

CHAIN SURVEY

The equipment consists of chain (30 m) or 100 ft. long, 10 arrows, (15 m) or 50 ft. metallic tape, ranging rods, required number of station flags, plumb bob, field book, pencil, and eraser.

In the ideal chain survey, no angular measurements are made. The stations are so located that the area, to be surveyed, is divided up into triangles and all the sides of these triangles are measured. The triangle is chosen, as it is the only plane figure whose shape can be fixed merely by linear measurements. However, for the survey to be accurate, the triangles should be "well conditioned", i.e., the angles should be as near to 60° as possible. This condition is essential for obtaining clear intersections when the survey is plotted. Acute intersections cannot be pinpointed, and the ill conditioned triangle or triangles in which any angle is less than 30° are to be strengthened by *"tie lines"*, which may either be perpendiculars from any angle to the opposite sides, or any line which connects any angle to any known point on the opposite side. Such tie-lines will serve as a check on the accuracy of the plotting of the survey.

Three men are normally required for chain survey : (*a*) Leader, (*b*) Recorder (*c*) Follower. (The work of follower and recorder can also be combined if necessary.).

Procedure : Flags preferably of conspicuous colours, attached to poles are planted at the various stations. The follower takes his stand at the starting station and opens the chain in the manner described later and gives one of the handles to the leader, who also takes the plumb bob, arrows and a ranging rod.

In order to open the chain, a firm grip is taken of both the handles with the left hand and a few links are paid off, while the rest of the folded chain is gripped in the right hand. Now the unopened portion of the chain in the right hand is thrown forward, in the direction of the line to be measured, while the left hand is slightly jerked backwards. The chain opens out but remains folded at the 50th link i.e. the mid link. The follower now throws one of the handles to the leader who takes it up, moves in line with the forward station, and stretches the chain to its full length. The follower aligns the leader's ranging rod with the forward station. The chain is now stretched in line with the rod and the arrow planted alongside the handle. The follower leaves the chain and accompanies the recorder, who walks along astride the chain noting the measurements, while the follower assists him in taking offsets to objects or other features seen on either side of the chain line with the metallic tape. The measurements are recorded in the manner given in Fig. 1.1.

The follower, in all subsequent operations, picks up the arrows so that by counting the number of arrows, both with the leader and follower, a mutual check, on the distance measured, is obtained.

Note : (1) The various shapes of the tags on the chain—the 10 ft. and 90 ft. tags have one point. The 20 and 80 ft. tags have two points. The 30 and 70 ft. tags have three points. The 40 and 60 ft. tags have four points while the 50th feet tag is circular.

(2) The double line in centre of the field book (Fig. 1.1) represents the chain line, and the figures noted between the double line, denote chain distances. Figures noted outside indicate the offset distances of respective points from the corresponding points on the chain line.

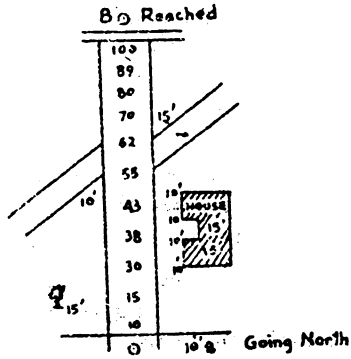


Fig. 1.1

(3) In recording the distance of a point to be 'off-set" the follower places the end of the tape at the point. The leader pays of the tape and then holding it taut and level over the chain describes an arc and obtains the minimum tape reading as well as the chain reading (at the tangential point).

(4) The interior filling in or the location of details, of the survey is done either with the help of extra tie lines or by means of subsidiary compass and tape surveys.

Method of closing the chain is as follows : The chain is pulled by the middle tag (*i.e.* the 50 ft. tag) so that the two handles come together. The two links on either side of the 50th tag, are held together in the left hand, while the alternate pair of links are picked up two together at a time with the right hand folded upwards and held in the left hand.

Note : It is a good and convenient practice for the follower to align the leader with the forward stations by a show of hands. Shouting is tiresome and leads to confusion. The right hand is used for indicating movement towards the right side and the left hand for movements towards the left. In case, the same hand is employed to indicate movement in both directions confusion will arise. While winding the metallic tape with the right hand, the tape is made to pass between the forefinger and the second finger of the left hand in which the case is held, so that any twists in the tape automatically straighten out.

COMPASS AND TAPE (OR CHAIN) SURVEY

This is the method most frequently used by geologists in mapping and in preliminary mineral investigations, in small areas or for interior filling.

Instruments required : (1) Compass, which may be (*a*) Prismatic- compass, (*b*) Clino compass, (*c*) Miner's dial, (*d*) Brunton compass; (2) Chain and arrows or tape; and (3) Ranging rods etc. as in Chain Survey.

Procedure : The angular measurements are made with the compass and recorded either as whole circle bearings or as quadrant bearings and the distances are measured, with the chain or tape. It is always advisable to close such traverses, so as to come back to the starting point. This will enable the surveyor to judge the magnitude of errors, if any, involved in the survey. If, on plotting, the errors are small, then the closing error could be distributed in the manner described later.

Note : The compasses are of two types :

A. Those in which the needle moves (relatively) over a fixed graduated scale, *e.g.*, Clino compass, Brunton compass, and Miners' dial. In these instruments there is an interchange of the east and west points so as to compensate the relative movement of the needle to the stationary scale so as to enable quadrant bearing to be obtained directly. Quadrant bearings are expressed in the following manner, *e.g.* NW, NNW, WNW, or ENE, NE, NNE, etc. or N 30° E, N 20° W, S 10° E, S 5° W, *i.e.* always in relation to the N or S direction.

The whole circle forward bearing is the reading of the north end of the needle on the 360° scale. For example in (Fig. 1.2), the forward bearing of AB is 90°, *i.e.*, bearing from A to B, while the back bearing from B to A is 270°. If during the course of mapping it is required to locate one position A, with reference to a known point B, then the back bearing from the unknown point A to the known point B is measured and the distance plotted on the map from the known point B or if on the other hand the position of A is known and it is required to locate further points, then the forward bearings are determined and the distances plotted. Frequently in practice, intersection of 2 or more plotted directions are used for a location of greater accuracy.

B. Those instruments, in which the graduated scale moves along with the needle, the bearing, with reference to the magnetic north is obtained directly by reading the graduations, *e.g.*, prismatic compass. These are the whole circle bearings and these are recorded in the following manner—1° to 360°. *Note* :—The instrument must be level or else inaccuracies will result. In instruments that have liquid immersion for damping the movement of the disc, air-bubbles when formed in the liquid chamber disturb the accuracy. Compass surveys are normally not recommended in areas susceptible to magnetic influence.

Compass traverse (alternative method)

It is easier to complete the traverse by observing both the bearings of the forward line and the backward line from each station as indicated earlier. The difference between the values of the bearings of the forward and the backward stations gives the included angle at the instrument station. Thus it is possible to plot the traverse with the value of the included angles.*

The closing error in a compass traverse should not exceed 1 in 300.

DISTRIBUTION OF CLOSING ERROR IN A COMPASS TRAVERSE

If on plotting a compass traverse, a small closing error AA′ is obtained (Fig. 1.2), it may be reasonably assumed that this is due to discrepancies in both angular and linear measurements, and if the error is not large, *e.g.*, 1 : 1000, it may be distributed proportionally between the distances AB, BC, CD, DE, EA without seriously affecting the angles of the traverse individually.

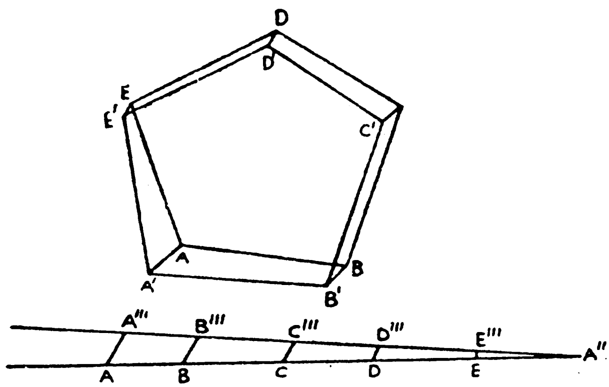


Fig. 1.2

Let AA′ (Fig. 1.2) be the closing error of the traverse which is to be distributed equitably. Draw any line AA″ and on it mark of lengths AB, BC, CD, DE, EA″, proportional or equal to AB, BC, CD, DE, EA′, respectively of the traverse. From A draw A A″ equal and parallel to AA′. Join A‴ to A‴. Similarly draw BB‴, CC‴ DD″, EE‴, parallel to AA‴ meeting A″ A‴ in B‴, C‴, D″, E‴. Draw lines parallel to AA, *i.e.* the closing error from B, C, D, and E of the traverse plan and plot lengths, BB″, CC″, DD″, EE″, respectively on these. Join A′B′C′ D′E′. The figure A′B′C′D′E′ represents the traverse after distributing the error.

See addendum

COMPASS SURVEY IN THE PRESENCE OF ROCKS CONTAINING MAGNETIC MINERALS

In the case of compass survey of an area where there is magnetic attraction *e.g.*, in the vicinity of large basic intrusions, magnetic ore bodies or in a mine where there is a large amount of iron work, like rails, pipes etc. or in an electrified part of a mine, the following procedure may be adopted :

1. The traverse is started from a point where there is no magnetic attraction, *i.e.* where the back and fore readings on a straight line are in agreement or show no deflection, the difference being 180°.

2. The survey is continued and back sight and fore sight readings are taken at each station. The following example (Fig. 1.3) will serve as a guide to computations. It will be seen that where there is a difference between the back and fore sight readings, the extent of the difference is the amount of correction to be applied. The same procedure is applied for the rest of the traverse.

Where the bearings (whole circle) have been recorded, it is possible to calculate the included angles in a traverse as can be seen from the example given below :

If f=the forward bearing at the station (2) and b=the bearing (forward) at the previous station (1) then the included angles can be computed from the formula :

$$180° - f + b \text{ or,}$$

where $f > 180°$ then, $360° - (f + b)$.

It may be pointed out that if there is a difference between the back sight and the foresight. on the same straight line, the amount of the difference, is the amount of the correction to be applied to the succeeding foresight reading got at the station from which the back sight was taken. It should also be noted whether the correction is to be applied in the clockwise direction or reverse. In the following example, the computation has been done in terms of whole circle bearings. (fig. 1.3).

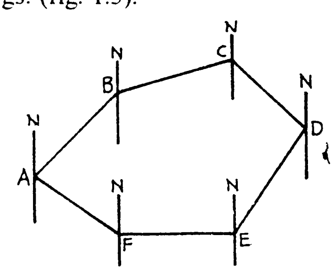


Fig. 1.3

In case whole circle bearings are used, the computations are easier, as it will be noticed from the examples given below:

Line	Observed bearing FB	Observed bearing BB	Correction	Corrected bearing FB	Corrected bearing BB
AB	15°20′	195°20′	At B = 0	15°20′	195°20′
BC	19°33′	199°33′	At C = 0	19°33′	199°33′
CD	108°09′	282°25′	At D = +5°44′	108°09′	280°09′
DE	273°11′	88°35′	At E = + 4°36′	278°55′	98°55′
EF	92°14′	276°50′	At F = + 4°36′	96°55′	276°50′
FG	350°11′	170°11′	At G = 0	354°47′	174°47′

Line	Observed bearing FB	Observed bearing BB	Correction	Corrected bearing FB	Corrected bearing BB
AB	50°	230°	At B = 0	50°	230°
BC	130°	308°	At C = + 2°	130°	310°
CD	178°	357°	At D = + 1°	180°	360°
DE	303°	122°	At E = + 1	304°	124°
EF	306°	126°	At F = 0	307°	727°

BRUNTON COMPASS

The Brunton compass is one of the most popular survey instruments for geological work. It can be used for the following purpsses :

1. As a compass or clino compass, where (a) the line of sight is horizontal or slightly depressed, (b) where the line of sight is steeply inclined or depressed.
2. As an Abney level.
3. As a prismatic compass.

1. (*a*) For measuring angles in azimuth or horizontal angles, Fig 1.3a., the body (B) of the Brunton is held horizontally while the sight vane (V) is made vertical and the lid (L) with the mirror is opened out at say 135° or 140°.

The instrument is held in this position at waist level, and the object aligned or sighted by turning round and not by moving the hands. The intersection is perfect when the centre line of the image of the slot in the sight vane (V), conicides with the black centre line of the mirror on the inside of the lid (L). Now with the observer's eye (E) overlooking the mirror, horizontality is checked up by the bulls' eye level (circular). The bearing in azimuth is obtained either as whole circle or quadrant bearing by reading the north end of the needle.

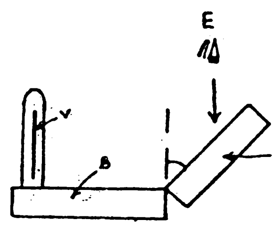


Fig. 1.3a

The above method is followed where the line of sight does not make an angle of more than 45° above the horizontal and not more than a few degress below the horizontal.

(*b*) Where the line of sight is inclined at steep angles, 15° from the horizontal, the instrument is held in the reverse position as shown in Fig. 1.3b, with the sight van (V) towards the observer and the lid (L) with the mirror away.

Observations are made by sighting the object through the sight vane (V) and the hole (h) in the lid (L) Fig. 1.3b.

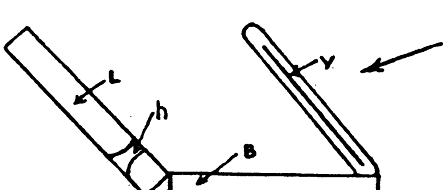


Fig. 1.3b

2. For obtaining the general angle of dip of a formation the mirror lid (L) is opened out fully and the upper flat edge of the instrument is aligned with bedding plane or contact plane of a conformable series. Then the bubble tube is brought to the centre of its run by operating the lever at the back of the instrument. The reading of the vernier gives the angle of dip.

Where it is intended to us the Brunton compass as an Abney level, to measure vertical angles, the sight vane (V) is opened out fully and made level with the body (B). The upper digit of the sight vane carrying the peep hole (P) is bent at right angles to the sight vane. The lid (L) is kept at an angle of 45° (Fig. 1.3c). The peep hole (P) on the sight vane (V) is aligned with hole (h) in the lid (L). The instrument is now held in the vertical position (edgewise) as shown in Fig. 1.3c and the object is sighed through the peep hole (P) and the hole (h). At the same time the long bubble attached to the vernier is brought to the centre of its run. The instrument is now read at the vernier to obtain the angle of elevation or depression.

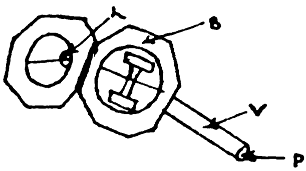


Fig. 1.3c

3. For using the Brunton compass as a prismatic compass, the instrument is opened out as shown in Fig. 1.3d. The observer sights the station through the peep holes 'P' and 'h'. The bearing is obtained from the reflection in the mirror.

The infilling of details, in a compass traverse, is done, as in chain survey, by taping off-set distances from the traverse line and recording the details in the same manner.

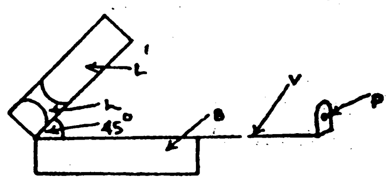


Fig. 1.3d

TUNNEL SURVEY WITH BRUNTON COMPASS

During the course of geological work it sometimes becomes necessary to survey tunnels driven in connection with engineering projects, *e.g.*, railway, electricity, irrigation, or road projects etc. In addition, surveys of inclined shafts, adits, levels, drives, crosscuts, raises and winzes etc. and other openings made in connection with mining and mineral exploratory operations are to be surveyed geologically.

Figure 1.4 gives the perspective view of an underground opening *e.g.*, a drive or tunnel.

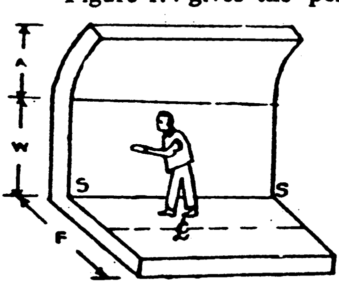


Fig 1.4

In geological surveying of tunnels, the most popular instrument used is the Brunton compass. The main stations are fixed by either theodolite traverse or by plane table traverse (using the Beaman's stadia arc). After this the geological details, such as mapping of rock types, location of structures, mineralised zones seepage zones etc. are done by means of Brunton compass.

There are two methods adopted for mapping tunnels. For purposes of normal underground geological surveying, all details are recorded at breast height. The surveyor (geologist) moves along the centre line of the tunnel holding the Brunton at breast height, and notes the bearing of the strike of all linear or planar features. The direction and amount of plunge and dip of such features are also recorded simultaneously.

Thus a planar (dyke, fault, joint etc.) feature oblique to the centre line of the tunnel will be recorded at the point where it crosses the centre line at breast height. The direction and amount of dip are also recorded.

While carrying out geological mapping of tunnels (Fig 1.4a) for engineering purposes, a modified practice is adopted. In general, the features on the floor of the tunnel F are covered largely by debris and dirt and hence all features visible on the walls W and in the arch are recorded. The mapping is carried out at spring level S. In this method, three separate plans will result, one each for the two walls below the spring line, and one for the arch.

Where a planar feature, *e.g.*, bedding plane, joint plane, foliation plane, fault plane, is encountered in the tunnel : (Fig. 1.4a).

(*a*) When the planar feature is vertical and is oriented at right angles to the traverse line, *e.g.*, ADECB then its projection on the floor of the tunnel AB and on the walls AD & BC lies in a straight line AB. (Fig. 1.4b(*a*).

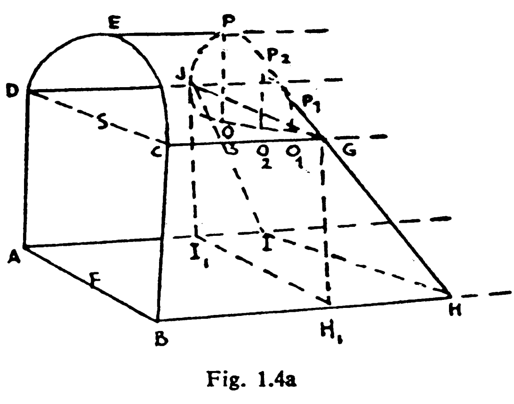


Fig. 1.4a

(*b*) When the planar feature is inclined, but the strike is normal to the traverse line, *e.g.* HPJI the projections on the walls are inclined and the angle of inclination of the trace in respect to the floor corners will be equal to the dip. The traces of the planes on either wall are symmetrical with reference to the traverse line (Fig. 1.4b) but will be off set from HI to H_1I_1.

(*c*) When the planar feature is vertical but it cuts the traverse line at an angle (*i.e.* oblique to it) then the trace on the walls bear the same angular relation to the corners and the trace line is off set. (Fig.1.4b (*b*)).

(*d*) When the planar feature is oblique to the traverse line and is at the same time dipping at an angle <90° the traces on the walls make an angle Q equal to the dip and show an off set. (Fig.1.4b(*c*))

The recording of the features on the walls is relatively simple, but the recording of certain plunging linear features on the arch or dipping planar features on the arch, requires careful judgement. Fig. 1.4*a* illustrates the procedure with respect to plotting dipping planar structure. The planar feature is HGPJI, and the portion GPJ is above the spring level. When projected to a horizontal plane at spring level the curve GO_1, O_2 O_3J will be obtained.

The following are some of the important points to be noted in connection with the geological examinations of tunnels made for mineral exploration and engineering projects, where the 3 dimensional mapping is undertaken.

(1) The dip or plunge of the planar features, is to be recorded. (2) The rake of a linear feature is to be shown. (3) In recording the features present on the tunnel walls imagine that the walls are hinged to the tunnel floor and that they open out on either side. Thus all linear traces on the walls will rotate about the hinge as axis, when the walls are opened, maintaining the same angle as the dip.

Plan of Tunnel (3—dimensional) showing the orientation of planar features in respect of floor 2-2 and spring line (1) and (1), (Fig. 1.4b).

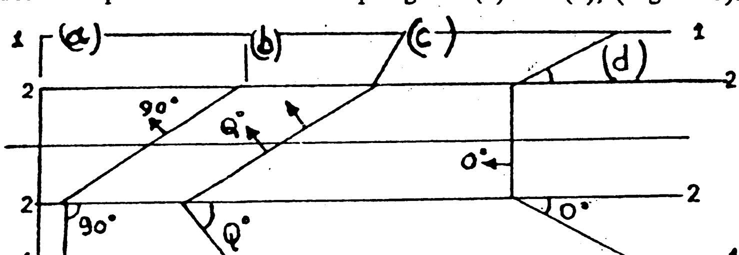


Fig. 1.4b

Figure 1.4*b* (*a*) shows a trace of a vertical linear feature, which is at right angles to the traverse line, as projected on the floor of the tunnel.

Figure 1.4*b* (*b*) shows the trace of a vertical linear feature, which is at an angle to the traverse line, as projected on the floor of the tunnel.

Figure 1.4 (*c*) Shows the trace of inclined linear feature, which is dipping at an angle Q as shown by the arrow, as projected on the floor of the tunnel.

Figure 1.4*d* shows the trace of an inclined linear feature, which is dipping at an angle O in the direction of the arrow, as projected on the floor of the tunnel.

In general, it may be stated that the procedure adopted is to record the geological features by projecting these on a plane on the floor level of the arch. It involves three stages :

(1) Projecting the features present on the roof of the arch, to a plane at ground or floor level.

(2) Projecting any feature present on the wall (say left side) at ground or floor level. This operation may be achieved by imagining the spring lines of the arch 2—2, from where the wall starts on Fig. 1.4*b* to form a hinge with the floor of the tunnel. Next, imagine the wall to open outwards, so that all linear features rotate about the hinge line, but yet remain attached to their respective points of intersection with the hinge line.

(3) Projecting of the features present on the right wall of the tunnel is done by adopting a similar procedure in respect of these, as discribed in (2) above.

SOME HINTS ON TRAVERSING

A. Ranging lines

(*a*) For measuring off-set distances the follower takes the end of the tape and holds it on the point to be off-set, while the recorder standing astride the chain stretches the tape and swings it over the chain, circumscribing a circle so as to find out the minimum reading, which gives the perpendicular, tangential or off-set distances.

(*b*) In order to range a line at right angles to the traverse line from any given point on it, any one of the following methods may be employed.

(1) The optical square or surveyor's cross.

(2) An angle measuring instrument like the theodolite or dial.

or (3) Advantage may be taken of simple geometrical relationship such as those given later.

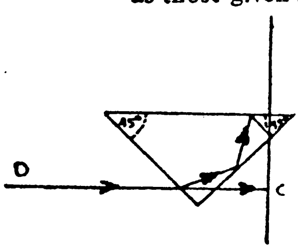


Fig. 1.5

(1) Optical square is an instrument in which a right angled prism with a mirrored side is employed to obtain reflections of objects at right angles in any one direction. Figure 1.5 shows the principle involved in the construction of the instrument.

There are two models of the surveyor's cross. One is a very elemenary device in which there are two sight varies at right angles or two

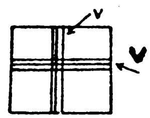


Fig. 1.6

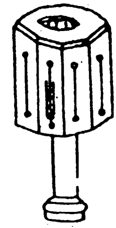


Fig. 1.7

V-shaped grooves at right angles to each other, on a flat square piece of wood (Fig. 1.6). The other type of cross consists of an octagonal hollow right prism made of brass, with pairs of sighting slits at right angles to each other (Fig. 1.7).

(2) The theodolite and dial are costly angle measuring instruments and these are not employed in preliminary work.

(3) Geometrical methods—Case (i) (Fig. 1.8). If P is the point from which the perpendicular is required, measure AP=30 ft. with the chain.

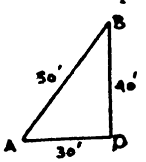


Fig. 1.8

Then take a length AB=50 ft. on the chain, or the helper takes hold of the 50th tag and a follower measures a further length of 40 ft., *i.e.*, the end of the 90th link, which is brought to rest on P. BP is perpendicular to AP at P.

Case (ii) (Fig. 1.9). When using a 100 ft. chain, measure off PA=PB=25 ft. on either side of the point P on the traverse. Then take hold of C the 50 ft. tag and rest the ends (handles) of the chain one at A and the other at B. Stretch the chain at point C. The point C lies on the prependicular to AB at P.

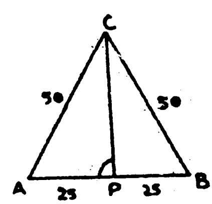


Fig. 1.9

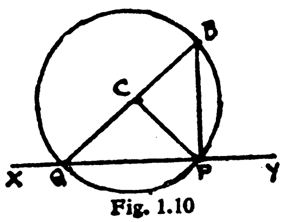


Fig. 1.10

Case (iii) (Fig. 1.10). From any point C away from the traverse line XY, draw a circle with any convenient radius CP (15 or 20 ft.) to cut the traverse at Q and P. Now draw QC to pass through the centre and meet the circle at B. BP is perpendicular to XY.

It is to be pointed out that methods (i) and (ii) may also be employed in the construction of angles of 60° or 30°.

B. Avoiding obstacles

(*a*) By ranging a line CD (Fig. 1.11) parallel to the traverse line AB.

Make AO=OD and

OB=OC so that

ABCD is a

parallelogram.

Then CD=AB.

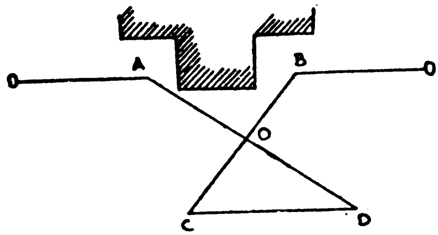


Fig. 1.11

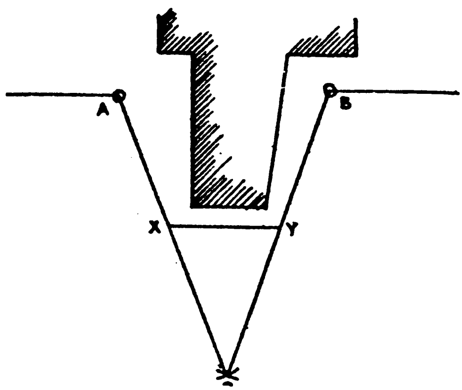


Fig. 1.12

(*b*) Construct triangle ACB to be isoceles, so that AC=CB. Make AX=BY so as to avoid the obstracle (Fig. 1.12). Further $\frac{AC}{CX}=\frac{AB}{XY}$, and since values of AC, CX and XY can be measured, the value of AB can be calculated from the above equation.

(*c*) Range line AX at right angles to OA at A by any one of the methods already explained.

similarly	range	,,	XY	,,	,,	,,	AX at X	,,
	range	,,	YB	,,	,,	,,	XY at Y	,,
and	range	,,	BP	,,	,,	,,	YB at B	,,

and continue the traverse (Fig. 1.13).

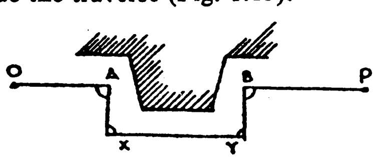


Fig. 1.13

C. Obtaining distances of inaccessible points

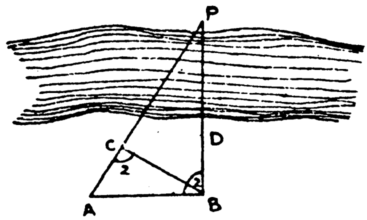


Fig. 1.14

1. Let P be the inaccessible point (Fig 1.14).

Peg lines PCA and PDB. From B draw AB perpendicular to PB, to meet PCA in A. Make CB perpendicular to PA. Since triangle ABP and triangle ABC are similar.

Therefore

$$\frac{CA}{CB} = \frac{AB}{BP}$$ (where CA, CB and AB are known and BP is got by calculation).

2. Range any line PQ. Peg any point B, and make an aligenment PBC. Range any other line parallel to PQ. (Fig. 1.15), *e.g.* by one of the methods given earlier.

Let line QB meet RC in R.

RCQP is a parallelogram

RC=PQ.

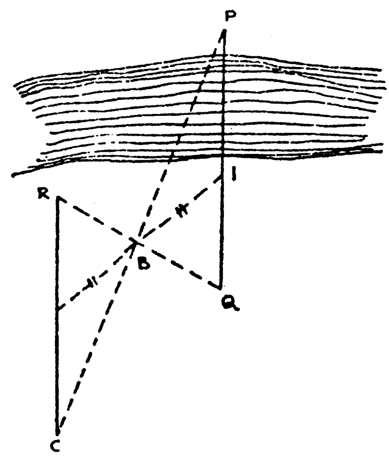


Fig. 1.15

3. The method suggested in (Fig. 1.16) may also be employed.

$$\frac{YO}{AO} = \frac{PB}{AB}$$ where YO, AO and AB are known and PB is got by calculation.

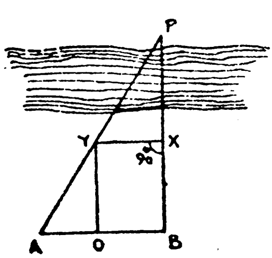


Fig. 1.16

D. Double ranging

In case where a high ground intervenes and one of the stations B is not visible from station A (Fig. 1.17) the following procedure will be useful. Two persons P and Q get to the top of the high ground, so that both can see each other. P can see station A and Q can see station B. P aligns Q with station B. This operation is continued until P and Q are in a line with A and B.

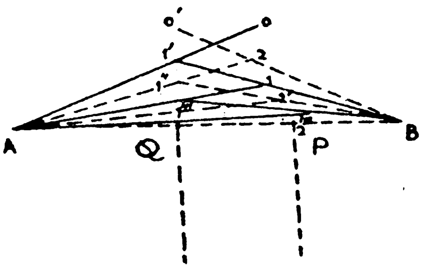


Fig. 1.17

There Q shifts successively from O′ to 1′, 1″ and 1‴ consequently P shifts from position O to 2, 1,2″ and 2‴, until both P and Q are finally in line with A and B.

PLANE TABLE SURVEY

Equipment : Plane table, tripod-stand (telescopic legs preferable), alidade, spirit level, trough conpass, drawing paper (non-shrinking), drawing pins (or adhesive tape), two or three ordinary pins, and a plumb bob, if available; chain with arrows, plotting scale.

Theory : The plane table is considered to be a point or at least of negligible dimensions, when compared to the area to be surveyed. Thus no accurate centering or placing over stations, is necessary, and in practice (scale 100′ = 1 inch) rough centering is sufficient, except where the area to be surveyed is comparable in respect of that of the plane table.

Procedure :

1. Open out the stand and adjust the legs to convenient length. Plant the legs firmly in the ground.
2. Attach the table board to the stand by means of the fly nut, but do not tighten. Fix the drawing paper, by means of drawing pins, or better with strips of adhesive tape, to the top of the board.
3. Align one edge of board parallel to any two legs of tripod by slight rotation.
4. Place bubble on the board parallel to the edge just aligned with the pair of legs in operation (3) above.
5. Move the third leg back and forth, or left and right (side-ways) *i.e.* in two directions at right angles, till the bubble comes to rest in the middle.
6. Check by placing the bubble in a position at right angle to the original. Fix this third leg also. Repeat operation (5) above if necessary.

If the stand is provided with a ball and socket head or (Johnson head) the levelling is simpler. The capstan nut for clamping the ball and socket joint is loosened and the table is levelled with the bubble tube, and then clamped. Now the fly nut at the bottom is loosened slightly and after placing the alidade on the ray for back sighting, the table can be oriented.

Note : (*a*) Accurate levelling of the board is essential so as to obtain accurate intersection of the various stations. If levelling is bad the wire in the sight vane will not coincide with a vertical rod at a station but will cut across it, and no definite intersection can be got.

(*b*) The centering of the table over the station point may be done by means of the plumb bob or by dropping a stone from beneath the plotted position of that station (on the board) on to the station point (on the ground). A discrepancy of 2″ or 3′ is allowable depending on the extent of area. This depends on the scale of the survey *i.e.* this error should not be of a magnitude which can be plotted, *e.g.* one can not plot 4″ on a scale of 1 inch=100ft. It also depends on the length of sights. In case of short sights a large error in centering will cause a variation in angle.

(c) Mark the north line after orienting the table (see procedure below).

(d) In the case of light wooden alidades, a pin planted at the station point is used as a pivot for turning the alidade and sighting the next station.

Plane table traverse : If ABCDE be the traverse (Fig. 1.18) the table is fixed at A, centered, and levelled by means of the bubble tube placed in two directions at right angles, and clamped in such a manner so as to accomodate the plan of the area, on the table when plotted to the scale required. It is a good practice to draw the north line by means of the trough compass (see adjustments above).

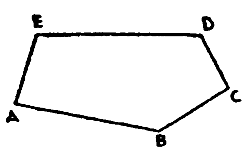


Fig. 1.18

Now station B is sighted with the alidade, and the ray drawn from A to B by using a fairly hard pencil. The distance AB is measured and the position of B plotted on this ray to scale. The table is now removed and set up at station B. After the necessary adjustments, explained earlier (i.e. centering, levelling etc.), A is sighted back along BA. This is done by loosening the fly nut at the bottom of the table, so as to release the board and by placing the alidade along BA, so that the peep-hole is on the side of B. Looking through the alidade the station A is sighted back, and the board is tightened up in the position.

Thus the table is oriented. Station C is now sighted and ray BC drawn and the distance BC measured and plotted to scale. The traverse is similarly continued onwards, to station C, D, and E.

Location of Points by Intersection and Resection

Triangle of error

(*a*) The location of points on the traverse is accomplished by means of intersecting rays generally two or sometimes three drawn from two or three stations respectively depending on the importance. This is known as intersection. If the table orientation has been perfect the intersection of the rays is at the same point. Very often the third ray when drawn gives rise to a small triangle and the object will have to be located in respect of this "triangle of error".

(*b*) Quite accurate location can be obtained by taking a ray from a station to the object measuring the distance and then plotting the same. This location is then checked up by a similar measurement from another station. This procedure is however likely to be more time consuming.

Sometimes, it becomes necessary to locate one's position on the plan, during the course of a plane-table survey. This can be accomplished only when three stations are visible. This method of location is called "resection". The plane table is set up at the desired point, and by inspection a point is selected on the plan so as to correspond as closely as possible to the instrument station. Three rays are then drawn passing through the three visible stations. Most often the three rays will not meet at a point, and a triangle known as, "the triangle of error" will be formed (Fig. 1.19). The following rules are helpful in fixing the position of the new station. (*a*) If the new station point lies on or near the great circle, passing through the three stations in the field, its position becomes indeterminate. (*b*) If the new station lies within the triangle formed by the field stations, the strength of determination increases

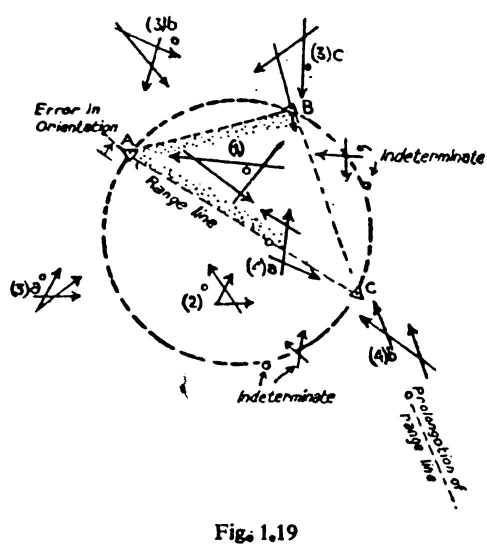


Fig. 1.19

as its position approaches the centre of gravity of the great triangle *i.e.*, formed by the field stations. (*c*) If the new station lies outside the great circle, its strength of determination will be weak (Fig. 1.19).

The following rules are handy in solving the triangle or error.

1. When the new station is within the great triangle formed by the field stations, then its location on the table is within the triangle of error, and its approximate position will be represented by a point whose perpendicular distances from the three sides are proportionate to the respective distances of the three stations sighted, from the new location.

2. When the new station is outside the triangle, then its position in the plan will be outside the triangle of error, but then its location on the plan will be, to the same side of the ray to the most distant station, as well as to that of the rays to the other two stations, *e.g.*, if the point is to the left of the ray to the most distant station, then it will also be found to the left of the other two rays.

3. When the new station lies within any of the three segments of the great circle, passing through the three stations, then its location would be as far distant from the median through one of the points (of the great triangle) as the point of intersection of the other two rays.

Note: A plane table in combination with a dumpy level is very useful for contouring in a moderately hilly terrain. In a rugged country a plane table with telescopic alidade equipped with Beaman stadia arc is to be preferred (for description of Beaman stadia arc see under stadia survey and tacheometric levelling).

A plane table can also be used in triangulation. The method is given under Triangulation.

Resection by transfer

Resection is sometimes facilitated when (*a*) the angles between the rays drawn are not too acute or small, and (*b*) when the possible location is not too near the great circle.

In this method the plane table is set up and levelled. A piece of tracing paper is fixed to the board by adhesive tape, at any convenient place. On the tracing paper a point is marked. With this point as the origin, rays are drawn to at least three triangulation stations, terminating at the edge of the tracing paper, and properly indexed.

The tracing paper is now detached from the board placed over the place (station) and moved in such a manner that the rays drawn on it are in perfect alignment with the corresponding points alreay plotted cn the place. When this is achieved the point of origin (on the tracing paper) is pricked through with a fine needle and transferred to the plan below.

The point thus obtained is checked up by orienting the table in the usual manner *i.e.*, (*a*) by loosening the fly nut, (*b*) aligning a ray and back sighting, (3) locking the fly nut, and (4) checking up correspondence between other stations.

THEODOLITE AND MINER'S DIAL SURVEYS

Permanent adjustment : It is taken that the instrument is in perfect adjustment and hence this has been dealt with in the appendix.

1. Open the stand and spread the legs of the tripod keeping the head céntrally over the station point. Further centring over the station may be done by dropping a small stone through the centre of the opening at the top of the head. See that the head is level and remove any shake or "headache" by tightening the fly nuts, and also see that shoes are firmly planted. Fix the cap to the leg or place it in the box.

2. Note the lie of the instrument in the box.
   (*a*) See which is the tribrach end and which is the objective end.
   (*b*) Note the position of the vernier clamps and the vertical circle.

3. Hold the uprights (supporting Y) firmly with the left hand, take hold of the tribrach with the right hand and lift the instrument gently out of the box.

4. Place the instrument on the stand, so that the opening in the tribraoh fits on the screw in the head of the stand. Rotate the tribrach in an anticlockwise direction, until a slight click indicates that the screws are engaged. Now tighten up in the clockwise direction.

5. Remove plumb bob from box and suspend it to the hook below central spindle of the instrument.

6. Bring the three foot screws to the centre of the run.

7. Roughly centre the instrument over the station by manipulating the sliding mechanism in the tribrach.

8. Clamp the tribrach by tightening the large fly nut at the bottom.

9. Bring the bubble on the horizontal plate ovèr any two foot screws and level by rotating both the foot screws inwards or outwards to an equal amount, simultaneously.

10. Turn the plate through 90°, so that the bubble is now over the third food screw, now operate the single foot screw and bring the bubble to the centre.

Repeat operations 9 and 10 until the bubble remains in the centre in all positions of the plate.

11. Remove any parallax by focussing the cross hairs of the telescope. Also check up by moving the eye from side to side.

Further manipulations for normal transits : Note on which side the vertical circle lies, *i.e.*, to the right or left of the line of sight.

(*a*) Clamp the body (*i.e.* lower clamp). (*b*) Loosen the vernier clamp. (*c*) Looking into the vernier 'A' rotate the plate and bring the vernier to read as near zero as possible. (*d*) Clamp vernier and then with the help of the vernier tangent screw, make the reading exactly zero. (*e*) Loosen body clamp and loosen telescope vertical arc clamp and sight the back station, so that the horizontal plate yet reads zero, when sighting the back station. *(f)* Clamp body, and intersect back station accurately by means of body tagent. (*g*) Release vernier clamp and rotate the telescope in a clockwise direction and sight the forward station. Clamp vernier and make an accurate intersection with the help of the vernier tangent. Read horizontal angle recorded by the vernier.

Note : (*a*) Always intersect the lowest point of the flag placed at stations, as the upper portions are liable to deflections.

(*b*) While recording vertical angles, always make a note whether it is positive or negative.

In the case of the modern "microptic" instruments the reading of both the horizontal and vertical circles are projected on to an auxiliary telescope and can be read simultaneously.

(Specimen book—see appendix).

VERNIERS

There are two types of verniers. (*a*) The verniers in which the smallest vernier division or subsidiary scale division is less than the smallest main scale divison, are known as forward reading verniers. (*b*) Those verniers in which the least vernier scale division or subsidiary scale division is greater than the main scale division, are called the retrograde or backward reading verniers. All modern instruments, however, employ the forward reading vernier so that the vernier is read in the same direction as that of the main scale. When handling any instrument, always determine the least count of the vernier or the value of the smallest division of the vernier.

In verniers for angle measuring instruments, the least count is calculated in the following manner. Suppose a degree of the main scale is divided into three equal parts or 20 minute divisions and if 59 of these are taken and again divided into 60 parts, the least count of the vernier will be 1/60×20 min. or 20 seconds. The graduations on the verniers are marked minutewise, *i.e.*, 0 to 20 minutes. Now to read the vernier the following example may be cited.

Suppose in a certain reading of the instrument the arrow indicating the zero is located between 152° to 153°. Suppose the arrow is beyond the 2nd small main scale division from 152°. This shows that the value of the angle is greater than 152° 40′. If now it is found that the 1st small vernier division after the graduation marked "15" on the vernier, (*i.e.* 16th division) coincides with a main scale division, then the value of the angle is read as 152° 55′ 20″, *i.e.* , 152° on the main scale, plus 40′ on the main scale plus 15′ 20″ on the vernier scale.

Definition of terms used : In a theodolite, when the vertical circle is to the left hand side of the operator or the line of sight, the position is called *face left*, when it is on the right hand side, the position is known as *face right*. *Transisting* or *changing* face consists in rotating the telescope through a vertical angle of 180° and then rotating the horizontal plate through 180° so that the telescope points in the same direction as before. For accurate work, repeat measurements of all angles made, first on face left and then on face right or vice versa. The readings so obtained should be recorded in a suitable tabular form. For further computation and calculation (see appendix 3).

The microptic tacheometers : are instruments similar to (Watts type) theodolite. These instruments differ from normal theodolites in that they have an arrangement for measuring distances.

The instrument is kept either in a box or sometimes in an all-steel shell. In the later case, the clamps holding down the shell to the base are loosened and the shell removed. Next the holding down clamps fixing the instrument to the base plate are loosened and the instrument set free.

The stand can be levelled with its own bull's eye bubble after placing it over the station point. The instruments are attached by means of the hollow nut, found in the head of the tripod. In some instruments it is possible to do preliminary centring by means of a prism arrangement attached to the tribrach. With the help of this device it is possible to see the ground stations and thus do the centring. After this the plump bob is attached to the other end of the hollow nut in the head of the tripod.

The other temporary adjustments are similar to those for normal tacheometers and theodolites.

The small circular plane mirror at the side is for reflecting light for illuminating the various scales, viz. vertical and horizontal. These scales in the case of these microptic instruments, are totally enclosed and are not visible. The images of these two scales are projected for convenient viewing by means of reflecting prisms, and can be seen in the auxiliary telescope placed on the main telescope. The image of the micrometer vernier scale can also be seen in the auxiliary telescope. This vernier is of the forward reading type and is common to both the horizontal and vertical scales (and designated H & V respectively) which are graduated in 20 minutes. The vernier is graduated in the same manner as in normal transits or theodolites and has a least count of 20 seconds; but it is used in a slightly modified fashion.

The vernier is operated by rotating the knurled nut on the right hand side of which the word "micrometer" is engraved. It will also be noticed that movements of the vernier from 1 to 20, after clamping the horizontal and vertical plates, produces a translation of 20 minutes on other circles. This shows that the least count of the instrument is 20 seconds. It is a good practice to set the vernier at zero at the start of each setting of the theodolite.

In replacing the instrument, the telescope is clamped in the vertical position and the base plate is placed in a position to fit on to the clamps provided.

In the case of the microptic theodolite (Watt's type) the least division (least count) of the main scale is 20′, and that of the vernier is 20″. There is a micrometer vernier which is common to both the circles, i. e. horizontal and vertical. After completing the temporary adjustments the micrometer vernier is made to read 'zero' by rotating the micrometer milled nut on the upright. After intersecting the station (either in azimuth or in the vertical), the auxiliary telescope readings are noted correct to the nearest 20′ on the lower side, e. g., if while reading the horizontal scale the index point lies between 189° 20′ and 189° 40′, it is moved towards 189° 20′. Now the micromer is rotated so that the micrometer vernier reading which was initially at zero now increases, whereas the index point moves towards 189° 20′, where further movement is stopped. If now the micrometer vernier reads 12′ 20″, this is added to the initial reading of the horizontal scale that is 189° 20′ + 12′ 20″ = 189° 32′ 20″. The same manipulations are carried out with respect to the vertical scale readings also.

FIXING OF STATIONS AND TRAVERSING

(*a*) Traverse stations for surface surveys are usually indicated by a small cement posts one foot long and eight inches square which are planted flush with the ground level. A cross made on the top (near the centre) of the exposed portion denotes the actual station point. Sometimes when permanent marks are not required it is convenient to drive a ½ inch pipe or peg to mark stations. The station flag is placed on the station point. When not in use a cairn or a stone heap is built up over the station points.

(*b*) In underground surveys, as stations are likely to be disturbed when placed along roadways, the station pegs are nailed on to the roof. A plumb bob is suspended from a hook or eye in the peg and the string or line is illuminated from behind and sighted though the telescope (of the theodolite etc).

Methods used in traverse surveys

If bearings are obtained with the Miner's dial the readings can be recorded as :

1. Whole circle bearings calculated with reference to the north. The values are recorded as 35°, 197°, 352°.
2. Quadrant bearings are calculated either with reference to the north or south and further indicated by east or west sector e. g. the values are recorded as N 20°E, S 30°W etc.

In the case of both theodolite and dial traverses, all angles are measured by the following methods :

1. Back and fore sight.
2. Double fore sight.
3. Continuous bearing (not recommended at the earlier stages).

1. Back and fore sight

The instrument is set up at stations 1, 2, 3, 4 etc. and the angular measurments made in the manner indicated by the arrows (Fig. 1.20) presuming the instrument to be right handed. The angles are read on both verniers, and recorded separately. In the Fig. 1.20, X is the eyepiece end and O is the objective end. The bearing. of any one of the lines is also measured.

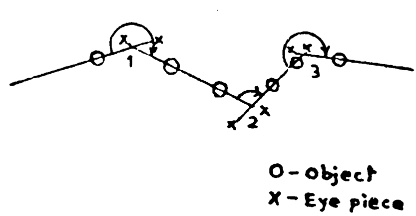


Fig. 1.20

The azimuth of all the lines is calculated with reference to that of any one line, by adopting the following procedure.

Add the measured angle to the measured or assumed azimuth of the preceding line, if the sum exceeds 360° subtract 360° and this will not affect the azimuth.

Example

Vernier Readings	measured or Assumed Azimuth
187° 15′	187° 15′
98° 20′	285° 35′
256° 09′	181° 44′
160° 12′	341° 56′

2. Double foresight method (This is the method commonly adopted).

This instrument is set up at station, 1, 2, 3 etc. and angular measurements taken as indicated by arrows, assuming the instrument to be right handed (Fig. 1.21).

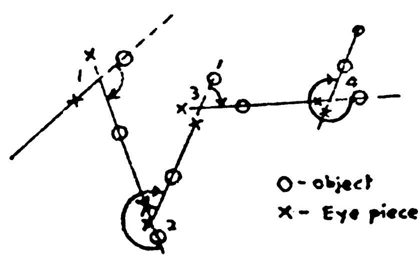


Fig. 1.21

Calculations of azimuths when the bearing of any line of the traverse is known, is done as follows :

Add the value of the forward bearing of the *preceding line* to the measured value of the included angle, deduct 180° if the sum is more than 180°. If on subtraction the balance is more than 360° further subtract 360°. When the sum is less than 180° add 180°.

Note : Fore sights are taken by using the plate or vernier clamp and tangent screw, and back sights are taken by handling the body clamp, and tangent. In the case of dial, looking in the S-N direction is fore sight, and N-S direction is back sight.

Station			Horizontal angle	Calculated bearings	Remarks
Back	On	Fore			
A	B	C	127° 14′	92° 00′	Bearing of AB=92° as measured in the field.
B	C	D	220° 27′	39° 14′	
C	D	E	92° 34′	79° 41′	
D	E	F	98° 12′	352° 15′	
E	F	G	130° 46′	270° 27′	
F	G	H	215° 09′	221° 13′	
G	H	A	115° 17′	256° 22′	
H	A	B	80° 21′	191° 39′	

Total 1080° = (180 × 8) − 360

The following is an example showing the method of calculation:

Method of Calculating Bearings

Bearing of HA	92° 00′
∠B	127° 14′
	219° 14′
	180° 00′
Bearing of AB	39° 14′
∠C	55° 27′
Bearing of BC	94° 41′
∠D	77° 34′
	172° 15′
	180° 00′
Bearing of CD	352° 15′
∠E	108° 12′
	450° 27′
	180° 00′
Bearing of DE	270° 27′
∠F	39° 46′
	310° 13′
	180° 00′
Bearing of EF	121° 13′
∠G	315° 09′
	436° 22′
	180° 00′
Bearing of FG	256° 22′
∠H	115° 17′
	371° 39′
	180° 00′
Bearing of GH	191° 39′
∠A	80° 21′
	272° 00′
	−180° 00′
Bearing of HA	92° 00′

3. Continuous bearing method

In this method (Fig. 1.22) the instrument is set up at sation A. The vernier is brought to read 0°00′ 00″. The trough compass is attached to

Y support of the telescope by the studs provided. The body clamp is released and the instrument rotated until the centre point marked on the compass coincides with the vertical mark at the end of the trough. Now the telescope is pointing north.

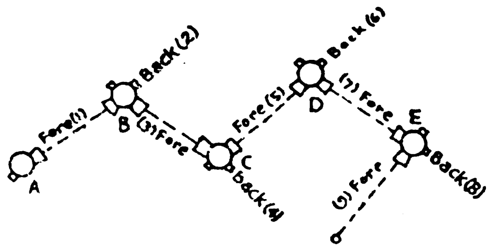


Fig. 1.22

The body is clamped, then the vernier clamp is released and the instrument is turned in a clockwise direction to sight the fore station B. The station is accurately intersected by vernier tangent. The instrument now records the bearing of AB.

Next the instrument is carried carefully and set up at station B. The reading on the vernier is the value of the fore bearing of AB. Body clamp is released and station A is intersected by using body clamp and tangent The telescope is now transitted, plate clamp released and station C is intersected using the plate tangent screw. The instrument now records the bearing of BC. Thus, when similar operations are repeated at each station, the forward bearings are automatically recorded by the instrument. This method is. however, not recommended as it requires additional care. Any accidental shifting of the reading of the instrument at any stage especially when transporting the instrument from station to station will effect the readings recorded. .

STADIA OR TACHEOMETER SURVEY

A tacheometer is a distance measuring telescope in which besides the normal vertical and horizontal wires, there are two extra horizontal cross wires (generally short) on the vertical cross wire, one of which is placed above and the other symmetrically below with respect to the horizontal wire. These short cross wires are known as the stadia wires.

Principles and theory

In this method no actual distances or heights are measured but calculated from readings obtained from the tacheometer and graduated staff. Hence this method is handy when a survey of moderate accuracy (say 100 ft. = 1 inch) 1 : 1200 is required in a rugged country. The principle involved is illustrated by Figure 1.21 where A is a pin hole and no optical device is employed.

This is the basic principle of the subtence bar commonly used by surveyor's for obtaining the distances to inaccessible objects.

In triangle XAY (Fig. 1.23)

XY$=i$, AZ$=a$

angle XAY$=\theta$

$$\text{Tan}\frac{\theta}{2}=\frac{XZ}{AZ}=\frac{i}{2}\times\frac{1}{a}$$

$$=\frac{i}{2a} \qquad ...(1)$$

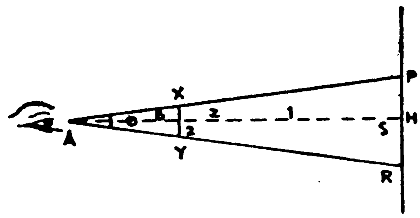


Fig. 1.23

In triangle PAR

PR$=h$, angle PAR$=\theta$

$$\tan\frac{\theta}{2}=\frac{PB}{AS}=\frac{h}{2}\times\frac{1}{AS}, \quad 2AS=\frac{h}{\tan\theta},$$

$$AS=\frac{h}{2\tan\frac{\theta}{2}}.$$

Substituting the value of $\tan\frac{\theta}{2}$ from (1)

we have.

$$AS=\frac{h}{2}\times\frac{2a}{i}=\frac{ha}{i} \qquad ...(2)$$

θ is known as anallatic angle, h is known as generating number, $\frac{a}{i}$ is stadia coefficient $=k$ (constant).

Case (i)= When a telescope is used instead of a pin hole, but the sights are horizontal (Fig. 1.24). As fh/i $AS/h=f/i$ where $f=$ focal length of the telescope, K is known, as the stadia coefficient Fig. 1.24.

f, focal length by construction;

C, is known as the instrument constant.

Since distances are large when compared to the focal length

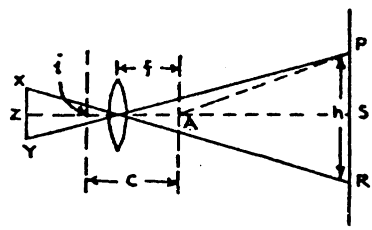


Fig. 1.24

$$\frac{AS}{h}=\frac{f}{i}$$

$$AS=\frac{hf}{i}$$

where $\frac{f}{i}=$K or stadia coefficient.

$$AS=Kh \qquad ...(3)$$

But the horizontal distance

$$=C+AS$$

$$=C+Kh \qquad ...(4)$$

($Kh+C$) is known as the distance function, K stadia coefficient, and C instrument constant and h the intercept or generating number. Thus the distance MC can be determined. In most instruments the distance, between the stadia ring and the object, is made equal to 0.5 f and hence $C=1.5\,f$. Kh or stadia constant is generally made equal to 100. In case, the values of the constants are not given by the makers, these can be determined in the field in the manner described later.

Case (*ii*). When a telescope is used and the observations are made with the telescope in the inclined (Fig. 1.25) position.

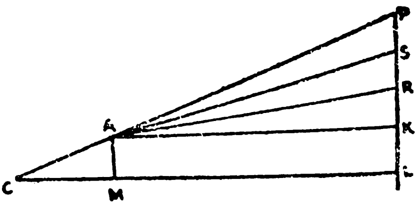


Fig. 1.25

M=Centre of instrument,

CA=C or instrument constant $\angle PAK=\alpha$ or angle of inclination

PR=h or generating number.

$\angle PAR=2\,\theta$ or anallatical angle, $\angle SAR=\theta$

(Horizontal distance) $CL=CM+ML=C\ \text{Cos}\alpha+Kh\ \text{Cos}^2\alpha$......(5)

or $=(C+hK)\ \text{Cos}^2\ \alpha$, since C Cos α is approximately=C..............(6)

(vertical height) $SL=KL+KS=C\ \text{Sin}\ \alpha+hK\ \text{Sin}\alpha\ \text{Cos}\ \alpha$.......(7)

or $=(C+hK)\ \text{Cos}\alpha\times\text{Sin}\alpha$, since C Sin$\alpha$ in approx.=C ....(8)

$C+hK$=distance function.

For horizontal distances equation (6) may be taken as $(C+kh)$ $\text{Cos}^2\alpha$. The value of $\text{Cos}^2\alpha$ is obtained from tables. For obtaining heights or elevations, equation (8) is taken as $(C+Kh)$ Sinα Cosα. The value of (Sinα Cosα) is obtained from tables. The quantity $(C+Kh)$ is known as distance function.

The value of $C+Kh$ is calculated in the manner described later and the values of $\text{Cos}^2\alpha$ and (Sinα Cosα) are got either from the tables which have already been computed for various values of α, called stadia or tacheometer tables or from graphs. These may also be obtained by means of a special slide rule. The value can be easily got from the scientific calculator.

Use of stadia or tacheometric tables

(Tacheometric tables by Louis & Caunt have been used in the following calculations.)

Example : If the distance function is 658.5 and the vertical angle is 15° 10′, then refer to the tables at 15° or 135° down the vertical column at 10′, and horizontally for partial "distances", and "heights" indicated at Dt. at Ht. respectively in the table. The partial distances are given in units from 1 to 10. The corresponding figures will have to be multiplied by 1000, 100, or 10 etc. if the distance function contains units of these orders, e. g., the value for partial distance of 6 is 5·589 hence for 600, it is 558·9. The others are similarly computed.

Distance function	Dt.	Ht.
600	558.9	151.5
50	37.3	10.1
8	7.5	2.0
0.5	0.5	.1
658.5	604.2	163.7

Distance is 604.2 and the corresponding vertical height in the height column is 163.7.

Thus in the above example, if 100 is the reduced level of the instrument station, and the height of instrument is 4.9 ft. and the reading of the middle cross-wire is 6.4 ft. on the staff intercepted then, the height of the staff station will be $100+4.9+163.7=262.2$ ft. (For derivation of the formula see tacheometric levelling). If the angle of sight exceeds 30° even then the tacheometric table can be employed in a modified form. If D be the distance function, and the angle of inclination the horizontal distance is equal to D $\sin(45°-\alpha)\cos(45°-\alpha)+D/2$ and the corresponding vertical height is equal to D $\cos^2(45°-\alpha)-D/2$. Therefore the *horizontal distance* is obtained by referring to the stadia tables for $(45°-\alpha)$ and obtaining the height given there and adding D/2 to it. The corresponding *vertical height* is got by obtaining the horizontal distance given in the table for $(45°-\alpha)$ and subtracting D/2 from it.

Example : If the distance function (D) be 420 ft. and the angle of inclination 38° 20′ the table is turned to $(45°-38°\ 20')=6°\ 40'$, and the corresponding height and distance is noted as 48.4 and 414.3. Hence the required values are :

Horizontal distance$=48.4+420/2=48.4+210=258.4$ ft. and
Vertical height$=414.3-420/2=414.3-210=204.3$ ft.

This rule in general applies to angles not exceeding 45°. For angles exceeding 45°, the angle corresponding to $(\alpha-45°)$ is referred to in the tables, and the required horizontal distance is obtained by subtracting from D/2, the value obtained from the table for height. The vertical height is obtained by subtracting D/2 from the value of the horizontal distance as given in the table.

The introduction of electronic calculators has not only reduced the time factor involved in the computations but it has provided an additional advantage, in that the calculations required in connection with techeometric/stadia surveys can to a large extent be carried out in the field. Further, one need not buy the telescopic alidade equipped with the Beaman Stadia arc, but for plotting the distances and heights, in the field, an ordinary telescopic alidade with the vertical circle graduated in degrees and fractions of a degree together with a 'scientific' calculator can do the job just as efficiently as its Beamen arc counterpart. With this calculator the values of $\cos^2$ can be found out as well as the value of the tan of the angle. In case, an ordinary calculator is used, then the values of the cos can be xeroxed as well as the values of the tan from a book of mathematical tables and placed in a transparent plastic envelope, back to back for reference in the field. It may be mentioned that the telescopic alidade should have the stadia lines engraved together with the cross hairs. The values of $\cos^2\theta$.*

Determination of instrument constant and the stadia co-efficient

The instrument is racked up to the solar focus (i. e. focus at infinity) and the distances (1) between the diaphragm and the centre of the objective (2) between the objective and the centre of the theodolite, are measured with a steel scale and the sum of these gives the instrument constant i. e. (1) plus (2).

In order to determine the stadia coefficient K the theodolite is set up at a station. From the station point a length equal to the instrument constant is measured out. From this latter point, distances equal to 50, 100, 150, 200 ft. and so on are measured and pegged out. Keeping the instrument horizontal, stadia intercepts are booked.

$$K=\frac{\text{distance}}{\text{intercept}}=\frac{50}{h}=\frac{100}{h}=\frac{150}{h} \text{ etc. } h=\text{intercept value.}$$

Thus the value of K is got.

Tabular columns for the recording of field reading and office work are shown in Appendix 4 and *4a*.

Self reducing tacheometers

In these instruments instead of the normal stadia lines, certain curves etched on a glass plate are introduced into the field, in the focal plane of the eyepiece. In the Zeiss instrument for example; the lowest horizontal curve is designated the base curve (1) and it has a companion curve (2) running left to right. The intercept on the staff between curves (1) and (2) gives the factor for horizontal distance. Similarly there are other curves designated −10 or −20 etc. (3) The intercept on the staff between the base curve (1) and curves (3) give the reduction factor for the vertical height.

See addendum.

For obtaining the horizontal distance the staff intercept between curves (1) and (2) is multiplied by 100, and in the case of vertical height the staff intercept between (1) and (3) is multiplied by the factor inscribed on the curve used, e.g., $\pm$ 10 or $\pm$ 20.

Example : If the intercept between curves (1) and (2) is 573 m.m. the horizontal distance $=0.573\times 100=57.3$ metres.

If the intercept between curves (1) and (3) is 652 m.m. and the number inscribed on the curve is 10, then the vertical height is $0.652\times 10=6.25$ metres.

The formula used in working out the distance curves are given below :

$$a=\frac{F\cos^2\alpha}{K\pm\frac{1}{2}\sin^2\alpha}$$

where a is intercept between the distance curve and base.

$$h=\frac{\frac{1}{2}\,F\sin^2\alpha}{K\pm\sin^2\alpha}$$

where h is the intercept between the height curve and the base.

f= equivalent focal length of the system

α = the angle of elevation or depression

K = instrument constant.*

Telescopic alidade with beaman stadia arc

This instrument also utilises the same primary principle as the tacheometer, *i.e.*, measuring distances by stadia intercepts. The telescopic alidade is very convenient when used in conjunction with the plane table for topographic surveying and geological mapping. The telescope is mounted on a scale rule which enables the proper plotting of direction of rod shot stations. The theory and the method of computation of stadia measurements have already been discussed in connection with the teacheometer. The same principles apply here also.

The Beaman stadia arc is a mechanical accessory to the alidade, which obviates the reading of vertical angles and eliminates laborious calculations of stadia and instrumental data, as well as any reference to tacheometric tables.

The accuracy of the method compares favourably with that of the tacheometric survey and for normal topographic-geological survey of mines and mineral deposits on moderately large scale, say upto 100 feet=1 inch, considerable accuracy can be achieved, specially in areas where the topography is moderately rugged.

Use of telescopic alidade with beaman stadia arc for levelling

The telescopic alidade, with Beaman stadia arc in combination with

See addendum.

the plane table, is an ideal instrument for topographic contouring. It has an advantage over the tacheometer, as intermediate staff stations can be straightaway plotted on the plane-table sheet, with their reduced levels and the contour lines interpolated and sketched, in the field. The difference in elevation, between the plane-table station and rod station, is added to or subtracted from the reduced level of the plane-table station, depending on whether the shot is a positive (up-slope) or negative (down-slope) one, and the reduced level of the staff station is calculated, as shown in Figs. 1.32 and 1.33.

The Beaman arc is a specially graduated vertical arc attached to the horizontal axis of the telescope. In addition to the scale of degrees in the vertical arc, there are two scales of Beaman factors, one marked "Hor" or "H" scale and the other "Vert" or "V" scale. The principle involved in the construction of these scales is the same as in teacheometer and shown in Figs. 1.26 and 1.27.

The horizontal distance $H=(C+Kh)\ Cos^2\alpha$

The vertical height $V=(C+Kh)\ Sin\alpha\ Cos\alpha$

As indicated earlier the instrument constant C is very small and in ordinary geological and topographical surveys, it need not be taken into account in the solution of the formulae. The formulae are thus simplified to

H or Horizontal distance $=C+hK\ Cos^2\alpha$ is modified to

$H=Kh\ Cos^2\alpha$

V or Vertical height $=C+hK\ Cos\alpha\ Sin\alpha$ is modified to

$V=Kh\ Sin\alpha\ Cos\alpha$, where h is the staff intercept and K is the stadia coefficient.

In most telescopic alidades, the telescope is so constructed that the stadia coefficient K is 100.

Thus $H=\text{Rod intercept}\times 100\times Cos^2\alpha$

$V=\text{Rod intercept}\times 100\times Sin\ \alpha\ Cos\alpha$

In some types of Beaman arc the values 100 ($cos^2\alpha$) and 100 ($Sin\alpha\ Cos\alpha$) are directiy obtained from the scales as "Hor" factor and "Ver" factor respectively. These are described later as 'B' type arcs. In other types, the values are obtained by very simple arithmetical manipulation of the "Vert" and "Hor" scale readings. These are described as 'A' type arcs.

Adjustment

(1) With the table levelled, and oriented with reference to the back station, or with reference to some triangu'ation station, clamp the tele-

scope and level it. Measure the height of the instrument over the station point.

(2) Set the Beaman arc with the scales exactly at the zero point (either 0 or 54 as the case may be), by means of the tangent screw. In the American type alidades, the right tangent screw or gradimeter screw is used for this adjustment, and this screw is not disturbed during further operations.

(3) Sight the telescope towards stadia rod, taking care to see that the edge of the alidade rule does not move from the station point on the table. For accurate work, it is better to prick holes at the station position and slide alidade carefully, without the aid of a fixed pin, on the station point, as is done in the case of the open-sight alidade method.

(4) Raise or lower the telescope, by the clamp and tangent screws, until the vertical cross-hair falls on the staff. Fix either the top or bottom stadia wire on a whole number of the staff and quickly read all the three intercepts.

(5) Level the Beaman arc, by observing the bubble attached to it, and by operating its tangent screw; and note the "Hor" and "Vert" scale readings.

(6) Calculate the horizontal distance, difference in elevation and reduced level by following the methods given earlier.

The instrument readings and calculations are recorded as shown in Appendix (5 & 5a).

(7) Without shifting the instrument, draw the rays to staff shot stations. The horizontal distance, to a particular station is plotted and the reduced level noted against the points.

(8) When a number of staff stations are plotted, the contour lines may be interpolated and sketched in accurately, by observing the slope of the ground, and topographic details in the field.

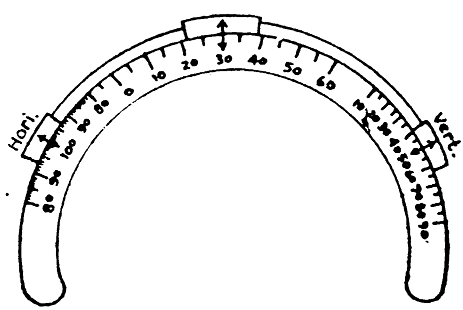


Fig. 1.26

'A' type arc : The horizontal scale divisions are proportional to $100\,(1-\text{Cos}^2\alpha)$ and the vertical scale divisions are proportional to $100\,(\text{Sin}\,\alpha\ \text{Cos}\,\alpha)$. The scales are so constructed that when the index mark of the "Hor" scale coincides with 0 or 100, the index mark of the "Vert" scale coincides with 50.

(1) In some American instruments the divisions on the "Hor" scale either read 90 on either side with 100 in the middle or 10 on either side with 0 in the middle. The "Vert" scale reads from 10 to 90 with 50 in the middle (Fig. 1.26).

or

(2) In English (Watts microptic alidade) instruments, the divisions on the "Hor" scale read from 0 to 20 and the "Vert" scale 0 to 40 (Fig. 1.27). The readings on each scale have the sign prefixed, viz. +ive or—ive. In this instrument the indices of both scales read 0 simultaneously.

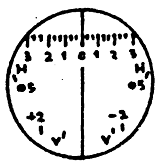


Fig. 1.27

'B' type arc : Although the "Hor" and "Vert" scales move as one unit on the same arc, they are in separate quadrants—one on the left and the other on the right. The zero points of these scales are 100 and 50 respectively. The divisions (in some American make alidades used by the U.S.G.S.) are 80 to 100 on the "Hor" scale with 80 in the middle and 10 to 90 on the "Vert" scale as in the case of the 'A' type arcs.

Calculation of distances and heights (tacheometric levelling)

1. **'A' type arc.**

(*a*) Horizontal distances
$=$staff intercept $(h)\times 100$
$-$(staff intercept $h\times$horizontal scale reading H)
$=h\,(100-\text{H})$.

Since the H or horizontal scale reading is deducted from 100 this type of scale is called the percentage deduction scale.

(*b*) Vertical height (+ or —)
$=$staff intercept (h) $\times$ vertical scale reading (V) $=h\times\text{V}$.

In case, the Horizontal scale (H) reads 90 on either side with 100 in the middle, the computation is done thus, $(h\times 100)\,\dfrac{\text{H}}{100}$.

Thus this type of scale is also the percentage scale, as the Horizonal scale reading H gives the percentage directly.

Note. In the American type alidades, the vertical scale reading is obtained by noting the reading of the scale index-50 (algebraic). In Watts' microptic alidades, this is directly read on the arc. The "Vert" reading may be plus or minus, depending on whether the staff reading is up-slope or down-slope.

2. **'B' type arc.**
   (*a*) Horizontal distance = staff intercept (h) $\times$ Horizontal scale reading (H) $= h \times$ H.
   (*b*) Vertical height (+or—) = staff intercept (h) $\times$ Vertical scale reading (V) $= h \times$ V.

'A' type Arc (Watts' Microptic alidade)

(*a*) For obtaining reduced level :

(1) Looking through the telescope intersect the staff at any convenient point and bring the bubble to the centre of its run.

(2) Move the telescope up or down so that the middle cross hair reading coincides with a whole number on the "V" scale or Vertical scale either + or —.
Note the reading on the (V) scale and the corresponding staff reading (M), of the middle cross wire at this position.

(3) Note stadia reading (i.e. intercept) (S) between the top and bottom stadia lines.
Thus the R.L. of the middle cross hair point is given by (V) $\times$ (S)
The R.L. of the ground point is given by (V) $\times$ (S)—(M).

Example :

Suppose the middle cross wire reading on the "V" scale is 4.00
The middle cross wire reading (M) on the staff is 7.10
The stadia intercept (S) is 5.20.
The R.L. of the middle wire intersection on the staff is $4.00 \times 5.20 = 20.80$
R.L. of the ground point is $20.80 - 7.10 = 14.70$.

(*b*) For obtaining horizontal distances :

(1) Read the horizontal scale "H" (H) when reading the "V" scale.

(2) Obtain stadia intercept (S)

(3) The stadia constant (C) = 100

Thus the horizontal distance is obtained from

$$(S)\times(C)-(S\times H)=S(C-H)$$

Suppose the "Hor" scale reading is 8.00 ; and S=5·20 as in the previous case, then the horizontal distance,

$$=S(100-H)$$
$$=5{\cdot}20(100-8{\cdot}00)$$
$$=5{\cdot}20\times92$$
$$=478\ 40$$

'B' type Arc :	Staff intercept	5·20
	"Hor" reading	92·00
	"Vert" reading	4·00

Horizontal distance $=5{\cdot}20\times\frac{92}{100}=47{\cdot}84.$

Vertical height $=5{\cdot}20\times4=20\ 80.$

The difference in elevation between the instrument station and the staff station is calculated as below:

Calculation of elevation of a point in relation to the instrument station = staff intercept (S) × Vertical scale reading (V) − Middle cross-hair reading (M) + Height of instrument (HI).

Example : Suppose when sighting a stadia X

staff intercept is	5·20
"Vert" reading is	4·00
middle cross-hair reading is	7·10
height of instrument is	4·20

then the elevation of X above the initial station is

$$(5{\cdot}20\times4{\cdot}00)-(7{\cdot}10)+4{\cdot}20)=29{\cdot}82$$

(See Tacheometric levelling)

LEVELLING AND CONTOURING*

Levelling is the process of determining the relative heights of points, above or below, an assumed horizontal datum ; when this is applied to surfaces or areas it is known as contouring. Levelling carried out on an alignment is known as profiling. In some cases, the sea level is assumed to be the datum. In practice, the level of any one point is taken to be the standard, and the relative difference in levels of all other points, known as reduced levels, in relation to this are then worked out.

For approximate levelling for use with prismatic compass and Brunton compass surveys (1) an ordinary mason's spirit level or (2) the Abney level or (3) the Bruton alone may be employed with advantage or (4) an aneroid barometer calibrated as an altimeter, can be employed.

However, when a higher degree of precision, in contouring, is

*See addendum.

required, the plane table with telescopic alidade and Beaman's stadia arc, is to be used. The plane table can also be used in conjunction with a levelling instrument or level (to be described later) for this purpose. Tacheometric levelling as described earlier can also be resorted to.

(1) **Mason's level or spirit level** : A makeshift sightvane and peep hole are attached to either end of the bubble as shown in Fig. 1.28, and the instrument is mounted on a light tripod or stand and placed over a starting station. After levelling the bubble, the level may be rotated and radial sights are taken on to a graduate staff or scale held at various spots. Hence, if the level of the instrument station and the height of instrument are known the relative levels of the other points can be derived. The method of calculating the relative or reduced levels of stations has been given at the end of this chapter.

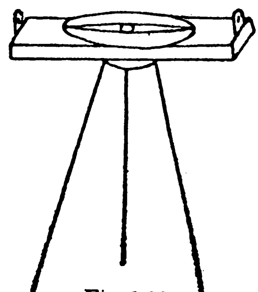


Fig. 1.28

(2) **Abney level** : This instrument (Fig. 1·29) consists of a sighting tube (1) to which is attached a semicircular scale (5). The line joining the zero of the scale to its centre is at right angles to the axis of the tube. To the vernier (4) of the scale is attached a bubble tube (3). The bubble is in the centre of its run, when the vernier reads zero, and, hence the sighting tube is also horizontal. On the top of the sighting tube is an opening, and below the opening and within the tube is fixed an inclined mirror (6), with a horizontal centre line. The bubble tube (3) is not cased at the top and bottom, and this naked portion is just over the mirror. The instrument is so constructed that when looking through the sighting tube the image of the bubble can be seen in the mirror.

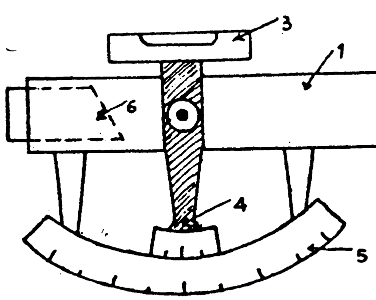


Fig. 1.29

When sighting the station point, through the tube subsequent adjustment of the vernier scale to set the bubble back on the centre of the mirror, is affected by rotating the vernier. The movement is registered either as an angle of elevation or depression, on the vernier.

The Abney level is handy for rough contouring as the vernier could be set at zero, and sights taken on to a graduated staff held at various points (no stand or sight vanes are required) in the manner described earlier for the use of the mason's level. If the level of the observer's eye above ground level is measured, the relative heights of points can be computed.

In case, only the heights of a few inaccessible points are required, and the distances are great, the method indicated below can be employed (Fig. 1.30).

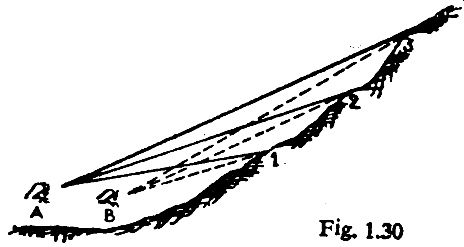


Fig. 1.30

Here the distance AB is measured, A and B are selected either on a level portion of the ground, or they are points whose reduced levels have been determined beforehand. The vertical angles to stations 1, 2 and 3 are measured first from A and then from B. The heights can be obtained either by geometrial constructions or by trigonometrical calculations. An approximate contouring can also be attempted with this method but this will be a tedious process. An alternative method of contouring with an Abney level is to run traverses across the area with a prismatic compass. The stations on the traverse are placed at such intervals, so that the Abney level at the back station, can be conveniently employed to sight a target on the staff placed at the forward station. The targets placed on the staff are at the height of the observer's eye from the ground. The slope distances between stations are taped. The heights or depression can be obtained either from tables given in Appendix (1) or can be calculated.

Levelling instruments

There are various modifications of the levelling instruments. Some are merely improved models while others are adaptations of the same instrument for a particular purpose.

One of the earliest forms of the levelling instrument is the Y-level. The telescope is supported on short Ys. This instrument is obsolete. The dumpy level is an improved model. The telescope is attached to a vertical spindle which fits into a cup. This instrument is very popular because of its simplicity and robustness of construction.

Further modifications and improvements have resulted in the manufacture of the reversible types—the Cooke reversible or the Zeiss reversible. The instruments are capable of being easily adjusted or corrected (see permanent adjustment) but they are not as robust as the dumpy level.

In addition to these models are the quick set levels in which there are several modifications. In the simpler models, the telescope is supported on a ball and socket arrangement with a single graduated drum screw for levelling the instrument along the axis of the telescope, *i.e.*, line of collimation.

Finally, there are the instruments known as gradiometers, which are particularly useful in projects involving the pegging out of graded alignments —as in laying roads or railroads; on surface or underground, or canals. The instrument is similar to the quick set type, but in addition the drum screw placed below the telescope is graduated in gradients also, *e.g.*, 1 in 100, 1 in 50 etc.

Some levels are also equipped with stadia device, for tacheometric measurements of distances.

Dumpy level : The most popular instrument for levelling is the dumpy level. This in combination with the plane table is useful for all ordinary purposes. The level sections can be run, either along radial lines from a known point or run in two sets of intersecting parallel lines at right angles to each other, constituting a grid. The reduced levels of several points are determined along these lines and the contours interpolated. Sometimes, it is more convenient to locate points of known reduced levels by trial and error.

Note : As the level is not a distance measuring instrument normally, hence it is not necessary to place it over stations.

Adjustment (temporary) : Adjustments required to be carried out for setting up the instrument, for carrying out observations are called temporary adjustments, as opposed to adjustment required to be made for setting right any defects in the instrument. The following are the various steps in setting up the dumpy level.

(1) Open out the stand, place it firmly on the ground and remove the cap.

(2) Note the lie of the instrument in the box, so as to enable its quick replacement.

(3) Support the instrument by the tribrach (as far as possible) and lift it out of the box.

(4) Place the instrument on the stand and screw in, after making sure that the threads have engaged (see Theodolite).

(5) Bring the foot screws to the middle of the run.

(6) Press the legs "home", *i.e.*, plant them firmly.

(7) Place the longer bubble, which is parallel to the axis of the telescope, in alignment with the two legs of the tripod stand.

(8) Now move the third leg back and fore and sideways, so as to bring the bubble to the middle of the run.

(9) Place the bubble next over the third leg, *i e.*, in a direction at right angles to its original position. Repeat as (7) above, by pressing one of the other two legs, so as to move the bubble to the centre.

(10) When the bubble is fairly steady, the free leg is also pressed down and the instrument is finally levelled by means of its three foot screws (see instructions regarding theodolite adjustment given below).

Theory and procedure

The process of levelling may be considered analogous to making measurements of the depth of water in a lake by taking soundings. In this process, it is assumed that the surface of water is horizontal and so it can be taken as a plane of reference. Once the levelling instrument is set up, and adjusted, (as stated earlier) the line of collimation describes a horizontal surface, when the instrument is rotated about its axis. This forms the surface of reference or datum for measurements of the relative difference or variations in levels.

*To start with, the staff is placed vertically at the control point or datum point. Then staff is placed over the other points, and readings are taken similarly. An increase in staff readings, over that noted for the control point, indicates a fall in ground level, while a decrease in the staff reading (from that of the control point) signifies a rise in ground level, assuming that the height of the instrument is the same at all stations.

While continuing levelling operations, over long distances, it is often necessary to shift the position of the instrument. When the levelling instrument is shifted to a new place, the staff is fixed at the last point sighted. After completing the temporary adjustments at this point, the staff is again sighted back and the new reading recorded for

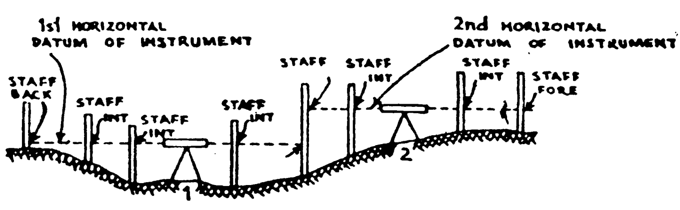


DIAGRAM EXPLAINING CORRELATION OF BACK & FORE READINGS IN LEVELING

Fig. 1.31

the same point. By these operations, the correlation between the horizontal datum of the instrument at the first setting with the new horizontal datum of the second setting is established. The last reading of the first setting, taken for connecting up is thus recorded as 'fore' and the first reading, at the same point, from the new setting is recorded as 'back', while the other intermediate readings are booked as 'inter' (Fig. 1.31).

Rise and fall method

Starting with the initial point, the relative, rise or fall in ground level is calculated by applying the principles stated earlier, *i.e.*, increasing staff reading shows fall in ground level and vice versa. Then assuming any one point as reference standard, reduced levels for successive points or obtained by either adding or subtracting the 'rise' and 'fall' value as the case may be.

The levelling data are recorded in the field, in the following tabular form in columns 1 to 7, and the reduced levels of the various points calculated at the office later on in the manner of the example given on next page.

In the table given below the same example is worked out using the rise and fall method.

Station	*Sight*			*Rise*	*Fall*	*Reduced*
	Back	*Inter*	*Fore*			*level*
	7.20					100.00
		8.62			1.42	98.58
		10.43			3.23	96.77
		9.32			2.12	97.88
		11.71			4.51	95.49
		14.60			7.40	92.60
	6.42		14.92		7.72	92.28
		8.92			2.50	89.78
		6.44			0.02	92.62
		4.11		2.31		94.59
		3.20		3.22		95.50
	9.28		2.10	4.32		96.60
		8.48		0.78		97.38
		6.56		2.72		99.32
		5.72		3.56		100.16
		4.10		5.18		101.78

Height of Instrument Method: This method of levelling is also commonly used by surveyors. In this method, the initial back sight is taken to a known Bench Mark (B.M.). The staff reading on the B.M. is noted and this is added to the B.M. height to obtain the instrument height. For example, if the B.M. height is 10 m and the staff reading is 1 m at the point, then the height of instrument is 11 m. If from this value, the reading on the foresight or inter sight, is deducted, the reduced level of the fore or inter sight point, is obtained. For example, if the fore or inter reading is 1.3 m, then the reduced level is 11 m − 1.3 m or 9.7 m. The fig 1.31a illustrates the principle of the procedure.* The method of recording the data is given in the table below.

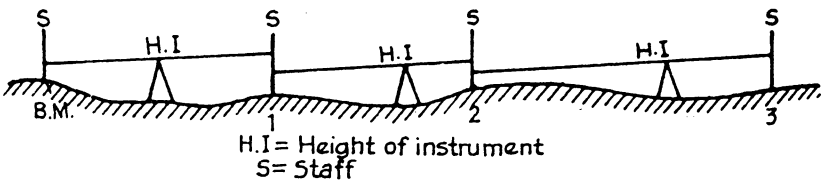


Fig. 1.31(a)

Back	Height of Inst	Inter	Fore	Reduced level
7.20	107.20			100.00
		8.62		99.58
		10.43		96.77
		9.32		97.88
		11.71		95.49
		14.60		92.60
6.42	98.70		14.92	92.28
		8.92		89.78
		6.44		92.26
		4.11		94.59
		3.20		95.60
9.28	105.88		2.10	96.60
		8.48		97.40
		6.56		99.32
		5.72		100.16
		4.10		101.78

Tacheometric levelling : The tacheometer can also be employed for purposes of levelling and contouring. The principle of the tacheometer has already been explained, while dealing with stadia or tacheometer survey methods Two cases arise, while computing the reduced levels from the field data. These are solved in the following manner.

See addendum.

Case (i)—When the vertical angle is positive, *e.g.*, (Fig. 1.32).

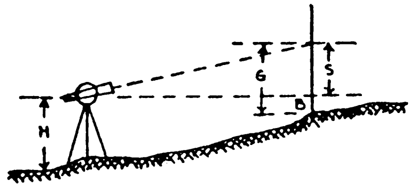


Fig. 1.32

To obtain the reduced level of B with reference to the reduced level of A ; the following formula may be employed :

R.L. of B=(R.L. of A+H+S)−G

where H=height of instrument at A

S=vertical height as obtained from tacheometric calculation (or from the Beaman arc).

G=is the middle wire reading.

Case (ii)—Where the vertical angle is negative *e.g.*, (Fig. 1.33).

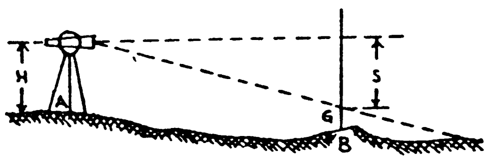


Fig. 1.33

To obtain the reduced level of B with reference to the reduced level of A ; the following formula may be used.

R.L. of B=(R.L. of A+H−S)−G.

=(R.L. of A+H−S+G).

Procedure for tacheometric levelling

1. Set up the tacheometer on the traverse stations or triangulation stations (see adjustments).
2. (*a*) Before sighting the staff stations, adjust the horizontal plate so as to make the verniers read zero with reference to a known field station.

   (*b*) Note the height of the instrument axis above ground station.

   (*c*) The above procedure should be repeated after setting up the instrument at every station.
3. Direct the telescope on the staff stations and read the (*a*) horizontal angle, (*b*) vertical angle, (*c*) stadia intercept, *i.e.*, top and bottom stadia wire readings, and (*d*) middle wire readings.
4. Calculate the horizontal distance, difference in elevation and

reduced level of the rod stations with reference to the tacheometer station.

5. Plot the bearings or azimuths between the stations following the "co-ordinate method" to be described later.
6. Plot the reduced levels of various stations, and interpolate the contour lines, at the required intervals, by calculation as well as by sketching in the field, where minor details can be brought in.

Note : (*a*) As the speed in reading the staff is of great importance it is advisable to adjust the vertical arc tangent in such a manner so that either the top or bottom stadia wire coincides with a whole figure on the staff, *i.e.*, 2 feet or 4 feet so that the other intercepts are read quickly. If readings are not taken quickly, the staff is likely to oscillate and cause inconvenience and delay.

(*b*) Steps described under paragraphs 4, 5 and 6 are carried out in the office.

TRIANGULATION SURVEYS

Triangulation method of surveying, which is often followed in theodolite, tacheometric surveys and plane table surveys, is generally utilised where considerable accuracy is required. Triangulation method is fast and enables horizontal control over large map areas. It is best adopted in regions, where long sight lines from many points are possible, especially in areas of rugged relief, as in hilly terrain.

Principle : In triangulation, map locations of ground points are obtained by the intersection of two or more rays, taken to the point from two different instrument stations. Briefly, the most simple triangulation method involves the following sequence of operations :—

(1) Fixing the base line by measuring its length and direction. The base line fixes the scale of the map as well as its orientation with respect to the sheet. The line should be measured on an open level ground and the two ends must be visible from each other as well as from at least three other triangulation stations. If the area is large, the base line should be located somewhere in the central portion, so that the net need not be extended too far in any one direction. If the area is greatly elongated, the base line should be oriented as far as possible in the direction of the larger dimension, and the measurement is accurately done with invar steel tape. A chain may be used in case of triangulation with prismatic compass. Where the country is uneven, stadia measurement may be made as a check on tape measurement. For very accurate theodolite triangulation,

the base line is measured with invar steel band applying corrections to compensate for (*a*) extension, due to tension or pull, (*b*) thermal expansion and (*c*) sag.

This method of measurement is outside the scope of normal geological work, and it is not required for use in connection with the methods of survey used in geological work.

(2) In the field, the location of any point, which is visible from both base line points, is obtained by sighting the unknown point from the base line points and drawing the two rays. The intersection establishes the relative location of the unknown point in question and horizontal distance from other known points. It is sometimes advisable to check the accuracy of intersections by a third interscctiog ray from a third station.

(3) Establishment of the elevation of the base line station is done by taping up to known bench mark or trigonometrical station of known R. L. and by running a fly level traverse.

(4) Establishment of a base net is done by sighting two known stations, one on either side of the base line, and sighting back from those to the base line stations, as well as on to each other; as shown in Fig. 1.34.

A triangulation net is established and extended over the area to be mapped, in precisely the same manner as in the case of establishing the base net. From stations in the base net, rays are drawn towards all visible stations, and these are located by the intersection of rays (Fig. 1.34). The relative strength of these locations, however, depends on the angles made by the intersecting rays and also on the number of rays drawn to each of these points. The location is strongest, when this angle is 90° and weakest if the angle less than 30° is greater than 150°.

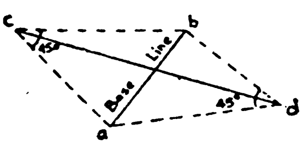


Fig. 1.34

There are four types of triangulation nets as shown in Fig. 1.35 (*a*) Chain of triangles, (*b*) Chain of quadrilaterals, (*c*) Chain of polygons,

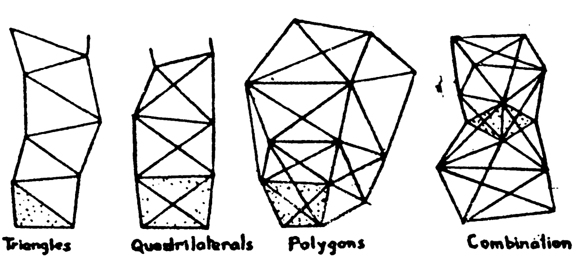


Fig. 1.35

and (*d*) Combination net. The chain of triangles is the weakest construction of the four and the combination net is the strongest.

A check base is established, when the triangulation system has been greatly extended from the base net (as shown in Fig. 1.36), in order to check up if the system is "out of scale" or orientation. The stations, at the two ends of the check base line, are located by intersection from three or more triangulation stations, and the distance measured accurately with the steel tape. If the work has been carried out accurately, the measured

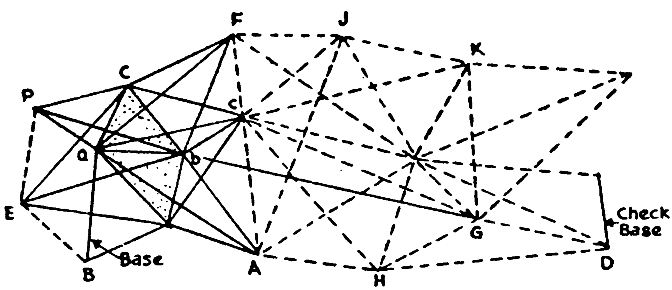


Fig. 1.36

distance and the scaled distance, as obtained from the map, should agree.

The recording of instrumental data and calculations are shown in Appendix 3 for theodolite surveys, Appendix 4 tacheometer surveys and 5, 5a, for telescope alidade Beaman arc, and plane table surveys respectively.

Note : Triangulation survey with a theodolite is different in principle, since the distances are not measured or plotted in the field, as in the plane table surveys. In the case of theodolite triangulation, only a base line is measured, with the greatest precision and all angles determined accurately with a theodolite. The area is divided into triangles. Starting with the base line, the lengths of all the sides of all traingles are mathematically computed. A check base, forming the side of one of the computed triangles, at some distance, is measured, and this serves as a check on the accuracy of the field measurements and computations.

In the case of plane table surveys, however, the traingles must be "well-conditioned", i. e., the angles should not be very acute. and should be as near 60° as possible. This factor is not of such prime consequence, in the case of theodolite or tacheometer triangulation surveys.

Interior filling of details in surveys

The filling in of interior details—topographic, geologic etc.—in most surveys, except in accurate theodolite triangulation, or traverse, is generally done either by compass and tape survey, or by compass and pace survey. The compass used, may by prismatic compass, Brunton compass or clinometer compass.

The measurement and recording of details in chain and compass traverse is shown in Figs. 1.37 and 1.38. Offsets, taken on either side of the chain or compass traverse, are measured with a tape. Subsidiary traverses shown in dotted line may also be made from the main traverse

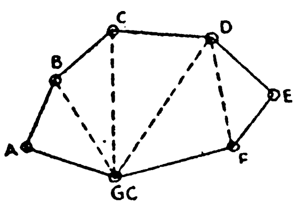


Fig. 1.37

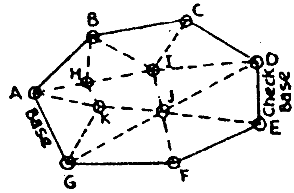


Fig. 1.38

shown in firm lines for filling in further details. The same procedure is adopted again for the purpose as explained under chain survey.

Theodolite traverse and plane-table surveys

In the case of theodolite and plane-table surveys, subsidiary traverses are made to cover the areas within easy reach of each rod-station, and such details are generally obtained by again employing the compass and pacing method or by compass and tape method. For such subsidiary traverse initial stations are radially located at short distances from each rod station. In a rugged country, requiring accurate work, it is advisable to have a larger number of such staff stations or rod shot stations, as these form, known points at close intervals, distributed all over the area. If carefully executed and plotted, pace and Brunton compass survey, (Fig. 1.39 & 1.40) or pace and clino-compass survey, for such work is as accurate as compass and tape measurement for map scales which are most commonly used, say upto 200 ft. to an inch. A specimen of the staff man's field notes, is shown below :

From station A—Station 1

"At contact of granite and manganese ore. Ore 5 ft. thick low grade consisting mainly quartz, subordinate manganese ore.

(Specimen 3/Rs/64).

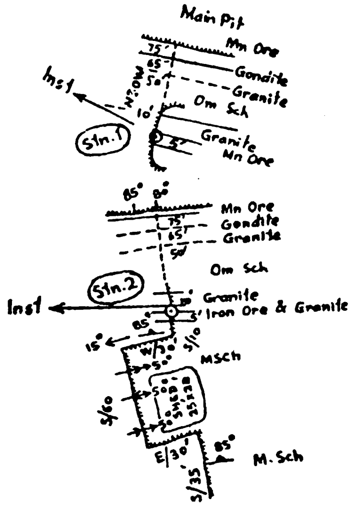


Fig. 1.39

From station A—Staff station 2

On the same contact as at stn. 1 and at the top of a vertical cutting (Fig. 1.39).

Staff station 51

On top of qzt. hillock due NNE of stn. A. Qzt. bed 55 ft. thick-finely bedded with a core of massive wide recrystallised quartz".

(Specimen 2/RS/64) (Fig. 1.40).

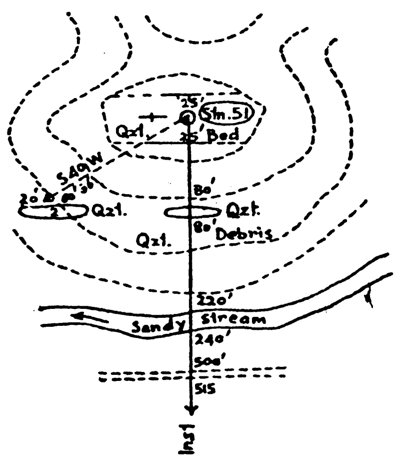


Fig. 1.40

DETERMINING TRUE NORTH

Shadow method—is sufficient for most purposes and the following is the procedure. The principle involved depends, on the symmetry in the length and direction of the shadow cast by a vertical linear object placed in the sun, during the course of the day. The sun is a bright star, moving round the celestial pole and, hence transits the celestial meridian once each day like every other star.

The traverse, after being plotted, is fixed to a level table (or plane table after levelling), and placed over a point P, the location of which is known, and which is free from interfering or extraneous shadows. The traverse is oriented with reference to the field.

Concentric circles of varying radii (Fig. 1.41) are drawn round this point P, at invervals of 1/4th inch upto 3 inch radius, and at intervals of ½ inch upto 6 inches radius. A straight steel pin, about 1/8 inch thick and 5 inches long is planted firmly and vertically at the plotted point P. The of the pin may be checked, in four directions at right angles to each other, either with the right angle of a setsquare or by a plumb bob. The measurements are started in the morning and completed in the evening, say between 8 A. M. and 4 P. M. As the tip of the shadow cast by the pin, just touches a particular concentric circle, the point of contact is marked on the circle by a cross. Thus, at the end of the day, when all such crosses are joined, two lines AP and BP meeting at the point P are obtained. The magnitude of the angle between the two lines, depends on the latitude of the place, and also on the season of the year. The bisector of this angle gives the direction of the true north.

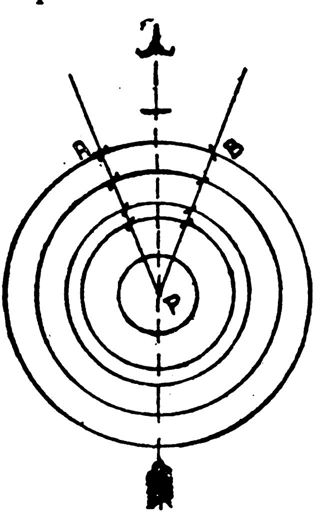


Fig. 1.41

Equal altitude method : When a theodolite is available, this method can be employed and greater precision obtained. This is also simple in operation. In this method, advantage is taken of the fact that when a star (or the sun) is sighted at a certain altitude when it is rising, and again at the same altitude when it is setting, and if directions of these two positions in the horizontal plane i. e. azimuths are known, the medial position gives the celestial meridian or the north-south line passing through that point. The theodolite is set up and temporary adjustments done, on any station in the traverse. The horizontal plate, say vernier A, is made to read zero. By releasing the body clamp A station B is sighted. At night a frosted lamp or beacon is required to be held behind the rod, planted at the station B for sighting.

Now the horizontal plate, vernier clamp and the vertical circle clamp, are released, and the star is sighted. The clamps are tightened, and finer intersection completed with the respective vernier tangents. The readings of both, the horizontal and the vertical verniers are recorded. Both the verniers are continuously read and the readings recorded, at suitable time intervals. It will be noticed, that the value of the vertical angles reach a maximum (until the azimuth of celestial meridian is crossed) and then decrease on the other side, where vertical angles, of equal value to those measured earlier, will be obtained. If the horizontal angle, subtended by two equal altitude positions, is bisected; the direction of the bisector, gives the position of the meridian, thus fixing the true north. The observations, on a suitable star, should be started, as early as possible, in the evening after dusk when the particular star is clearly visible, and continued until the early hours of the morning without a break. If the sun is observed, special equipment, known as solar attachment, will be required for reading vertical angles (in our latitude) and a grey filter will have to be employed to protect the eyes.

PLOTTING OF SURVEYS

It may be observed that angles can be measured with great accuracy, in the field, whereas distances cannot be measured with the same degree of precision. But reverse is the case with regard to office work, where distances can be plotted with a greater degree of precision than angles. Hence in the case of the most accurate surveys e. g., theodolite triangulation, only a few distances are measured accurately, in the field, and all angles are measured with a precision instrument. However, the survey is plotted, in the office, by a method which involves only lengths.

In general, the accuracy of the method of plotting employed, should be commensurate with the degree of accuracy of the survey. Thus for a compass survey, the plotting may be done by means of plotting scales and protractor. For chain surveys, the plotting scale may be employed.

Coordinate Method

In the case of a theodolite traverse, the plotting should invariably be done, by the co-ordinate method described below :

(*a*) The bearings or azimuths of various lines, in the traverse are calculated, as indicated earlier; vide, theodolite and Miner's dial traverses, or the bearings may be obtained directly, by the use of the continuous bearing method.

(*b*) The whole-circle bearings thus obtained are then converted to quadrant bearings.

(*c*) If the length of any line and its quadrant bearing are known, taking one end to be the initial point, the co-ordinates are determined for the other end. The co-ordinates so obtained are called the partial co ordinates. Similarly partial co-ordinates may be calculated for any number of points, in the traverse. When the same initial point is taken for calculating the coordinates of the next and the succeeding points, then the co-ordinates so obtained will be known as total co-ordinates. The

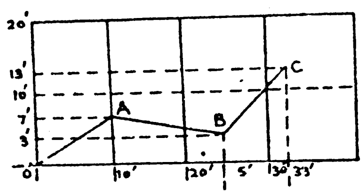


Fig. 1.42

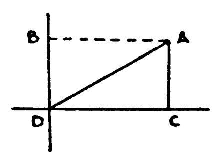


Fig. 1.43

principle involved in the calculation of partial and total coordinates is explained in Figs. 1.42 and 1.43.

The partial co-ordinates of A are + 10′ and + 7′,

for B, are +15′ and +4′,

for C, are + 8′ and +10′ ;

whereas the total co-ordinates are

for A, +10′ and +7′,

for B, +25′ and +3′, and

for C, +33′ and +13′.

The co-ordinates on the Y axis or vertical axis are called latitude, and the co-ordinates on the X axis or horizontal axis are called departure. Both latitude and departure can be easily calculated as shown below (Fig. 1.43).

Partial latitude of A

$=\text{Cos } \theta \times AD$

Partial departure of A

$=\text{Sin } \theta \times AD$

where θ is the quadrant bearing of the line AD.

The partial co-ordinates of all the points or stations in a traverse are calculated in a similar manner.

The total co-ordinates are thus computed from the partial co-ordinates by mere algebraic summations, i. e., paying heed to sign ; whether positive or negative in respect of the point of origin. An example of the procedure involved is given in table on page 58*.

The total coordinates have been computed by reference to tables known as traverse tables, in which the values of partial latitude and departure for various values of quadrant bearings have been calculated for various distances from 1 to 1000 ft.

Checks on angular measurements and coordinate calculations

1. In the case of a closed traverse, the accuracy of the field work can be checked, since the sum of the included angles=2 X right angles−360°, where X is the number of included angles. If there is a small angular error, this may be distributed giving special weightage to (*a*) the amount or magnitude of the angle, (*b*) difficulties in measurement at any particular point etc.

2. In a closed traverse, the calculations of the latitude and departure may be arithmetically checked. The total latitude and departure should be equal to zero, since the traverse terminates at the point of origin. Further, the sum of northings=sum of southings and the sum of eastings=sum of westings. It should be noted, that this check is only in respect of the computations, and not in regard to the accuracy of the field work.

The scale of chords

This method is at times employed for plotting angles. In order to construct an angle of required magnitude at a point on straight line, to start with, a circle, of radius equal to the length of the 60° chord of the scale, is constructed (Fig. 1.44). Then from the intersection point of the

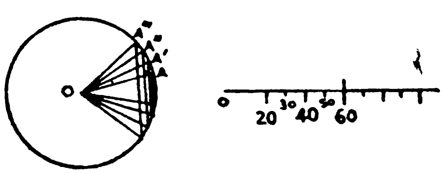


Fig. 1.44

*See addendum.

circle and the straight line, a length of chord equal in value to that of the desired angle is marked of on the circle (on required side of the straight line). This point on the circle, when joined to the centre gives the angle required.

Principle of the scale of chords : In any circle, the length of the chord subtended by an angle of 60° is equal to the radius (Fig. 1.44).

In the scale of chords, length of the chords subtended by various angles at the centre of the circle, are utilised to constitute a scale. Thus AB is the length of a chord which subtends an angle of 30° at the centre, AB′ is also the length of a chord subtending angle of 40° at the centre of the circle.

These lengths are marked as 30°, 40°, 50°, and 60° on the scale.

Thus with the scale of chords, since the values of the angles are plotted as linear measurements some amount of accuracy can be achieved.

Preparation of Mine Plans: The underground mine plan is a map which shows the position of the workings projected on to the horizontal plane. Since coal seams generally dip at low angles the common practice is to prepare the plan according to the above convention. This is mostly the case in regard to the coal mines of India. In the case of the metalliferous lodes, however, where the dips are often very high, being of the order of 70 to 80 degrees, and where the dip and strike are also inconsistent, plans for each level are drawn separately, placed one below the other, in the correct relative orientation and traced out, taking into account the dip of the ore body. In case the dips are very steep, the offset is increased so that the plan of each level is well separated, as shown in figure (1.45A). Sometimes combining the level plans in their correct orthographic position, in order to obtain the composite plan, is very useful in the identification of geological structures, which otherwise will be diffused in the individual level plans. This aspect is brought out in the figure (1.45B).

As opposed to the mine plan the practice in all underground metal mines, is to prepare longitudinal sections of the ore body, in which the levels, raises, winzes shafts etc., are projected on to the vertical plane at right angles to the dip. Thus the contortions seen in the level plans, due to the variation in strike, are eliminated and the levels appear as straight lines, as shown in figure (1.46). The longitudinal section is best suited for depicting structural features, such as faults, joints, bedding, wall rock contacts etc. It is, however, to be noted that the true dips of such features is to be converted to the apparent dip corresponding to that of the ore body, while

Tables known as traverse tables have been worked out for various values ∠BDA = 0 (bearing) and for various lengths of DA BD. Thus at a glance the partial latitude and partial departure of any given point for any given bearing can be determined by referring to the table

Sl. No.	Line	Distance in feet	Horizontal angle	Calculated whole circle bearing	Quadrant bearing	Partial latitude		Partial departure	
						N	S	E	W
A	HA	–	–	92° 0′	S 88° E	–	16.3	522.7	–
A	AB	523	127° 14′	39° 14	N 39° 14 E	271.1		221.3	
B	BC	350	55° 27′	94° 41	S 85° 19′ E		61.9	754.5	
C	CD	757	77° 34′	352° 15′	N 7° 45 W	768.7			104.6
D	DE	776.7	108° 12′	270° 27′	N 89° 33 W	4.7			595.1
E	EF	595.1	39° 46′	121° 13′	S 41° 13 W		246.7		226.2
F	FG	328.0	315° 09′	256° 22′	S 76° 22′ W		108.4		447.0
G	GH	460.0	115° 17′	191° 39′	S 11° 39′ W		609.2		125.6
H	HA	622.0	98° 39′	92° 0′	S 38° E				
						1044.5	1042.5	1498.5	1498.5

The following calculations will have to be done before computing the co-ordinates.

plotting. A variant of the longitudinal section is what is known as the inclined longitudinal section. This is in reality the plan of the ore body in the plane containing the dip and strike. (See Underground Mining Plan and Opencast mine Plan.)

SURVEY EQUIPMENT WITH REFERENCE TO THE PROBLEM

In general, the instruments chosen will be in keeping with the accuracy required and the time available for completion of the survey :

Instrument employed and method	Problem involved or conditions
1. Chain Survey.	It is employed when no angle measuring instrument is available. It can be carried out with the help of untrained hands. The country should be more or less flat. The method can also be employed for the interior filling of theodolite surveys. Scale of plotting, 1 inch = 100 ft.
2. Compass Survey.	It is good for ordinary preliminary work. Distances are, in general, obtained by taping or pacing. As such it cannot be employed for accurate work. Scale of plotting is generally 4″ = 1 mile. It can be used for interior filling of chain surveys and for normal geological mapping, on large scale. It can also be used in stope surveys underground (with taping).
3. Plane table Survey.	It is useful in flat or gently undulating topography. It is as accurate as chain surveys and can be usefully employed for interior filling of theodolite surveys. When telescopic alidade, equipped with Beaman stadia arc is available, the method can be applied even in fairly rugged terrain. It is recommended for detailed mapping and proving of mineral deposits—underground mine surveys, geochemical surveys, and geophysical surveys.

4. Theodolite Survey.	Traverses are required for accurate work—when extreme accuracy is required, then theodolite triangulation is essential. Tacheometric surveys are as accurate as chain surveys but the accuracy is relatively higher in rugged country. The method is useful in contouring hilly country. It is useful in proving mineral deposits, mines surveys, and correlation surveys, etc.
5. Abney Level.	Useful in contouring in conjunction with compass surveys or in the interpolation of heights in such surveys in rugged country. It can be used also with chain surveys.
6. Dumpy Level.	Very useful in accurate contouring of theodolite and plane table surveys in a flat or gently undulating topography.

Instruments and mineral deposits

General scheme giving a few examples of mineral deposits and the methods of survey.

Type or nature of deposit	Instruments or methods to be employed
1. Limestone and clay in a flat country.	If no high order of accuracy is required the compass and pacing with abney level will do. If a higher order of accuracy is wanted, then plane table with dumpy level, will be necessary.
2. Limestone or coal in flat country.	Theodolite traverse with dumpy level will be required.
3. Metalliferous lode in plane country (Gold or copper or manganese etc.).	Theodolite traverse, or plane table or chain with dumpy level will be required.
4. Metalliferous lode in a hilly country.	Tacheometer survey (or Telescopic alidade with Beaman stadia arc) may be employed.

5. Iron ore occurring as a capping on a flat topped hill.	Theodolite. Normally no compass survey should be employed. Traverse with tacheometer and levelling by stadia or dumpy level will be satisfactory.
6. Mine survey for coal or metal.	Theodolite traversing or Dial survey may be done. Underground traverses may also be done with plane table with Beaman stadia arc attachment.
7. Stope survey.	Compass and tape survey (Brunton compass is convenient) will be best suited.

Note :—(1) In general the more precious or valuable a metal is, the more accurate should be the method of survey chosen.

(2) And the nature of the work should also be considered *i.e.*, whether preliminary or detailed investigation.

Appendix 1

Horizontal Distance in Feet with Corresponding Slope-Distance

Slope in degress	Horizontal distance														
2	0.99	1.99	2.99	3.99	4.99	5.99	6.99	7.99	8.99	9.99	19.99	29.98	39.98	49.97	99.94
4	0.99	1.99	2.99	3.99	4.98	5.98	6.98	7.98	8.98	9.98	19.95	29.93	39.90	49.88	99.76
5	0.99	1.99	2.99	3.98	4.98	5.98	6 97	7.97	8.97	9.96	19.92	29.89	39.85	49.81	99.62
6	0.99	1.99	2.98	3.98	4.97	5.97	6.96	7.96	8.95	9.95	19.89	29.84	39.78	49 73	99.45
8	0.99	1.98	2.97	3.96	4.95	5.94	6.93	7.92	8.91	9.90	19.81	29.71	39.61	49.51	99.03
10	0.98	1.97	2.95	3.94	4.92	5.91	6.89	7.88	8.86	9.85	19.70	29.54	39.39	49.24	98.48
12	0.98	1.96	2.93	3.91	4.89	5.87	6.85	7.82	8.80	9.78	19.56	29.34	39.13	48.91	97.81
14	0.97	1.94	2.91	3.88	4.85	5.82	6.79	7.76	8.73′	9.70	19.41	29.11	38.81	48.51	97.03
15	0.97	1.93	2.90	3.86	4.83	5.79	6.76	7.73	8.69	9.66	19.32	28.98	38.64	48.29	96.59
16	0.96	1.92	2.88	3.84	4.81	5.77	6.73	7.69	8.65	9.61	19.23	28.84	38.45	48.06	96.13
18	0.95	1.90	2.85	3.80	4.76	5.71	6.66	7.61	8.56	9.51	19.02	28.53	38.04	47.55	95.11
20	0.94	1.88	2.82	3.76	4.70	5.64	6.58	7.52	8.46	9.40	18.79	28.19	37.59	46.98	93.97
22	0.93	1.85	2.78	3.71	4.64	5.56	6.49	7.42	8.34	9.27	18.54	27.82	37.09	46.36	92.72
24	0.91	1.83	2.74	3.65	4.57	5.48	6.39	7.31	8.22	9.14	18.27	27.41	36.54	45.68	91.35
25	0.91	1.81	2.72	3.62	4.53	5.44	6.34	7.25	8.16	9.06	18.13	27.19	36.25	45.31	90.63
Slope distance	1	2	3	4	5	6	7	8	9	10	20	30	40	50	100

Horizontal Distance in Feet with Corresponding Slope Distance—Continued

Slope in degrees	Horizontal distance													
26	0.90	1.80	2.70	3.59	4.49	5.39	6.29	7.19	8.09	8.99	17.98	26.96	35.95	44.94	89.88
28	0.88	1.77	2.65	3.53	4.41	5.30	6.18	7.06	7.95	8.83	17.66	26.49	35.32	44.15	88.29
30	0.87	1.73	2.60	3.46	4.33	5.20	6.06	6.93	7.79	8.66	17.32	25.98	34.64	43.30	86.60
32	0.85	1.70	2.54	3.39	4.24	5.09	5.94	6.78	7.63	8.48	16.96	25.44	33.92	42.49	84.80
34	0.83	1.66	2.49	3.32	4.15	4.97	5.80	6.63	7.46	8.29	16.58	24.87	33.16	41.45	82.90
35	0.82	1.64	2.46	3.28	4.10	4.91	5.73	6.55	7.37	8 19	16.38	24.58	32.77	40.96	81.92
36	0.81	1.62	2 43	3.24	4.05	4.85	5.66	6.47	7.28	8.09	16.18	24.27	32.36	40.45	80.90
38	0.79	1.58	2.36	3.15	3.94	4.73	5.52	6.30	7.09	7.88	15.76	23.64	31.52	39.40	78.80
40	0.77	1.53	2.30	3.06	3.83	4.60	5.36	6.13	6.89	7.66	15.32	22.92	30.64	38.30	76.60
42	0.74	1.49	2.23	2.97	3.72	4.46	5.20	5.94	6.69	7.43	14.86	22.29	29.73	37.16	74.31
44	0.72	1.44	2.16	2.88	3.60	4.32	5.03	5.75	6.47	7.19	14.39	21.58	28.77	35.97	71.93
45	0.71	1.41	2.21	2.83	3.53	4.24	4.95	5.66	6.36	7.07	14.14	21.21	28.28	35.35	70.71
Slope distance	1	2	3	4	5	6	7	8	9	10	20	30	40	50	100

Appendix 2

Vertical Height in Feet with Corresponding Slope Distance

Slope in degrees	Vertical height														
2	0.03	0.07	0.10	0.14	0.17	0.21	0.24	0.28	0.31	0.35	0.70	1.05	1.40	1 74	3.49
4	0.07	0.14	0.21	0.28	0.35	0.42	0.49	0.56	0.63	0.70	1.40	2.09	2.79	3.49	6.98
5	0.09	0.17	0.26	0.35	0.44	C.52	0.61	0.70	0.78	0.87	1.74	2.62	3.49	4.36	8.72
6	0.10	0.21	0.31	0.42	0.52	0.63	0.73	0.84	0.94	1.05	2.09	3.14	4.18	5.23	10.45
8	0.14	0.28	0.42	0.56	0.70	0.83	0.97	1.11	1.25	1.39	2.78	4.18	5.57	6.96	13.92
10	0.17	0.35	0.52	0.69	0.87	1.04	1.21	1.39	1.56	1.74	3.47	5.21	6.95	8.68	17.36
12	0.21	0.42	0.62	0.83	1.04	1.25	1.45	1.65	1.87	2.08	4.16	6.24	8 32	10.40	20.79
14	0.24	0.48	0.73	0.97	1.21	1.45	1.69	1.93	2.18	2.42	4.84	7.26	9.68	12.10	24.19
15	0.26	0.52	0.78	1.03	1.29	1.55	1.81	2.07	2.33	2 59	5.18	7.76	10.35	12.94	25.88
16	0.28	0.55	0.83	1.10	1.38	1.65	1.93	2.20	2.48	2.76	5.51	8.27	11.03	13.78	27.56
18	0.31	0.62	0.93	1.24	1.55	1.85	2.16	2.47	2.78	3.09	6.18	9.27	12.36	15.45	30.90
20	0.34	0.68	1.03	1.37	1.71	2.05	2.39	2.74	3.08	3.42	6.84	10.26	13.68	17.10	34.20
22	0.37	0.75	1.12	1.50	1.87	2.25	2.62	3.00	3.37	3.75	7.49	11.24	14.98	18.73	37.46
24	0.41	0.81	1.22	1.63	2.03	2.44	2.85	3.25	3.66	4.07	8.13	12.20	16.27	20.34	40.67
25	0.42	0.84	1.27	1.69	2.11	2.54	2.96	3.38	3.80	4.23	8.45	12.68	16.90	21.13	42.26
Slope distance	1	2	3	4	5	6	7	8	9	10	20	30	40	50	100

Vertical Height in Feet for Corresponding Slope Distance—Continued.

Slope in degrees	Vertical height													
26	0.44	0.88	1.31	1.75	2.19	2.63	3.07	3.51	3.95	4.38	8.77	13.15	17.53	21.92	43.84
28	0.47	0.94	1.41	1.88	2.35	2.82	3.29	3.76	4.22	4.69	9.39	14.08	18.78	23.47	46.95
30	0.50	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	10.00	15.00	20.00	25.00	50.00
32	0.53	1.06	1.59	2.12	2.65	3.18	3.71	4.24	4 77	5.30	10.60	15.90	21.20	26.49	52.99
34	0.56	1.12	1.68	2.24	2.80	3.35	3.91	4.47	5.03	5.59	11.18	16.78	22.37	27.96	55.92
35	0.57	1.15	1.72	2.29	2.87	3.44	4.01	4.59	5.16	5.74	11.47	17 21	22.94	28.68	57.36
36	0.59	1.18	1.76	2.35	2.94	3.53	4.11	4.70	5.29	5.88	11.76	17.63	23.51	29.39	58.78
38	0.62	1.23	1.85	2.46	3.08	3.69	4.31	4.93	5.54	6.16	12 31	18.47	24.63	30.78	61.57
40	0.64	1.29	1.93	2.57	3.21	3.86	4.50	5.14	5.78	6.43	12.86	19.28	25.71	32.14	64.28
42	0.67	1.34	2.01	2.68	3.35	4.01	4.68	5.35	6.02	6.69	13.38	20.07	26.77	33.46	66.91
44	0.69	1.39	2.08	2.78	3.47	4.17	4.86	5.56	6.25	6.95	13.89	20.84	27.79	34.73	69.47
45	0.71	1.41	2.12	2.83	3.53	4.24	4.95	5.66	6.36	7.07	14.14	21.21	28.28	35.35	70.71
Slope distance	1	2	3	4	5	6	7	8	9	10	20	30	40	50	100

Appendix 3

Table for Recording Theodolite Traverse Field Data

Station from	Station to	Face right		Face left		Mean value of angles	Distance	Remarks
		Vernier A	Vernier B	Vernier A	Vernier B			

Appendix 4

Tabular Form for Recording Field Data in Tacheometer or Stadia Surveys

I Back sation	II On station	III Fore station	IV Height of Inst.	V Horizontal angles	VI Vertical angles	VII Staff readings on		VIII Remarks regarding Geology or Topography
						Stadia lines	Centre line	

Appendix 4a

Tabular Form for Calculating Horizontal Distances and Reduced Levels in Tacheometry

I Station Point	II Azimuth (Horizantal) angle	III Generating Number h = staff intercept	IV Distance function (Kh + c)	V Horizontal distance $D=(kh + c) \cos^2\theta$	VI Vertical heigh $(D\times \cos\theta \sin\theta)$	Reduced levels	
						VII of Inst.	VIII of Point

APPENDIX 5

Tabular Form for Recording Plane-table Telescopic Alidade Survey Data (with Beaman arc).

Sl. No.	Inst. Station	Ht. of Inst. (HI)	Rod Station	Rod Inter (RI)	Hor. Beam (HR)	Hor. Distance (HR×RI)	Vert. Beam (VR)	Cal. Diff. in Elev.	Net Difference in Elevation	R. L. of Instrument	R.L. of Rod Station
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(10)	(12)
1											
2											
3											

APPENDIX 5a

Tabular Form for Reporting Triangulation Data (Telescopic Alidade).

Sl. No.	Inst. Station.	Ht. of Inst.	Field Station	Rod Int.	Rod Read	Vert. Angle	Measured Hor. Dist.	Cal. Diff. in Elevation	Net Diff. in Elevation.	R. L. of Inst. Station	R. L. of Field Station
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1											
2											
3											

BORE HOLE SURVEYING METHODS

Introduction : Apart from logging bore holes by the various techniques, such as (a) lithological logs obtained by inspection and examination of cores and cuttings, (b) Caliper logs, (c) geophysical data e.g. S.P. and resistivity, (d) γ-ray data, (e) dip meter reading etc., the geologist is often required to know some essentials of the methods used in bore hole surveying.

Deep bore holes, and sometimes holes exceeding 1000 ft. are liable to deflection, either due to faulty drilling techniques or due to certain geological factors. Some of the chief causes for deviation of the diamond drill bore holes are, (a) the presence of rocks of differential hardness dipping a steep angles. In such a case, a steeply inclined bore hole, after passing through softer rocks, is deflected on touching a relatively very hard formation, and tends to follow the dip of that formation, *e.g.*, an inclined hole after passing through shales or phyllites may be deflected if it touches a steeply dipping band or hard quartzite; (b) jointing and fracturing present in the rocks, also cause deviation; (c) sudden variations and excessive pressures, applied during drilling, either due to increased rate of feeding or increased rock resistance, are another common cause. In general calyx holes, using shot bit, are more liable to deflection than diamond drill holes. A deflection, in the case of a shot drill hole, causes greater abrasion of the walls and dilution of the sample since the shot, leave the bottom of the hole and comes to the sides. The same is true of diamond drill holes to a lesser extent. A deviation will cause greater abrasion of the walls when the reamer is used; and the bit and reamer tend to cut downwards more than forwards. The result will be that the hole is more elliptical than circular; when the downward cutting is considerable. It may also be necessary, at times, to deflect a bore hole from a certain depth. In both cases, the survey of the bore hole becomes essential so as to know what exactly is happening at depth. Under these circumstances, the driller may not be aware of the deflection. Sometimes deflection, of a bore hole, is necessary when additional geological data is to be obtained. In such cases, the bore hole is deflected with a purpose. Bore holes, when once deflected may take the form of large arcs, or spirals with a large pitch. The deflection may be to either side, upwards or downwards or a combination of all these possibilities. Surveying of bore holes, thus becomes essential if the geological data obtained by drilling is to be assembled in the correct perspective, or correlated with data obtained by drilling in the adjacent areas. It is in fact, a method of underground surveying, under special circumstances, when the geologist has no direct access, to the traverse.

In the methods of bore hole surveying, so devised, distances (i.e. depths either vertical or inclined distances) are measured by means of the drill rods. The inclination of the bore hole, at the point of measurement, is indicated by a plumb bob arrangement, either directly or indirectly. A magnetic compass or a gyro compass is employed for obtaining the bearing of the bore hole at the point of measurement. All data, i.e., the inclination and bearing of the bore holes are recorded, either directly or photographically.

Inclinometer

The inclinometer is a simple apparatus for measuring the angle of the hole, at various depths. This consists essentially of a glass cylinder, protected by a metallic case. The cylinder is partially filled with a solution of HF of suitable concentration, generally 5%–10% and the inclinometer assembly is lowered into the bore hole to the desired depth, where it is kept sufficiently long to enable the HF to etch the glass container. The inclination of the bore hole is obtained by measuring the angle, which the itched ring left by meniscus of HF, makes with the axis of the glass cylinder.

It is safer to use the instrument in which the principle of electrolytic deposition of copper on iron is utilised. If a copper sulphate solution is placed in a iron cylinder and lowered into the drill hole, copper is deposited on the iron, and the reading of the angle, made by the plane of the meniscus, to the axis of the cylinder gives the dip. Instead of a cylinder of iron two iron sheets, welded at right angles to each other, can be fitted into a suitable glass cylinder, filled with $CuSO_4$ solution and lowered into the drill hole. These fins of iron plates show the deposition of copper. From the inclination of marks left by the copper deposit, the dip or angle of the hole can be worked out.

In the earlier methods, the inclination of the bore hole was obtained by freezing a gel contained in a glass cylinder, in which a plumb bob was suspended.

Photographic method

In the photographic methods, the apparatus (Fig. 1.45) used comprises a metallic cylinder, (C) which is attached to the end of the drill rods (T). At the bottom of the cylinder, is a spring loaded marking device or scriber (S) which makes a deep linear mark on the top of the stub at bottom of the hole (before drilling). Within the cylindrical housing, as a sensitised disc of photographic paper (P), over which is a free moving magnetic needle attached to a graduated transparent scale (D). Immediately above the centre of the transparent disc is a plumb bob (B). The centre of the transparent disc and the point of suspension of the plumb bob are in alignment when the equipment is vertical. On the same alignment is, a collimation hole, (H) above which is a small battery operated electric lamp (L). The shadows of the magnetic needle, and the plumb line fall on the photographic paper, and thus the direction of the deviation and amount are recorded. A continuous record is possible, where the disc of photographic paper is replaced by a roll of paper moved by a electric motor.

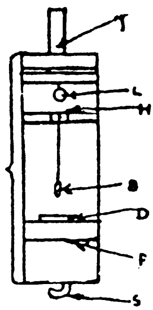


Fig. 1.45

In modern bore hold surveying instruments, a gyro-compass is employed instead of a magnetic needle. This makes the equipment suitable for work in areas where magnetic influence is very pronounced.

The gyroscope has been in use, for well over 60 decades, inmarine navigation and is also used for direction finding in aircrafts. The gyro compass is essentially gyroscope or spinning disc. In effect, the gyro is very much like the spinning top. Just as the upper end of the top's axis, while spinning steadily and after standing vertically, shows a circular movement about of the vertical axis, the gyro also behaves similarly. This movement of the axis is akin to that of the earth's axis and is known as precession.

Gyro Compass

Latter modifications of the equipment have utilised the photographic devices increasingly. The gyro compass was first available for use, only in the large diameter bore holes, but now models for use in holes upto about 2 diameter are available. The instrument is housed in a cylindrical shell with a diameter of $1\frac{3}{4}$". Once the gyro compass is set in motion, and corrected for precession, the axis maintains a constant direction irrespective of the movement of the housing. Thus the gyro device is proof, against effects of deflection as well as that caused by extrancous magnetism, and provides the basis of directional correlation in azimuth. A plumb bob can be included in the assemblage, for indicating the inclination of the hole. Continuous photographic recording, of the position of the gyro axis and the plumb bob simultaneously, provides a very clear picture of the deflections and deviations (Fig. 1.46)

Gyros can be classified under two main types:

(a) Free gyro, and

(b) Compensated or counter balanced gyro.

(a) The free gyro Fig 1.46 comprise a free spinning wheel (W) mounted on the axis 3–3. The arrangement is mounted on a universal joint as shown in Fig 1.46 or gimbals or in a fluo-liquid suspension. The last principle of suspension resembles the mounting of the human eye ball, within the

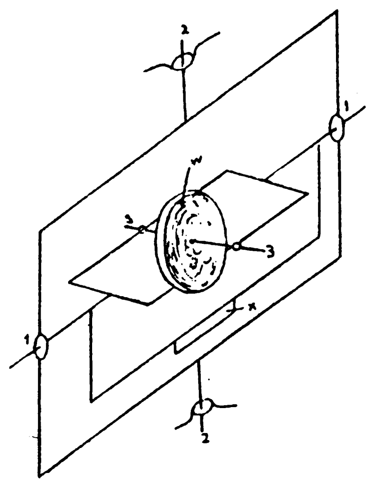


Fig. 1.46

eye socket. Thus the spinning wheel (W) can move in two mutually perpendicular directions 1–1 and 2–2.

When the wheel (W) is made to spin on 3–3 axis, at a high velocity, the entire set up shows a slow rotation about 2–2 axis, called the gyroscopic precession of the axis, and the movement has been attributed to conservation of angular momentum.

(b) In the counter balanced gyroscopic set up, this precession is either arrested or compensated either by applying a torque, or by a small electric motor which induces a controlling torque, on the rotating spinning wheel (W). This type of gyro is also called the north seeking type. as it compensates the rotation and precession, in the manner described below :

If the gyro axle 3—3 is oriented in a random position, making an angle with the true north, then the torque is impressed on the spinning wheel (W) about 3—3. This is mainly caused by the rotation of the earth from west to east, which makes the wheel (W) tilt on 1—1. This tilt initiates a precession. If, however, the axle 3—3 can be maintained in a horizontal position, by the use of a pendulum (X) attached to the 1—1 axis, then precession is about the 2—2 axis. The precession stops once the 3—3 axis aligns itself with the N—S (north-south) direction.

In the earlier bore hole surveying instruments, a free gyroscope, together with a plumb bob type angle (inclination) measuring unit, were employed, to give the direction and inclination of the bore hole, at various points. The measurements were, made continuously on a 16 m.m. camera film. The entire equipment was lowered into the bore hole, by wire line, and the time taken for recording each exposure was only ½ minute. The spool contained enough film length, for even the deepest bore hole.

The above equipment was suitable for bore holes of diameters exceeding 6⅝ inch (OD). In the later models, a miniature gyro, employing an electro-mechanical device, for expensating precession, is utilised. The instrument can be used in 3 inch diameter bore holes, and very soon the manufacture of instruments, for use in holes of BX size can be expected.

The Masas compass consists of a combination of both the inclinometer, using a dilute solution of hydrofluoric acid (about 12 parts of water to 1 part of acid), and a compass needle. The instrument comprises a 6 inch long glass tube, with a median partition (1) between the upper (2) and a lower portion (3) Fig.1.47. The lower chamber which is fitted with a gutta-percha stopper, is partially filled with H.F. solution. The upper chamber is filled with a solution of 1 2/3% gelatine. In the chamber is a combination which consists of a small magnetic compass (*b*) fitted with a guard ring, (*c*) suspended from a float (*a*). Tha combination is placed in a bronze casing, which is attached to the string of rods and lowered into the bore hole. Sufficient time is allowed, for the gelatine to set, and for the acid to etch the glass. The setting time for the gelatine varies from 20 minutes to about 50 minutes. The relation between the level (surface) of the galatine, to the axis of the compass tube, gives the angle of inclination. The orientation of the tube axis, in relation to the compass

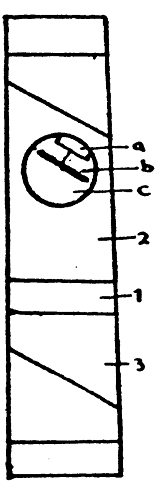
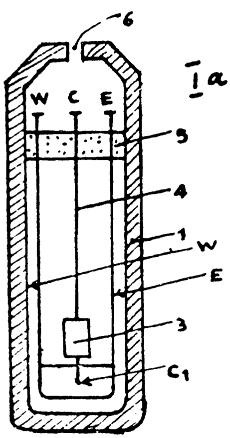


Fig. 1 47

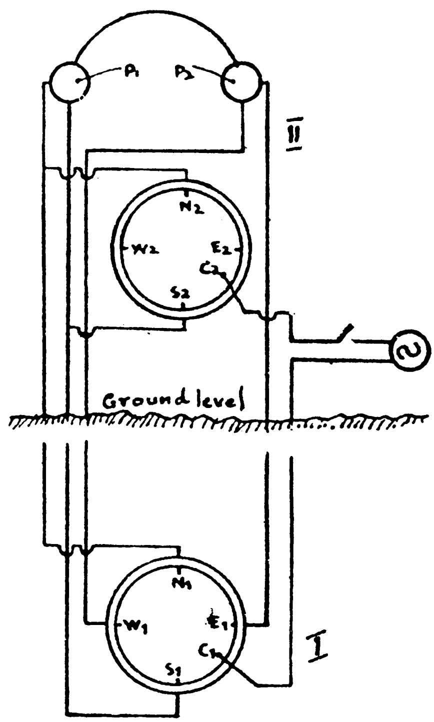
(magnetic) needle, gives the bearing of the inclined hole.

The Briggs Clinophone Fig. 1.48 is an instrument which measures deflections with a high order of accuracy. The instrument consists of two portions, I and II, the former is lowered into the bore hole while the latter remains on the surface. The first portion Fig. 1.48 I*a* consists of a brass (cylindrical) body (1) with a bakelite cup at the bottom (2). In the cup are four terminals $N_1W_1S_1E_1$ Fig. 1.48 I. A plumb bob (3) in suspended from a insulate plug (5) by means of the wire (4) which terminates in point C_1. The terminals in the cup are connected to connectors N, W, S, and E in the insulator plug (5), and C_1 to C_2.

The portion II (on the surface) Fig. 1.48 II consists of an insulator cup in which there are four corresponding terminals N_2, W_2, S_2 and E_2 immersed in an electrolyte as well as additional point C_2. A 5-wire lead connects the corresponding terminals of II to I (at the connectors attached to the insulator (5) N_1 and N_2, S_1 and S_2, W_1 and W_2, E_1 and E_2, and C_1 and C_2).



I
Fig. 1.48



II
Fig. 1.48

N_1, N_2, S_1, S_2 are connected to the ear phones P_1 while E_1, E_2, W_1, W_2 are connected to the ear phone P_2. The moving terminals C_1 and C_2 are connected to an alternating frequency generator (of audio frequency range).

Since the N—S and E—W terminals are separately connected to the ear phones P_1 and P_2, they form, as it were, reference coordinate for the location of the moving terminal C_1 attached the plumb bob (3). In operating the instrument, sufficient time is allowed for the plumb bob to come to rest. The alternating frequency generator is started up, and the switch P is closed when the operator wears the ear phones. The terminal C_2 in Fig. 1.48 II *i.e.*, upper portion of the instrument, is moved from place to place, within the cup, and the intensity of the tone in the ear phone is noted. It is only at one point that the volume of the note, generated by both ear phones, is identical. The location of C_2 in respect of $N_2W_2S_2E_2$, in portion (II) will be identical to that of C_1 in respect of $N_1W_1S_1E_1$ in the portion (I) of the instrument. Thus the inclination and the bearing of the hole can be made out.

Bore hole surveying is an essential feature of all drilling programmes, where deflections in angle and deviation, in the bearing, are expected. Jointed rock, or rock formations, in which the units are of varying degrees of hardness, can cause deflections and deviations. Joints, steeply dipping sills of basic rock, and faults are other causes of deflections. Whatever, the cause, the geologist is always at a loss as he is not sure of the exact structural orientation. Due to deflections, there is an apparent variation in the amount of dip, as seen from the cores, and the ore body or oil reservoir may be missed completely if such errors are not eliminated. Thus surveying becomes essential. With the help of the surveying data, the location of the end of the core hole, in relation to the collar, can be obtained. The depth of the various points, the inclination and direction, as obtained, are plotted and this gives a complete picture. Fig. 1.50 gives the section along the bore hole whereas Fig. 1.49 gives the plan of bore hole.

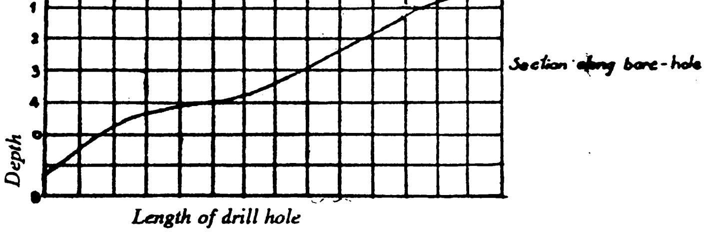


Fig. 1.49

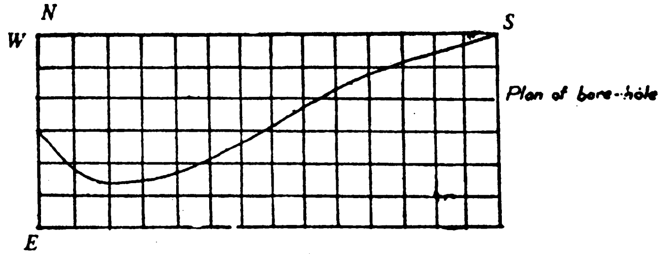


Fig. 1.50

MISCELLANEOUS CALCULATIONS AND ADJUSTMENTS

Corrections to be applied to distance measurements with steel tape for base line measurements.

(a) Slope correction.

If Sc = Slope correction
L = Slope length
h = Difference in level between the two ends
D = Horizontal distance

the $Sc = D - L + -\frac{h^2}{2L} - \frac{h^4}{8L^3} - \frac{h^6}{16L^5} - \frac{5h^8}{128L^7}$

(Tables are available for this).

(b) Tension correction.

If the additional tension applied is T lbs.
S = Cross sectional area in sq. in.
L = Length of tape in ft.
C = Increase of length due to extension of tape in ft.
E = Young's modulus

$C = \frac{L \times T}{E \times S}$; Young's mod. of steel = 28,000,000.

(c) Sag correction.

Sa = Sag correction
C = Actual length
L = Sagged length
h = Sag
t = Tangential tension at the bottom of the sag.
W = Weight per foot lengh of tape.

$$Sa = -\frac{W^2L^2C}{24t^2} = \pm \frac{W^2L^3}{24t^2}$$

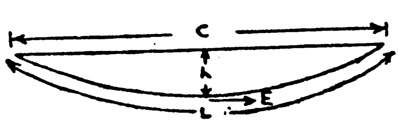


Fig 1.51

(d) Temperature correction.

This depends on the coefficient of linear expansion of steel, which is normally provided by the makers.

Permanent adjustments

These adjustments are required to be carried at intervals, after a period of use for verifying that the performance of the instrument is upto the desired standard.

A. **Dumpy level :** After carrying out the temporary adjustments, it is essential to check ;

1. whether the axis of the bubble tubes, lies in a plane, perpendicular to the vertical axis of the instrument,
2. whether the horizontal cross hair is really horizontal,

and 3. whether the line of sight is parallel to the axis of the bubble tube.

For the purpose of carrying out the various adjustments,

1. The instrument is set up firmly on a ground which is nearly level, and temporary adjustment is carried out. If the long bubble, on the telescope, does not remain in the centre of its run, on a rotation of 180°, between two foot screws, then half the error is corrected by raising or lowering the capstan nut, at one end of the bubble tube and the remaining half adjusted by moving the foot screws. This procedure is repeated until the bubble does not move from the centre.

The telescope is now turned through 90° and brought over the third foot screw. If the bubble leaves the centre of the tube, half the discrepancy is adjusted by the capstan nut, at the end of the tube, and the remainder corrected by moving the single foot screw. It may be necessary to repeat the operations, until horizontality is achieved, when the bubble remains in the central position during a complete rotation through 360° of the telescope.

2. In order to make the horizontal cross wire actually horizontal, any point, within the field of view and lying on the cross hair is selected. The telescope is moved on the horizontal axis and the movement of the point, in relation to the cross hair, is observed. In case, the point is found to move away from the cross hair during rotation, the cross hair is not horizontal. The four set screws holding the ring and containing the cross hairs are loosened and the ring is rotated in the required direction, so as to eleminate the observed "tilt", and the point under observation moves on the cross wire, when the telescope is moved.

3. In this adjustment, the line of sight is made parallel to the axis of the bubble tube, so that when the instrument is levelled, the line of sight will lie on a horizontal plane.

For this purpose, two stations A and B are pegged down on a nearly level stretch, about 300 ft. apart. The instrument is set up in a central position, between A and B and temporary adjustments completed. A levelling staff is set at A and the reading taken. Similary, the reading on a staff placed at B is also noted. Since A and B are equidistant, from the instrument and exactly 180° apart, the difference in level can be readily obtained as seen from the Fig. 1.52. Thus the readings obtained indicate the relative levels of A and B.

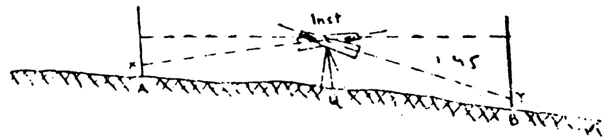


Fig. 1.52

Now the instrument is moved to a point P in alignment with A and B, Fig. 1.52 (*a*) The staff at A is sighted, and its reading (X_1) noted. Since the difference in level between A and B is known the corresponding reading on the staff at (Y_1) can be calculated.

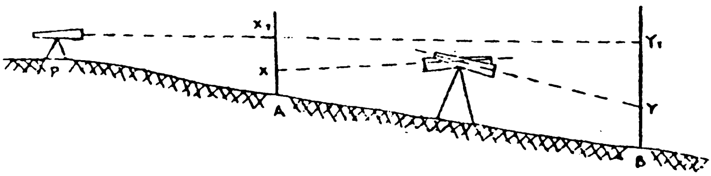


Fig. 1.52(*a*)

The cross hair ring is, now, so adjusted that the intersection coincides with X_1 on the staff at A, and Y_1 on the staff at B.

Example

Instrument on station Q.
Readings on the staff at A = 5.32
Readings on the staff at B = 7.54
Difference in elevation between A and B = 2.22
Instrument at station P.
If the reading on the staff at A = 8.42
Then the reading on the staff at B should be 8.42+2.22 = 10.64.

B. Theodolite : The accuracy of the instrument depends on its condition also, hence regular checking is necessary for rectifying any defects.

1. To make the axis of the plate bubbles horizontal, *i. e.*, lie on a plane normal to the vertical axis of the instrument.

The adjustment is identical, to that described for the dumpy level (adjustment 1) ; except that the adjustment is carried out in respect of both the plate bubble tubes, which are arranged at right angles to each other.

2. (*a*) To make the vertical cross hair vertical to the horizontal axis of the instrument. This is an important adjustment since the theodolite is an angle measuring instrument.

(*b*) To make the horizontal cross hair really horizontal. This adjustment is also identical to that carried out on the dumpy level (adjustment 2), (Figs.1.51 and 1.52)

The instrument, after adjusting the bubble tubes, is focussed on any convenient mark at about 100 or 200 ft. away, for adjusting the vertical cross hair.

After focussing a point on the vertical cross wire the telescope is moved up and down, on the horizontal axis. If the point moves away from the vertical cross wire, then the ring holding the cross wire is adjusted, so that coincidence is maintained between the point sighted, and the cross wire at all positions of the telescope.

3. To make the axis of collimation, coincident with the optical axis of the telescope, so that the image of the object formed by the objective lies at the intersection of the cross wires.

The instrument is set up and adjustments (1) and (2) described above are completed. Keeping the telescope nearly horizontal, station (A), at a distance of about 250 ft. or 200 ft., is sighted and brought into focus, without parallax. Both body and vernier plates are clamped. The telescope is transitted and made more or less horizontal. Any station (B), at a distance of about 200 ft., is fixed in the alignment of the cross wires. The body clamp is released and station A sighted back with body clamp and tangent. The telescope is again transitted. If the instrument is in adjustment, station B should again lie on the cross wires, if not, another station (C) is fixed at about the same distance from the instrument, say about 200 ft. away. The distance between the two stations B and C is measured and at ¼ this distance station C_1 is located ($CC_1 = \frac{1}{4}BC$). The screws, holding the cross wire ring, are loosened and the vertical cross wire is moved to coincide with C. The procedure is repeated, and if on transitting station B falls on the cross hair, the instrument is in adjustment.

4. To make the horizontal axis of the telescope truly horizontal. The instrument is set on a nearly plain ground and adjusted as described above.

A point (Q) at a height of about 50 ft. above the instrument is sighted say *e. g.*, the top of a building. The telescope is lowered and the

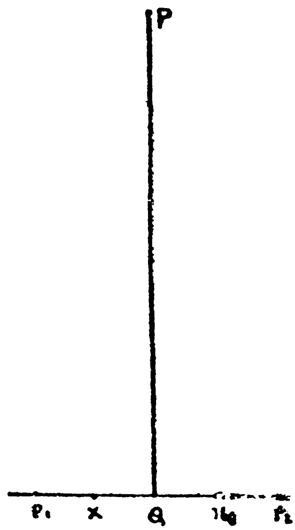


Fig. 1.53

point is transferred to the ground (P_1). The instrument is transitted and the point Q is sighted again, and the telescope lowered to intersect the ground. If the point (P_2) is not coincident with P_1 the instrument requires adjustment.

The adjustment is affected by means of the screws, found on the standard, just below the bearing of the horizontal axis. The screw is raised or lowered so that the vertical cross wire is moved from P_2 to X_2.

The horizontal distance P_1P_2 is measured and $\frac{1}{4}$ P_1P_2 is taken from P_2 so as to fix point X_2.

The procedure is repeated, so that on transitting the point P, when sighted and transferred to the ground, is constant at Q'.

5. The bubble tube fixed to the vertical circle is now adjusted, as indicated for the adjustment of the longer bubble tube (under dumpy level). Now, when the reading on the vertical circle is "O", then the bubble tube on the telescope should be in the middle of its run.

NOTES ON INSTRUMENTS AND ACCESSORIES

(1) The Automatic Level

Sketch showing the principle of an automatic level.

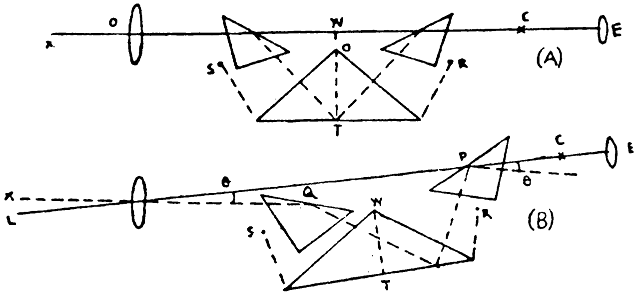


Fig. 1.54

In the self adjusting level, the coarser adjustment is carried out by means of the 3 foot screws and the bull's eye or circular bubble. When this is completed, the line of sight remains horizontal, even though the axis of the telescope is tilted. Figure (1.54) above illustrates the principle.

O is the objective. W is a pendulum with counter weight.

E is the eye piece. C is the position of the cross wire.

P, Q and T are reflecting prisms.

Prisms P, Q and T are enclosed in a darkened box, as also the pendulum. The prism T is suspended by wires of equal lengths, from the box at points S and R, while prisms P and Q are adjusted so as to be in the axis of the telescope.

When the telescope is horizontal, the line of sight coincides with the axis of the instrument, as shown in Fig. 1.54 A, and the line connecting the points of suspension is horizontal, *i. e.*, they lie on a line parallel to OE, the line of collimation.

When the telescope is tilted by $\theta°$, Fig. 1.54 B the two prisms P and Q move together through angle θ, while prism T moves relatively but also through θ. Since the reflecting surface of T is parallel to the points of suspension R and S, which are fixed to the dark box lying within the telescope tube, any movement of the telescope produces the same displacement of the reflecting surface T.

Thus in Fig. 1.54 B if the axis of the telescope is tilted by $\theta°$, the reflecting surface T tilts by $\theta°$ so that the line of sight OX remains horizontal. The angle between the line of sight XO, and the axis of the telescope is also θ.

(2) Levelling : Sensitivity of the bubble tube is an important factor in levelling.

In order to determine the sensitivity of the bubble the following procedure is adopted.

A levelling staff is held at any convenient distance.

The instrument is levelled such that the bubble is 3 to 4 divisions away from the centre. The reading of this end of the bubble is noted, and the reading on the staff is recorded.

The instrument is again adjusted in such a manner that the bubble is made out of centre by 3 or 4 divisions in the opposite direction. The staff reading is recorded again.

$$\text{Sensitivity} = S = \frac{1}{0.00000485} \frac{d}{Ln}$$

where d = difference in staff readings.

L = distance between centre of the instrument and staff.

n = total number of divisions through which the bubble has moved.

S = sensitivity in seconds per bubble division.

Note :—Difference between the 3 foot screw and 4 foot screw levels is, that for any setting of the stand or tripod the instrument height is not changed by adjustment, in the case of a 4 foot screw instrument. This is not so with the 3 foot screw instrument.

(3) Optical micrometer or telemeter comprises a thick circular disc of glass in which both the surfaces are optically plane and are parallel to each other (*i.e.*, plano-parallel).

The glass plate is mounted diametrically within a cylindrical holder, on which it is capable of being rotated by means of a graduated drum or micrometer (Fig. 1.55 A).

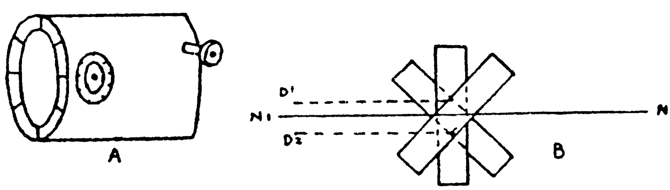


Fig. 1.55

When the micrometer is attached to the telescope, the surfaces of glass plate are normal to and, intercept the line of sight. Since the surfaces of the plate are parallel, the line of sight is, deviated parallel to itself according to the laws of refraction as shown in Fig. 1.55 B.

N_1N is the normal ray, D_1 and D_2 are deviated rays.

(4) **Horizontal stadia:** In the instrument, so far described the stadia hairs (or wires) are horizontal and are placed across the vertical cross wire. Instruments have been made, where vertical stadia

hairs are placed on the horizontal cross wire. When using such instruments for stadia measurements. (1) The staff is held in a horizontal position. (2) It is made level. (3) It is at right angles to the line of sight. (4) It is fixed at the same height as that of the instrument above ground level at the point.

It wili be seen that the stand for the staff is of a special design. In the tripod stand, the rod carrying the staff forms one of the legs and this is planted over the station point. This vertical leg is also used for intersection and for obtaining the angles in azimuth, as well as vertical angles, as no staff is placed over the station. Levelling of the horizontal staff is carried out by means of the level tube or bubble provided.

(5) **Distance wedge:** The dislance wedge is an attachement (Fig. 1.56), which enables horizontal stadia measurements. It consists of two thin wedges of glass, making an achromatic combination. The wedge covers a narrow strip in the field of view of the objective. When viewing

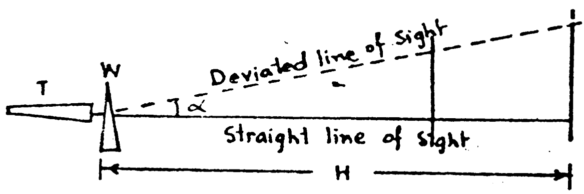


Fig. 1.56

through the telescope it is seen that, due to the presence of the wedge, the images of objects found, either to the left or right of the line of sight, are super imposed on those lying in the field of view. In effect, the wedge serves to (1) deflect or deviate the line of sight, (2) to super-pose the images of the objects on the deviated line of sight on to those of objects on the straight line of sight.

Viewed in azimuth (Fig. 1.51) any point Y appears along side X. The glass wedge is so constructed that, if H is the distance from the wedge to an object X and Y is an object (at distance D from X) the deviated image of Y falls on X.

In most instruments the ratio

$$D/H=\frac{1}{100}$$

$$H=D\times 100.$$

The distance wedge is used with a special staff, which has a vernier attached to it. When sighting, through the telescope on the station (*i.e.*, the vertical leg of the staff stand) and the staff, it will be seen that one end of the staff is superposed on the other end, due to deviation caused by the wedge. Thus for, *e.g.*, 1.00m may appear against 7.00m. or 2.00m may appear against 6.00m etc. etc. In the first case the stadia reading is 6.00m. In the 2nd case the stadia reading is 4.00m. etc. Thus the horizontal distances are 600m and 400m. respectively. The vernier is used to read up 0.01m.

(5a) In some distance wedges a plane parallel glass plate is provided. The plate is placed in front of the wedge and can be rotated about a vertical axis. The effect of this rotation, is to cause the deviated ray (passing through the wedge) to move parallel to itself as shown in Fig. 1.57.

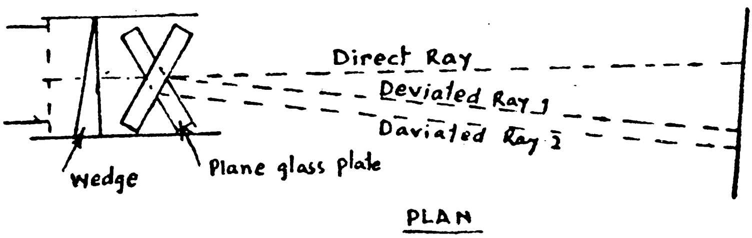


Fig. 1.57

The formula for horizontal distance is

$$H=100S\ \cos\theta$$

The formula for vertical height is

$$V=100\ \sin\ \theta$$

where, S = Stadia reading

θ = is the angle of elevation or depression of the telescope (*i.e.* vertical arc reading).

The distance wedge is more accurate than normal stadia measurement as

(1) both ends of the staff are read simultaneously,

(2) neither cosine corrections nor addition constants are needed as the staff is held horizontal.

(6) **Notes on Watts Microptic Theodolite :** The auxiliary tube on side is the reading microscope. The field of view presents, both the horizontal scale reading designated by "H", and the vertical circle reading designated "V".

The upper window, gives directly the main scale graduations which are at 10 minutes intervals. The lower window shows 2 sets of readings, (*a*) the scale of the top read directly in minutes and can be approximated to the nearest minute while (*b*) the scale below reads directly in seconds. Thus the reading indicated in Fig. 1.58 is 72°06′30″.

The vertical scale is also similarly divided.

The micrometer scale seen in the lower window is calibrated thus :

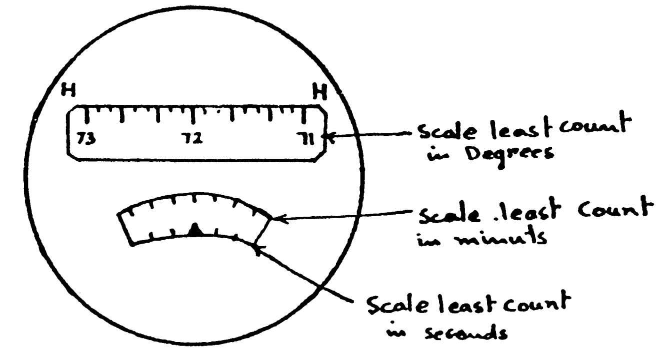


Fig. 1.58

the outer circle reading in minutes is divided into 60 parts, and each division is subdivided into 10 parts, *i e.*, 1 subdivision = 1 minute, as the main scale division is 10 minutes. The lower part of the scale is also divided into 60 parts, *i.e.*, reading directly in 1″.

The micrometer is so adjusted, that when the reading of the scales is "0" minutes and seconds, *i.e.*, the readings in the lower arcuate scale, are zero and the reading on the upper main scale is a complete number of 10 minntes, *e.g.*, 28°10′00″, 28°20′00″, 28°30′00″ etc. Consequently, any reading other than "zero" on the minutes and seconds scales indicates a departure of main scale from coincidence with the indicator mark. The micrometer scale is actuated by a knob, the movement of which causes the main scale reading to be altered so as to coincide with the nearest 10′ reading. This operation also involves a change in the minute and seconds scales from which the angular value can now be read off in minutes and seconds.

(7) **Self reducing tacheometers** : In these instruments the usual stadia wires are replaced by a set of tacheometric curves. When looking through the telescope, the tacheometric curves are seen to occupy one half

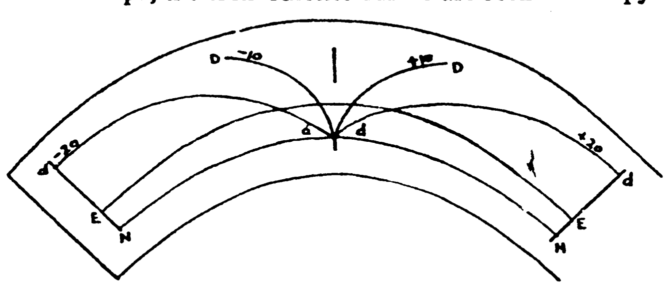


Fig. 1.59

of the field of view. The curves are etched on a transparent glass laminate, housed in the vertical supports, in the same manner as the vertical arc (Fig. 1.59).

The tacheometric curves comprise the reference curve (N-curve), which constitutes the "zero", for both the horizontal measurements and the vertical measurements. Alongside N-curve, at some distance, is the E-curve on which measurements for horizontal distances are made. The curves for measuring vertical height, consist of 2 pairs of companion curves, designated $+$D or $-$D for the lower range (*i.e.*, ± 10 factor) and $+$d or $-$d for the higher range (*i.e.* ± 20 factor).

The N-curve which is concentrically placed, with reference to the horizontal axis of the telescope, is so adjusted that it represents the horizontal reference line, and forms a tangential contact with the horizontal cross wire.

The curve E can also be identified, (in the field of view) as being nearly concentric to curve N, while the curves $\pm$D and $\pm$d bear a marked cross cutting relationship to both the E and N curves. (Fig. 1.59).

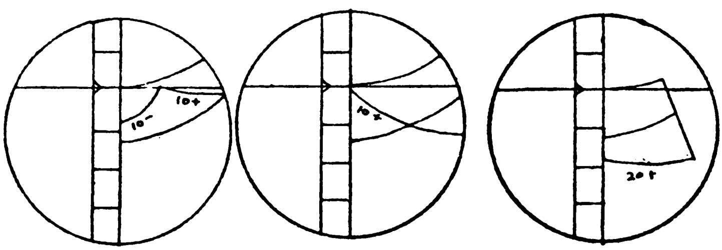


Fig. 1.60

The basis for working out the E, D and d curves in relation to the N curve is as follows :

Intercept between N and E $= i \cos^2\theta$

" " N and D $= i \sin\theta \cos\theta$

" " N and d $= i \sin\theta \cos\theta$

where i is the distance of separation of the stadia hairs

when $f/i = 100$

or $i = f/100$

$\theta =$ vertical angle.

Note : (*a*) the N curve is tangential to the horizontal cross wire,

(*b*) when an angle of depression is indicated, only the negative counterpart of the d or D curves comes against the staff, (Fig.1.60).

(c) when an angle of elevation is indicated, the positive counterparts of the same curves abuts against the staff.

(d) The N curve is tangential to the horizontal cross wire.

(e) Intercept on the staff subtended by the N and E curves gives the horizontal distance factor.

(f) The intercept between N curve and D or d curve gives the vertical height.