UNIT 1 THEODOLITE TRAVERSING

Structure

1.1 INTRODUCTION

The introduction of theodolite as an essential equipment for any exhaustive, accurate and extensive survey exercise like triangulation and precise measurement of horizontal and vertical angles, contouring and even measuring linear distances under difficult terrain conditions has already been covered in the first course on survey.

You were introduced with the details of various elements of a theodolite instrument, the setting of the instrument at survey station, its temporary and permanent adjustments etc. which enable you to use theodolite for normal survey exercise. The simple traversing using chain and compass, plane table and with the theodolite was introduced in Elements of Surveying in previous semester. However, the principle of traversing, the problems associated with general traverse surveying processes and the error adjustments are explained here in greater details.

In this unit, you will be introduced with more intricate details of the instruments, their prominent commercial variance and recent developments. The details of temporary and permanent adjustments required in an instrument and their importance etc. are explained in greater details with emphasis on traverse adjustments and computations. Having undergone through this study, the student will be able to understand the basic principles of traverse surveying, the correct way to record the observations in traverse table field work, checks and errors, omitted measurements and methods to account for them and the computations involved.

With the study of this unit, you will be able to appreciate the advantages and intricacies of accurate surveying using a precision instrument like theodolite.

Objectives

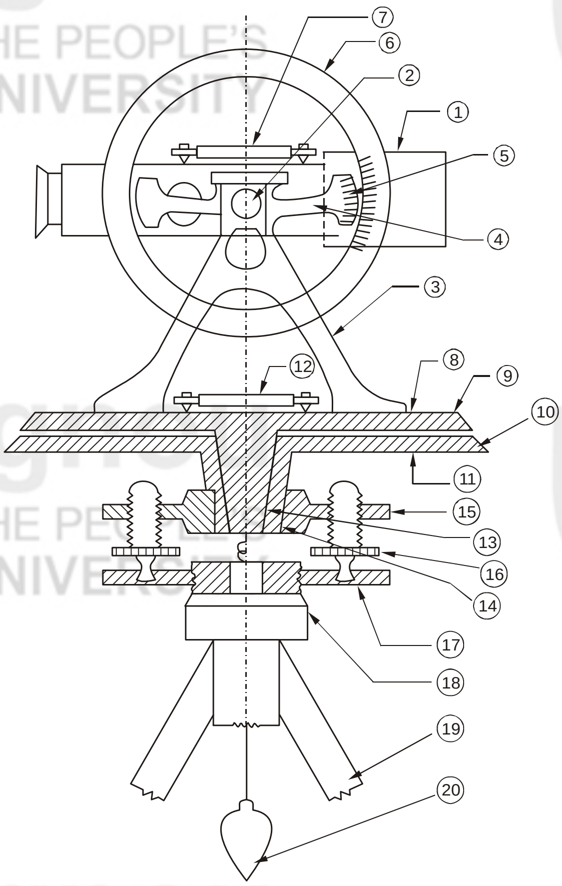
After studying this unit, you should be able to

- conceptualize about adjustments,
- understand various methods of traversing,
- understand traverse adjustments and measurements, and
- conceptualize about traverse computations.

1.2 INSTRUMENTS

Optical Theodolite

The basic construction of the general transit theodolite was described in Elements of Surveying in Unit 6. This type of general theodolite is also termed as direct reading theodolite. The readings in this type of instrument are read directly either by eye or with the aid of a low power Microscope, e.g. scale readers against the verniers or using micrometer microscopes. However, it was discovered later that it is possible to etch much finer lines on glass rather than on brass or silver. The light can pass through glass scales and can be refracted by a system of lenses and prisms along almost any desired path. It is possible to present the much fine readings of the scales to a microscope attached to the telescope barrel or mounted on the index arm (Figure 1.1).



(1) Telescope
(2) Trunnion (Horizontal) Axis
(3) Index Standards
(4) Index Arms
(5) Vertical Circle Vernier
(6) Vertical Circular Scale
(7) Spirit Level
(8) Upper Horizontal Vernier Plate
(9) Horizontal Circular Vernier
(10) Horizontal Circular Scale
(11) Lower Horizontal Plate
(12) Spirit Level
(13) Inner Axis
(14) Outer Axis
(15) Levelling Head
(16) Levelling Screws
(17) Foot or Tribach Plate
(18) Tripod Head
(19) Tripod Legs
(20) Plumb Bob

Figure 1.1

The possibility to etch very fine lines on glass also implies that the circular scales can be greatly reduced in size. In some instruments only 50 mm dia circular scales are used, with same accuracy which was achieved by 900 mm diameter scales. The representative typical reading along with the micrometer reading is shown in Figure 1.2 upto an accuracy of half of a second.

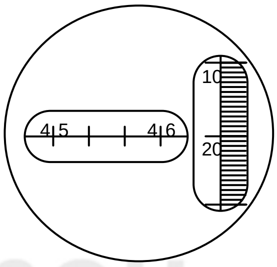


Figure 1.2

In standard optical theodolites, only one end of each scale is read as opposed to the two vernier readings of the direct reading theodolites. However, in more accurate type of instruments, each scale is read at opposite ends of a diameter and also the mean of these two readings with the help of special optical devices.

The advantages of optical theodolite are its smaller and lighter sizes, and the speed with which the observations can be taken and recorded.

Gyro Theodolite

A gyroscope is a device which is constrained to lie in a horizontal plane by suspending it (Figure 1.3) and then spun. The earth's rotation causes the oscillation of gyroscope's axis and brings it in the direction of the true north. The gyro attachment can be mounted on a theodolite. It is attached with Ni-Cd batteries and electronic device to spun the gyro spinner. The attachment is suspended on a thin metal tape and hangs like a plumb bob spinning at about 22000 rpm about an horizontal axis. The spinning plane is maintained in its original position by the rotation inertia influenced only by earth's spinning motion. Thus, the earth's gravity and spinning inertia keeps the spin axis oscillation until it takes the direction of meridian plane. However, the gyro axis takes a long time to come to this equilibrium position.

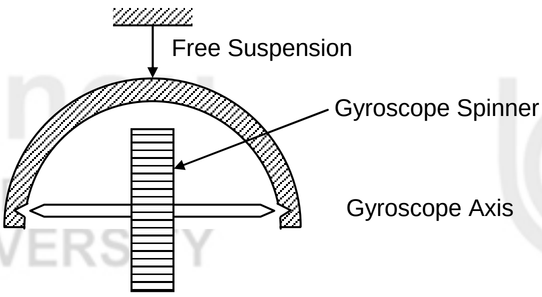


Figure 1.3 : Gyroscope

Gyro theodolites have many advantages in field astronomy. Good weather and accurate clock times are required during field astronomy survey readings for azimuth. The compass readings are liable to gross errors due to local disturbances in earth's magnetic fields. Complex and laborious calculations increase the chances of computational errors while gyro mounted theodolites can give very accurate azimuth readings, within a standard deviation of

± 20 seconds in less than 20 minutes.

1.3 ADJUSTMENTS

1.3.1 General

The basic operations required in any surveying exercise undertaken with a theodolite are discussed in detail in **Elements of Surveying** course. There are two types of operation required for adjustment of any theodolite, e.g. **temporary** and **permanent**. Temporary adjustments are those which are required to be undertaken at every new set up of the instrument at each survey station before starting to make any observation. (Section 6.3 of Elements of Surveying). These include

(a) Setting up,

(b) Levelling the instrument at site, and

(c) Focusing the eyepiece and object lenses, i.e. eliminating the parallax.

Fixed relationships also exists between the fundamental axis of the instrument. These basic instrument axes are

(a) Vertical axis,

(b) Plate levels axis,

(c) Line of collimation,

(d) Trunnion axis or horizontal axis, and

(e) Azimuthal axis or bubble line of the altitude level.

These relationship are established with the help of instrument adjustments known as permanent adjustments. Once made, they remain to hold for long periods for many settings of the instrument (Unit 6 of Elements of Surveying).

1.3.2 Temporary Adjustments

As stated earlier temporary adjustment consists of

(a) Setting,

(b) Levelling, and

(c) Parallax removal.

Setting

The vertical axis of the instrument shall be located exactly above the survey station position marked by a peg permanently fixed in ground. The top of the peg is normally marked with a cross by permanent paint. In normal theodolites, a hook is placed in the centre of tripod stand representing the position of vertical axis of the instrument. A plumb bob is suspended from this hook with the help of a strong thread.

The instrument assembly is set on the firm ground and tripod legs are manipulated to be approximately over the station point. The legs are then moved sideways and/or radially to bring plumb bob exactly over the cross junction on peg while maintaining tribach horizontal. In more refined theodolites, optical plummet is used for centering in place of plumb bob assembly for better accuracy. A centering plate mounted on tripod can also be used for rapidly centering the instruments.

Levelling

To ensure that the horizontal circle does lie in a true horizontal plane which is normal to vertical axis of the instrument, the theodolite is levelled. This is done with the help of leveling screws and plate bubbles. Normally, the instrument has three leveling screws and two plate bubble tubes. The upper plate of the instrument is rotated until one of the bubble tube is parallel to the line joining two leveling screws. While the second bubble tube will be normal to this line. The bubble of the first tube is brought to central position by moving the corresponding pair of leveling screws simultaneously. The third screw is then manipulated to bring bubbles in second bubble tube midway of its run. This movement may cause disturbance in position of first bubble. The process of leveling is then iterated until bubbles of both the tubes remains locked up in central position in all rotations of upper horizontal plate. This will ensure perfect horizontality of horizontal circle and makes instrument's vertical axis truly vertical.

Parallax Removal

It consists of focusing of the eyepiece and object lens so that the foci of the eyepiece and object lens coincide the cross hairs plane. As a first step, a piece of white paper is placed in front of the object lens and eyepiece screw is manipulated to move eyepiece in or out of instruments tube until the cross hairs are distinctly and clearly observable. This process ensures that eyepiece is locked in focused condition. As a next step, the telescopic tube is directed towards a distinct object and the focusing screw is turned until the object's image appears sharp and clear. This step may be required every time the distance between the object and instrument changes while making observation. This ensures that the image of object is formed in the plane of the cross hairs.

1.3.3 Permanent Adjustments

As explained in Elements of Surveying (Unit 6), the fundamental axes of the theodolite can be identified as :

- vertical axis,
- axes of plate levels,
- line of collimation (also known as line of sight),
- trunnion axis (or horizontal axis or transverse axis), and
- bubble line of the altitude level (or azimuthal axis).

For an instrument to give reliable and accurate observations, certain definitive relationships must exist between the above fundamental axes of the instrument. These relationships must also be maintained during the entire surveying exercise. It may be noted that these relationships are the properties of the instrument and do not change with survey station positions.

The relationships which must exist between fundamental axes of the instrument can be listed as follows :

(a) The plate levels axis is normal to vertical axis.

(b) The horizontal axis is normal to vertical axis.

(c) Line of collimation must be perpendicular to horizontal axis.

(d) The telescope's axis must be parallel to line of collimation.

In addition to above relations, the well adjusted theodolite should also meet following requirements to make the instrument working easily and smoothly.

(e) The only movement of one part relative to another should be along a circular arc. There should not be any backlash, whip or looseness.

(f) The verniers of a vernier type theodolite shall be diametrically opposite to each other. The vertical circle vernier should read zero when the instrument is levelled.

(g) The geometric centres of vertical circle and trunnion axis should coincide as should the geometric centres of the axis of the horizontal plates and vertical axis of the instrument.

The new instruments are checked for all these requirements before marketing. However, old instruments get wears and tears during usage and require to be serviced by competent instrument mechanics at regular intervals if high degree of accuracy is to be maintained.

Some procedures to be adopted for testing these requirements and subsequent adjustments where necessary will now be described.

Horizontal Plate Level Test

This shall be conducted to test that vertical axis of the instrument is truly vertical when the horizontal plate spirit level bubbles are central. It must be noted that horizontal is an important reference plane when the results of one station are related to observations made from other stations. It is necessary, therefore, that upper and lower horizontal plates are oriented along this plane. The manufacturer always ensures that the vertical axis and horizontal plates are mutually orthogonal.

To start any adjustment, it is essential that diaphragm in the telescope is truly vertical, to ensure that vertical and horizontal hairs on diaphragm are truly vertical and horizontal respectively. The instrument is erected and levelled carefully on a firm ground. A well defined object is sighted, e.g. the electric pole or corner of a building. Both horizontal and vertical rotations of telescope are clamped in this position and the telescope is rotated in vertical plane by corresponding tangent screw. If the sighted line moves along the vertical hair, the verticality of vertical cross hair is ensured. If not the diaphragm screw is loosened and diaphragm rotated to ensure verticality. Then the screw is retightened. For test (a), clamp the lower plate. With levelled instrument, rotate the telescope through 180^o in a horizontal plane. The plate spirit bubbles must remain central to ensure that horizontal plate is truly horizontal. If it is not so, then adjustment is required.

Adjustment

Bring the axis of the telescope in line parallel to the line between two leveling screws. The telescope spirit level (altitude level) is centralized using the vertical circle clamp and tangent screws. If the spirit level is on the index arm, the bubble is centralized using levelling screws.

Turn the telescope about the vertical axis through 90^o and centralize the relevant spirit level bubble using the third leveling screw. Repeat the process until the bubble remains central in these two positions.

Next, rotate the telescope horizontally through 180^o. If the bubble does not remain central, carefully note the deviation of the bubble (say *n* divisions). The bubble is then returned half way to the centre ($n/2$ divisions) with the help of corresponding levelling screws. The telescope spirit level bubble is centralized using clip screws or the vertical circle tangent screw. Clip screw is used in case of index arm spirit level.

Turn the telescope through 90° until the bubble is over the third leveling screw and centralize it using only this screw. The entire process, as above, is repeated until no further adjustment is contemplated. The plate spirit level bubbles are now centralized by adjusting the capston headed screws used for fixing the levels to horizontal plate. When the above adjustment is completed, all the bubbles will traverse during a complete revolution of the telescope ensuring that the instrument's vertical axis is truly horizontal. It must be emphasised here that the rotation of telescope through 180° had caused a deviation of "*n*" divisions. This is termed as apparent error.

It is twice the value of the actual error in the level axis. Hence, it may be noted that correction was made only for half the value of apparent error (*n*/2 divisions).

After performing this adjustment, one more test may be conducted to ascertain that both the inner axis and outer axis of the instrument are parallel. In the adjusted instrument, the lower plate is unlocked while the upper (vernier) plate is clamped. If in this position the bubble does not traverse during 180° rotation, it indicates that outer axis is not vertical. If the error is large the instrument cannot be adjusted and warrant repairing.

Collimation Test

This test is conducted to check whether the line of collimation coincides with the optical axis of the telescope. It simultaneously checks whether the line of sight is perpendicular to trunnion axis or not. If the line of sight passing through cross hair intersection does not coincide with the optical axis and is not perpendicular to trunnion axis observational errors will creep in (Figure 1.4).

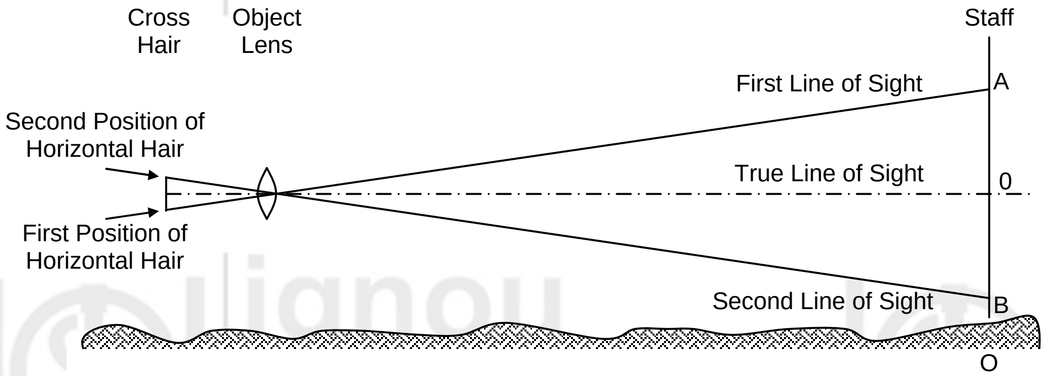


Figure 1.4 : Collimation Test (Horizontal Hair)

We can have four sub-tests under collimation test, namely

(a) Horizontal hair, angular displacement,

(b) Vertical hair, angular displacement,

(c) Horizontal hair, lateral displacement, and

(d) Vertical hair, lateral displacement.

Test (a) : Horizontal Hair, Angular Displacement

Erect the instrument on a firm ground and level it. Clamp the vertical motion. A staff is sighted at both the sides of the field of view using the upper plate tangent screw. Both the readings are same if the horizontal cross hair is truly horizontal, otherwise it is rotated and requires adjustment.

Loosen the capstan headed diaphragm screws, if adjustment is required. Rotate the diaphragm until both the above readings are same. Tighten the screws. If the cross hairs are etched on glass, this adjustment will ensure that vertical cross hair is also truly vertical.

Test (b) : Vertical Hair, Angular Displacement

In any case, whether the cross hairs are etched or not, this test must be carried out for better accuracy. A plumb line is hung in the field of view of telescope and verticality of cross hair is checked against this plumb line. If it does not coincide and the horizontality of horizontal cross hair is already checked and adjusted, the diaphragm under test is rejected and replaced. Tests (a) and (b) are repeated till, for a particular diaphragm, both these tests are simultaneously satisfied.

Test (c) : Horizontal Hair, Lateral Displacement

As shown in Figure 1.4, a staff is placed at about hundred meters from the instrument which is erected and levelled on a firm ground. Clamp all the rotations and record the staff reading (say '*A*') and the corresponding vertical angle.

Rotate the telescope through 180^o both horizontally and vertically. If the new staff reading for same vertical angle reading, as previously measured, is "*B*" and if "*B*" does not change with reading "*A*", lateral displacement adjustment of horizontal hair is required.

Adjustment

Slacken the diaphragm screws and move the horizontal hair vertically to intercept the staff reading at $(A + B)/2$, i.e. equal to $(OA + OB)/2$. Tighten the diaphragm screws once again and repeat tests (a) to (c). Iterate the test till $OA = OB$.

Test (d) : Vertical Hair, Lateral Displacement

Select a nearly level firm ground. Set and level the instrument at an instrument station *S*. Place a ranging pole or staff at location *A* nearly 100 m away from stations (Figure 1.5), clamp the horizontal rotation.

Turn the telescope through 180^o and place a second ranging rod *B* on the line of sight *SA* such that $SB \approx SA$. Place a measuring staff horizontally on ground at *B* normal to line of sight *SB* and note the vertical hair intercept at *B*.

Now, unclamp horizontal movement and rotate the telescope through 180^o and sight the station *A*. Swing the telescope through 180^o in vertical rotation and sight the staff placed at *B*. Note the vertical hair intercept once again which might be "*C*". If intercept *C* coincides with intercept *B*, the vertical hair is correctly aligned. If not, adjustment is required.

Adjustment

The deviation *CB* in the vertical intercept is recorded. After loosening the diaphragm screws, the vertical hair is moved laterally until staff intercept *D* is sited such that $CD = CB/4$.*

* [*In order to move the diaphragm, one screw of diaphragm is loosened while the diametrically opposite screw is tightened. The cross hair ring will move towards the loosened screw*.]

Test (d) is repeated until no adjustment is needed, i.e. *C* coincides with *B* ($CB = 0$).

After all the adjustments indicated above, i.e. from test (a) to test (d), these are repeated until no additional adjustment is required.

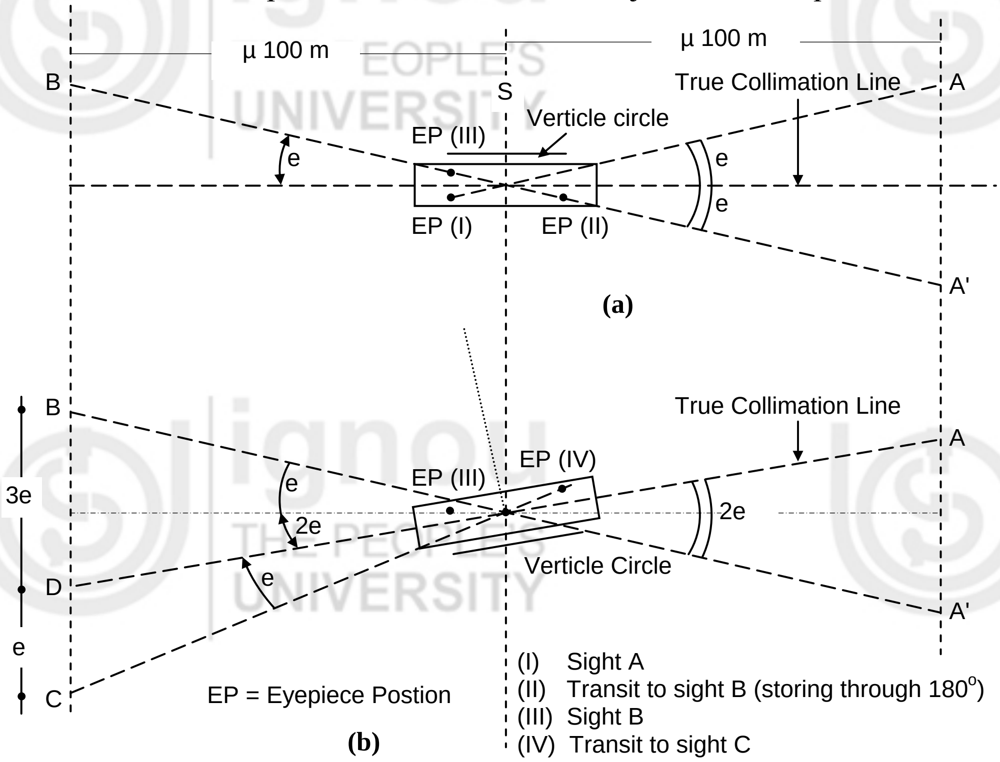
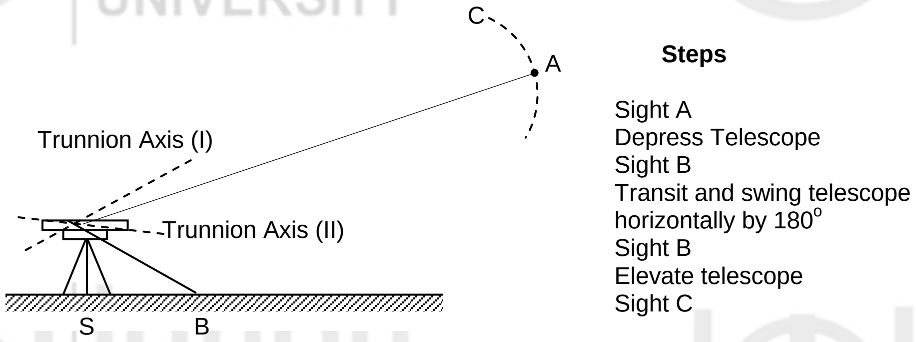


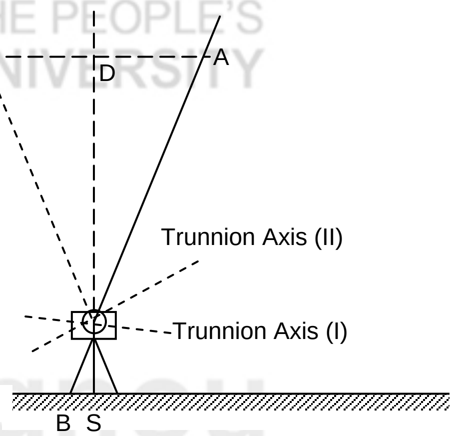
Figure 1.5 : Collimation Adjustment

Horizontal Axis Test

When the vertical axis of the instrument is adjusted for its true verticality, the trunnion axis shall be horizontal. This is essential for line of sight of telescope to trace the arc in a vertical plane when the telescope is swing in a vertical plane.



(a) Front View



(b) Side View

Figure 1.6 : Trunnion Axis Test

Set up the instrument at a station (say "*S*") and level it carefully. Sight a well defined point *A* at a considerable elevation, e.g. top of a pole or minaret. Clamp the horizontal plate's rotation. Rotate the telescope in a vertical plane and sight a position "*B*" on ground near to instrument station.

Transit the telescope, i.e. rotate the telescope horizontally through 180° and sight *B*. Clamp the horizontal plate movement. Elevate the telescope and sight *C*, an imaginary point at same elevation as *A*. If trunnion axis is horizontal, imaginary point *C* will coincide with real point *A*. If it is not so, then adjustment is required.

Adjustment

Using the trunnion axis adjustment screw the line of sight of telescope is moved in the direction of *D*, at a point midway between points *A* and *C*, i.e. $CD = 1/2\ AC$. Repeat the test until *C* coincides with *A*.

Telescopic Spirit Level Test

The axis of telescope must be parallel to line of collimation. This ensures that the line of collimation is horizontal when the telescope bubble is central. This test is essential when the theodolite is used as a level or is used for measuring vertical angles.

(a) A fairly level ground is selected and two pegs are driven along a line *AB*, where *A* and *B* are nearly 100 m apart at positions as shown in Figure 1.7. Select first instrument station (say S_1) as close to Peg *A* as possible and read the staff position at *B* (reading *a*).

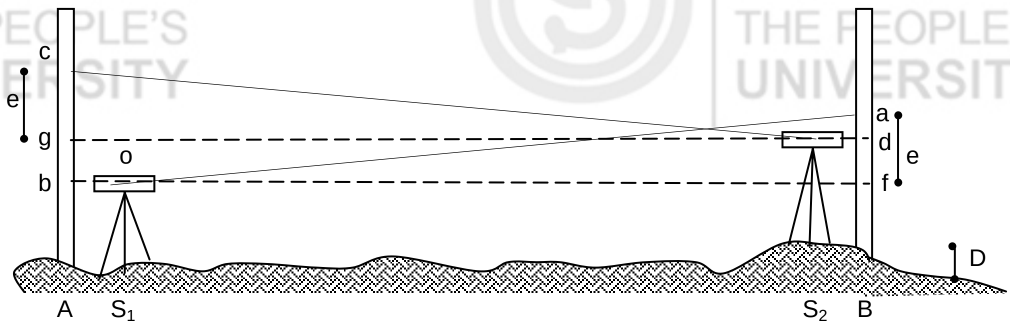


Figure 1.7 : Telescope Spirit Level Test

(b) View the staff held at *A* through the object glass (reading *b*). Through the eyepiece, cross hairs cannot be seen, reading *b* can be read with reasonable accuracy due to proximity.

(c) Remove the theodolite from station S_1 and reset it at station S_2 as close to peg at *B* as possible and read the staff held at *A* (reading *c*). Also read the staff held at *B* through object glass (Reading *d*).

(d) If $(a - b) = (d - c) = D$, the true level difference between *A* and *B*, then the axis of telescope coincides with the line of collimation, if not, corrective adjustment is required.

If $(a - b)$ is not equal to $(d - c)$ then let $e = (a - f)$ be the difference between line of collimation and horizontal in a distance *AB*. Then $(a - b) = D + e$ or $D = a - b - e$. *bf* is the true line of collimation at S_1

or the horizontal line of sight and *bo* is assumed to be negligibly small as compared to distance *AB*.

Similarly, $D = d - c + e$

where $e = c - g = a - f$

Hence, $2D = (a - b) + (d - c)$

or, $$D = \frac{\{(a - b) + (d - c)\}}{2}$$

and $e = D + c - d$

Adjustment

Check that instrument is still near *B*, and bubble is still central. Manipulating diaphragm screws ensures that reading is now "*g*" where $g = c - e$, i.e. the cross wires coincide with reading '*g*'.

Repeat the entire test procedure until $(a - b) = (d - c)$. It may be noted that the vertical circle reading has been set to zero at the start of this test before the instrument was levelled at S_1 and vertical movement clamped.

Index Error Test

The previous test is conducted to determine that telescope level is central when line of collimation is parallel to telescope level and reading of vertical circle is zero. The index error test is done to ensure that when the line of collimation is horizontal and vertical circle reading is zero when index arm bubble is centralized. The procedure of the test will depend upon the position of the altitude spirit level. This could be on telescope, index arm of vertical circle. The clip screw and tangent screws could also be mounted on one index frame or separately on both arms in different models and makes of theodolites.

Some of these conditions are described below :

(a) **Spirit level is on telescope :** Clip and tangent screws are on one frame.

Test

Set the vertical circle reading to zero using clamp and tangent screw. Level the instrument using the telescope spirit level and leveling screws. Swing the telescope through 180^{o} in horizontal plane. If the bubble does not remain central, adjustment shall be made.

Adjustment

Centralize the bubble, half of its run using clip screws and remaining half by using leveling screws. Repeat, till bubble does not move.

(b) **Spirit level is on index arm :** Clip and tangent screws on one frame.

Test

Level the instrument using horizontal plate levels. Set the vertical circle reading to zero using clamp and tangent screws. Centralize the index arm level using the clip screw. Reading of a staff is noted which is placed nearly 100 m from instrument. Rotate the telescope by 180^o and set the vertical circle reading to zero again using tangent screw. Swing the telescope now 180^o horizontally and level the horizontal plate level once again using leveling screws only. The staff reading in this position shall be same as the first reading if instrument is correctly adjusted.

Adjustment

If not, set the telescope's cross hairs to intersect mean staff reading of the two already taken using clip screws and centralize the index arm spirit level using spirit level adjusting screws. Repeat the test procedure till perfection.

(c) **The Clip Screw and Tangent Screw are on Separate Index Frame**

Test

If the spirit level is on telescope, test is conducted similar to Test (a) except that centralize the level tube bubble using leveling screws.

If the spirit level is on index arm, the test procedure is exactly similar to Test (b).

Adjustment

Telescope cross hair is set to the mean reading on the staff using vertical circle tangent screw and vertical scale reading to zero using the clip screws. The bubble of spirit level is brought to centre of its run using the screws of the spirit level.

1.4 TRAVERSING

1.4.1 General

The simple basic principle of traverse surveying is that if the distances and angles between successive survey stations are measured, their relative positions can be plotted on survey maps. A survey line may be represented on plan by two rectangular coordinates if its length and bearings are known. In general, the magnetic meridian *N-S* axis is taken as *Y* coordinate axis while *E-W* is chosen as *X*-axis. Distances measured along *Y*-axis are termed latitudes while those along *X*-axis as departures or longitudes. The known length and bearings of a line are together termed as "course" of the line.

The length or linear distances can be measured by chain, tape, tacheometer or by any recently developed electronic methods of measurements. The bearings, i.e. angles, are measured by compass, theodolite or electronic equipment. These measurements are then plotted to scale by method of coordinates, thus giving the location of main traverse lines on map. These traverse lines can then be used for plotting the details by measurement of offsets to the details.

It is necessary to select a reference direction, particularly at first survey station. This could be same natural prominent land mark. However, in most of the cases true meridian, (*N-S*) or magnetic meridian, is chosen as basic reference direction.

It may, however, be noted that this meridian direction varies with time and station location requiring necessary corrections.

1.4.2 Types of Traverse

A traverse is generally classified as

(a) closed, or

(b) open traverse.

When the location of the first and last station coincides, so that a complete circuit is made (Figure 1.8(a)) or when the coordinates of the last station and first station are known (Figure 1.8(b)) so that survey work could be checked and balanced, the traverse is known as closed traverse.

A traverse is termed open when it does not form a closed polygon (Figure 1.8(c)). It consists of a series of lines extending in the same general direction, so as not to return to the starting station.

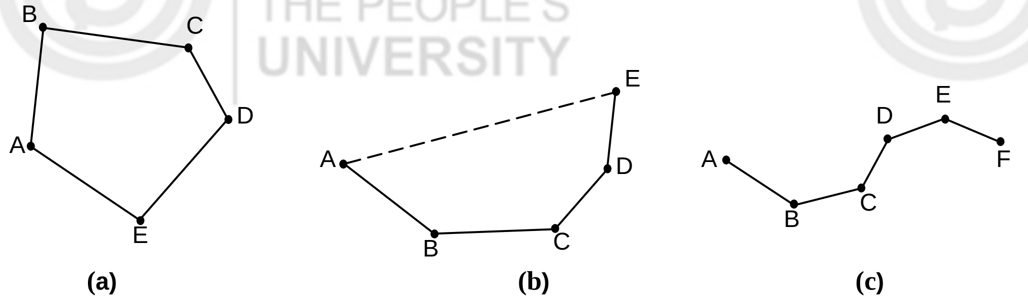


Figure 1.8 : Types of Traverse

1.4.3 Methods of Traversing

A close traverse method of surveying can be employed for land surveys of moderately large areas. It is also used for locating areas like woods, lakes etc. While open traversing is more suitable for survey of a long strip of land, e.g. road or railway routes, river valley etc. For very large areas geodetic survey and triangulation is used.

The basic methods for determining the directions of the survey line in any type of traversing could be by :

(a) chain angles, e.g. chain traversing

(b) free or loose needle method

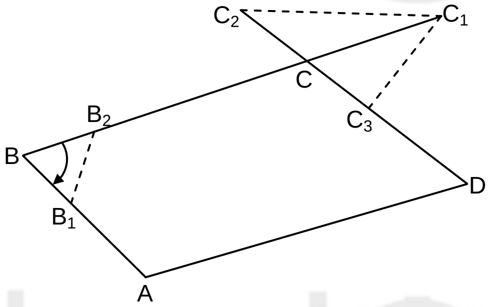
(c) the fast needle method and

(d) direct measurement of angles between successive lines.

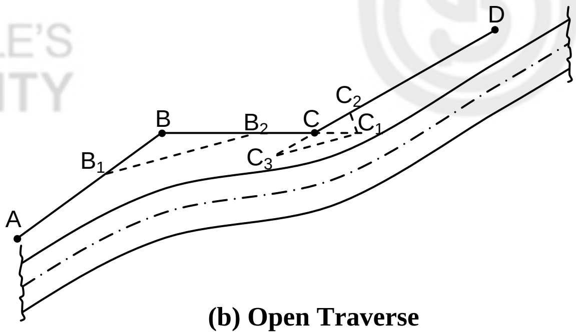
Chain Traversing

Chain angle method is used in chain traversing where all the survey work is accomplished by using only chain and tape. The angle between the successive lines can be decided by measuring the length of the tie lines with chain or tape. Angles so determined are termed as **chain angles**. Tie lines should have sufficient length to ensure accuracy in measurements. However, angle measurements so obtained are less accurate than those made using angle measuring instruments like compass or theodolite.

The tie lines could be internal like B_1B_2 or external, e.g. C_1C_2 (Figure 1.9). The distance $B_1 B_2$ of the internal tie line is obtained after fixing the positions of B_1 (colline AB, measuring BB_1) and of B_2 (on line BC, measuring BB_2). For external tie line, line BC is extended upto C_1 (measuring CC_1 along BC) and line DC is extended up to C_2 (measuring CC_2). External tie line length $C_1 C_2$ is measured to fix angle BCD. As a check measure length of alternate tie line $C_1 C_3$ and distance CC_3.



(a) Closed Traverse



(b) Open Traverse

Figure 1.9 : Chain Angle Method

To obtain the value of angle α (∠ CBA) in Figure 1.9, BB_1 is chosen equal to BB_2. Then

$$\sin\frac{\alpha}{2} = \frac{B_1 B_2}{2} / BB_1 = \frac{B_1 B_2}{2BB_1}$$

or $$\frac{\alpha}{2} = \sin^{-1}\left(\frac{B_1 B_2}{2BB_1}\right)$$

or $$\alpha = 2\sin^{-1}\left(\frac{B_1 B_2}{2BB_1}\right) \quad \ldots (1.1)$$

The chain angle method is not preferred except in exceptional circumstances only, when survey is to be conducted while angle measuring instruments are not available. The measurement is prone to errors where even a small error in measuring $B_1 B_2$ will be magnified greatly at location of ends of survey line AB and BC. It is against the first principle of surveying of working from whole to part.

Free or Loose Needle Method

The bearing of each line is taken with respect to the magnetic meridian at each survey station with the help of an angle measuring instrument like prismatic compass. Loose or free needle refer to magnetic needle mounted freely on frictionless pivot in the compass.

Fast Needle Method

Theodolite is used for measuring horizontal angle to determine the bearing of the line. The theodolite used for this purpose is fitted with a magnetic needle. This method is more accurate than the compass bearings, as theodolite is a more precise and sensitive instrument.

Angle Measurement Method

Theodolite is employed for measuring horizontal angles between the survey lines. These angles could be with reference to

(a) an already fixed reference line whose bearings are known,

(b) included angles between successive lines, or

(c) deflection angles between successive lines (Figure 1.10).

The details of measuring these angles with the help of compass is described in detail in Elements of Surveying, Unit 3, and using theodolite in Unit 6.

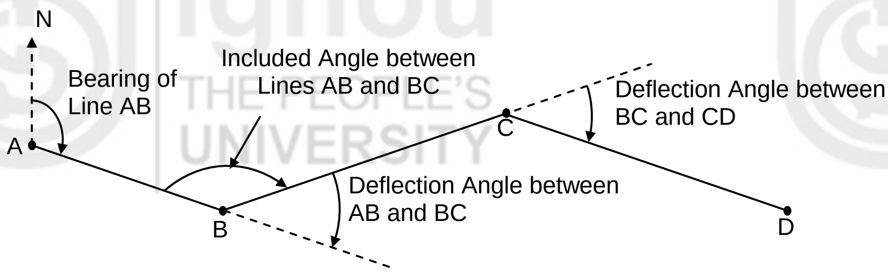


Figure 1.10 : Horizontal Angles of Survey Lines

1.4.4 Field Work in Traversing

When a definitive framework is required for detailed survey the traverse is usually preferred. The examples of this could be to plot the outlines of small land areas or water bodies, where details can be surveyed with reference to main traverse lines. In land development, traversing is used as reference framework for marking the details. Large land areas can also be mapped in flat densely wooden areas.

The field work involved in traverse surveying has to be carried out in a planned way. The basic steps depend on the extent of information which can be obtained before starting the actual surveying. As a preliminary step of survey the existing maps of the area under consideration are collected for getting as much information as possible. If no reliable maps are available an outline reconnaissance survey has to be conducted. This consists of taking photographs of all the salient features and conducting a rapid rough survey using compass and estimating distances without actual distance measurements with accuracy. This will help in locating the most suitable positions of possible survey stations to be used for precision surveying subsequently. The selected survey stations must be visible and approachable from several of the other selected stations and such that maximum number of details and salient features can be measured from the survey lines joining these stations. The chosen stations are then marked with a wooden or metallic pegs. Stations which are of permanent or semi-permanent nature should be marked with a concrete block.

Once stations are finally chosen, these should be marked with signals such as ranging rods, ranging poles and in particular circumstances with elaborate mast. The actual survey can then be started by measuring the angles and distances. While every care is undertaken to ensure that the measurements are made and recorded as accurately as possible, it should also be noted that degree of accuracy to be achieved should be as uniform as possible for all the measurements. It would be wastage of time and effort to measure angles to the accuracy of 0.1 sec by geodetic theodolite, if distances are to be measured using chain laid on ground.

The choice of instruments and methods to be used for linear and angular measurements will mainly depend upon the degree of precision required, which depends upon the purpose of survey. As a general rule, if $\delta\theta$ is error permitted in angular measurement and '*n*' cm in measuring a linear distance of *l* cm, then for the degree of precision to be same, following relationship should be satisfied.

$$\tan \delta\theta = \frac{n}{l} \qquad \ldots (1.2)$$

1.5 TRAVERSE COMPUTATIONS

1.5.1 Traverse Tables

As soon as the angles and distances are measured in field, these should be recorded in a tabular form, for subsequent calculations and use. The recording and results of calculations are usually set out in a traverse table. The most commonly used form of traverse table preferred in practice is called Gale's Traverse Table in Table 1.1.

The computations involved in a traverse survey are explained with the help of an illustrative example. The specimen traverse is shown in Figure 1.11. It has six stations *A*, *B*, *C*, *D*, *E* and *F*. The reference direction is *Y*, which is normally the magnetic or True North direction. True North is used in geodetic surveying while magnetic north is used in normal traversing after suitable corrections for local attraction. Parallel reference directions are drawn in Figure 1.11 at *A*, *B*, *C*, *D*, *E* and *F*. The orthogonal *X*-axis in this case will be East-West. Some salient feature of permanent nature is selected as origin such that coordinates of first station *A* are (X_A, Y_A) relative to origin *O*. Angle $Y'_A AB$ is measured (= α) which may be the bearing of line *AB* if Y'_A is north and X'_A is east. The lengths L_{AB}, L_{BC}, L_{CD}, L_{DE}, L_{EF} of the traverse line, and bearings $\beta_1, \gamma_1, \delta_1$ etc. are measured in the field and converted to included angles β, γ, δ etc. In theodolite traversing, these can be obtained directly. It may be noted that angles are always measured with reference to previous survey line in a **clockwise** direction.

For plotting the survey map with same accuracy as used in measurements of length and angles, these measurements must be used to obtain the coordinates of survey stations. Direct plotting of angles by scale and protractor cannot give this degree of accuracy. The plotting errors will become cumulative in these cases.

The absolute coordinates of survey stations with reference to origin are obtained by first computing the coordinates at each station with respect to the preceding one. These are termed latitudes and departures as explained in earlier unit. The absolute coordinates will then be

$X_A = X_A : Y_A = Y_A$ at station A

$X_B = X_A + X_{AB} : Y_B = Y_A + Y_{AB}$ at station B

$X_C = X_A + X_{AB} + X_{BC} \quad : \quad Y_B = Y_A + Y_{AB} + Y_{BC}$ at station C

$X_D = X_A + X_{AB} + X_{BC} + X_{CD} : \quad Y_A + Y_{AB} + Y_{BC} + Y_{CD}$ at station D and so on.

It may be noted that *X* measured towards east is + ve and towards west is – ve. Similarly, *Y* measured in the direction of north is + ve and towards south is – ve.

In Figure 1.11, X_{AB}, X_{CD} and $X_{DE,}$ Y_{AB}, Y_{BC}, Y_{CD} and Y_{FA} are positive while X_{BC}, X_{EF} and X_{FA}, Y_{DE}, and Y_{EF} will have negative numerical value. The computations are shown in Table 1.2 in tabular form. The angles β_1, γ_1, δ_1 . . . are the reduced bearings at station *B*, *C*, *D* . . . etc.

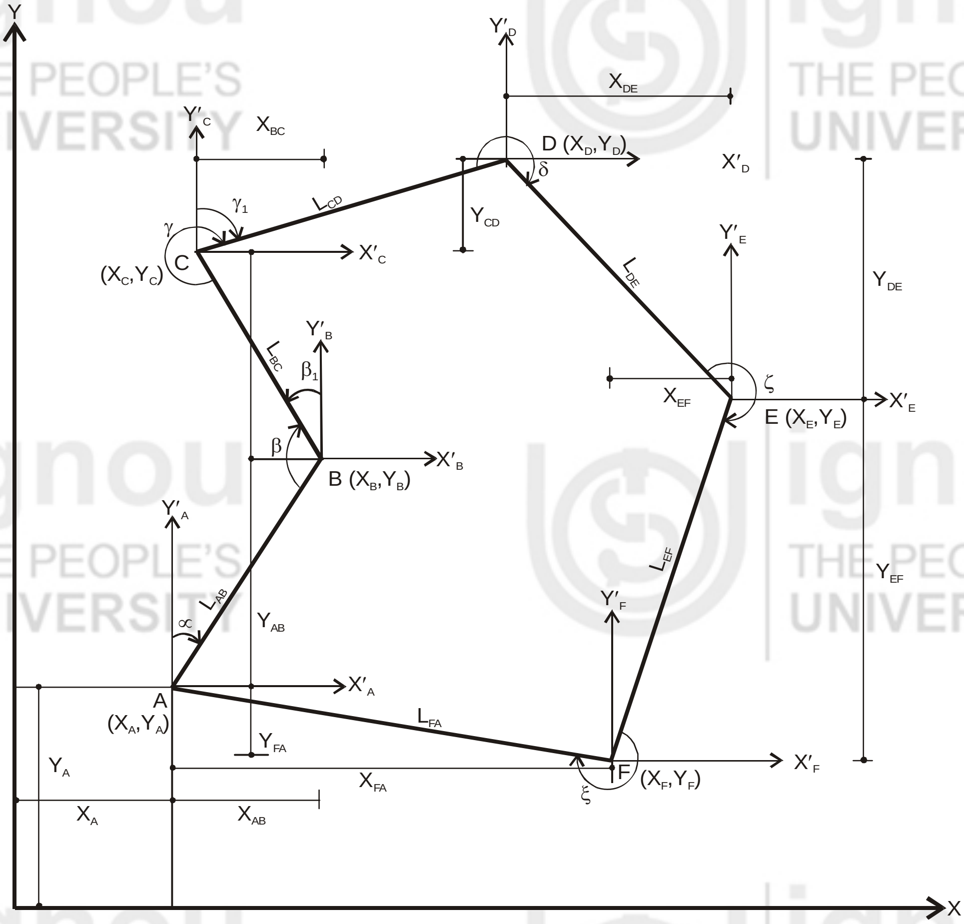


Figure 1.11 : Specimen Traverse

Table 1.2 : Computation of Coordinates

Station	**Coordinates**			
	Local		**Global**	
	Departure	**Latitude**	***X***	***Y***
A			X_A	Y_A
B	$X_{AB} = L_{AB} \sin \alpha$	$Y_{AB} = L_{AB} \cos \alpha$	$X_B = X_A + X_{AB}$	$Y_B = Y_A + Y_{AB}$
C	$X_{BC} = L_{BC} \sin \beta$	$Y_{BC} = L_{BC} \cos \beta$	$X_C = X_B + X_{BC}$	$Y_C = Y_B + Y_{BC}$
D	$X_{CD} = L_{CD} \sin \Upsilon$	$Y_{CD} = L_{CD} \cos \Upsilon$	$X_D = X_C + X_{CD}$	$Y_D = Y_C + Y_{CD}$
E	$X_{DE} = L_{DE} \sin \delta$	$Y_{DE} = L_{DE} \cos \delta$	$X_E = X_D + X_{DE}$	$Y_E = Y_D + Y_{DE}$
F	$X_{EF} = L_{EF} \sin \zeta$	$Y_{EF} = L_{EF} \cos \zeta$	$X_F = X_E + X_{EF}$	$Y_F = Y_E + Y_{EF}$
A	$X_{FA} = L_{FA} \sin \xi$	$Y_{EF} = L_{FA} \cos \xi$	$X_A = X_F + X_{FA}$	$Y_A = Y_F + Y_{FA}$

Check $X_F + X_{FA} = X_A$: $Y_F + Y_{FA} = Y_A$ (Numerical cross check)

1.5.2 Checks in Linear Measurements

The results as tabulated in Gale's traverse table must be of specified accuracy. To achieve this desired degree of accuracy it is necessary that these should be checked wherever possible. If the survey has been conducted properly, that is, if all the linear and angular measurements are precisely measured, the algebraic sum of all the departures and latitudes as obtained in Table 1.2, i.e. sums of second column and sum of third column should be independently zero. In other words the coordinates X_A and Y_A of station A, as given in first row and as obtained in last row of Table 1.2, should be numerically equal.

This generally will not happen and a value δD in Σ col 2 and δL in Σ col 3 in Table 1.2 will be obtained which is not zero. This is termed as linear error in closure δE where :

$$\delta E = \sqrt{(\delta L)^2 + (\delta D)^2} \qquad \text{. . . 1.3(a)}$$

or

$$\delta E = \sqrt{\{(\Sigma\ L)^2 + (\Sigma\ D)^2\}} \qquad \text{. . . 1.3(b)}$$

The magnitude of δE will provide the degree of error indicating the level of accuracy achieved. It is usual to refer it as accuracy ratio. Where accuracy ratio AR is

$$AR = \frac{\delta E}{\sum_{i=1}^{n} L_i} \qquad \text{. . . 1.3(c)}$$

where $\sum_{i=1}^{n} L_i$ = sum of the lengths of traverse lines or the parameter of the surveyed traverse. The AR value will vary from area to area and from one method of traversing to the other. Depending upon the nature of survey and desired accuracy, AR will range from 1 in 5000 to 1 in 10000.

If the accuracy ratio achieved in a traverse survey is larger than the permissible limit, i.e. if its value is less than 1 in 5000 (say), the entire survey in the field need to be re-conducted and repeated. However, if it is within the permissible limit (more than 1 in 5000 say), the correction is sought to be applied and readings of latitudes and departures as obtained in Table 1.2 are adjusted by distributing the closing error throughout the traverse. The adjustment process is known as balancing the traverse.

Traverse Balancing

There are several alternative methods of balancing of traverse. These are arbitrary method, Bowditch rule (compass rule), transit rule, least square method, Crandall's method etc.

The Crandall's and least square methods are based on theory of probability and are more complex hence not generally used in practice, while in the arbitrary method the latitude and departures are adjusted arbitrarily on the judgement of the surveyor. For example, if in the opinion of the surveyor one or more of the traverse sides may not have been measured as precisely as others, because of particular practical difficulties or obstructions in the field, the whole of the larger part of linear error of closure may be assigned to that side or sides, arbitrarily depending purely on surveyors perception.

However, it is observed that all the traverse lines are measured linearly and angularly with same precision, it is common practice to apply either the Bowditch rule (compass rule) or the transit rule.

In transit rule, the adjustment to latitude (or departure) are applied in proportion to their lengths. Thus, longer a latitude (or departure), the greater is its adjustment, i.e.

$$\delta X_i = \frac{\Delta X_i}{\sum_{i=1}^{n} X_i} \cdot X_i \qquad \text{. . . 1.4(a)}$$

and $$\delta Y_i = \frac{\Delta Y}{\sum_{i=1}^{n} Y_i} \cdot Y_i \qquad \ldots 1.4(b)$$

where δX_i (δY_i) are adjustment in departure (latitude) in i^{th} side, $\triangle X$ ($\triangle Y$) are total closing error in departure (latitude), X_i (Y_i) are departure (latitude) of side I, while ΣX_i and ΣY_i are sum of columns 2 and 3 in Table 1.2.

It is preferable to apply this method when linear measurements are less precise than angular measurements.

The compass rule or Bowditch rule is applied when both angular and linear measurements have similar precision.

In this method

$$\delta X_i = \Delta X_i \cdot \frac{L_i}{\sum L_i} \qquad \ldots 1.4(c)$$

and $$\delta Y_i = \Delta Y_i \cdot \frac{L_i}{\sum L_i} \qquad \ldots 1.4(d)$$

where L_i and ΣL_i are lengths of traverse line i and the perimeter of traverse, i.e. sum of all the lengths of traverse sides.

The differences in the above two sets of corrections are relatively small. The calculations are simple and results are fairly accurate. For precision surveying like geodetic surveying and triangulation, more precise methods like Crandall or least square method is adopted.

The corrections are carried out in tabular form and the results of the computations along with corrected coordinates are recorded as shown in Gale's traverse table (Table 1.1).

As a field check, all linear measurements should be repeated if possible in opposite direction of traverse, compared to first measurement. If situation permits these could be checked by tacheometric methods using a theodolite at either of the stations.

1.5.3 Checks in Angular Measurements

A check of angular error of closure is available in closed traverses with n stations. The internal angles should sum to $180\,(n-2)^o$ and sum of external angles should be $180\,(n+2)^o$, where n is number of sides of the traverse.

Due to problem of the field observations and in instruments there will always be some discrepancy, however small it may be. This is termed as angular error of closure (E). If the closing error is relatively large and more than the permissible limit, the surveying exercise is required to be repeated. The permissible limit is normally taken as $\delta\alpha \times \sqrt{n}$ where $\delta\alpha$ is the least count of measuring instrument and n is number of sides in the traverse. If the closing error (E) is small, it is distributed either equally among the stations if the traverse sides are nearly equal. The angles so corrected and adjusted shall satisfy the conditions of internal or external angles of the traverse. If, however, some of the traverse lines are too short relative to others, the angular corrections are advised to be applied to the angles adjacent to these lines preferably in ratio of their lengths. This is because centering errors are more likely to occur on short lines. It is important to take cross bearings wherever possible. This will help in localizing any large errors.

Some other angular checks to be applied in case of closed traverses could be as follows :

Deflection Angles

The algebraic sum of the deflection angles of a traverse should be equal to 360°. It is important to follow same sign convention in this process. For example, right hand deflection angle can be taken as positive while left hand as negative, or vice-versa.

Bearing

The accuracy of traversing can be checked by comparing the fore bearing of the last line with its back bearing observed at initial station.

1.5.4 Checks in Open Traverse

In an open traverse, an attempt is made at closure even if an extra station has to be introduced otherwise the measurements as a whole cannot be checked. Some checks could be as follows :

(a) Cut off lines between certain intermediate stations can be run. Let there is an open traverse *ABCDEFGH* . . . (Figure 1.12). *AE* and EM are cut off lines, thus dividing the open traverse into two closed traverses *ABCDE* and *EFGHKM*. The linear and angular measurements of each part of the traverse can now be checked. The traverse *ABCDE* is checked by observing the direction of *AE* both at *A* and at *E* and observing whether the difference between these bearings is 180° and also by measuring distance *AE*. Similarly, traverse EFGHKM can also be checked.

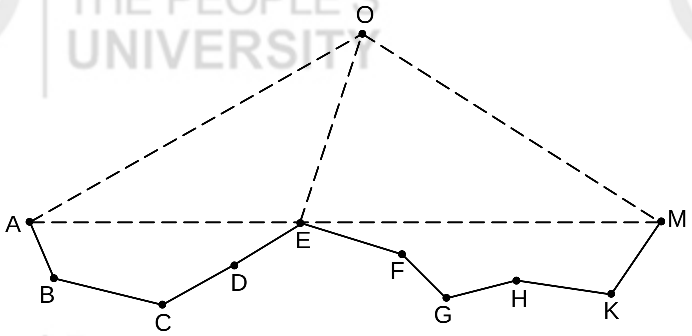


Figure 1.12 : Checking Open Traverse

(b) Well defined prominent object (say, *O*) lying on one side of the traverse is chosen. The bearings of object *O* is taken at intervals. Let the bearing of object *O* in Figure 1.12 is taken from stations *A*, *E* and *M*. The coordinates of object *O* can be computed from measurements of traverse *ABCDEOA*. Bearing of line *MO* can then be obtained from the coordinates of station *M* and object *O*. This computed bearing of *MO* can then be checked with the actual observed bearing of line *MO* from station *M*. The other part of the traverse *EFGHKM* can then be carried out and coordinates of *O* are computed once again from the new traverse measurements. The two computed values of coordinates of *O* are then compared for the accuracy of traverse *A* to *M*.

The methods of open traverse checking as described in (a) and (b) are used for normal survey work wherever possible. However, for precision surveys, particularly when length of the open traverse is very large, the angular errors can be determined by astronomical observations for azimuth at regular intervals during the progress of the traverse.

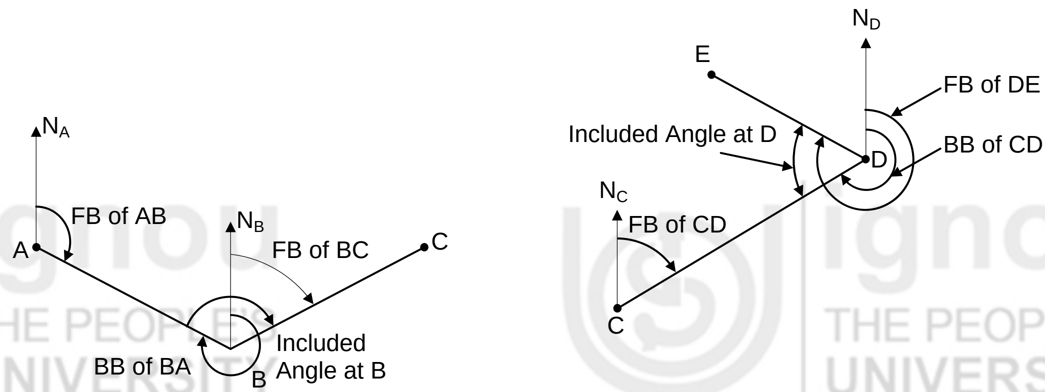
1.5.5 Other Computations

As described in unit on theodolite in Elements of Surveying (Unit 6) and else where in present unit on traverse surveying, angular measurements are made with the help of compass and/or theodolite. These angular measurements, whether these are whole circle bearings, included angles (interior or exterior) or deflection angles, are used to compute the value of other angles. The procedure followed can be described as follows.

Method of Included Angles

If the traverse survey is made by the method of included angles and the whole circle bearing of the initial line is measured, the bearings of other traverse lines can be computed as follows

To the whole circle bearing of any line (known) add the included angle between that line and the next line, measured in clockwise direction.



(a) **(b)**

Figure 1.13

If the sum is greater than 180^o subtract 180^o, and if the sum is less than 180^o add 180^o. The result will be the whole circle bearing of next line.

Let the fore bearing of line *AB* = whole circle bearing of line *AB* = 130^o in Figure 1.13(a) and included angle between line *AB* and *BC* is 110^o. Then adding the two values 130 + 110 = 240^o, which is greater than 180^o, hence reduce this value by 180^o (i.e. 240^o – 180^o = 60^o). Thus, the whole circle bearing of line *BC* will be 60^o which is fore bearing of line *BC* at station *B*.

Similarly, let the WCB of line *CD* (i.e. FB of *CD* at *C*) is 70^o while included angle between lines *CD* and *DE* measured clockwise if 60^o. The total is 70^o + 60^o = 130^o, which is less than 180^o. Add to this 180 to obtain the WCB of line *DE* (180^o + 130^o = 310^o), i.e. fore bearing of line *DE* (Figure 1.13(b)).

Example 1.1

In a closed traverse survey *ABCDE*, the observed bearing of line *AB* is 120^{o}30′0″ (Figure 1.14). The included angles measured are as follows.

Station	A	B	C	D	E

Included Angles	76°49′00″	150° 20′40″	98°20′30″	102°15′40″	112°14′10″

Calculate the bearings of remaining sides of the traverse.

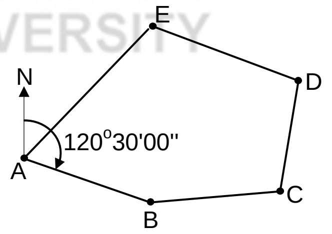


Figure 1.14

Solution

Bearing of line AB	=	120°30′0″
Add ∠B		150°20′40″
	=	270°50′40″>180°
Subtract 180°		–180°
Bearing of line BC	=	90°50′40″ (a)
Add ∠C		98°20′30″
	=	189°11′10″ >180°
Subtract 180°		– 180°
Bearing of line CD		9°11′10″ (b)
Add ∠D		102°15′40°
	=	111°26′50″ <180°
Add 180°		+ 180°
Bearing of line DE	=	291°26′50″ (c)
Add ∠E		112°14′10″
	=	403°41′00″ >180°
Subtract 180°		– 180°
Bearing of line EA	=	223°41′00″ (d)
Add ∠A		76°49′00″
		300°30′00″ >180°
Subtract 180°		– 180°
Bearing of line AB		120°30′00″ (e)

The computed bearing of line *AB* is same as observed value of bearing of line *AB*. Hence, the accuracy of measurement and calculations is cross checked.

Method of Deflection Angles

If the traverse is run by measuring the bearing of initial line and deflection angles, the whole circle bearings of remaining lines can be computed using the following procedure.

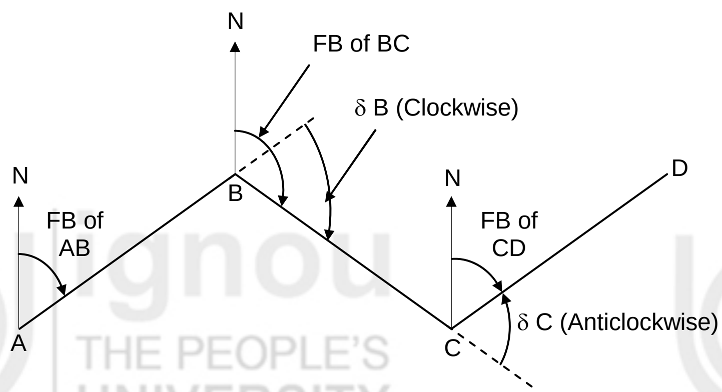


Figure 1.15

WCB of any line = WCB of preceding line ± δ, where δ is deflection angle taken + ve if deflection angle is clockwise (right) and – ve if it is counter clockwise (left). To the obtained WCB add 360^o if it is negative and subtract 360^o if it is more than 360^o to obtain true value of WCB of the line.

Bearing of line *BC* = Bearing of *AB* + δ*B*

Bearing of line *CD* = Bearing of *AB* – δ*B*

Check : Bearing of last line = FB of initial line + Sum of deflection angles.

Example 1.2

The following table gives the deflection angles in a traverse survey. The bearing of line *AB* is 120^{o}30′00″. Compute the bearings of remaining traverse line.

Station	A	B	C	D	E
Deflection Angles	103^o 11′00″ (anticlockwise)	29^{o}39′20″ (anticlockwise)	81^{o}39′30″ (anticlockwise)	77^{o}44′20″ (anticlockwise)	67^{o}45′50″ (anticlockwise)

Solution

Bearing of line *AB*	=	120^{o}30′00″	
Deduct δ*B*	=	– 29^{o}39′20″	
Bearing of line *BC*	=	90^{o}50′40″	(i)
Deduct δ*C*	=	– 81^{o}39′30″	
Bearing of line *CD*		9^{o}11′10″	(ii)
Deduct δ*C*		– 77^{o}44′20″	
	=	68^{o}33′10″ < 0	
Add 360^o		+ 360^o	
Bearing of line *DE*	=	291^o 26′50″	(iii)
Deduct δ*E*		67^{o}45′50″	
Bearing of line *EA*	=	223^{o}41′00″	(iv)
Deduct δ*A*		– 103^{o}11′00″	
Bearing of line *AB*		120^{o}30′00″	(v)

The computed bearing of line *AB* is same as given value.

Checked

Bearing of line *AB* = 120^{o}30′00″ – (29^{o}39′20″ + 81^{o}39′30″ + 77^{o}44′20″ + 67^{o}45′50″ + 103^{o}11′00″)
= 120^{o}30′00″ – (360^o) . . . (vi)

The value of bearing of line *AB* by rule of checking is – ve hence add 360^o. Hence, true bearing of line *AB* by rule of checking is

$120^o30'00'' - 360^o + 360^o = 120^o30'00''$ . . . (vii)

Included Angle and Deflection Angle

Conversion of included angles (θ) measured in clockwise direction from the back station to corresponding deflection angle (δ) can be achieved as follows

(a) Included angle $\theta > 180^o$:

then $\delta = \theta - 180^o$

(b) If $\theta < 180^o$

then $\delta = 180^o - \theta^o$

Check Σδ of a closed traverse is equal to 360^o

Example 1.3

Compute the deflection angles in a closed traverse whose included angles are given as follows :

Station	A	B	C	D	E	F
Included Angle (θ)	$50^o40'$	$191^o38'$	$103^o19'$	$79^o48'$	$220^o13'$	$74^o22'$

Solution

The traverse *ABCDEF* is sketched as shown in Figure 1.16.

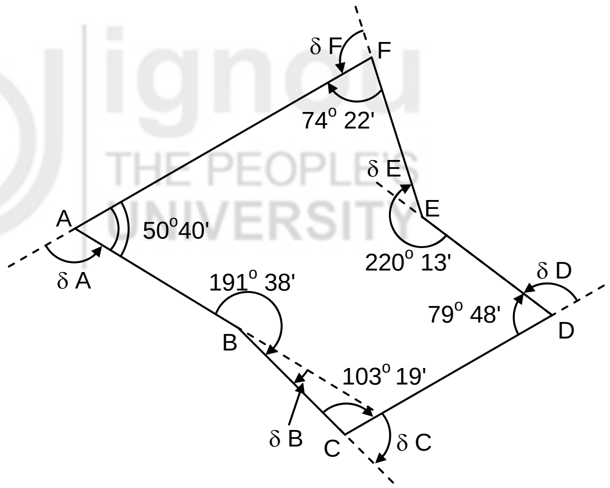


Figure 1.16

Deflection angle at station $B = 191^o\ 38' - 180^o$

$= 11^o\ 38'$ (+ ve) clockwise

Deflection angle at station $C = 180^o - 103^o19'$

$= 76^o\ 41'$ (– ve) counterclockwise

Deflection angle at station $D = 180^o - 79^o48'$

$= 100^o\ 12'$ (– ve) counterclockwise

Deflection angle at station $E = 220^o\ 13' - 180^o = 40^o\ 13'$ (+ ve) clockwise

Deflection angle at station $F = 180^o - 74^o22' = 105^o\ 38'$ (– ve) anticlockwise

Deflection angle at station $A = 180^o - 50^o\ 40' = 129^o\ 20'$ (– ve) anticlockwise

Σ (– ve) δ = $(100^{o}12' + 105^{o}38' + 129^{o}20' + 76^{o}\,41') = 411^{o}51'$

Σ (+ ve) δ = $(11^{o}38' + 40^{o}13') = 51^{o}\,51'$

Algebraic sum = $\left| 411^{o}51' + 51^{o}51' \right| = \left| -360^{o} \right| = 360^{o}$ (OK).

1.6 MISSED MEASUREMENTS

1.6.1 General

As described earlier, two measurements are required to be made for each traverse line, i.e. its length and bearing. With the help of these field measurements, the coordinates of each survey stations along global *Y*-axis (usually North) and global *X*-axis (usually East), can be determined for plotting the traverse map. The distances computed parallel to *Y*-axis (North) is called latitude while those parallel to *X*-axis (East) are termed as departures. A closed traverse is considered to be completely surveyed when the length and bearing of each of its sides are known as obtained by field observations.

It is, however, possible that some of these field measurements are accidentally omitted during the survey or could not be made due to certain unavoidable obstructions in the field. If these omissions or missed measurements are only one or two in number, these can be manipulated and obtained by calculations. The sides affected by these omissions are called **affected sides**. However, during these computations it has to be assumed that all field measurements were precise and accurate. There is no scope for computation of balancing or closing errors in such cases.

The common cases of missed measurements during field survey can be listed as follows.

(a) Only one side is affected, i.e.

   (i) Bearing of one side is unknown

   (ii) Length of one side is missing

   (iii) Length and bearing of one side is omitted.

(b) Two sides are affected, i.e.

   (i) Length of one side and bearing of another side is wanted, or

   (ii) Lengths of two sides were not recorded, or

   (iii) Bearings of two sides are missing.

These measurements in case of a closed traverse can be obtained using the principle that sum of the latitudes of all the traverse sides is zero and the sum of the departures of traverse side is also zero. Thus, from the above two equations two unknown measurements can be obtained. If the unknown measurements are more than two the problems is indeterminate. For computational work, following relationships are useful

(a) Latitude $Y_i = l_i \cos \theta_i$ and departure $X_i = l_i \sin \theta_i$ . . . 1.6(a)

where l_i and θ_i are length and reduced bearing of i^{th} traverse line.

(b) $\tan \theta_i = \dfrac{x_i}{y_i}$ and $l_i = \sqrt{(x_i^2 + y_i^2)} = y_1 \sec \theta_i = x_i \text{ cosec } \theta_i$ . . . 1.6(b)

1.6.2 Various Cases of Missed Measurements

Case 1 : When Bearing, or Length, or Bearing and Length of One Side is Missing

This case is explained by Example 1.4.

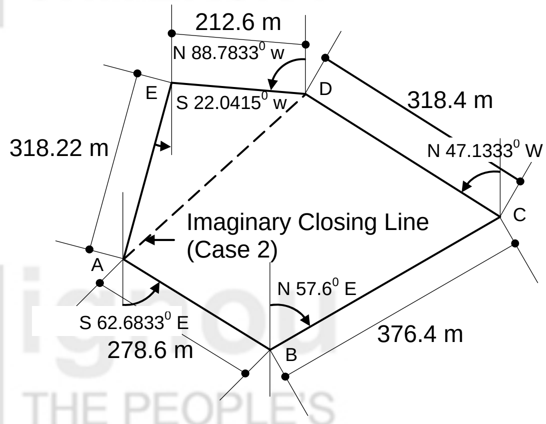


Figure 1.17 : Missed Measurements

Example 1.4

The field measurements of a closed traverse *ABCDE* are reproduced in the following table. Fill in the blanks.

Line	AB	BC	CD	DE	EA	EA Computed
Length (m)	278.6	376.4	318.4	212.6	?	– 318.22
Bearing (WCB)	$117^0\ 19'$	$57^0\ 36'$	$312^0\ 52'$	$271^0\ 13'$	?	

Solution

Reduce Bearing	S62.6833° E	N57.6000°E	N 47.1333°W	N 88.7833°W	–	
Latitude Y_i	– 127.85	+ 201.69	+ 216.61	+ 4.51	+ 294.96	– 294.96
Departure X_i	+ 247.53	+ 317.81	– 233.81	– 212.55	+ 119.42	– 119.42

$$l_{EA} = \sqrt{\{(-294.96)^2 + (-119.42)^2\}} = 318.22 \text{ m}$$

$$\tan \theta_{EA} = \frac{-119.42}{-294.96} = 0.4049 = S\,22.0415^\circ \text{ W}$$

Two values of reduced bearings θ_{EA} are obtained. In first or third quadrant the third quadrant selected because both latitude Y_{EA} and departure X_{EA} are – ve indicating S. W. quadrant.

Case 2 : When Length of One Side and Bearing of Another Side is Missing

In Case 1, the length and bearing of same line were missing. Now it is assumed that length of one survey line and bearing of another survey line are missing. To start with consider that line *j* and *k* are adjacent. Let *k* indicates line *DE* and *j* indicates line *EA*, two adjacent lines in Figure 1.17.

The problem is attempted to be solved first by neglecting the affected sides *DE* and *EA* and considering the traverse *ABCD* closed by an imaginary line

DA. Its length and bearing are computed using the procedure followed in Case 1.

Let in triangle *ADE*, the included angles are α, β and δ, respectively at *A*, *D* and *E*. Then the sine rule can be used to analyse this triangle, e.g.

$$\frac{DA}{\sin\delta} = \frac{AE}{\sin\beta} = \frac{DE}{\sin\alpha} \qquad \ldots (1.7a)$$

Since bearing of line *DE* is known and of *AD* calculated earlier, magnitude of β can be obtained.

Also, since length *EA* is known and *DA* computed earlier the expression

$$\frac{DA}{\sin\delta} = \frac{AE}{\sin\beta}$$

can be used to obtain δ

or $$\sin\delta = \frac{DA}{AE}\sin\beta \qquad \ldots (1.7b)$$

Having the values of β and δ, α can be obtained as

$$\alpha + \beta + \delta = 180^o \text{ or } \alpha = 180^0 - (\beta + \delta) \qquad \ldots (1.7c)$$

Finally, length *DE* can be computed as

$$DE = DA\,\frac{\sin\alpha}{\sin\delta} \qquad \ldots (1.7d)$$

Example 1.5

The survey records of a closed traverse are given in the following table. Fill up the missing entries

Line	AB	BC	CD	DE	EA	Computed DE	Value
Length	278.6	376.4	318.4	—	318.22	212.61	EA
Bearing	117°19′	57°36′	312°52′	271°13′	—		

Solution

In Figure 1.17 imaginary closing line *AD* closed the traverse *ABCDA*, omitting the effected sides *DE* and *EA*. L_{DA} and θ_{EA} are obtained by exactly following the procedure of Case 1.

Line	AB	BC	CD	Σ	Closing DA
Length	278.6	376.4	318.4		
Reduced Bearing	S62.6833°	N57.6000°E	N 47.1333°W		
Latitude y_i	– 127.85	+ 201.69	+ 216.61	+ 290.45	– 290.45
Departure x_i	+ 247.53	+ 317.81	– 233.37	+ 331.97	– 331.97

$$L_{DA} = \sqrt{\{(-290.45)^2 + (-331.97)^2\}} = 441.10 \text{ m}$$

$$\tan\theta_{DA} = \frac{-331.97}{-290.45} = +1.1430$$

θ_{DA} = S48.8164° W (in third quadrant, as both latitude and departure are – ve).

Check L_{DA} = *L* 90.45 sec 48.8164 = 441.10 m (OK)

Now *RB* of line *DE* = *N* 88.7833° W and of *DA* = *S* 48.8164° W

Hence $\beta = 180^o - 88.7833^o - 48.8164^o = 42.4003^o$

$$\therefore \quad \sin\delta = \frac{441.102}{318.22}\sin 42.4003^0$$

$= 0.9347$ or $\delta = 69.1778^o$ or $180^o - 69.1778^o = 110.8222^o$

And δ is selected as explained in figure of Example 1.4

$\alpha = 180^o - (110.8222^o + 42.4003^o) = 26.7775^o$

Then $$l_{DE} = DA\,\frac{\sin 26.7775^o}{\sin 110.8222^o} = 212.61 \qquad \text{(OK)}$$

Reduce Bearing of EA = 90° + 20.8222°

= 110.8222° (OK)

Case 3 : When Lengths of Two Sides are Missing

The lengths of two affected adjacent sides are omitted, i.e. l_j and l_k are missing.

In Figure 1.17, let adjacent sides *DE* and *EA* of close traverse *ABCDE* are the affected sides. This problem of missed measurements is attempted in a similar way as in Case 2, i.e. ignoring the affected side, the traverse *ABCD* is closed using an imaginary closing line *AD* and the length and bearing of the closing line is calculated.

Since the bearing of all the lines are given the magnitude of α, β and δ of the triangle *ADE* is computed and cross checked as $\alpha + \beta + \delta = 180^o$. Applying sine rule the lengths l_{DE} and l_{EA} can be obtained.

Example 1.6

Following table gives the site measurements of a traverse (Figure 1.17). Calculate the missed lengths.

Line	AB	BC	CD	DE	EA
Lengths (m)	278.6	376.4	318.4	?	?
WCB	117°19′	57° 36′	312°52′	271°13′	201°56′
Reduce Bearing	S62.683°E	N 57.6000° E	N 47.13333°W	N 88.7833°W	S 21.9305°W

Solution

The length and bearing of closing line *DA* are obtained by finding the latitude and departures of lines *AB*, *BC* and *CD* as in following Table.

Line	AB	BC	CD	E	Closing Line DA

Length (m)	278.6	376.4	318.4	–	–
R· Bearing	S 62.633°E	N 57.6000°E	N 47.1333°W	–	–
Latitude	– 127.85	+ 201.69	+ 216.61	+ 290.45	290.45
Departure	+ 247.53	+ 371.81	– 233.37	+ 331.97	– 331.97

$$l_{DA} = \sqrt{\{(-331.97)^2 + (-290.45)^2\}} = 441.10 \text{ m}$$

$$\tan \theta_{DA} = \frac{331.97}{290.45} = 1.1430$$

RB of DA = S 48.1864°W

RB of AD = N 48.8164°E

RB of EA = S 22.0415°W

RB of AE = N 22.0415°E

RB of DE = N 88.7833°W

RB of ED = S 88.7833°E

Here $\alpha = -22.0415^\circ + 48.8164^\circ = 26.77^\circ$

$\beta = -88.7833^\circ + (-180^\circ - 48.8164^\circ) = 42.4003^\circ$

and $\delta = 110.8222^\circ$

Check $\alpha + \beta + \delta = 179.9974^\circ = \approx 180^\circ$ (OK)

Knowing length, AB = 441.10

$$\frac{441.10}{\sin \delta} = \frac{l_{DE}}{\sin \alpha} = \frac{l_{AE}}{\sin \beta}$$

or $$l_{DE} = 441.10 \times \frac{\sin 26.7749^\circ}{\sin 110.8222^\circ} = 212.60$$

$$l_{AE} = 441.10 \times \frac{\sin 42.4003^\circ}{\sin 110.8222^\circ} = 318.22$$

Case 4 : When Bearings of Two Sides are Missing

Angles of two adjacent sides are missing, i.e. θ_J = ?, θK = ?

Similar to Cases 2 and 3, assume AE and DE are affected sides.

The procedure is similar to that followed in earlier procedures, except that area of triangle ADE is computed by following formula, i.e.

$$\Delta = \sqrt{\{s\,(s-a)\,(s-b)\,(s-c)\}}$$

Then angles $\alpha \cdot \beta$ and δ of triangle ADE are obtained by equating Δ to 1/2 × product of lengths of two sides multiplied by sin of angle between them, i.e.

$$\Delta = \frac{1}{2}\, l_{AE} \times l_{ED} \sin \delta \text{ or } \sin \delta = \frac{2\Delta}{l_{AE} \cdot l_{ED}}$$

$$= \frac{1}{2}\, l_{EA} \times l_{AD} \, \sin \alpha \;\text{ or }\; \sin \alpha = \frac{2\Delta}{l_{AE} \cdot l_{ED}}$$

$$= \frac{1}{2}\, l_{AD} \times l_{ED} \, \sin \beta \;\text{ or }\; \sin \beta = \frac{2\Delta}{l_{AD} \cdot l_{ED}}$$

Case 5 : When Affected Sides are Not Adjacent

Refer to Figure 1.18 of closed traverse *ABCDE*.

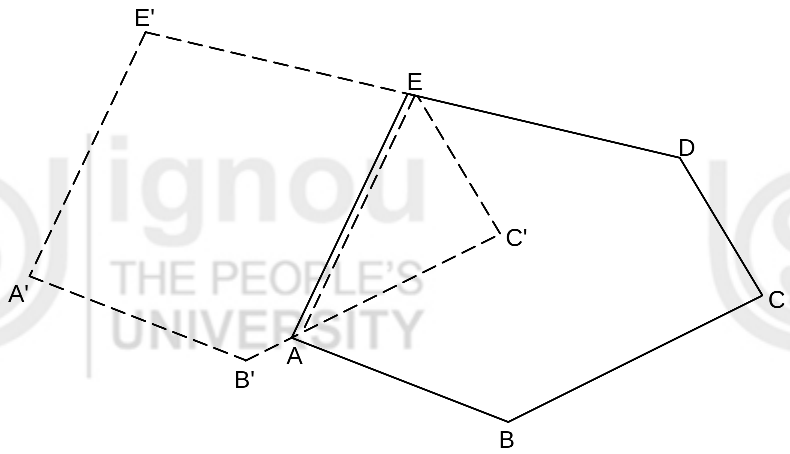


Figure 1.18 : Missing Dimensions Sides Not Adjacent

Let the sides affected are *EA* and *CD* which are not adjacent. In this case, any of the affected side say *EA* is shifted parallel to itself to a position adjacent to other (in this case *CD*). The known sides are shifted parallel to themselves. Thus, in order to form an imaginary close traverse with adjacent affected sides, shift the known sides *AB* and *BC* parallel to themselves. Thus, closing line *A′E′* and then the procedure of solutions from Case 1 to Case 4 can be repeated.

The method is based on the principle that length and bearing of a line remain unaffected when moved parallel to itself. In attempting such problems, it is advantageous to draw the traverse to scale.

SAQ 1

(a) What is gyro theodolite? Explain with Figure.

(b) What are the types of adjustment used in theodolite? Explain in details.

(c) What is collimation test? Explain with Figure.

(d) Explain basic principle of traverse survey? Explain types of traverse.

(e) Explain traverse computation with Figure.

(f) A traverse *ABCD* was supposed to be run but due to an obstruction between the stations *A* and *B*, it was not possible to measure the length and direction of the line *AB*. It was only possible to obtain the following data :

Line	AD	DC	CB
Length (m)	44.5	67.0	61.3
Reduced Bearing (RB)	N50° 20′E	S69°45′E	S30°10′E

Determine the direction and length of *BA*. Also, work out the perpendicular distance between *C* and *AB* along with the distance of the foot of the perpendicular from *C* on *AB* from *B*.

1.7 SUMMARY

Theodolite is a highly sensitive instrument for measuring angles, both horizontal and vertical. It can also be used for obtaining bearings of line with an attached compass. With vertical movements of the telescope locked in horizontal position, it can be used for levelling. For highly undulating grounds, it can be used for trigonometric levelling. Horizontal distances can also be measured, using the tacheometric diaphragms, fairly accurately.

Using theodolite for general survey work, it is required to be adjusted. The adjustment could be temporary or permanent. Temporary adjustments are needed at every instrument station, while permanent adjustments are required to assure the prescribed relationships between instrument's fundamental axis.

The land to be surveyed is measured by technique of traversing, which could be closed one or open. The main survey lines of traverse are so selected that the entire survey area is adequately covered. For getting the information and location of all salient ground features, secondary and tertiary survey lines are drawn with reference to main traverse sides and details measured by laying the offsets from these lines. The data so obtained from traverse survey is required to be manipulated with corrections and computation so that the accurate realistic survey maps are prepared. Survey maps are essential for all subsequent applications of survey exercises in civil engineering projects, like land measurements, fixing plot boundaries and locations contouring, drawing longitudinal and cross sections and earth work computations.

1.8 ANSWERS TO SAQs

Refer the relevant preceding text in the unit or other useful books on the topic listed in "Further Reading" given at the end to get the answers of SAQs.