PIE Tech

POLLACHI INSTITUTE OF ENGINEERING AND TECHNOLOGY

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Department of Civil Engineering

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CE3351- Surveying Levelling

RANGING

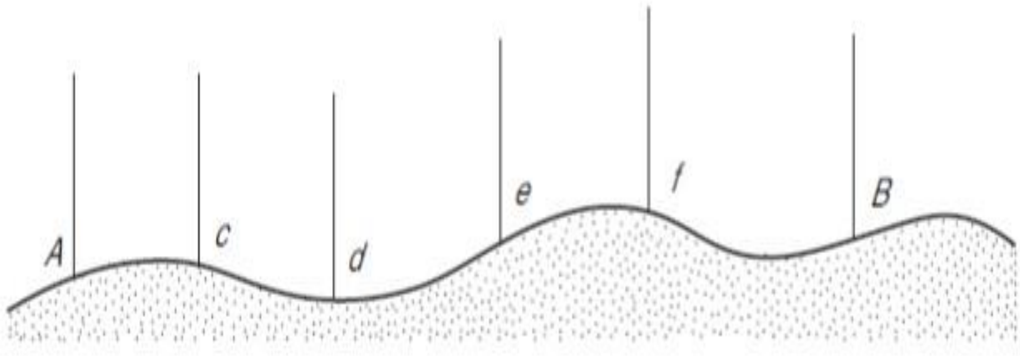
When the distance to be measured with the chain, between the two stations, is less than chain length and the ends are visible. But when the distance is too long and ends are not intervisible due to intervening ground, obstruction, etc., a number of intermediate points are established with the help of ranging rods. The process of establishing intermediate points on a survey line joining two stations in the field, so that the length between the stations may be measured accurately is known as ranging.

Ranging is of two kinds:

1. Direct ranging
2. Indirect ranging

Direct ranging

When ranging rods are placed on intermediate points along the chain line by direct observation from either end stations, the process is known as direct ranging (Fig.).



Direct ranging

Let A and B be two end stations and c, d, e, etc., be the intermediate points to be established. The procedure for marking the intermediate points is as follows.

Procedure

1. Ranging rods are erected vertically behind each end of the line.

2. A surveyor stands behind the ranging rods at the end stations A and B of the line.

3. One of the surveyors, say the surveyor at A, directs the assistant to hold a ranging rod vertically at arm's length from the point where the intermediate point is to be established.

4. The assistant is directed to move the rod to the right or left until the three ranging rods appear to be exactly in a straight line.

5. The surveyor at A then sits down and ensures that the bottom of all the three ranging rods are in the same line.

6. The surveyor then signals the assistant to fix the rod.

INDIRECT RANGING

When the end stations are not intervisible due to rising ground between them, or due to long distance between the ends, indirect ranging is done. The given points are inaccessible or are separated by an elevation making it impossible for one to be visible from the other, the following procedure is adopted:

1. Let A and B be the two end stations of a line with a rising ground between them and C and D the two intermediate points to be established on the chain line (Fig. (a)).

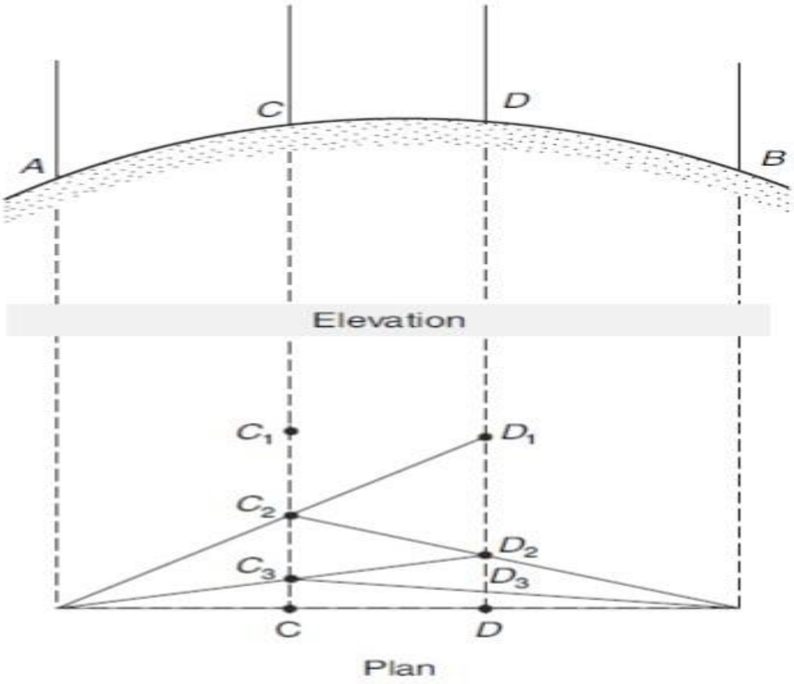
2. The two chainmen stand at C_1 and D_1, the chainman at C_1 can see both the ranging rods at D1 and B, and the chainman at D_1 can see both the ranging rods at C_1 and A.

3. Now the chainman at D_1 directs the chainman at C_1 to move to C_2 so as to be in line with A.

4. Then the chainman at C_2 directs the chainman at D_1 to move to D_2 so as to be in line with B.

5. By successively directing each other, the two chainmen proceed to the line AB and finally come at C and D exactly in the line AB.

6. C and D are the required intermediate points between A and B.



(a) Across a mountain

Indirect Ranging

LINEAR MEASUREMENT WITH CHAIN

On Smooth Level Ground

In measuring a distance that is longer than one chain length, it is necessary to mark chain lengths at intermediate points, and if the total measured distance is to be accurate, it is imperative that these intermediate points be on the line.

The following steps are followed in chaining a line longer than one chain length:

1. The follower places one of the handles of the chain in contact with the peg.
2. The leader takes the other handle of the chain, ten arrows, ranging rods and moves forward along the line.
3. After the chain is stretched completely along the line, the follower stands on one side of the line with the ranging rod touching the handle.
4. The follower directs the leader to come exactly in line. This can be achieved ensuring that the lower ends of all the three ranging rods are in same line.
5. The leader puts a scratch at the position of rod and inserts an arrow. He then moves forward with the chain handle, the remaining nine arrows, and the ranging rod, till the follower reaches the next peg point.
6. The follower places the handle of the chain in contact with the peg and the entire procedure is repeated till the line is chained.
7. In the end, if some fractional length remains, it is measured by counting the links.
8. During the process, the leader inserts the arrows and the follower picks them up at every chain length. After every tenth chain length the follower erects a ranging.

ON SLOPING GROUND

There are two methods by which the actual horizontal distance can be obtained.

Direct Method

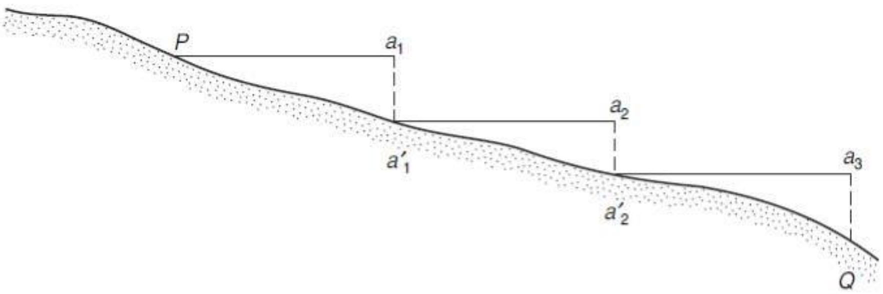
The process of chaining a line on sloping ground is as follows.

1. The follower holds the zero end of the chain at P on the ground while the leader holds its broken end a1 at a suitable length (say 20 or 30 links) horizontally, as shown in Fig.
2. The follower then ranges the leader in line with Q.
3. The leader transfers the end a1 to the ground by means of a plumb bob and marks the point a1 on the ground with an arrow.
4. The follower moves to a1 and holds the zero end of the chain at that point.

5. Steps 1 to 4 are repeated until the end Q is reached.

6. The horizontal distance PQ is the sum of all such measured distance:

PQ = Pa1 + a1 a2 + a2 a3 +……



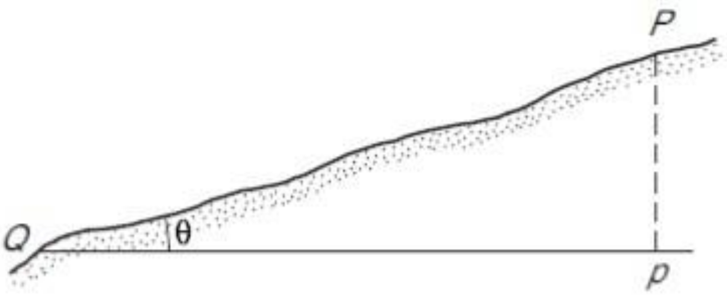
Direct Method

Indirect Method

Wherever the chain can be conveniently held on ground, it may be easier to measure a slope distance PQ as shown in Fig. 2.20 and then the corresponding horizontal distance PQ can be computed.

First Method

The angle PQp (Fig. 2.20) can be measured by a clinometer or on the vertical circle of the transit. Then, pQ = PQ cos θ



Second Method

The horizontal distance pQ may be found by applying hypotenusal allowance (Fig.) derived as follows.

Let = angle of slope of the ground.

pQ = p_1Q = 1 chain length

PQ = chain length sec θ

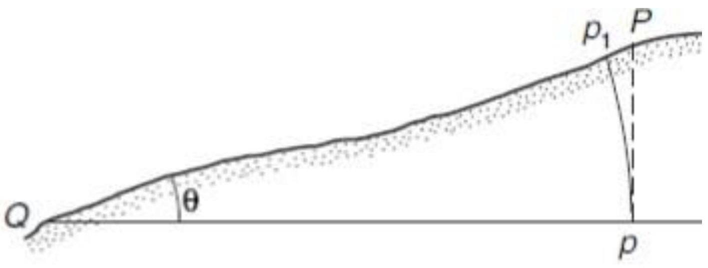
Hypotenusal allowance,

Pp_1 = chain length (sec θ – 1)

Therefore, for measuring a distance

on slope by this method, the chain is stretched in position p1Q with the arrow

placed in advance by an amount Pp1. The next chain length starts from P.



Third Method

Another method of measuring horizontal distance consists in measuring the slope distance l (PQ) and the difference in elevation h (Fig.) between the two points by a level. Required horizontal distance is pQ = $\sqrt{L^2 - h^2}$ and, slope correction = $h^2/2L$

COMPASS SURVEY - BASIC PRINCIPLES

The direction of the survey lines is measured with the help of an instrument known as compass. The direction of survey lines may be defined in two ways: (i) relative to each other, (ii) relative to some reference direction. In the first case, the directions are expressed in terms of angles between two consecutive lines, measured with a theodolite. In the second case, these are expressed in terms of bearings, measured with a compass.

Definitions

Meridian

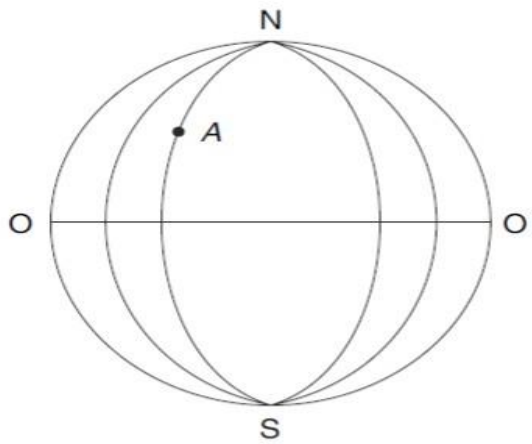
It is the fixed direction in which the bearings of survey lines are expressed.

Bearings.

It is the horizontal angle between the reference meridian and the survey line measured in clockwise or anticlockwise direction. The bearing is described either from north or south and the angle described is either east or west. The bearing of a line is obtained with the aid of whole circle bearing, quadrantal bearing (reduced bearing) and grid bearing (in geodetic survey).

True Meridian:

The true meridian passing through a point on the earth's surface is the line in which a plane passing through the given point (say A) and the geographic (true) north and south poles, intersects the surface of the earth. It represents the true north-south direction at the place.



The True Meridian

True Baring:

The horizontal angle measured clockwise between the true meridian and the line is called true bearing of the line.

Grid meridian

Grid meridian is the reference meridian for a country on a national survey map. The vertical grid lines on a national survey map indicate the direction of grid north. For survey of a country, the true meridian of a central place is regarded as the reference meridian. All the other meridians in the country are assumed to be parallel to the grid meridian.

Grid bearing:

The horizontal angle which a line makes with the grid (central) meridian is called grid bearing.

Magnetic meridian

It is the direction indicated by a freely suspended and balanced magnetic needle unaffected by local attractive forces. The location of the magnetic poles is constantly changing; hence the direction of magnetic meridian also changes. However, the magnetic meridian is employed as a line of reference on rough surveys.

Magnetic bearings:

The horizontal angle which a line makes with the magnetic meridian is called magnetic bearing. It varies with time.

Arbitrary meridian:

It is any convenient direction, usually from a survey station to some well-defined permanent object. The first line of survey at times is also taken as arbitrary meridian.

Arbitrary bearing: The horizontal angle measured with respect to the arbitrary meridian is called arbitrary bearing.

Azimuth:

When survey is done for a large area, i.e., when curvature of earth is accounted for (in geodesy), bearing of lines are sometimes reckoned as azimuth. The azimuth is called geographic if it is reckoned from the geographic meridian, and magnetic, if reckoned from the magnetic meridian in the same manner as that for bearings.

TYPES OF COMPASSES

Surveying compass may be classified as trough compass, tubular compass, prismatic compass and surveyor compass.

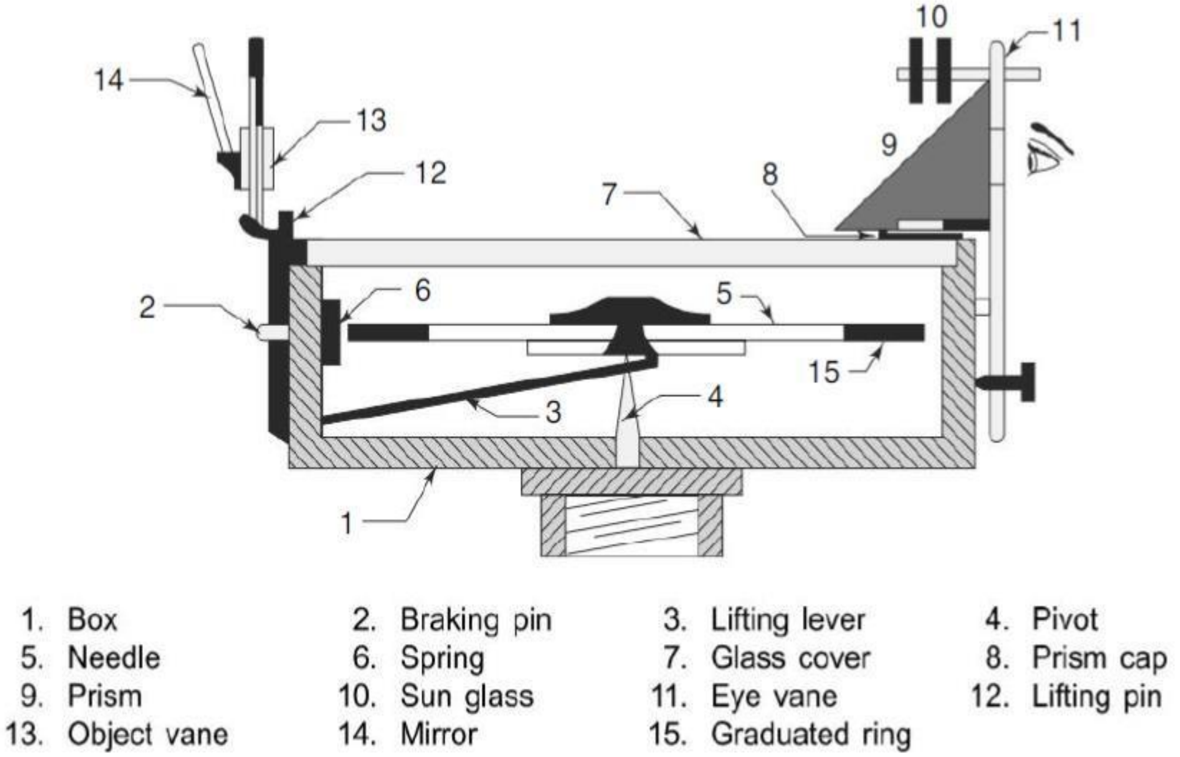
Trough compass

It consists of a long magnetic needle in a narrow rectangular box. The needle of trough compass consists of a long, narrow, magnetised bar of steel, pointed at both ends with the usual agate bearing at the centre. At each end of the box is a block of metal, on which is engraved a zero line and a very short graduated arc extending about 5° on either side of the zero mark. When it is used in conjunction with a plane table, the sides of the box are used as a ruler to plot the north direction. When fitted on a theodolite (generally attached by screws to the side of one of the standards), it is used to align the telescope in the meridian.

Tubular compass

A tubular compass is an improved version of a trough compass. In a tubular compass, the magnetic needle is contained in a tube, at one end of which an eye piece and a diaphragm carrying a glass plate with vertical rulings is fitted. This is nearly in the same plane as one end of the needle. The reticule being suitably illuminated by a reflector, the observer on looking through the eye piece, sees the end of the needle without any parallax.

Prismatic Compass



Prismatic Compass

1. It consists of a circular box about 100 mm in diameter.

2. There is a broad magnetic needle balanced on a hard steel pointed pivot (Fig.).

3. An aluminium ring, graduated to degrees and half degrees is attached to the needle. A prism is provided on the observer's side to read the bearing. The ring is graduated from the south end of the needle. The observations run clockwise round to 360° with zero placed at south as shown in Fig. This is done to facilitate direct reading of the bearings. The figures on the graduated ring are engraved inverted as they are viewed through the prism.

4. When the needle is balanced on the pivot, it orients itself in the magnetic meridian and the north and south ends of the ring face the N–S direction.
5. The object vane carries a vertical hair of fine silk thread attached to a suitable frame.
6. The sight vane consists of a vertical slit cut into the upper assembly of the prism. The two vanes are hinged at the box in diagonally opposite directions.
7. The object vane is sometimes provided with a hinged mirror which can be raised upwards or lowered downwards and can also be slided, if required, to sight the objects too high or too low. Figure 3.4 explains the use of the mirror.
8. Sunglasses are provided on the prism to sight luminous objects. The inverted figures in the graduated ring below the prism can be read erect after being reflected from the hypotenuse side of the prism, when the observer looks horizontally into the prism.
10. The two perpendicular faces of the prism are made convex, so that it also acts as a magnifier.
11. When not in use, the object vane may be folded on the glass lid. It presses against a lever which lifts the needle off the pivot, thus preventing undue wear of the pivot point.
12. Breaking pin, provided at the base of the object vane is used to dampen the oscillations of the needle to facilitate the reading.
13. A prismatic compass reads the whole circle bearing of the lines of objects directly.

SURVEYOR COMPASS

Surveyor compass acquires its name from its extensive use by surveyors. But the prismatic compass has now replaced it as it is light, compact, and handy. It is similar in construction to the prismatic compass except for a few differences as follows:
1. The graduated ring is attached to the circular box and not to the magnetic needle (Fig. 3.5).
2. The edge bar type magnetic needle floats freely over the pivot and is not attached to the ring. When the magnetic needle is lowered to its pivot, it will come to rest pointing north.
3. The eye vane consists of metal vane with a fine sight hole.
4. As the compass box is turned, the letters N, E, S, and W turn with it, but the needle continues to point towards the north and gives a reading which is dependent on the position of the graduated circle.
5. The 0° is placed at both north and south directions and 90° is marked at east and west directions.
6. The east and west markings are interchanged from their normal position as shown in Fig. to read the bearings in the proper quadrant. Suppose the compass is rotated to point N30°30 E. In reading the bearing, the north end of the needle will be found between the letters N and E or 30°30 from N towards E. If W had

been on the left in place of E, as one naturally expects it to be, the north end of the needle would fall between N and W, which might lead to the mistake calling the bearing to be NW instead of NE.

DRFINITION

Surveying may be defined as an art to determine the relative positions of points on, above or beneath the surface of the earth, with respect to each other, by measurements of horizontal and vertical distances, angles and directions. Surveying may also be defined as the science of determining the position, in three dimensions, of natural and man-made features on, above or beneath the surface of the earth.

CLASSIFICATION OF SURVEY

I. BASED ON ACCURACY DESIRED

1. Plane Survey

Survey in which the mean surface of earth is regarded as plane surface and not curved as it really is, is known as plane surveying. The following assumptions are made:

(a) A level line is considered a straight line and thus the plumb line at a point is parallel to the plumb line at any other point.

(b) The angle between two such lines that intersect is a plane angle and not a spherical angle.

(c) The meridians through any two points are parallel.

2. Geodetic survey

Survey in which the shape (curvature) of the earth's surface is taken into account and a higher degree of precision is exercised in linear and angular measurements is termed as geodetic surveying. Such surveys extend over large areas. The measurements must be made to the highest possible standard.

II. BASED ON PURPOSE OF SURVEY

1. Engineering Survey

Surveys which are done to provide sufficient data for the design of engineering projects such as highways, railways, water supply, sewage disposal, reservoirs, bridges, etc., are known as engineering surveys.

2. Defence survey

Surveys have a very important and critical application in the military. They provide strategic information that can decide the course of a war. Aerial and topographical maps of the enemy areas indicating important routes, airports, ordnance factories, missile sites, early warning and other types of radars, anti-aircraft positions and other topographical features can be prepared.

3. Geological Survey

In this both surface and subsurface surveying is required to determine the location, extent and reserves of different minerals and rock types. Different types of geological structures like folds, faults and unconformities may help to locate the possibility of the occurrence of economic minerals, oils, etc.

4. Geographical survey

Surveys conducted to provide sufficient data for the preparation of geographical maps are known as geographical surveys. The maps may be prepared depicting the land use efficiency, sources and intensity of irrigation, physiographic regions and waterfalls, surface drainage, slope height curve and slope profile and contours.

5. Mine survey

In this both surface and underground surveys are required. It consists of a topographic survey of mine property and making a surface map, making underground surveys to delineate fully the mine working and constructing the underground plans, fixing the positions and directions of tunnels, shafts, drifts, etc., and preparation of a geological map.

6. Archaeological survey

These are done to unearth the relics of antiquity, civilisations, kingdoms, towns, villages, forts, temples, etc., buried due to earthquakes, landslides or other calamities and are located, marked and identified. Excavations of the surveyed area lead us to the relics, which reflect the history, culture and development of the era. These provide vital links on understanding the evolution of the present civilisation as well as human beings.

7. Route Survey

These are undertaken to locate and set out the adopted line on ground for a highway or railway and to obtain all the necessary data. The sequence of operations in a route survey is as follows:

(a) Reconnaissance Survey

A visit is made to the site and all the relevant information is collected. It includes collection of existing maps of the area; tracing the relevant map portion over a paper; incorporating the details of the area, if missing, by conducting rough survey.

(b) Preliminary Survey

It is the topographical survey of the area in which the project is located. Sometimes an aerial survey is done if the area is extensive. It includes the depiction of the precise locations of all prominent features and fixing the position of the structure on the map.

(c) Control Survey

It consists in planning a general control system for preliminary survey which may be triangulation or traversing. For location survey, it consists of triangulation.

III. Based on place of survey

1. Land Survey

It consists of re-running old land lines to determine their lengths and directions, subdividing the land into predetermined shapes and sizes and calculating their areas and setting monuments and locating their positions. Topographical, city and cadastral surveys are some of the examples of land surveying.

(a) Topographical survey

This is a survey conducted to obtain data to make a map indicating inequalities of land surface by measuring elevations and to locate the natural and artificial features of the earth, e.g., rivers, woods, hills, etc.

(b) City Survey

An extensive survey of the area in and around a city for fixing reference monuments, locating and improving property lines, and determining the configuration and features of the land, is referred to as a city survey.

(c) Cadastral survey

This is referred to extensive urban and rural surveys made to plot the details such as boundaries of fields, houses and property lines. These are also known as public land surveys.

2. Hydrographic Survey

It deals with the survey of water bodies like streams, lakes, coastal waters and consists in acquiring data to chart the shore lines of water bodies. It also determines the shape of the area underlying the water surface to assess the factors affecting navigation, water supply, subaqueous construction, etc.

3. Underground Survey

This is referred to as the preparation of underground plans, fixing the positions and directions of tunnels, shafts and drifts, etc. This consists in transferring bearings and coordinates from a surface base line to an underground baseline. An example of this kind of survey is mine surveying.

4. Aerial Survey

When the survey is carried out by taking photographs with a camera fitted in an aeroplane, it is called aerial or photogrammetric surveying. It is extremely useful for making large-scale maps of extensive constructional schemes with accuracy. Though expensive, this survey is recommended for the development of projects in places where ground survey will be slow and difficult because of a busy or complicated area.

IV. Based on Instrument used

1. Chain Survey:

When a plan is to be made for a very small open field, the field work may consist of linear measurements only. All the measurements are done with a chain and tape. However, chain survey is limited in its adaptability because of the obstacles to chain like trees and shrubs. Also, it cannot be resorted to in densely built-up areas. It is recommended for plans involving the development of buildings, roads, water supply and sewerage schemes.

2. Traverse Survey:

When the linear measurements are done with chain and tape and the directions or angles are measured with compass or transit respectively, the survey is called traversing. In traversing, speed and accuracy of the field work is enhanced. For example, the boundaries of a field can be measured accurately by a frame work of lines along it forming an open traverse. On the other hand, in a densely populated area, the survey work can be carried out with a frame work of lines forming a closed traverse. A traverse survey is very useful for large projects such as reservoirs and dams.

3. **Tacheometry Survey:**

   This is a method of surveying in which both the horizontal and vertical distances are determined by observing a graduated staff with a transit equipped with a special telescope having stadia wires and anallatic lens. It is very useful when the direct measurements of horizontal distances are inaccessible. It is usually recommended for making contour plans of building estates, reservoirs, etc.

4. **Levelling:**

   This is a method of surveying in which the relative vertical heights of the points are determined by employing a level and a graduated staff. In planning a constructional project, irrespective of its extent, i.e., from a small building to a dam, it is essential to know the depth of excavation for the foundations, trenches, fillings, etc. This can be achieved by collecting complete information regarding the relative heights of the ground by levelling.

5. **Plane Table Survey:**

   It is a graphical method of surveying in which field work and plotting are done simultaneously. A clinometer is used in conjunction with plane table to plot the contours of the area and for filling in the details. This method of surveying is very advantageous as there is no possibility of omitting any necessary measurement, the field being in view while plotting. The details like boundaries, shore lines, etc., can be plotted exactly to their true shapes, being in view. The only disadvantage of plane tabling is that it cannot be recommended in humid climate.

6. **Triangulation Survey:**

   When the area to be surveyed is of considerable extent, triangulation is adopted. The entire area is divided into a network of triangles. Any one side of any of the triangles so formed, is selected and is measured precisely. Such a line is called baseline. All the angles in the network are measured with a transit. The lengths of the sides of all the triangles are then computed, from the measured length of the baseline and the observed corrected angles, using sine formula. $a/\sin A = b/\sin B = b/\sin C$

7. **Electro Magnetic Distance Measurement:**

   This is the electronic method of measuring distances using the propagation, reflection and subsequent reception of either light or radio waves. The examples of EDM instruments are tellurometer, geodimeter, distomat, etc.

8. **Total Station Survey:**

The electronic theodolites combined with EDMs and electronic data collectors are called total stations. A total station reads and records horizontal and vertical angles, together with slopes distances. The instrument has capabilities of calculating rectangular coordinates of the observed points, slope corrections, remote object elevations, etc. The surveys carried out using total station are called total station survey.

9. **Satellite Based Survey:**

Remote sensing and global positioning system (GPS) are the satellite-based surveys. Acquiring data for positioning on land, on the sea, and in space using satellite-based navigation system based on the principle of trilateration is known as GPS. Global positioning system uses the satellite signals, accurate time and sophisticated algorithms to generate distances in order to triangulate positions.

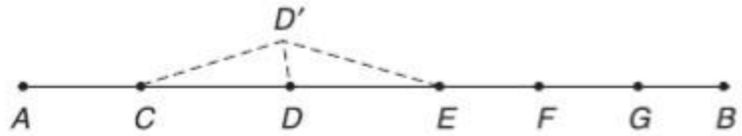
PRINCIPLES OF SURVEYING

There are two basic principles of surveying. These find their inherent applications in all the stages of a project, i.e., from initial planning till its completion.

1. To work from whole to part.
2. To locate a point by at least two measurements.

TO WORK FROM WHOLE TO PART

It is the main principle of surveying and a method violating the principle of working from whole to part should not be adopted until and unless there is no alternative. The main idea of working from whole to part is to localise the errors and prevent their accumulation. This can be explained by taking a simple example of measuring a horizontal distance AB, say about 120 m with a 20 m chain Fig.(a) The process consists in measuring the distance in parts, as the length of chain is smaller than the distance to be measured and is accomplished by the process of ranging. There can be two alternatives as follows.



(Fig.(a))

In one of the method also called the direct method, various points such as C, D, and E are established independently at a distance of about 20 m each with respect to the two end control points and the distance AB can be measured. As C, D, E, etc., are established independently with respect to the main control points, error, if any, introduced in establishing any intermediate point will not be carried in establishing the other points. For example, suppose that point D has been established out of the line AB, as D (Fig.(a)) and E, F, etc., have been established correctly. The actual distances DC and DE will be in error (D C and D E) but all other distances AC, EF, FG, etc., will be correct. Therefore, the error in this procedure is localised at point D and is not magnified. This method observes the principle of working from whole to part.

In the other method, a part, say AC, of the whole distance AB to be measured is fixed.

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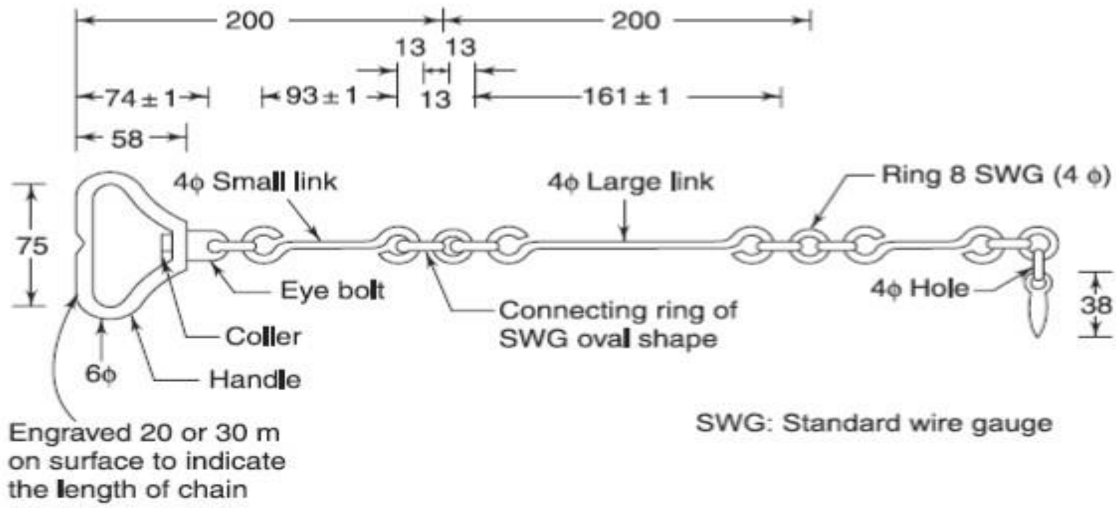
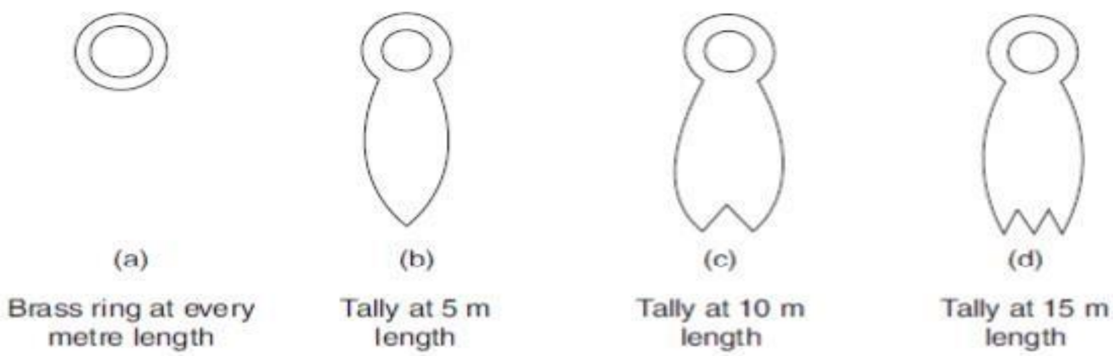
THE EQUIPMENT'S AND ACCESSORIES REQUIRED FOR CHAINING

(A) CHAIN

Gunter, revenue, engineer and metric chain are the various types of chains which are normally used for surveying. The chains are mostly divided into 100 links. While Gunter's chain is 66 ft. long (100 links), the revenue chain is 33 ft. long (16 links) and the engineer's chain is 100 ft. long (100 links). Metric chains are either 30 m (150 links) or 20 m (100 links) in length.



Details of Metric Chain



Ring and Tallies of a Chain

(B) TAPE

Tapes are available in a variety of materials, lengths and weights. The different types of tapes used in general are discussed as follows.

(a) Cloth or Linen Tape:

This is closely woven linen or synthetic material and is varnished to resist the moisture. These are available in lengths of 10 –30 m and widths of 12–15 mm. The disadvantages of such a tape include: (i) it is affected by moisture and gets shrunk; (ii) its length gets altered by stretching; and (iii) it is likely to twist and does not remain straight in strong winds.

(b) Metallic Tape:

It is a linen tape with brass or copper wires woven into it longitudinally to reduce stretching. As it is varnished, the wires are not visible. These are available in lengths of 20 –30 m. It is an accurate measurement device and is commonly used for measuring offsets. As it is reinforced with wires, all the defects of linen tapes are overcome.

(c) Steel Tape:

These are 1–50 m in length and are 6 –10 mm wide. At the end of the tape a brass ring is attached, the outer end of which is zero point of the tape. Steel tape cannot be used in ground with vegetation and weeds.

(c) Invar Tape:

This is made of an alloy of nickel (36%) and steel, having very low coefficient of thermal expansion (0.122 10 – 6/°C). These are available in lengths of 30, 50 and 100 m and in a width of 6 mm.

The advantages and disadvantages of an invar tape are as follows:

Advantages

1. Highly precise. 2. It is less affected by temperature changes.

Disadvantages

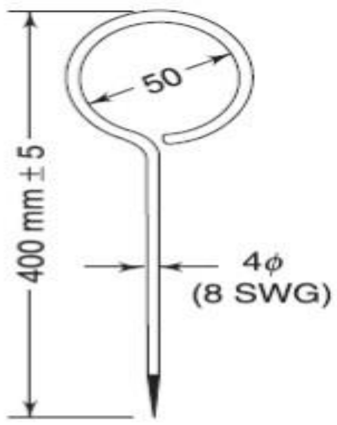
1. It is soft and so deforms easily. 2. It requires much attention in handling.

(C) PEGS

These are used to mark definite points on the ground either temporarily or semi permanently. The size of a peg depends on the use to which the pegs are to be put and the nature of the ground in which they are to be driven.

(D) ARROWS

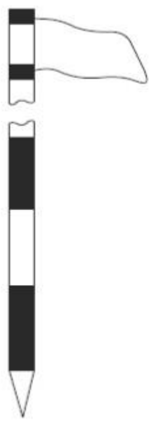
These are also known as chaining pins and are used to mark the end of each chain during the chaining process. These are made of hardened and tempered steel wire 4 mm in diameter. The length of arrow is kept 400 mm. These are pointed at one end whereas a circular ring is formed at its other end. As the arrows are placed in the ground after every chain length, the number of arrows held by the follower indicates the number of chains that have been measured.



Arrow

(E) RANGING RODS

These are also known as flag poles or lining rods. These are made of well-seasoned straight grain timber of teak, deodar, etc., or steel tubular rods. These are used for marking a point in such a way that the position of the point can be clearly and exactly seen from some distance away. These are 30 mm in diameter and 2 or 3 m long. These are painted with alternate bands of either red and white or black and white of 200 mm length. A cross-shoe of 15 mm length is provided at the lower end. A flag painted red and white is provided at the top.

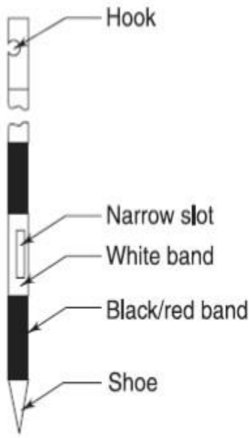


Ranging Rod

(F) OFFSET RODS

These are similar to ranging rods except at the top where a stout open ring recessed hook is provided. It is also provided with two short narrow vertical slots at right angles to each other, passing through the centre of the section, at about eye level.

It is mainly used to align the offset line and measuring the short offsets. With the help of hook provided at the top of the rod, the chain can be pulled or pushed through the hedges or other obstructions, if required.



Offset rods

PLUMB BOB

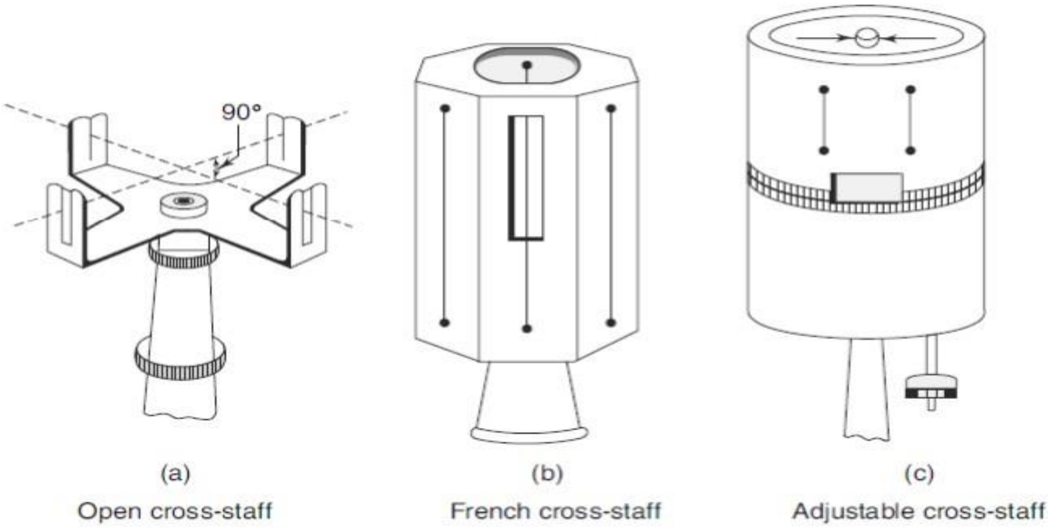
It is made of steel in a conical shape, as shown in Fig. It is used while measuring distances on slopes and in all the instruments that require centring. Before starting the work, it should be ensured that there are no undesirable knots in the thread of the plumb bob.

Plumb bob.

CROSS-STAFF

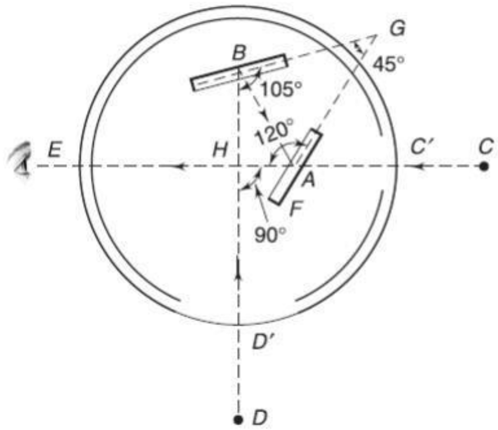
It is essentially an instrument used for setting out right angles. In its simplest form it is known as open cross-staff (Fig. 2.11(a)). It consists of two pairs of vertical slits providing two lines of sight mutually at right angles. Another modified form of the cross-staff is known as French cross-staff (Fig. 2.11(b)). This consists of an octagonal brass tube with slits on all eight sides. This has a distinct advantage over the open cross-staff as with it even lines at 45° can be set out from the chain line. The latest modified cross-staff is the adjustable cross-staff (Fig.).

It consists of two cylinders of equal diameter placed one above the other. The upper cylinder can be rotated over the lower one graduated in degrees and its subdivisions. The upper cylinder carries the vernier and the slits to provide a line of sight. Thus, it may be used to take offsets and to set out any desired angle from the chain line.



OPTICAL SQUARE

This is a compact hand instrument to set out right angles and is superior to the cross-staff. It is a cylindrical metal box about 50 mm in diameter and 12.5 mm in depth. Figure. shows the plan of its essential features. It has two oblong apertures C and D on its circumference at right angles to each other. E is a small eye-hole provided diametrically opposite to C.

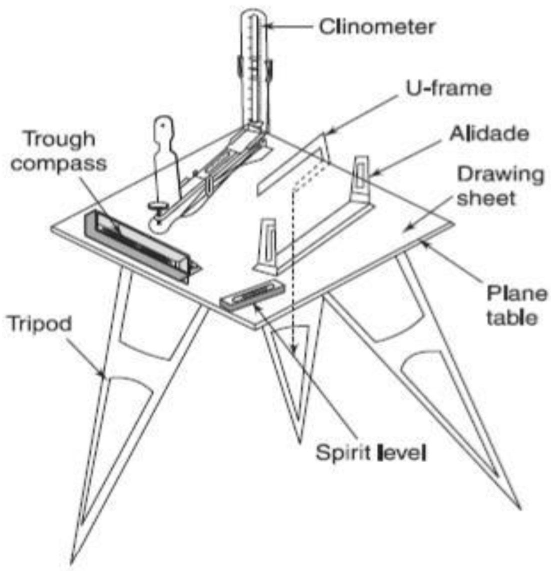


SETTING OUT A RIGHT ANGLE FROM CHAIN LINE

Suppose the optical square is required to set out a perpendicular from a point H on a chain line EC, to a curved boundary, as shown in Fig. 2.13. The surveyor stands at H with the optical square at the eye level and turns it until a signal at C is seen directly through the transparent portion of the horizon mirror. The curved boundary will also be visible through the silvered portion of the horizon mirror. The surveyor then directs the assistant at the curved boundary to move left or right until the signal D held by the assistant appears to coincide exactly with the signal C seen directly. The line HD will be the required perpendicular to the chain line EC.

PLANE TABLE AND ITS ACCESSORIES

The plane table is an instrument used for surveying by a graphical method in which the field work and plotting are done simultaneously. It is most suitable for small and medium scale-mapping (1:10,000 to 1: 2,50, 000). plane tabling is now not so universally used.



Plane table with accessories

A plane table is a drawing board mounted on a tripod. An alidade is used to plot the directions and a clinometer to measure the elevations. Accessories such as a plumbing fork or U-frame, trough compass, spirit level, drawing sheet and waterproof cover are also required for the field work. In using the plane table, a drawing sheet is mounted, with adhesives or pins, on the drawing board. Before commencing a plane table survey, the instrument stations are fixed to control the entire area.

The elevations of the points of observation are measured with an Indian clinometer or telescopic alidade. All the measurements made are plotted directly on the drawing sheet
instead of recording in the field book.

The principle used in plane table surveying is that an unknown point of interest can be established by measuring its directions from known points.

Advantages:

1. The observations and plotting are done simultaneously. Hence, there is no risk of omitting necessary details.
2. The errors and mistakes in plotting can be checked by drawing check lines.
3. Irregular objects can be plotted accurately as the lay of land is in view.
4. It is most rapid and useful for filling in details.
5. No great skill is required.
6. It is less costly than theodolite survey.

7. It is advantageous in magnetic areas, where compass survey is not reliable.

Disadvantages:

1. It is not suitable for work in a wet climate and in a densely wooded country.
2. The absence of measurements (field notes) are inconvenient, if the survey is to be replotted to some different scale.
3. It is heavy and awkward to carry and the accessories are likely to be lost.
4. It does not give very accurate results.

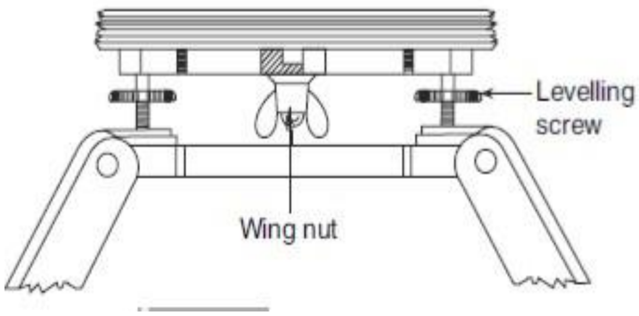
DESCRIPTION OF PLANE TABLE

A plane table instrument, as stated above, consists of a drawing board mounted on a tripod in a way so that the board can be levelled, rotated about a vertical. axis, and clamped in any required position. It also consists of an alidade and some accessories.

Board

The drawing board is carefully made of well-seasoned wood in a way to counteract the effect of warping and damages from weathering. The upper surface is kept smooth. The table at the centre of the underside, is attached to the tripod by means of a screw and wing nut (Fig. 8.3). By means of the wing nut, the table can be clamped in any position. Plane tables are available in the following different sizes.

Designation	Size (mm × mm)
B_0	1500 × 1000
B_1	1000 × 700
B_2	700 × 500
B_3	500 × 350

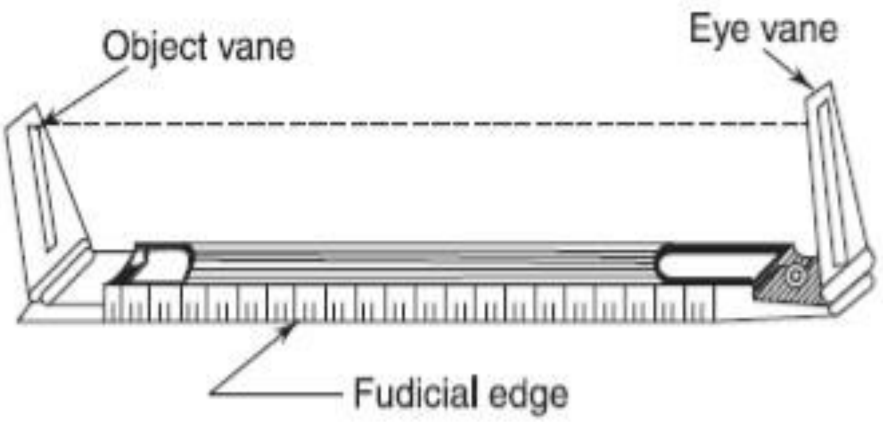


Tripod

An open frame type light tripod is usually provided. In the simplest form of plane tables, levelling of the board is achieved by manipulating the tripod legs and checking the horizontality of the board by means of two spirit levels fixed at right angles to each other in a block of wood. For a beginner it is rather difficult to keep the plane table level throughout the work, since even with a slight pressure on any side of the table, the level of the board is disturbed. In some of the other forms of the tripod heads, levelling screws (Fig.), or ball-and-socket joint (Fig.) is provided to facilitate levelling.

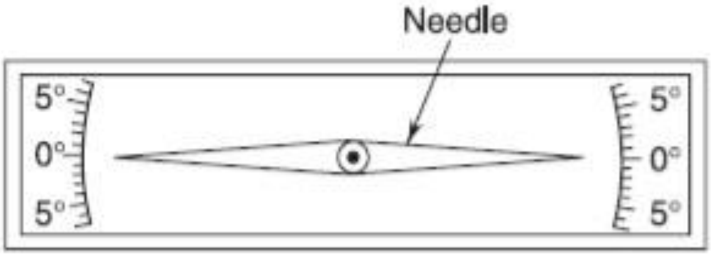
Alidade

It is a wooden or brass ruler of about 50–60 cm in length. It is also known as sight rule. Two vanes, the ‘object vane’ and the ‘sight vane’ (Fig.), are hinged at its two ends. It is essential that the plane of the vanes should be perpendicular to the underside of the alidade while the observations are made. These vanes should be folded over the alidade top surface, when not in use. The line of sight thus provided, is parallel to the ruling or fudicial edge of the alidade, but it is unnecessary that the line of sight be parallel to the fudicial edge, provided the horizontal angle between the two remains constant. A scale is attached to the bevelled fudicial edge so as to plot distances to the scale.



Trough compass

Usually it is 15 cm long (Fig.) and is provided to plot the magnetic meridian (N – S direction) to facilitate orientation of the plane table in the magnetic meridian. Although a trough compass is sufficiently accurate for field surveying, it is not precise owing to a parallax arising from the difficulty of ensuring that the eye is in the vertical plane of the needle. To overcome this difficulty the trough compass is modified. An eyepiece and a diaphragm are placed on one side of a tube having a magnetic needle inside. Such a compass is known as tubular compass. The diaphragm of the tubular compass consists of a glass plate with vertical rulings, which is in the same plane as one end of the needle. The observer, on looking through the eyepiece, sees the end of the needle without the parallax.

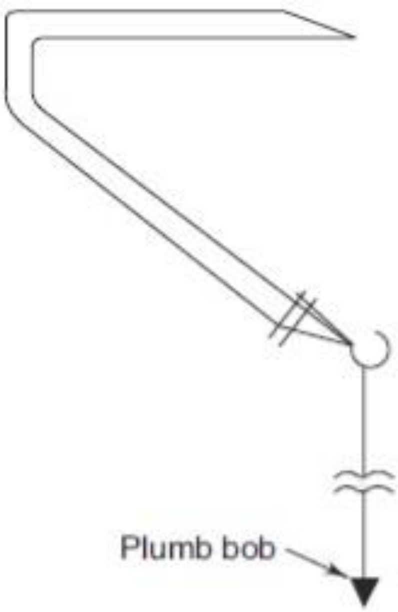


Spirit level

The essential condition in plane table surveying is that the board should be level. This is usually accomplished with a circular spirit level. It is placed on the board in two positions mutually at right angles and the bubble is centred in each position to make the board horizontal.

Plumbing fork

It is a hairpin-shaped brass frame (Fig.) having two arms of equal length. One end of the frame is pointed and is kept over the drawing sheet touching the plotted position of the instrument station. The other end of the frame carries a plumb bob. The position of the plane table is adjusted until the plumb bob hangs over the Plumb bob station occupied by the instrument. The use of a plumbing fork is justified only if the scale of plotting is large, the rays being short. However, for small-scale mapping, which is usually done with a plane table, the use of plumbing fork is a sheer waste.



SETTING UP THE PLANE TABLE

It includes the following operations: (1) Centring, (2) Levelling, (3) Orientation.

Centring

It is the operation of bringing the plotted station point exactly over the ground station. To achieve this the pointed leg of the plumbing fork is placed against the plotted point and the plumb bob is suspended from its other leg. Exact centring is important for large-scale mapping only. For small-scale mapping, an error in centring of about 30 cm is permissible.

Levelling

It is the operation of bringing the plane table in a horizontal plane. Set the plane table at a convenient height, which is elbow level, by spreading the legs. Level the board with the help of a spirit level.

Orientation

It is the operation of keeping the plane table parallel to the position it occupied at the first station. In such a condition all the lines plotted will be parallel to the corresponding lines on the ground. If the board position is different at successive stations, the relative positions of the plotted details will not remain the same as the relative positions of the details on the ground.

Consequently, the plotted work of the previous stations cannot be connected to that of the successive stations. It should be noted that during orientation the table is rotated and the plotted position of the instrument station is also disturbed and shifts relative to the ground stations except when the plotted point happens to lie on the vertical axis of the instrument.

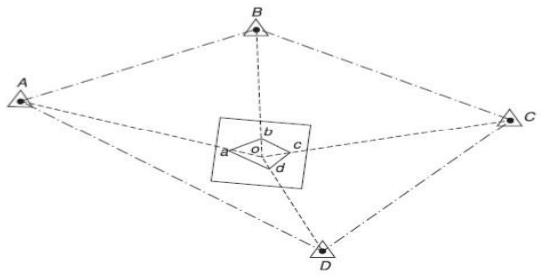
The operations of orientation and centring are therefore interrelated. Since accurate orientation is an essential condition, a compromise can be made with centring, though within permissible limits. Whenever an exact centring is required, for example, in large-scale surveys, repeated orientation and centring by shifting the table are necessary.

METHODS OF PLANE TABLE SURVEYING

The methods of surveying with a plane table are radiation, traversing, intersection and resection. In the figures illustrating these methods, capital letters such as A, B and C have been used to indicate the ground points and small letters such as a, b and c are their corresponding plotted positions on the drawing sheet.

RADIATION

In this method the instrument is setup at a station and rays are drawn to various stations which are to be plotted. The distances are cut to a suitable scale after actual measurements (Fig.).



Radiation with plane table

Procedure:

Select a station O such that all the other stations A, B, C and D are accessible and visible from O. Plot the N – S direction. Setup a plane table at O. Place the alidade at o and successively sight stations A, B, C and D. Draw rays from o to the stations and cut the distances oa, ob, oc and od to the chosen scale. Join a, b, c and

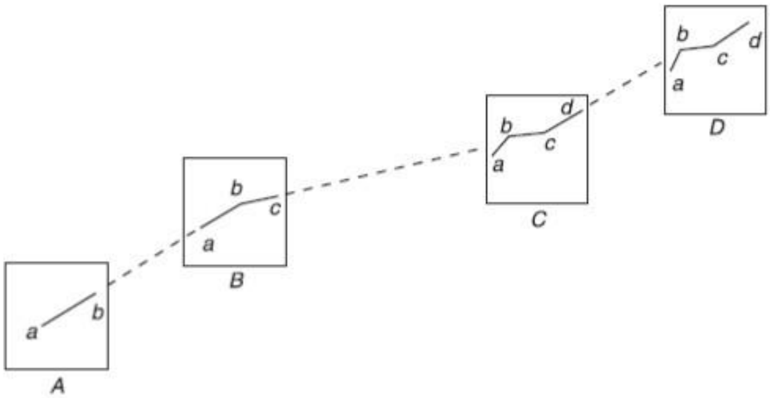
d. This method is suitable only when the area to be surveyed is small and all the stations are visible and accessible from the instrument station.

TRAVERSING

This method is similar to compass or theodolite traversing. The table is set at each of the stations in succession. A foresight is taken to the next station and the distance is cut to a suitably chosen scale.

Procedure:

Set up the plane table at the initial station A (Fig.). Transfer ground station A as a on the drawing sheet. Draw a ray aB along the fudicial edge with the alidade pivoted against a. Cut the distance ab to the selected scale. Shift and set up the table at B. Orient the plane table. Place the alidade at b and sight station C. Draw a ray bC along the alidade and cut the distance bc to the selected scale. The procedure is carried out till all the stations are traversed. It is most suited when a narrow strip of terrain is to be surveyed, e.g., survey of roads, railways, etc. This method can be used for traversing both the open as well as close traverses.



Traversing with plane table

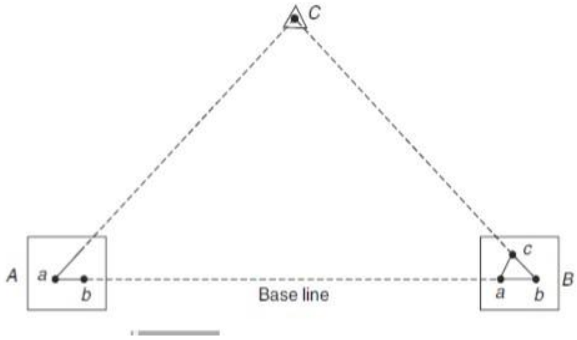
INTERSECTION

In this method two stations are so selected that all the other stations to be plotted are visible from these. The line joining these two stations is called base line. The length of this line is measured very accurately. Rays are drawn from these stations to the stations to be plotted. The intersection of the rays from the two stations gives the position of the station to be plotted on the drawing sheet.

Procedure:

Let A and B be the two accessible stations (Fig.), such that A and B can be suitably plotted. C is the station to be plotted by intersection. Place the plane table at A. Set it up. Plot the N – S direction. Transfer ground station A as a onto the drawing sheet. With the alidade centred at a, sight station B. Draw a ray aB and cut ab to a suitable scale. With the alidade at a, sight C also and draw a ray aC. Shift the table to B and set it up. Place the alidade at b and sight C. Draw a ray bC. The intersection of the two rays gives the position of station C as c on the plane table.

This method is very commonly used for plotting details. It is preferred when the distance between the stations is too large, or the stations are inaccessible, or the ground is undulating. The most suitable example is of broken boundaries which can be very conveniently plotted by this method.



Intersection with plane table

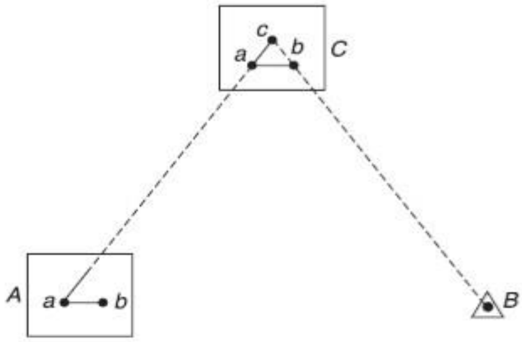
RESECTION

It is a method of orientation employed when the table occupies a position not yet located on the drawing sheet. Therefore, it can be defined as the process of locating the instrument station occupied by the plane table by drawing rays from the stations whose positions have already been plotted on the drawing sheet. The resection of two rays will be the point representing the station to be located, provided the orientation at the station to be plotted is correct, which is seldom achieved. The problem can be solved by any of the methods such as resection after orientation by back ray, by two points, or by three points. These methods are described in the sections to follow.

This method is employed when during surveying the surveyor feels that some important details can be plotted easily by choosing any station other than the triangulation stations. The position of such a station is fixed on the drawing sheet by resection.

Procedure:

Let a and b be the plotted positions of the two ground stations A and B. Station C is to be plotted (Fig.). Set up the table at A, with a above A. Keep the alidade along ab and orient the table so that B is bisected. Pivot the alidade at a, sight C and draw ray aC. Shift the instrument and set it up at C. Place the alidade along ca and rotate the table till it is oriented. With alidade pivoted against b, sight B and draw a back ray. The resection of this ray with the previous ray gives the position of station C as c on the drawing sheet.



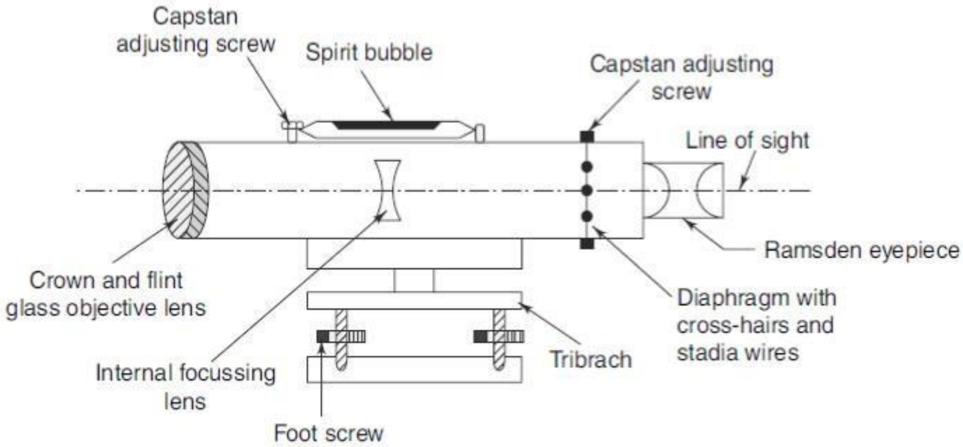
Resection by back ray

TYPESOFLEVEL

Thevarioustypesof levelsusedin surveyingaredescribed below.

Dumpylevel

Thisisthemostwidelyuseddirectlevellinginstrument.Theessentialfeaturesofthedumpylevelareshown in Fig. 6.9. It consists of a telescope which is rigidly fixed to its support. It can neither be rotated about its longitudinal axis nor can it be removed from its support. It is very advantageous when several observations are to be made with one set up of the instrument.



Dumpylevel

WYE-LEVEL

This is similar to the dumpy level except that the telescope in this is supported by two Y-shaped uprights (Fig. 6.10) fixed to a horizontal bar and attached to the vertical spindle about which the instrument rotates. ThetelescopecanbeliftedclearoftheY-supportsbyreleasingthetwoclampingcollarswhichfitacrossthe tops of the Y-supports. Wye-level has an advantage over dumpy level in that its adjustments can be tested rapidly. The disadvantage is that is carries many loose and open parts, which are liable to frictional wear.

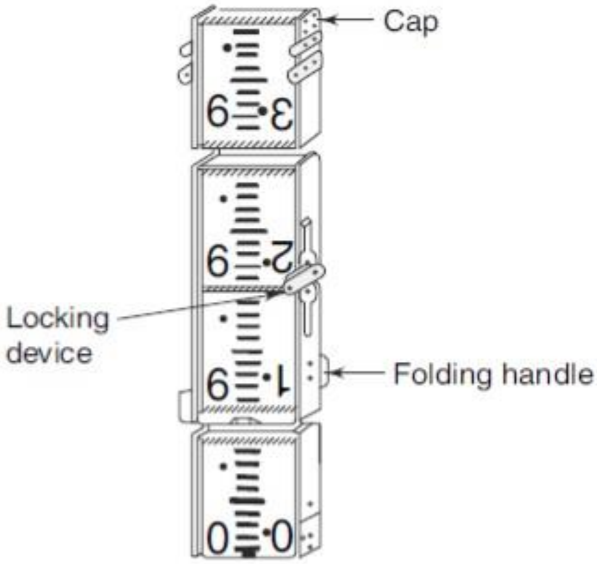
TILTINGLEVEL

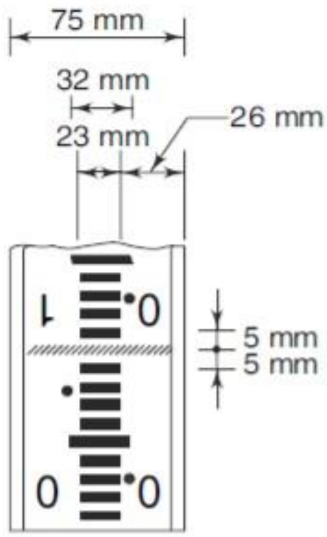
In this type of level the telescope can be rotated about a horizontal axis. It enables the surveyor to quickly centre the bubble and thus bring the line of sight into the horizontal plane.

LEVELLINGSTAFF

A levelling staff is a straight, rectangular, wooden rod graduated into metres and smaller divisions. The readinggivenbythelineofsightonalevellingstaffistheheightofthelineofcollimationfromthepointon whichthestaffisheldvertically.Thesemaybe 3-5minlength.Asolidstaffisusually3mlong,whereas a foldingstaffisgenerally4minlength.Thefoldingstaff(Fig.)ismadeoftwopieceseachof2mlength. The widthandthicknessofthestaffis75mmand18mm,respectively.Afoldingjointisprovidedtoconnectthe two pieces.

Eachmetrelengthofthestaffisdividedinto200divisionsof5mmeach.Thespacesindicatingthedecimetre reading are marked in red while all other spaces are marked alternatelyin black and white. The graduations are marked inverted (Fig.) so that they may appear erect when seen through the telescope.





LevellingStaff(FoldingType) **Graduationsof thestaff**

TEMPORARYADJUSTMENTS

Theseconsist ofsettingup,levelling, andelimination ofparallax.

SettingUp

Levelisnottobesetatanyfixedpointformakingtheobservationsasitiswithothersurveyinginstruments whicharetobesetuponstation,thepointofinterest.Therefore,settingupofalevelismuchsimple;centring is not required. While locating the level, the ground point should be so chosen that (a) the instrument is not toolowortoohightofacilitatereadingonabenchmark,(b)thelengthofthebacksightshouldpreferablybe not more than 98.0 m, and (c) the backsight distance and the foresight distance should be equal, and the foresight should be so locatedthatitadvancesthelineoflevels.Settingupincludesfixingtheinstrumentandapproximatelevelling by leg adjustment.

FixingtheInstrumentOverTripod:

Theclampscrewoftheinstrumentisreleased.Thelevelisheldintherighthand.Itisfixedonthetripodby turning round the lower part with the left hand and is firmly screwed over the tripod.

LegAdjustment:

Theinstrumentisplacedataconvenientheightwiththetripodlegsspreadwellapartandsoadjustedthatthe tripod head is as nearly horizonal as can be judged by the eye. Any two legs of the tripod are fixed firmly intothegroundand thethirdlegis movedrightorleftinacircumferential directionuntilthemainbubbleis approximately in the centre. The third leg is then pushed into the ground.

Levelling:

Theclampisloosenedandtheupperplateisturneduntilthelongitudinalaxisoftheplate levelisparallelto a line joining any two levelling screws, say A and B.

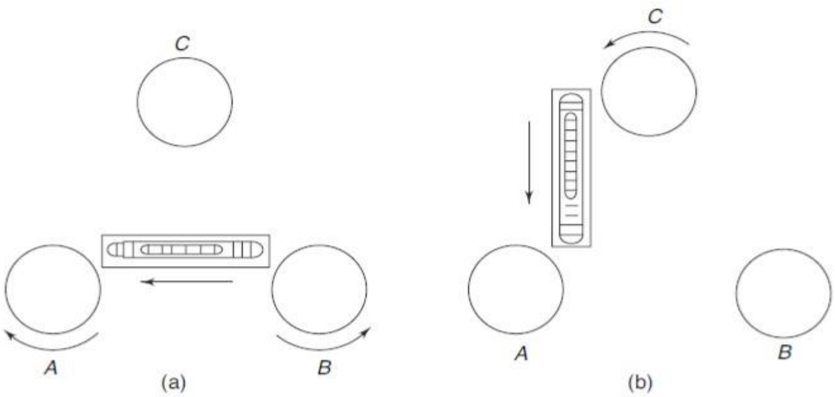
2. Thetwo-footscrewsareturneduniformlytowardseachotherorawayfromeachotheruntiltheplate bubble is central (Fig.(a)).

3. Thetelescopeis swingthrough 90°so thatitliesover thethird footscrew (Fig.6.22 (b)).

4. Thethird screwis turned untiltheplatebubble is central.

5. The telescope is swing again through 90° to its original position and the above procedure is repeated till the bubble remains central in both the positions.

6. Thetelescopeisnowswingthrough 180°.The bubbleshouldremain centraliftheinstrumentisinproper adjustment.



Levellingwiththree-footscrews

Eliminationof Parallax

Itconsistsoffocussingtheeyepiece andobjective ofthe level.

Focussingtheeye piece:

This operation is done to make the cross-hairs appear distinct and clearly visible. The following steps areinvolved:

1. Thetelescopeisdirected skywardsor asheetofwhitepaper is heldin frontof theobjective.

2. Theeyepieceis movedinor outtillthecross-hairappear distinct.

Focussingthe Objective:

Thisoperationisdonetobringtheimageoftheobjectintheplaneofthecross-hairs.Thefollowingstepsare involved:

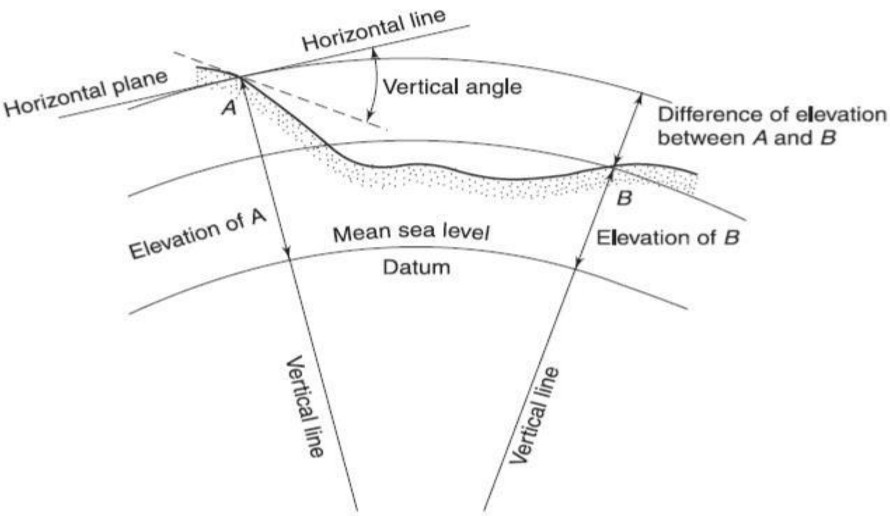
1. Thetelescopeisdirected towardsthestaff.
2. Thefocussingscrewisturneduntiltheimage appearsclear andsharp.

INTROUDCTION

The relative positionofa point interms ofthe verticaldistance, above or below another point is designated by its elevation. The elevation of a point may thus be defined as its vertical distance above or below a reference surface (datum) having zero elevation. Therefore, elevation of a point may be considered as its vertical coordinate. It is treated as positive if the point is above and as negative if the point is below the datum.Gradeandaltitudearethetwotermsfrequentlyusedasanalternatetothetermelevation.Gradeisan expressionofelevationinconstructionactivities, whereasaltitude isthe verticaldistanceofapoint inspace. Usually,sealevelisconsideredtobethestandarddatum,butsometimesanarbitraryassumedsurfaceistaken as the reference. The vertical heights of points above or below a datum are referred to as simply levels or reduced levels and the operation of determining the difference of elevation of points with respect to each other on the surface of the earth is called levelling.

DEFINITIONS

Someofthebasictermsdefined below.



Levellingterms

Levellsurface:

Asurfaceparalleltothemeanspheroidalsurfaceoftheearthiscalled levelsurface, e.g.,astill lake. Alevel surface is a curved surface, everypoint on which is equidistant fromthe centre of the earth. It is normalto the plumb line at all the points.

VerticalLine:

It is a line from any point on the earth's surface to the centre of the earth. It is commonly considered to be the line defined by a plumb line.

Levelline:

Itisalinelyingonalevelsurface.It isnormalto theplumb lineatallthepoints.

HorizontalPlane:

It is a plane tangential to the level surface at the point under consideration. It is perpendicular to the plumb line.

HorizontalLine:

Itis linelyinginthehorizontalplane. Itisastraightlinetangentialtothelevelline.

Elevation:

Elevationofapoint istheverticaldistanceaboveorbelowthedatum. Itisalso knownasreduced level (R.L.).

Axisof telescope:

Itisalinejoiningthe opticalcentreofthe objective tothe centreofthe eyepiece.

Line ofsight:

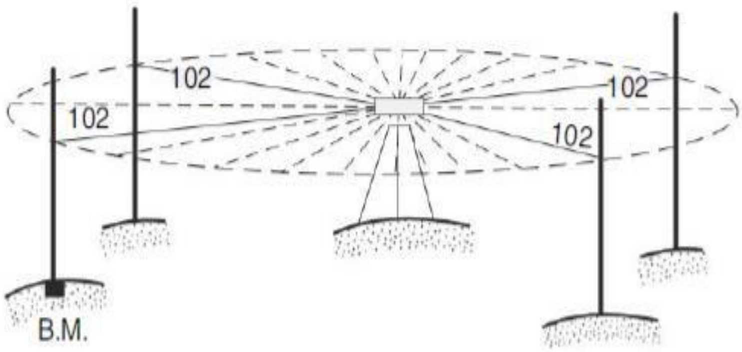
Itisalinejoiningtheintersectionofthecross-hairstotheopticalcentreoftheobjectiveanditscontinuation. Since in levelling the line of sight should remain horizontalwhile making the sights, the line of sight when horizontal is called the line of collimation.

Bubbletube:

Itisan imaginarylinetangentialtothelongitudinalcurveofthetubeatitsmid-point.

HeightofInstrument:

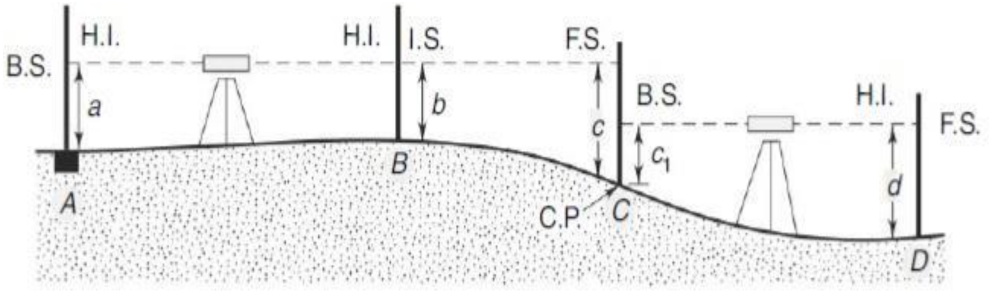
Itistheelevationoftheplaneofcollimationwhentheinstrumentislevelled,e.g.,theheightoftheinstrument is102 min Fig. It shouldbe notedthattheheightofaninstrument doesnot meanthe heightofthecentreof the telescope above the ground, where the level is set up.



HeightofthePlane of Collimation

Backsight:

It is a staffreading taken on a point ofknown elevation, e.g., a sight on a bench mark (station A) or on a changepoint, i.e., stationC. InFig. andc1, arebacksights. Itisthefirst staffreadingtakenafterthe levelis set up. It is also called plus sight.



Measurement ofsightwith level

Foresight:

Itisastaffreadingtakenonapoint whoseelevationisto bedetermined, e.g.,asight onachangepoint, i.e., stationC and D. InFig. c and d are fore sights. It is also called a minus sight. It is the last staffreading and denotes the shifting of the level.

IntermediateSight:

It is a staff reading takenon a point of unknown elevation between backsight and foresight, e.g., a sight on station B. In Fig. b is the intermediate sight.

ChangePoint:

Itisapoint,denotingtheshiftingofthelevel.BothF.S.andB.S.aretakenonthispoint e.g.,stationC(Fig.).

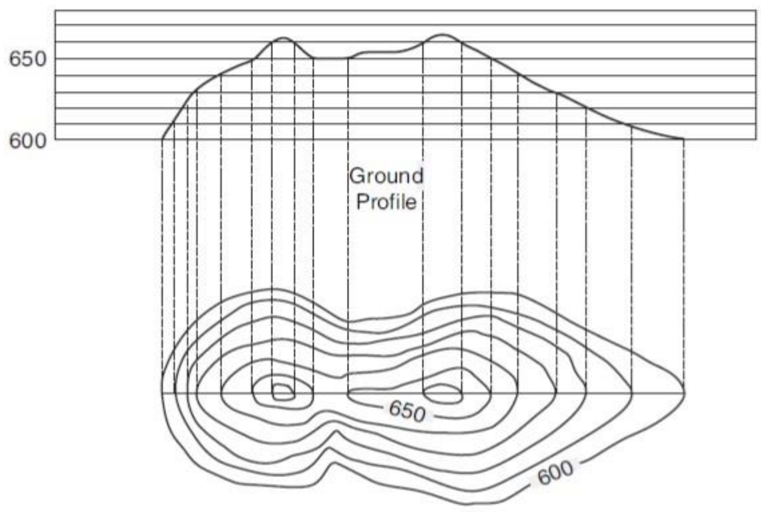
Station:

Apoint,whoseelevationistobedetermined iscalledstation.Thestaffiskept atthispoint,e.g.,A,BandC (Fig.). It is the apparent movement of the image relative to the crosshairs when the image formed by the objective does not fall in the plane of the diaphragm.

CONTOURING.

Acontourmaybedefinedasanimaginarylinepassingthroughpointsofequalelevation.Thus,contourlines on a plan illustrate the conformation of the ground.

A contour line may also be defined as the intersection of a level surface with the surface of the earth. The best method of representation of features such as hills, depressions, undulations, etc., A contour representation along with the ground profile is shown in Fig.From the contour representation of Fig. it is evidentthatsteepertheslopesofthesurfacethemorecrowdedareitscontourlines.Hence,contourlinesare usually found spaced on a map or plan with different densities.



Contourrepresentation

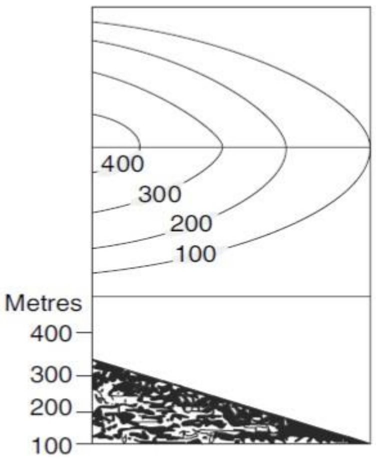
Contourinterval

The vertical distance between consecutive contours is termed as contour interval. It is desirable to have a constantcontourintervalthroughoutthemap.However,inspecialcases,avariablecontourintervalmayalso beprovided.Forexample,inIndia,thehighmountainregionalongthenorthernfrontiershadtobecontoured forsomepartsatdoublethenormalcontourintervalowingtoexcessiveaveragesteepness.Avariablecontour intervalis, as far aspossibleavoided since it givesa false impressionoftherelative steepnessoftheground in different parts of the map. Usually contour intervals are taken as 1 to 15 m.

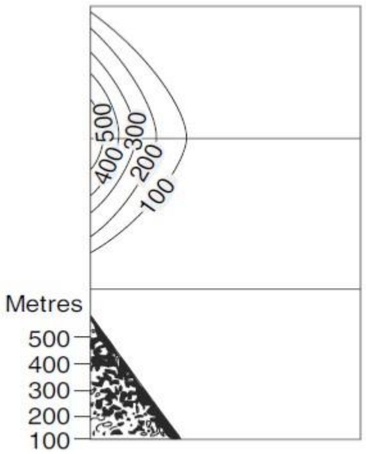
CHARACTERISTICSOFCONTOURLINES:

SLOPES

Aslope maybe gentle or steep. Agradient upto 1in2.5 (20°withhorizontal) is referredto as gentle slope (Fig.(a)),whereashighergradients(20°–45°withhorizontal)aretermedassteepslopes.Averysteepslope is termed as scrap. A high scrap is known as crag.



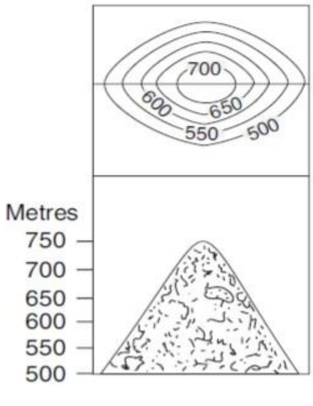
(a) A gentle slope



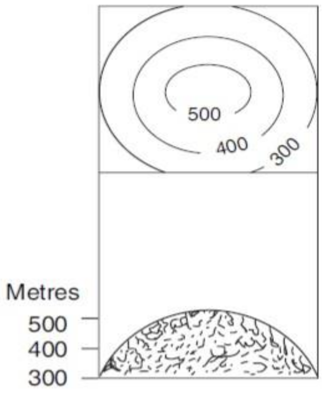
(b)Asteepslope

HIGH-LYING FORMS

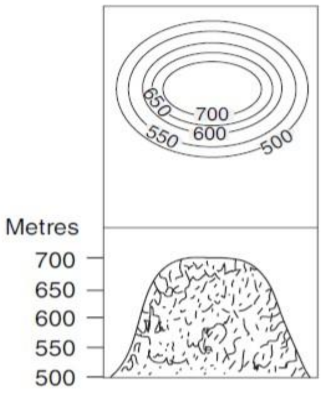
Thesearecharacterisedbyelevatedgrounds,forexamplehill,hillockandplateau.Hillsareelevatedground usuallywithapointedpeak.Thecontoursofahill(Fig.(c))areabitcircularinshapewithincreasingcontour values inwards. Hillocks are elevated lands, quite low in height with gentle side slopes (Fig.(d)). A plateau is a broad relief feature which has a relatively even surface at the top (Fig.(e)), but is conspicuously higher thanthe surrounding land. It isalso knownastable land. Sincethetopisalmost flat, veryfewcontourswill be there as compared to the sides which are often steep and thus have closely spaced contours.



(c) Hill



(d) Hillock

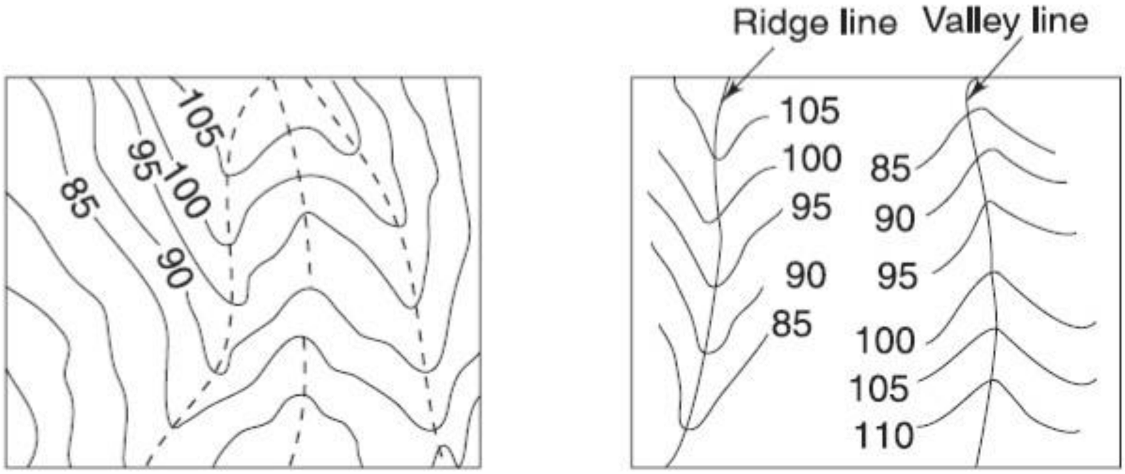


(e)Plateau

VALLEY LINE AND RIDGE LINE

The slopes of ravine intersect along a line referred to as the axis of the ravine, the line of discharge, or a valley line in case of a valley. The counter part of a ravine is a ridge—a convex form of terrain gradually decliningin one direction. Tworavines are usually separated by a more or less pronounced ridge (Fig.(f)).

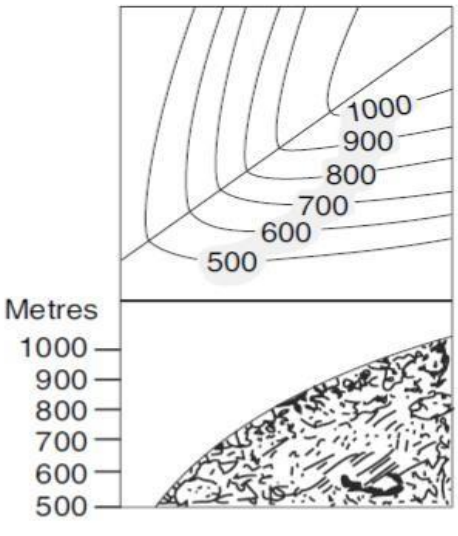
The line along whichthe slopes intersect is referredto asthe axis ofridge, the watershedorwatershed line. The watershed line is usually wavy.



(f) Valleyand Ridge line

SPUR

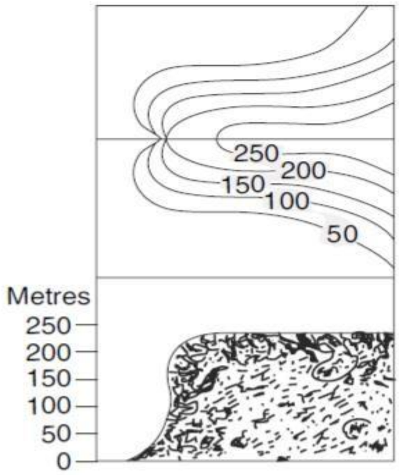
A part of land in formof tongue, which juts out froma hillyarea is known as spur (Fig. (g)). The contours are similar to that ofa valley, with a difference that here the counter values decrease towards the Vee.



(g) Spur

CLIFF:

Contourlinesofdifferentelevationscanunitetoformonelineonly inthecaseofaverticalcliff



(h) Cliff

Two contour lines of different elevations cannot cross each other. Two contour lines having the same elevationcannotuniteandcontinueasoneline Acontourlinesmustcloseuponitselfthoughnotnecessarily withinthelimitsofthemap. Contourlinescrossawatershedorridgelineatrightanglestheyformcurvesof U-shape round it withthe concave side ofthe curvetowardsthe higher ground. Contourlines cross avalley line a right angle they formsharp curves ofV-shape. The same contour appears on either side ofa ridge or valley, for the highest horizontalplane that intersects the ridge must cut it on both sides.

METHODSOFLOCATINGCONTOURS

The locationofa point intopographic survey involves bothhorizontalaswellas vertical control. Ingeneral the field method may be divided into two methods.

i. DirectMethod

ii. Indirectmethod

DirectMethod:

i) Inthedirectmethodthecontourtobeplottedisactuallytracedonthegroundonlythosepointsaresurveyed which happen to be plotted. After having surveyed those points they are plotted and contours are drawn through them.

ii) Themethodisslowandtediousandisusedforsmallareasandwheregreataccuracyisrequired.thefield work is two-fold

i. Verticalcontrol

ii. Horizontalcontrol

Indirectmethods:

i) In thismethod, someguidepointsare selected alongasystem of straightlinesandtheirelevationsare found. The points are then plotted and contours are then drawn by interpolation

ii) Theseguidepointsarenot,exceptbycoincidence,pointsonthecontourstobelocated.whileinterpolating, it is assumed that the slope between any two adjacent guide points is uniform,

The followingaresomeoftheindirectmethodsoflocatingthegroundpoints.

i) BySquares

ii) ByCrosssections

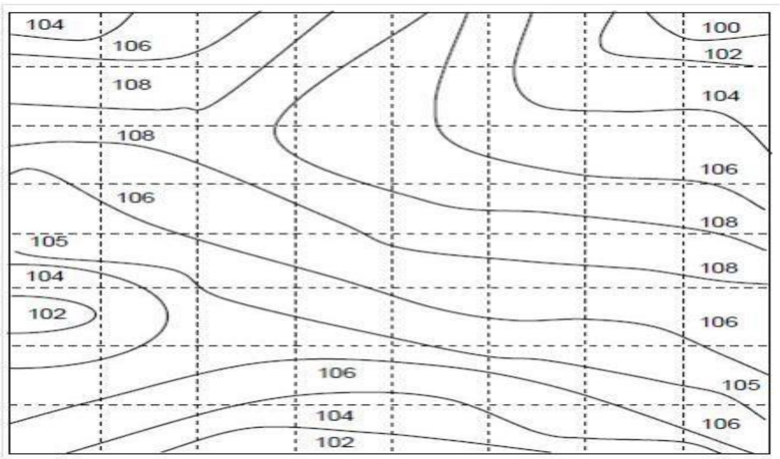
iii) ByTacheometricmethod

1.BySquares

The methodisusedwhentheareatobesurveyedissmallandthegroundis notverymuchundulating.

The are ai s divided into a number of squares. The size o tf he square may vary from 5t o 20 m depending upon the nature of the contour and contour interval. The elevations of the corners of the square are then determined by means of a level and a staff. The contour lines maythen be drawn by interpolation. It is not necessarythatthesquaresmaybeoft,he samesize.Sometimesrectanglesarealso usedinplaceofsquares. Whenthereareappreciablebreaksinthesurfacebetweencorners,guidepointsinadditiontothoseatcorners may also be used.

The squaresshould be as long aspracticable, yet smallenoughto conformto the inequalitiesotfhe ground and to the accuracy required. The method is also known spot levelling.



1. **ByCrosssections:-**

Inthismethod,cross-sectionsareruntransversetothecentrelineofaroad,railwayorcanaletc. Themethod is most suitable for railway route surveys. The cross-sections should be more closely spaced where the contours curve abruptly, a s in ravines or on spurs.

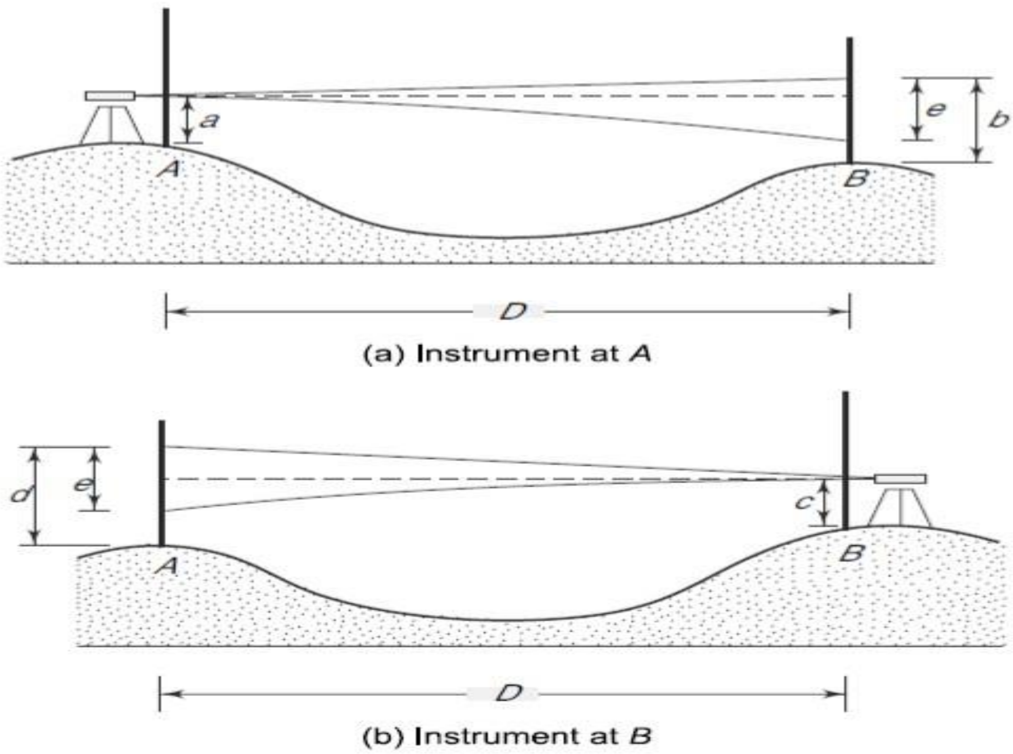
Thecross-sectionandt hepointscanthenbeplottedandtheelevationofeachpoinit smarked. Thecontour lines are interpolated on the assumption that the rei s uniform slope between two points on two adjacent contours. Thust,hepointsmarkedwithdotsarethepointsactuallysurveyedinthefieldwhilethepointsmarkedwith x on the first cross-section are the points interpolated on contours.

RECIPROCALLEVELLING

It is the operation of levelling in which the difference in elevation between two points is accurately determinedbytwosetsofreciprocalobservations.Thismethodisveryusefulwhentheinstrumentcannotbe set up between the two points due to an obstruction such as a valley, river, etc., and if the sights are much longer than are ordinarily permissible. For such long sights the errors of reading the staff, the curvature of earth, and the imperfect adjustments of the instrument become prominent. Special methods like reciprocal levelling should be used to minimize these errors.

Inthismethodtheinstrument issetupnearonepointsayA,ononesideonthevalley,andareadingistaken on the staff held at A(Fig. (a)) near the instrument and on the staff at B onthe other side ofthe valley. Let thesereadingsbeaandb,respectively.Thenearreadingaiswithouterror,whereasthereadingbwouldhave an error e due to curvature, refraction and collimation.

The instrument is then shifted near to Bonthe other side ofthe valleyand the reading is takenonthe staff heldat BandthatonA.Letthesereadingsbecandd(Fig.(b)).Thenearreadingciswithout error,whereas readingdwouldcontainanerroreduetothereasonsdiscussedabove.Lethbethetruedifferenceofelevation between A and B.



Reciprocallevelling

Inthe 1st case(Fig.(a)),h= (b –e)–a

Inthe 2^{nd}case (Fig.(b)),h= c–(d –e)

2h=(b– a) +(c – d)or h=1/2[(b– a) +(c –d)] and e

= 1/2[(b – a) – (c – d)]

Intheabovederivationsitisassumedthattheeffectofrefractionisthesamewhilemakingobservationsfrom boththestations. However, ifonlyonelevelisused,therewillbeatime lagintransferringthe instrumentto theoppositebank, duringwhichtimethevalueofrefractionmaychange. Therefore,toensurebetterresults, somesurveyorsrecommendtheuseoftwo levels, oneat eachbank, so that sightsaretakensimultaneously. Althoughthis willgive betterresults but each level may have a different collimationerror. The instruments should therefore be interchanged and the entire procedurerepeated. The meanofthe four values willbe the most probable difference in the level between the two points.

Example:1 Thefollowingnotesrefer tothereciprocallevelstakenwith onelevel:

Instrument station	*Staff readings on*		*Remarks*
	A	*B*	
A	1.03	1.630	Distance *AB* = 800 m
B	0.95	1.540	R.L. of *A* = 450 m

Find:

(i) True R.L.ofB

(ii) Combinedcorrectionforcurvatureandrefraction

(iii) Theerrorincollimationadjustmentofthe instrument.

Solution

(i) TrueR.L.ofB

Instrumentat A

Incorrect leveldifference betweenAandB=1.630 –1.03=0.600m Instrument at B

IncorrectleveldifferencebetweenAandB=1.540–0.95=0.59 m

Truedifferenceoflevel between Aand B=meanofthetwoincorrect differences

=0.6+0.59/2=0.595m(fallfromAtoB)

Theresultscanalsobeobtainedbyusingthe expression

$h=(b–a)+(c–d)/2$

$= (1.630–1.03)+(1.540–0.95)/2$ $=0.595m$

(ii) Combinedcorrectionforcurvatureand refraction

$=0.0673D2$

$=0.0673(800/1000)^2=0.043$

(iii) Errorin collimationadjustment

Reading of A = 1.03

mFallfromAto B=0.595m

Requiredreadingoflevellime=1.03+0.595=1.625m The

actual staff reading at B (touching horizontal line)

=1.625+0.043=1.668m

Buttheobserved readingatB=1.630 m

Errorincollimationadjustment=1.668– 1.630 =0.038m

Errorofcollimationisnegativesincetheobservedreadingislessthanthe actual.

CURVATUREANDREFRACTION

Curvature andrefraction effects should beaccounted for in preciselevelling work andalso ifthe sightsare too long. The effect of curvature is to cause the objects sighted, to appear lower than they really are. The effect ofrefraction is to make the objectsappear higher than they really are.

CURVATURE

Incaseofalongsightthehorizontallineisnotalevellineduetocurvatureoftheearth.Theverticaldistance betweena horizontal lineand the level linerepresents the effect of curvature of the earth.

InFig.let ABDbealevellinethroughA,andObethecentreofthe earth.Aistheinstrumentposition.AC, theline of collimation, will bea horizontal line. R istheradius ofthe earth.

Thecurvaturecorrection,C_c=BC

Now$OC^2=OA^2+AC^2$ or $(R+C_c)^2=R^2+D^2$

or$R^2+2R \times C_c+C_c^2=R^2+D^2$or $C_c(2R+C_c)=D^2$

or$C_c= D^2/2R+C_c$

Since$C_c$isverysmallascomparedtotheradiusoftheearthR, C_c =

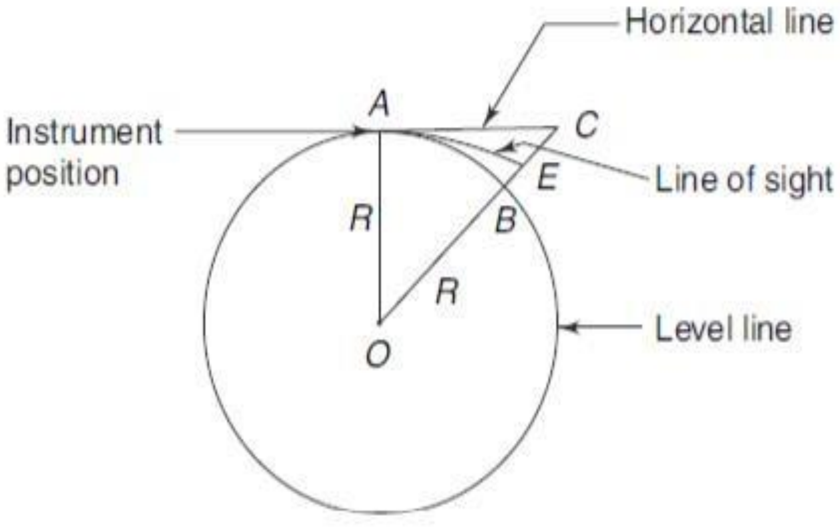
$D^2/2R$

Takingtheradiusoftheearthas6370km,

$C_c=0.0785D^2$

whereD=distanceinkm

Sincethecurvatureincreasesthestaffreading,thecorrectionisthereforesubtractive. True staff reading = observed staff reading – $0.0785\ D^2$



Curvatureandrefraction

REFRACTION

Refraction of the ray passing through the atmosphere from the signal to the observer is the main source of external error. The rays of light while passing through layers of air of different densities refract or bend down. These densities depend upon the temperature and pressure at all points along the track of the rays. Consequently, ray from a staff follows a curved path, let us say AE (Fig.). CE is the amount of refraction correctionandvariesconsiderablywithclimaticconditions.Theaveragerefractioncorrectioncan,however, be taken as 1/7th of the curvature correction.

Refractioncorrection=$0.0785D^2/7=0.0112D^2$ The correction due to refraction is additive.

COMBINEDCORRECTION

Since,theeffectofcurvatureandrefraction, when combined,isto maketheobjectssightedappearlow,the overall correction is subtractive.

Combinedcorrection=$0.0785D^2–0.0112D^2=0.0673D^2$ True staff reading = observed staff reading – $0.0673\ D^2$

ErrorduetocurvatureandrefractioncanbeeliminatedbyequalisingF.S.andB.S.distancesorbyreciprocal levelling. Foralengthofsightofabout400m,combinedcorrectionwillbe1cmandmaybeneglectedwhenrunning indirect levelling.

Example:1 Calculatethecombinedcorrectionforcurvatureandrefractionfora distanceof:(i)5km(ii)500 m.

Solution:(i)5km:$C_c=0.0673\times(5)^2=1.6825$m

(ii)500m:

$C_c=0.0673\times(500/1000)^2=0.016825$m

Example:2InordertofindthedifferenceinelevationbetweentwopointsAandB,alevelwassetuponthe line AB,50 m from Aand1300 m fromB. AandBbeingonthe same side oftheinstrument.Thereadings obtained on staff heldat Aand B were0.435 mand 3.950 m, respectively. Findthetrue difference in elevation between A and B.

Solution: Thecurvatureandrefractioncorrectionsareappliedonlyiftheobservationsaretakenforalength greater than 200 m. Therefore, correctionsare not appliedto the staffreadingat A.

The combined correction for curvatureand refraction at B = $0.0673\ D^{2=}0.0673\ (1.3)^2$ = 0.1137 m, Hence, corrected staff reading at B = 3.950 – 0.1137 = 3.8363 m

TruedifferenceinelevationbetweenBandAis=3.8363–0.435=3.40126m.

TACHEOMETRY

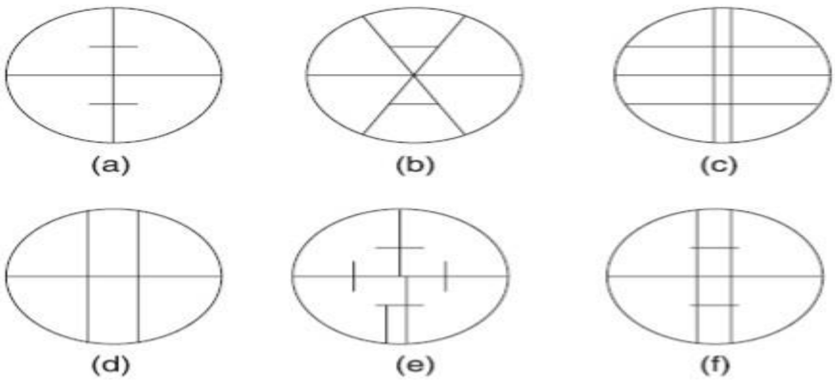
Tacheometry is defined as an optical distance measurement method. Though less accurate, this method of surveying is very rapid and convenient. The other names given to tacheometry are tachymetry or telemetry. It is particularly suitable for filling in details on topographical maps, preliminary location surveys (e.g., for railways, roadways, canals, reservoirs, etc.) and surveying steep grounds, broken boundaries and water stretches, etc. Also, on surveys of higher accuracy, it may be used to provide a ready check on distances measured with a chain or tape.

A tacheometer is essentially a transit theodolite, the diaphragm of which is furnished with stadia wires in addition to the cross-wire. Observations are made on stadia rod, usually a level staff but with a larger least count (1 cm), and horizontal as well as vertical distances are computed from these observed readings.

INSTRUMENTS USED

Tacheometer

It is a transit theodolite fitted with stadia diaphragm. The stadia diaphragm consists of two stadia hairs at equal distances, one above and the other below the horizontal hair of the cross-hair. Various types of stadia diaphragm are shown in Fig., but usually the arrangement shown in Fig. (a) is provided.



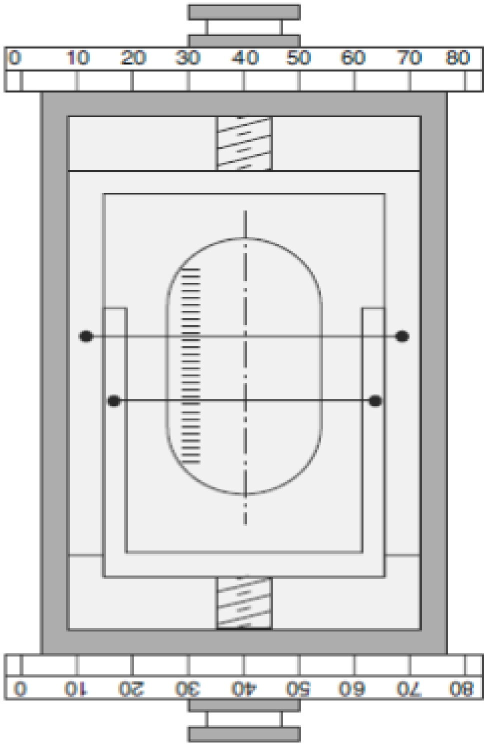
Stadia diaphragm

Essential characteristics:

1. The value of the multiplying constant should be 100.
2. The value of the additive constant should be zero.
3. The telescope should be fitted with an anallactic lens.
4. The magnification of the telescope should be 20 – 80 diameters.
5. Magnifying power of the eyepiece is kept high.

Subtense Theodolite

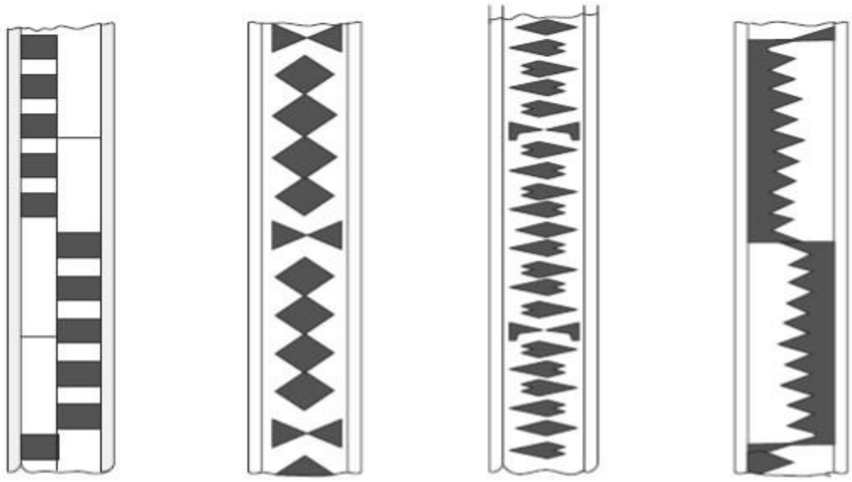
It is similar to a tacheometer but with a special diaphragm, as shown in Fig. The stadia hairs can be raised or lowered by a micrometre screw. The screw is provided with a milled head and a drum scale. The drum is divided into 100 parts and is read against a fixed index to 0.1 of a division by a vernier. Readings are, therefore, made to 0.001 of the pitch of the screw. A comb scale with teeth of the same pitch as that of the screw is provided to exhibit the number of complete pitches. The distance through which either stadia hair is moved from the middle one is measured by the number of turns made by the micrometre screw, the whole turns being read on the comb scale seen in field of view and the fractional part of a turn on the drum scale.



Subtense diaphragm

STADIA ROD

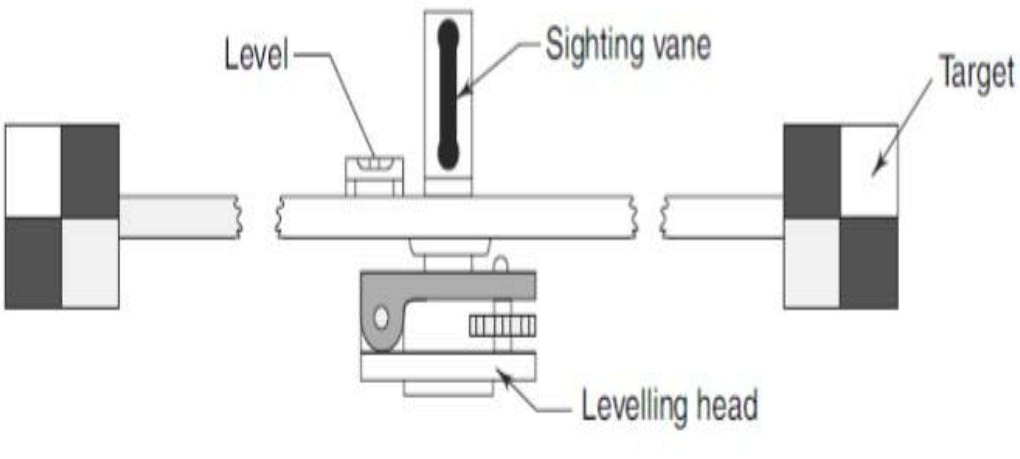
It is also known as vertical stave. It is a rod (Fig.) 5 - 15 m long, graduated in decimals of a metre. For small distances, say up to 100 m, an ordinary levelling staff may be used but beyond this stadia rod is used, since the graduations of an ordinary levelling staff become indistinct. There is a great variety of stadia rod patterns in common use. But, irrespective of the patterns, an observer should be able to read easily and accurately the staff intercepts through the telescope. The staff can be held either vertical or normal to the line of sight.



Stadia rods

SUBTENSE BAR

It is also known as horizontal stave. It is used for measuring both the horizontal as well as the vertical distances in places where chaining is impossible because of undulations and rough country. It is used to determine short distances of up to 200 m.



Subtense bar

It is a horizontal metal bar to which two targets are fixed at a known distance of 0.3 – 3.0 m apart. In India, the subtense bars are usually 3.5 m long. It is mounted on a tripod. A small spirit level is provided to level it. The alidade provides a line of sight perpendicular to the bar, which is thereby set normal to the line of measurement. After aligning and levelling the bar, it is clamped by the screw underneath the tripod top. The targets are usually 20 cm in diameter and are painted half red and half white with a 7.5 cm black centre. Sometimes targets are made square as shown in Fig. The targets are set apart at a known distance and the horizontal angle between them is read by a theodolite. The vertical angle to the bar is also read. Then the horizontal and vertical distances are computed.

TRIGONOMETRIC LEVELLING

Trigonometrical levelling is the process of determining the differences of elevations of stations from observed vertical angles and known distances, which are assumed to be either horizontal or geodetic lengths at mean sea level. The vertical angles may be measured by means of ·an accurate theodolite and the horizontal distances may either be measured (in the case of plane surveying) or computed (in the case of geodetic observations).

We shall discuss the trigonometrical levelling under two beads:

(I) Observations fur heights and distances, and

(2) Geodetical observations

In the first case, the principles of plane surveying will be used. It is assumed that the distances between the points observed are not large so that either the effect of curvature and refraction n1ay be neglected or proper corrections may be applied linearly to the calculated difference in elevation. Under this head fall the various methods of angular levelling for determining the elevations of particular points such as top of chimney, or church spire etc.

In the geodetical observations of trigonometrical levelling, the distance between the points measured is geodetic and is large. The ordinary principles of plane surveying are not applicable. The corrections for curvature and refraction are applied in angular measure directly to the observed angles. The geodetical observations of trigonometrical levelling have been dealt with in the second volume.

HEIGHTS AND DISTANCES

In order to get the difference in elevation between the instrument station and the object under observation, we shall consider the following cases:

Case 1: Base of the object accessible.

Case 2: Base of the object inaccessible: Instrument stations in the same vertical plane as the elevated object.

Case 3: Base of the object inaccessible: Instrument stations not in the same vertical plane as the elevated object.

BASE OF THE OBJECT ACCESSIBLE

Let it be assumed that the horizontal distance between the instrument and the object can be measured accurately. In Fig. 15.1,

P = instrument station

Q = point co be observed

A =centre of the instrument

Q' = projection of Q on horizontal plane through A

D = A Q ' = horizontal distance

between P and Q

h' = height of the instrument at P

h = QQ’

S = reading of staff kept ac B.M., with line of sight horizontal

a = angle of elevation from A to Q.

From mangle AQQ' ; h = D tan α

R. L. of Q = R. L. of instrument axis + D tan α

If the R.L. of P is known,

R.L. of Q= R. L. of P+ h' +D tan α **... (15.1)**

If the reading on the staff kept at the B. M. is S with the line of sight horizontal,

R.L. of Q = R.L. of B.M. + S + D tan α

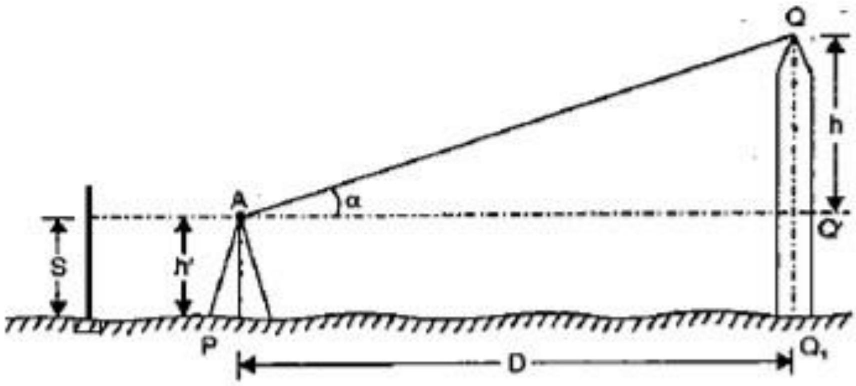


FIG. 15.1. BASE ACCESSIBLE

The method is usually employed when the distance A is small. However, if D large, the combined correction for curvature and refraction can be applied.

BASE OF THE OBJECT INACCESSIBLE:

Instrument Stations in the Same Vertical Plane as the Elevated Object.

If the horizontal distance between the instrument and the object can be measured due to obstacles etc., two instrument stations are used so that they are in the same vertical plane as the elevated object (Fig. 15.5).

Procedure

1. Set up the theodolite at P and level it accurately with respect to the altitude bubble
2. Direct the telescope towards Q and bisect it accurately. Clamp both the plates. Read the vertical angle α_1.
3. Transit the telescope so that the line of sight is reversed. Mark the second instrument

station R on the ground. Measure the distance RP accurately. Repeat steps (2) and (3) for both face observations. The mean values should be adopted.

4. With the vertical vernier set to zero reading, and the altitude bubble in the centre of its run, take the reading on the staff kept at the nearby B.M.

5. Shift the instrument to R and set up the theodolite there. Measure the vertical angle α_1, to Q with both face observations.

6. With the vertical vernier set to zero reading, and the altitude bubble in the centre of its run, take the reading on the staff kept at the nearby B.M.

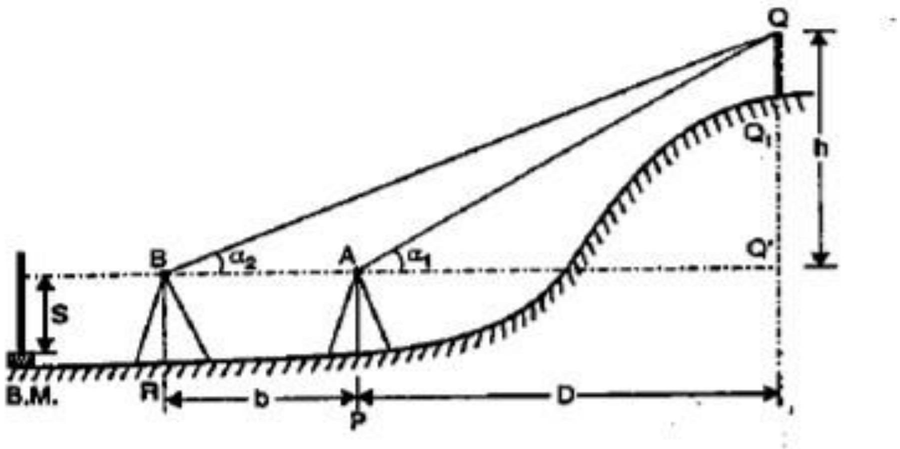


FIG. 15.5. INSTRUMENT AXES AT THE SAME LEVEL

In order to calculate the R.L. of Q. we will consider three cases:

(a) when the instrument axes at A and B are at the same level.

(b) when they are at different levels but the difference is small, and

(c) when they are at very different levels.

(d) instrument axes at the same level (Fig. 15.5)

Let h= QQ'

a, = angle of elevation from A to Q

a, = angle of elevation from B to Q.

S = staff reading on B.M., taken from both A and B, the reading being the same in both the cases.

b = horizontal distance between the instrument stations.

D = horizontal distance between P and Q

From triangle AQQ', $h = D \tan \alpha_1$ ... (1)

From triangle BQQ', $h = (b + D) \tan \alpha_1$ ... (2)

Equating (1) and (2), we get

' $D \tan \alpha_1 = (b + D) \tan \alpha 1$, or $D (\tan \alpha_1 - \tan \alpha_2) = b \tan \alpha_2$, or

$$D = \frac{b \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} \qquad \text{...(15.2)}$$

$$h = D \tan \alpha_1 = \frac{b \tan \alpha_1 \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = \frac{b \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)} \qquad \text{...(15.3)}$$

R.L. of Q=R.L. of B.M. +S+h

b) Instrument axes at different levels (Fig. 15.6 and 15.7)

Figs. 15.6 and 15.7 illustrate the cases, when the instrument axes are at different levels. If S, and S, are the corresponding staff readings on the staff kept at B.M., the difference in levels of the instrument axes will be either (S2 - S1) if the axis at B is higher or (S1, - S2,) if the axis at A is higher.

Let Q' be the projection of Q on horizontal line through A and Q" be the projection on horizontal line through B.

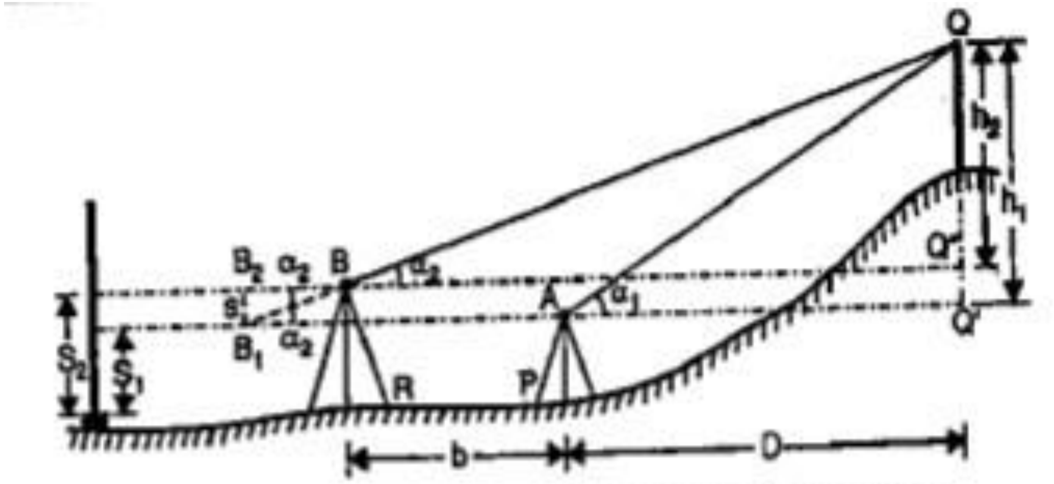


FIG. 15.6. INSTRUMENT AT DIFFERENT LEVELS.

Let us derive the expressions for Fig. 15.6 when S2, is greater than S1,

From triangle QAQ', h, = D tan**α1**, ... (1)

From triangle BQQ", h, = (b + D) tan**α1**, ... (2)

Subtracting (2) from (1), we get

(h1 – h2) = D tan**α1**- (b +D) tan**α2**,

H1, - h2, =difference in level of instrument axes = S2- S1, = s (say)

s = D tan**α1**, - b tanα2 - D tan**α2**

or D (tan**α1**, - tan**α2**,) = s + b tan**α2** or $$D = \frac{s + b \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = \frac{(b + s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} \quad \text{...[15.4 (a)]}$$

Now $$h_1 = D \tan \alpha_1$$

$\therefore$ $$h_1 = \frac{(b + s \cot \alpha_2) \tan \alpha_1 \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = \frac{(b + s \cot \alpha_2) \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)} \quad \text{...[15.5 (a)]}$$

Expression 15.4 (a) could also be obtained by producing the lines of sight BQ backwards to meet the line Q'A in B1 • Drawing B1 B2, as vertical to meet the horizontal line Q" B in B2, it is clear that with the same angle of elevation if the instrument axis were at B1, the instrument axes in both the cases would have been at the same elevation. Hence the distance at which the axies are at the same level is AB_1 = b + BB2 = b + s cot **α2.**

Substituting this value of the distance between the instrument stations in equation 15.2

we get,

$$D = \frac{(b + s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2}$$ which is the same as equation [15.4 (*a*)].

Proceeding on the same line for the case fig. 15.7 where the instrument axis at D is higher, it can be proved that

$$D = \frac{(b - s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} \quad \text{...[15.4 (b)]}$$

and

$$h_1 = \frac{(b - s \cot \alpha_2) \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)} \quad \text{...[15.5 (b)]}$$

Thus, the general expressions For D and h_1 can be written as

$$D = \frac{(b \pm s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} \quad \text{...(15.4)}$$

and $$h_1 = \frac{(b \pm s \cot \alpha_2) \sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 - \alpha_2)} \quad \text{...(15.5)}$$

Use + sign with s cot **α2** when the instrument axis at A is lower and - sign when higher than at B.

R.L. of Q = R.L. of B.M. + S1 + h1,

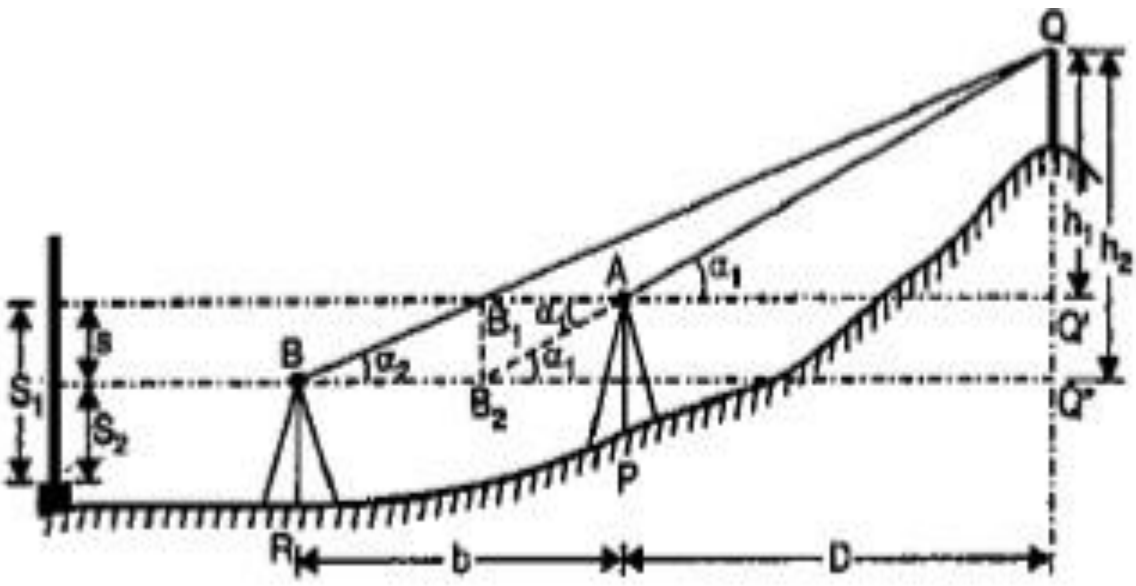


FIG. 15.7. INSTRUMENT AXES AT DIFFERENT LEVELS.

(c) Instrument axes at very different levels

If S_2 -S_1 or s is too great to be measured on a staff kept at the B.M., the following procedure is adopted (Fig. 15.8 and 15.9):

(1) Set the instrument at P (Fig. 15.8), level it accurately with respect to the altitude bubble and measure the angle **α,** to the point Q.

(2) Transit the telescope and establish a point R at a distance b from P.

(3) Shift the instrument to R. Ser the instrument and level it with respect to the altitude bubble, and measure the angle α_2, to Q.

(4) Keep a vane of height r, at P (or a staff) and measure the angle to the top of the vane for to

reading r, if a staff is used (Fig.15.9), vertical projection of Q. Thus, AQQ' is a vertical plane. Similarly, BQQ" is a vertical plane, Q" being the vertical projection of Q on a horizontal line through B. PRQ, is a horizontal plane, Q1, being the vertical projection of Q, and R vertical projection of B on a horizontal plane passing through P. θ_1 and θ_2 are the horizontal angles, and α_1 and α_2 are the vertical angles measured at A and B respectively.

From triangle AQQ', $QQ' = h_1 = D \tan \alpha_1$ ...(1)

From triangle PRQ_1, $\angle PQ_1R = 180° - (\theta_1 + \theta_2) = \pi - (\theta_1 + \theta_2)$

From the sine rule, $$\frac{PQ_1}{\sin \theta_2} = \frac{RQ_1}{\sin \theta_1} = \frac{RP}{\sin [\pi - (\theta_1 + \theta_2)]} = \frac{b}{\sin (\theta_1 + \theta_2)}$$

$$\therefore \quad PQ_1 = D = \frac{b \sin \theta_2}{\sin (\theta_1 + \theta_2)} \quad \text{...(2)}$$

and $$RQ_1 = \frac{b \sin \theta_1}{\sin (\theta_1 + \theta_2)} \quad \text{...(3)}$$

Substituting the value of D in (1), we get

$$h_1 = D \tan \alpha_1 = \frac{b \sin \theta_2 \tan \alpha_1}{\sin (\theta_1 + \theta_2)} \quad \text{...(15.6)}$$

$\therefore$ R.L. of Q = R.L. of B.M. $+ s + h_1$

As a check, $$h_2 = RQ_1 \tan \alpha_2 = \frac{b \sin \theta_1 \tan \alpha_2}{\sin (\theta_1 + \theta_2)}$$

If a reading on B.M. is taken from B, the R.L. of Q can be known by adding h_2 to R.L. of B.

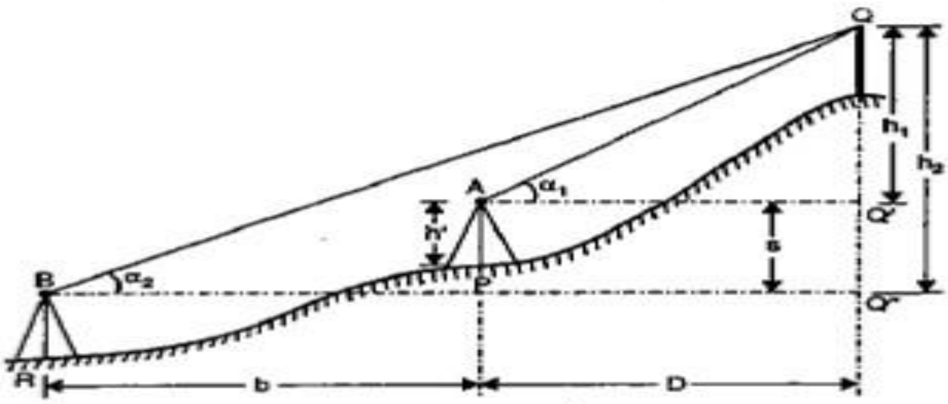


FIG. 15.8. INSTRUMENT AXES AT VERY DIFFERENT LEVELS.

TANGENTIAL METHOD

In this method, stadia hairs are not used to bisect the staff for observations. Two vanes at a constant distance apart are fixed on the staff. Each vane is bisected by the cross-hair and the staff reading and vertical angle corresponding to each vane are recorded.

This method is preferred when the telescope is not equipped with a stadia diaphragm. Since in this method two manipulations of the instrument and two sights are required for one set of observations, there are more possibilities of error as compared to the stadia and subtense methods of tacheometry. Though the results do not differ much, however, the tangential method should definitely be regarded as inferior to the other two methods of tacheometry.

There are three cases for deducing distance and elevation formulae depending upon the nature of the vertical angles.

DISTANCE AND ELEVATION FORMULAE

Both the Angles are Angles of Elevation Refer to Fig. 7.21. Let

D = distance between instrument station O and staff station P

V = vertical distance between the instrument axis and the lower vane

s = distance between the vanes—staff intercept

θ_1 = vertical angle to the upper vane B

θ_2 = vertical angle to the lower vane C

O' = position of instrument axis

r = height of lower vane C, above the foot of the staff at P and

h = height of the instrument.

From triangle $O'KB$, $\quad V + s = D \tan \theta_1$

From triangle $O'KC$, $\quad V = D \tan \theta_2$

From the above equations

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

or

$$D = \frac{s}{\tan \theta_1 - \tan \theta_2}$$

Elevation of station P = elevation of instrument axis + $V - r$

= elevation of station $O + h + V - r$

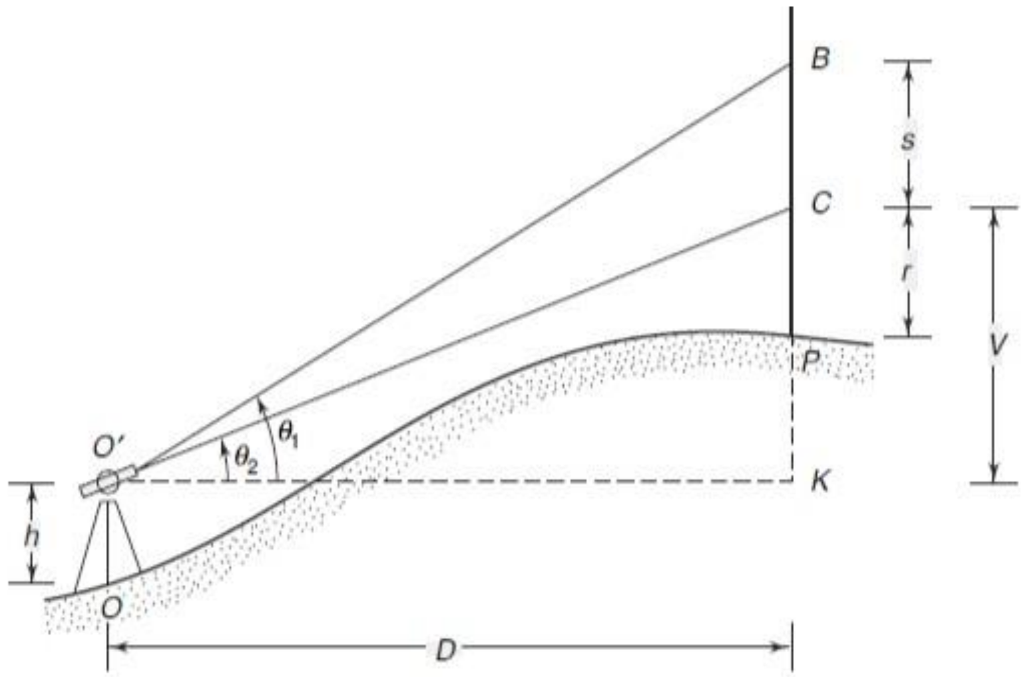


Fig. 7.21 *Tangential method (Elevation angles)*

BOTH THE ANGLES ARE DEPRESSION:

From triangle O' KC (Fig. 7.22),

$V = D \tan \theta_2$

From triangle O' KB,

$V - s = D \tan\theta_1$

From the above equations

$$D \tan \theta_1 + s = D \tan \theta_2$$

or $$D (\tan \theta_2 - \tan \theta_1) = s$$

or $$D = \frac{s}{\tan \theta_2 - \tan \theta_1}$$

But $$V = D \tan \theta_2$$

$$= \frac{s \tan \theta_2}{\tan \theta_2 - \tan \theta_1}$$

Elevation of staff station P = elevation of station $Q + h - V - r$

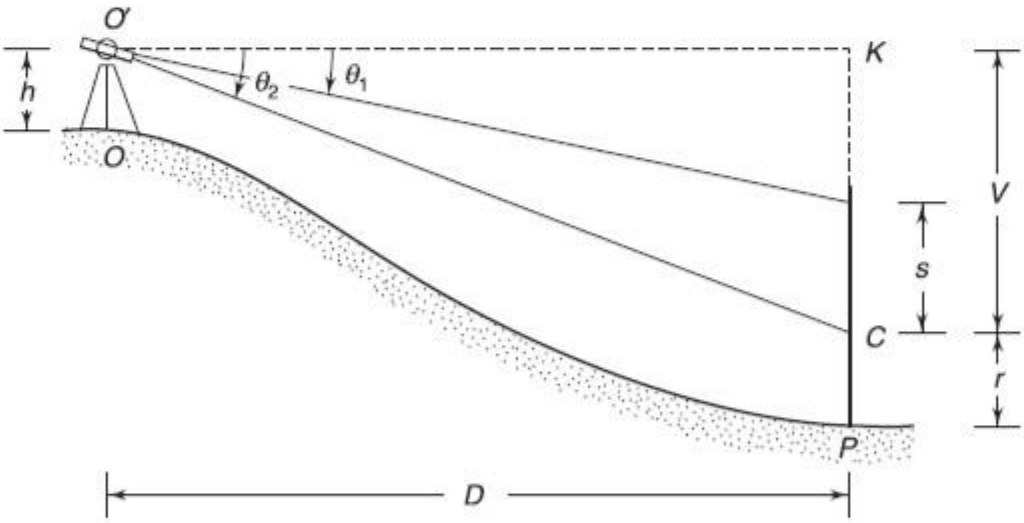


Fig. 7.22 *Tangential method (Depression angles)*

One Angle is Angle of Elevation and the Other Angle is Angle of Depression From triangle $O'KC$ (Fig. 7.23),

$$V = D \tan \theta_2$$

From triangle $O'KB$, $\quad s - V = D \tan \theta_2$

From the above equations

$$s = D \tan \theta_1 + D \tan \theta_2$$

or
$$D = \frac{s}{\tan \theta_2 + \tan \theta_1}$$

But
$$V = D \tan \theta_2$$

$$V = \frac{s \tan \theta_2}{\tan \theta_1 + \tan \theta_2}$$

Elevation of staff station P = elevation of station $O + h - V - r$

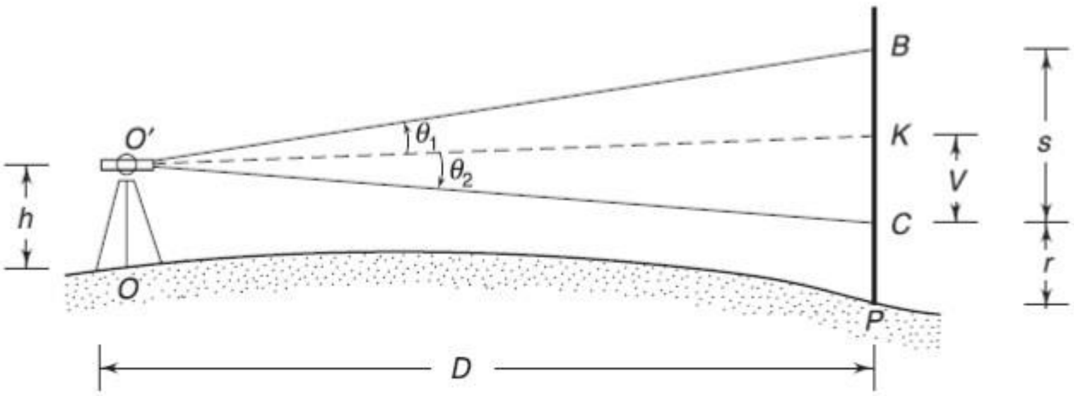


Fig. 7.23 *Tangential method (One angle of elevation and the other angle of depression)*

Example 7.14 In the tangential method of tacheometry, two vanes were fixed 2 m apart, the lower vane being 0.5 m above the foot of the staff held vertical at station A. The vertical angles measured were $+1°12'$ and $-1°30'$. Find the horizontal distance of A from the instrument, if the height of line of collimation is 100 m. Also find the R.L. of A.

Solution

$$D = \frac{s}{\tan \theta_1 + \tan \theta_2} = \frac{2}{\tan 1°12' + \tan 1°30'} = 42.433 \text{ m}$$

$$V = D \tan \theta_2 = 42.433 \tan 1°30' = 1.111 \text{ m}$$

$$\text{R.L. of } A = 100 - V - 0.5 = 100 - 1.111 - 0.5 = 98.388 \text{ m}$$

MEASUREMENT OF HORIZONTAL ANGLE

Horizontal angles are measured on the horizontal circle of a theodolite by operating the upper clamp, the lower clamp, and the upper and lower tangent screws. It should be remembered that with both the clamps set, the upper plate, the lower plate, and the telescope are immobile with respect to the levelling head and tripod. With the upper clamp tight and the lower clamp loose, the two plates cannot move in relation to each other, but the telescope can sweep through 360° in the horizontal plane. With the lower clamp set and the upper one loose, the same 360° sweep is allowed, but this time the upper plate moves relative to the lower plate. This simplifies the measurement of horizontal angle between any given pointing's of the telescope.

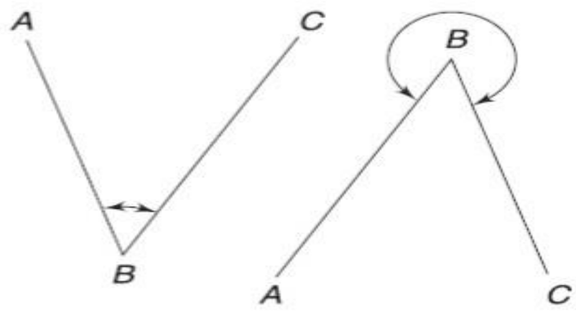
To measure a horizontal angle, say ABC, the following procedure is followed:

1. Set up the instrument over B and level it.

2. Loosen the upper clamp and turn the upper plate until the index (the arrow) of the vernier A, nearly coincides with the horizontal circle. Clamp both the plates with the upper clamp.

3. Turn the upper slow motion (tangent) screw so as to make the two zeros exactly coincident.

4. Loosen the lower clamp and direct the telescope to sight station A (Fig.).



The approximate bisection of the station is done by sighting from over the telescope through a pin-and-hole arrangement provided over its top. Clamp the plates by the lower clamp.

5. Bisect station A exactly by using the lower slow motion (tangent) screw. Exact bisection is done by bringing the station mark exactly at the intersection of horizontal and vertical hairs. The vertical circle clamp and slow-motion screws are used to achieve this.

6. Check the vernier A. It should be 0-0. Note the reading of the vernier B. It should be 180°.

7. Unclamp the upper plate, swing the telescope clockwise and bring the station C in the field of view. Tighten the upper clamp and bisect C accurately using the upper slow motion (tangent) screw.

8. Read both the verniers. The reading on vernier A directly gives the value of angle ABC. From the reading on vernier B, subtract 180° to get the value of angle ABC. Take the mean of the two values to get the value of angle ABC.

9. Change the face of the instrument and repeat the procedure. Thus, a second value of the angle ABC is obtained. The average of the two values is the required horizontal angle. A horizontal angle is measured either by the method of repetition or by reiteration.

Method Of Repetition **Measurement of Horizontal and Vertical Angle**

To measure an angle by repetition, between two stations, means to measure it two or more times allowing the vernier to remain clamped each time at the end of each measurement instead of setting it back to 0° every time when sighting at the previous station. Thus, an angle reading is mechanically multiplied by the number of repetitions. The value of the angle observed is obtained by dividing the accumulated reading by the number of repetitions. Generally, six repetitions are done, three with the telescope normal and three with the telescope inverted.

1. To measure an angle, say ABC, by the method of repetition, set up the instrument at B and level it. The telescope should be in normal position.

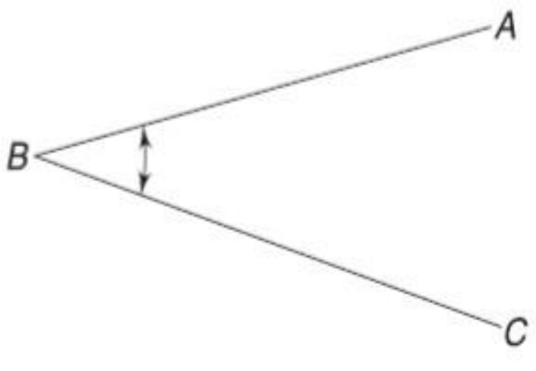
2. Loosen the upper clamp and turn the upper plate until the index (the arrow) of the vernier A coincides with the zero (or 360°) of the horizontal circle. Clamp both the plates with the upper clamp.

3. Turn the upper slow motion (tangent) screw so as to make the two zeros exactly coincident.

4. Sight station A (Fig. 4.10). Tighten the lower clamp and bisect station A exactly by the lower tangent screw. Read both the verniers.

5. Unclamp the upper plate and swing the telescope clockwise. Bisect station C by the upper clamp and tangent screw.

1. Read both the verniers. Take the average to get angle ABC. 7. Unclamp the lower plate and swing the



Horizontal angle by repetition

2. Telescope clockwise and bisect station A accurately by using the lower clamp and lower tangent screw.

3. Read both the verniers. Check the vernier reading. It should be the same (unchanged) as that obtained in step 6.
4. Release the upper plate by using the upper clamp and tangent screw and bisect station C accurately (the telescope is turned clockwise). The vernier will read twice the angle ABC.
5. Repeat the process for required number of times, say three times, and find out the value of angle ABC.
11. Repeat the above procedure with the face changed and calculate the angle ABC.
12. The average of the two values of angle ABC thus obtained with face left and face right gives a precise value of the horizontal angle.

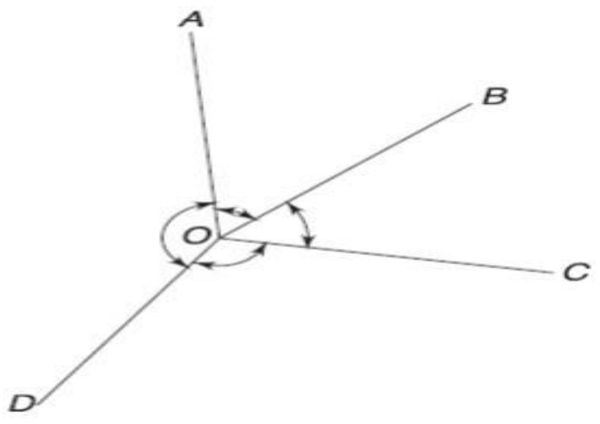
Advantage:

1. The errors of graduations are minimised by reading the angle on different parts of the graduated circle.
2. Personal errors of bisection are eliminated.
3. The errors due to eccentricity of the centres and that of the verniers are eliminated, by reading both the verniers.
4. Error due to the line of collimation not being perpendicular to the transverse axis of the telescope is eliminated as both the face left and face right readings are taken.

Method Of Reiteration

This method of measuring a horizontal angle is preferred when several angular measurements are to be made at a station. All the angles are measured successively and finally the horizon is closed. The final reading on vernier A should be same as the initial zero. If not, the discrepancy is equally distributed among all the angles.

1. To measure angles AOB, BOC, COD and DOA (Fig.), set up the instrument at O and level it.
2. Set the vernier A to read zero using the upper clamp and tangent screw.
3. Direct the telescope towards A and bisect it exactly using the lower clamp and lower tangent screw. Read the two verniers A and B.
4. Unclamp the upper plate, swing the telescope clockwise and bisect B accurately, using the upper clamp and upper tangent screw. Read both the verniers.
5. Similarly, bisect stations C, D and finally A, and read both the verniers in all the cases. The last reading on vernier A should be 360°. If not, the discrepancy is noted and distributed.
6. Transit the telescope, swing the instrument in anticlockwise direction with face right and repeat the whole procedure.



Horizontal angle by reiteration

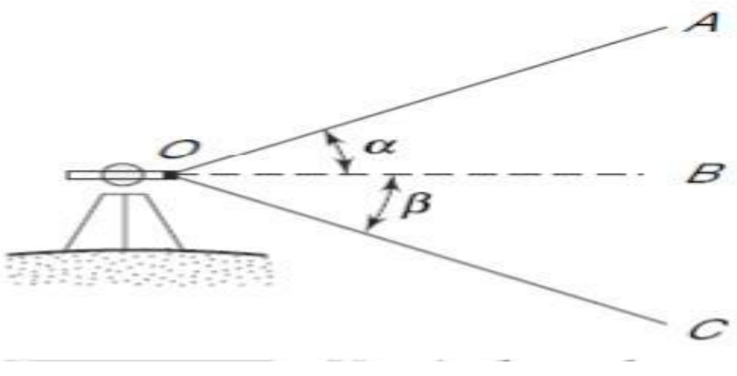
Comparison of Method of Repetition and Reiteration

The method of repetition is preferred for the measurement of a single angle and when accuracy is desired beyond the least count of the instrument with a coarsely graduated circle. On the other hand, the method of reiteration is preferred in triangulation, where a number of angles may be required at one point by the instrument with a finely graduated circle. By exercising appropriate precautions, instrumental errors can be eliminated theoretically, for either of the methods. Though the method of repetition appears to be better, it is more time consuming and chances of personal errors are more and even many repetitions may yield ordinary results.

MEASUREMENT OF VERTICAL ANGLE

A vertical angle may be defined as the angle subtended by the line of sight and a horizontal line at a station in the vertical plane.

If the point to be sighted is above the horizontal plane, the angle is called the angle of elevation (+) and if the point is below it, the angle is called the angle of depression (–).



Vertical angle Measurement

1. Suppose AOB (α), the vertical angle, is to be measured (Fig.). Set up the instrument at O and level it.

2. Using the upper clamp and upper tangent screw, set the zero of the vertical vernier to the zero of the vertical circle. Check the bubble of the altitude level which should be central. It not, bring it to the centre with the help of the clip screw. This will ensure that the instrument is in adjustment.

3. Loosen the vertical circle clamp and rotate the telescope in a vertical plane and bring station A in the field of view. Bisect it accurately with the vertical clamping and tangent screws. Read both the verniers C and D on the vertical circle.

4. Change the face and repeat the procedure.

5. The average of the two observations gives the value of the required angle.

Errors

The sources of error in angular measurement may arise from imperfections in the adjustments and construction of the theodolite. The errors arising from imperfect adjustment of a theodolite are as follows:

Vertical Axis error (α): Axis not vertical in an observation, either from imperfect plate level adjustment, or settlement of the instrument.

Lateral collimation error (β): Line of collimation not perpendicular to the horizontal axis.

Horizontal axis error (γ): Horizontal axis not perpendicular to the vertical axis.

Vertical collimation error (δ): Line of altitude bubble not parallel to the line of collimation when the verniers of vertical circle read zero.

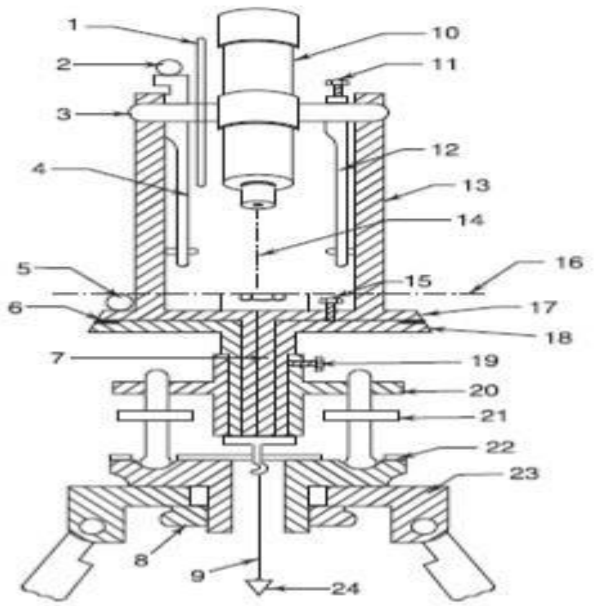
The above errors are often co-existent, wholly or in part in any given case. The defects in construction are usually those of eccentricity and graduations. The former can be eliminated by taking the mean of the two vernier readings, whereas the latter can be minimised by taking observations over different portions by the horizontal scale.

Transit or theodolite is an instrument used to measure horizontal and vertical angles. Depending upon the facilities provided for reading of observations the theodolites may be classified as simple vernier theodolite, micrometre theodolite, optical (glass arc) theodolite and electronic theodolite. Though the first two are obsolete and all the modern theodolites are of the optical or electronic digital type, this chapter mainly discusses the vernier theodolite.

CLASSIFICATION: Theodolites may be classified into transit and non-transit theodolites.

Transit Theodolite

A theodolite is said to be a transit one when its telescope can be revolved through 180° in a vertical plane about its horizontal axis, thus directing the telescope in exactly opposite direction. The various parts of the transit theodolite are shown in Fig. The vertical circle is rigidly fixed to the telescope and rotates with the telescope (Fig.).



1.	Vertical circle	2.	Altitude bubble
3.	Horizontal axis	4.	Vernier arm
5.	Plate bubble	6.	Graduated arc
7.	Levelling head	8.	Clamping nut
9.	Vertical axis	10.	Telescope
11.	Vertical circle clamping screw	12.	Arm of the vertical circle clamp
13.	Standard	14.	Line of sight
15.	Upper plate clamping screw	16.	Axis of plate bubble
17.	Upper plate	18.	Lower plate
19.	Lower plate clamping screw	20.	Tribrach
21.	Foot screw	22.	Trivet
23.	Tripod top	24.	Plumb bob

Fig. 4.1 *Transit theodolite*

NON-TRANSIT THEODOLITE

A theodolite is said to be a non-transit one when its telescope cannot be revolved through 180° in a vertical plane about its horizontal axis. Such theodolites are obsolete nowadays. Examples are Y-theodolite and Everest theodolite.

READING A THEODOLITE

A theodolite has two verniers A and B placed on the opposite sides of the upper plate (i.e., they are placed at a difference of 180°). The main scale and vernier of a typical theodolite as graduated are shown in Fig. The main scale is graduated from 0° to 360° in degrees and minutes. Each degree part is tested and divided into three equal parts. Hence, the minimum reading that can be read from the main scale is 20'. The vernier scale is graduated into minutes and seconds. Each minute division is divided into three equal parts. Hence, the least reading that can be read from the vernier scale is 20".

DEFINITIONS

The following are the definitions of the terms that will frequently be used in the measurement processes:

1. *Transit:* It is also called plunging or reversing. This is the operation of revolving the telescope through 180° in a vertical plane about its horizontal axis, thus making it point exactly in the opposite direction.
2. *Face right:* When the vertical circle of a theodolite is on the right of the observer, the position is called face right and the observation made is called face right observation.
3. *Face Left:* When the vertical circle of a theodolite is on the left of the observer, the position is called face left and the observation made is called face left observation. By taking the mean of both face readings, the collimation error is eliminated.
4. *Swinging telescope:* Revolving the telescope in the horizontal plane, about its vertical axis is called swinging. A right swing means clockwise rotation of the telescope, whereas a left swing means anticlockwise rotation of the telescope. By taking the mean of the right swing and the left swing observations, the effects of error due to friction or backlash in the moving parts is eliminated.
5. *Telescope normal:* The telescope is said to be normal or direct when its vertical circle is to the left of the observer and the bubble is up.
6. *Telescope inverted:* The telescope is said to be inverted when its vertical circle is to the right of the observer and the bubble is down.
7. *Horizontal axis:* It is also called the trunnion axis or transverse axis
8. *Vertical axis:* It is the axis about which the telescope can be rotated in a vertical plane.

   It is the axis about which the telescope can be rotated in a horizontal plane.
9. *Axis of telescope:* It is the line joining the optical centre of the object glass to the centre of the eyepiece.
10. *Line of sight:* It is an imaginary line joining the intersection of cross-hairs to the optical centre of the objective and its continuation.

GALE’STRAVERSE TABLE

Traversecomputationsareusuallydoneinatabularform.OnesuchformisGale’straversetable(Table5.1) and is widely used because of its simplicity. The following steps are involved in theodolite traversing and these are illustrated in Table 5.1.

1. Inthecaseoftheodolitetraversing, the included anglesareadjustedto satisfythegeometricalconditions, i.e., the sum of the included angles should be (2n ± 4) 90°, where n is the number of sides of the closed traverse.Theplussignisusedwhentheanglesareexteriorangles,andtheminussignwhentheyareinterior angles. In the case of compass traversing, the observed bearings are adjusted for local attraction.

2. Fromtheobservedbearingofa line, e.g., lineAB inTable5.1,thewholecircle bearingsofallotherlines are calculated and then these bearings are reduced to those in the quadrantal system.

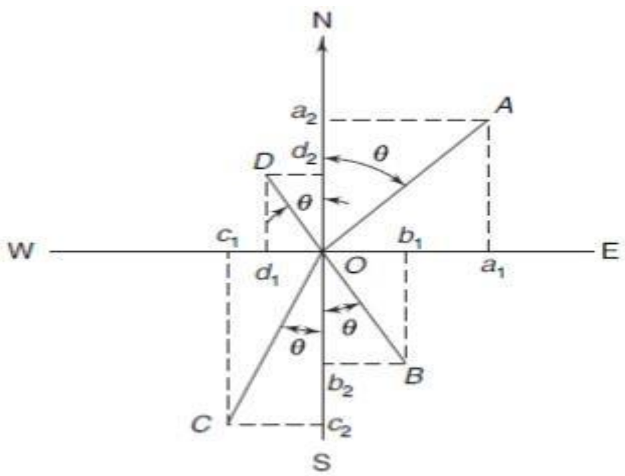
3. Fromthe lengths and computed reduced bearings of the lines, the consecutive coordinates, i.e., latitudes and departures are worked out.

4. Acheck isdonetofindoutwhetherthealgebraicsumoflatitudesandthealgebraicsumofdeparturesare zero. Ifnot, acorrection is applied using the transit rule. Inthe case ofa compass traverse, the correction is applied by Bowditch rule.

5. The independent coordinates are then worked out from the consecutive coordinates. The origin is so selectedthattheentiretraverseliesinthenorth-eastquadrant.Thisisdonetofacilitateplottingofthetraverse on a sheet with the left-hand bottom corner of the sheet as the origin.

LATITUDESANDDEPARTURES

The latitudesanddeparturesofthetraverse linescanbecalculatedifthereducedbearingsand lengthsofthe lines are known. Northing Oa2 and easting Oa1 are taken as positive, whereas southing Ob2 and westing Od1 are taken as negative, as shown in the Fig.



LatitudeofOA	=	northing	=	lcosθ(+)
DepartureofOA	=	easting	=	lsinθ(+)
LatitudeofOB	=	southing	=	lcosθ(–)

DepartureofOB	=	easting	=	$l\sin\theta(+)$
LatitudeofOC	=	southing	=	$l\cos\theta(-)$
DepartureofOC	=	westing	=	$l\sin\theta(-)$
LatitudeofOD	=	northing	=	$l\cos\theta(+)$
DepartureofOD	=	westing	=	$l\sin\theta(-)$

Here,l,θarethelengthandreducedbearingoftherespectiveline.

ERRORPROPAGATION

In surveying measurements, there are all possibilities for errors to creep in, irrespective of the precautions and precision exercised. This necessitates a proper control, assessment and distribution of errors. Error, mistake and discrepancy are few terms frequently encountered in surveying measurements. True error may be defined as the difference in measured and true value of a quantity. when a number of measurements are tofulfilaknowncondition,trueerrorcanbeobtained.Forexample,thetheoreticalsumofexteriorincluded anglesinatraverseofNsidesis(2N+4)×90°.Here,theerrorinthesumofobservedanglesisknown,but the error in individual angles is not known.

The observed difference between two like measurements each of which may have an error, of a quantity, is termedas *discrepancy*. A discrepancyshould not beregarded as an error. Asmall discrepancybetween two measurements does not reflect that error is small. Whereas, a large discrepancy indicates a mistake in the observations.

*Mistakes*aretheerrorsarisingfromcarelessness,inexperience,poorjudgmentandconfusionoftheobserver.

NATUREOFERROR: Errorsinameasurementmaybepositiveifthemeasurementistoolarge,ornegative ifthesameistoosmallascomparedtoitstruevalue.Errorsareclassifiedassystematicerrorsandaccidental errors.

SYSTEMATIC ERROR: The error due to sag of a tape supported at its ends can be calculated and subtracted from each measurement. However, the tape can be supported throughout its length at short intervals and the sag error may be reduced to a negligible quantity. It always has the same magnitude and signsolongastheconditionsremainthesame,andsuchanerroriscalled*constantsystematicerror*.Whereas, if the conditions change, the magnitude of the error changes and is known as *variable systematic error.*A systematic error follows a definite mathematical or physical law and, therefore, a correction can always be determined and applied. These errors are also known as *cumulative errors.*

ACCIDENTAL ERROR:

Thesearetheerrorsduetoacombinationofcausesandarebeyondthecontrolofthesurveyor.Thesecanbe positiveornegative.Erroneouscalibrationofachainisanexampleofan*accidentalerror*.Thereisinreality nofixedboundarybetweentheaccidentalandsystematicerrors.Everyaccidentalerrorhas somecause,and if the causes were perfectly understood and the amount and sign could be determined, it would cause an accidental error, but would be classed as systematic. On the other hand, a constant or systematic error may bebroughtintotheaccidentalclasswhollyorpartiallybyvaryingtheconditions,instruments,etc.,such that the sign of the errors is frequently reserved.

LAWS OFACCIDENTAL ERRORS

Accidental errorsfollow thelawofprobabilityanditisbecauseof theseaccidentalerrors arealsoknownas *probableerrorsorstandarderrors*.Thislawdefinestheoccurrenceoferrorsandwhenexpressedintheform of an equation, it can be used to compute the probable value or the probable precision of a quantity. The probable error is the number of errors numerically greater than it is the same as that those less than it. It is alwayswrittenaftertheobservedquantitywiththeplusandminussign,e.g.,25°42’30”plusorminus3.16”. Probable error serves two important purposes:

(i) asameasureof theprecision ofanyseries of observations, and

(ii) asameansofassigningweightstotwoormorequantities,andthustofindtheweightedmeanor the most probable value of each.

PRINCIPLEOFLEASTSQUARES

The principle of distributing errors bythe method of least squares is of great help to find the most probable value of a quantity which has been measured for several times, perhaps by different methods and different observers and in calculating the trust worthiness of such a value. In the method of least squares, the discrepanciesorerrorsofthediscrepantobservationsareassumedtobeofaccidentalnatureonly.According to this principle, the most probable value of a quantity is the one for which the sum of the squares of the errors is a minimum.

Themain objects of themethod ofleast squares are

(i) todetermine thebest valueswhich canpossiblybeobtained fromagivenset of measurements,

(ii) todeterminethe degreeof dependencewhichcanbeplaced uponthesevalues,

(iii) toenableustotracetotheirsourcesthevariouserrorsaffectingthemeasurements,and consequently,

(iv) toincreasetheaccuracyoftheresultbyapropermodificationofthemethodsandtheinstruments used.

TRIANGULATIONADJUSTMENT

The most accurate method is that of least squares but is very complicated since all the angles are simultaneously involved. However, using an approximate method, the adjustment can be achieved by adjusting angles, stations and figures separately. After adjusting the triangulation figure, the sine rule is applied for computing sides. Then the positions of the points are determined by calculating the geodetic coordinates.

ANGLEADJUSTMENT

Manyobservations are made for a single angle; for example, face leftand face right, vernier A and vernier B, and reading an angle on different partsof the scale. The correction to be applied is directlyproportional totheweight andalsotothesquareoftheprobable error.Theanglescanbe measuredwithequal orunequal weights. Inthe former case, the most probable value is the arithmetic meanofthe observations, whereas in the latter case, it is the weighted arithmetic mean of the observed angles.

STATIONADJUSTMENT

The stationadjustment consistsofdetermining the most probable valuesofthe angles measured at astation so as to satisfy the geometric consistency. The various conditions can be

(i) closingthehorizon,

(ii) measuringtheangleswithequalorunequalweights,and

(iii) measuring different angles at a station individuallyor in combination. Inthe first case, the error ifanyisdistributedequallytoallthethreeangles. Inthesecondcaseit isdistributedinverselyas the respective weights. Whereas in the last case, normal equations are formed and are solved simultaneously.

(iv) FIGUREADJUSTMENT

(v) In any system of triangulation, determination of the most probable values of the angles so as to fulfil the geometrical conditions are called figure adjustment. There can be a number of geometrical conditions which the angles should fulfil, but since all the measured angles are affected byerrors, theynever will meet allthe conditions perfectly. Therefore, it is necessaryto adjusttheanglesso astoobtainthe best possibleand most probable value. The best solutioncan be obtained by the method of least squares, also known as the rigid method, which is a little complex and therefore, the adjustments are usually done by an approximate method. The geometrical figures encountered in triangulation are a triangle, a quadrilateralor a polygon with a central station.

(vi) ADJUSTMENTOFATRIANGLE

(vii) A triangleis the basicfigure of any triangulation system.All the three angles of a triangle areadjusted. Some of the rules for applying corrections to the observed angles are as follows. Let

(viii) A,B,C=anglesofthe triangle

(ix) n=numberofobservationsforanangle

(x) w=weightofthe angle

(xi) d=discrepancy(errorofclosure)

(xii) c=correctiontoobservedangle

TRIANGULATIONFIGURESORSYSTEMS

Triangulationfiguresmaybedefinedasasystemconsistingoftriangulationstationsconnectedbyachainof triangles. The complete figure iscalled triangulationfigure ortriangulationsystem. The most commontype offiguresusedinatriangulationsystemaretriangles,quadrilateralsandpolygons.Allofthesefiguresshould fulfil the rigid **geometric conditions** given as follows:

1. Thesumofthe interior anglesshould be(2n – 4) ×90°, wheren is the number ofsidesofthe figure. The averagenumberofsecondsbywhichthesumofanglesdeviatesfrom180°,plustherequiredsphericalexcess is known as triangular misclosure.
2. Ifalltheanglesaremeasuredatastation,their sumshouldbe360°.
3. Thelengthofsidescalculatedthroughmore thanone routesshouldagree.

Itisimpossibletofulfilallthegeometricalconditions,owingtotheerrors,untilthefieldmeasurementshave been adjusted.

CLASSIFICATION

The classification ofa triangulation system is based uponthe degree ofaccuracyrequired, the extent ofthe area to be surveyed, length of the base, length of the sides, and triangular misclosure.

PrimaryorFirst-orderTriangulation

Afirstordertriangulationisthehighest-ordertriangulationandisemployedforverylargeareas,forexample, for the earth’s figure, for obtaining the most precise control in map ping, and for small-scale mapping.

It consists of forming large, well-conditioned triangles. Precise instruments are used for observations and every possible refinement is exercised. The following are the general specification of the primary triangulation.

1.	Average triangle closure	:	Less than 1 second
2.	Maximum triangle closure	:	Not more than 3 seconds
3.	Length of base line	:	5 to 15 kilometres
4.	Length of the sides of triangles	:	30 to 150 kilometres
5.	Actual error of base	:	1 in 300,000
6.	Probable error of base	:	1 in 1,000,000
7.	Discrepancy between two measures of a section	:	10 mm $\sqrt{\text{kilometres}}$
8.	Probable error of computed distance	:	1 in 60,000 to 1 in 250,000
9.	Probable error in astronomic azimuth	:	0.5 seconds

Secondaryorsecond-orderTriangulation

A second-order triangulation is employed for running a second series of triangles by fixing points at close intervals inside the primary series oftriangles. It consistsof forming small, well-conditioned triangles with less precise instruments. the general specification of the second-order triangulation are:

1.	Average triangle closure	: 3 sec
2.	Maximum triangle closure	: 8 sec
3.	Length of base line	: 1.5 to 5 km
4.	Length of sides of triangles	: 8 to 65 km
5.	Actual error of base	: 1 in 150,000
6.	Probable error of base	: 1 in 500,000
7.	Discrepancy between two measures of a section	: 20 mm $\sqrt{\text{kilometres}}$
8.	Probable error of computed distance	: 1 in 20,000 to 1 in 50,000
9.	Probable error in astronomic azimuth	: 2.0 sec.

TertiaryorThird-orderTriangulation

A third-ordertriangulation is employed for running in a third series oftriangles, by fixing points inside the secondary triangles at short intervals to furnish horizontal control for details on a topographic survey. The trianglesareofthesmallest size incomparisonwiththeothertwoorders oftriangulation. Thespecifications for a third order triangulation are

1.	Average triangle closure	: 6 sec
2.	Maximum triangle closure	: 12 sec
3.	Length of base line	: 0.5 to 3 km
4.	Length of sides of triangles	: 1.5 to 10 km
5.	Actual error of base	: 1 in 75,0000
6.	Probable error of base	: 1 in 250,000
7.	Discrepancy between two measures of a section	: 25 mm $\sqrt{\text{kilometres}}$
8.	Probable error of computed distance	: 1 in 5,000 to 1 in 20,000
9.	Probable error in astronomic azimuth	: 5 sec.

HORIZONTALCONTROL

Every survey, from mapping a continent to a small plot of land, depends upon a carefully measured frameworkwhichisthereaftertreatedasfreefromerror.Subsequently,thedetailsarefilledintheframework by less elaborate methods. The fixation of a framework for a survey is known as *horizontal control*.

Horizontalcontrolusuallyconsistsofacombinationoftriangulationandtraverse.Formostofthesurveysof small extent, e.g., plane surveys, where direct linear measurements are impossible, triangulation is most suited. It is also suitable when long sights are taken.

Intriangulation,anumberoflinesofsightarerequiredateachstation.Whenthesightsarelong,thestations areelevatedbybuildingtowers.Incasethedistancesareshort,theexpenseoftowersoffsetsanysavingand traverse becomeseconomical. Triangulation is most suited for hillyareas, whereastraversing issuitable for flat areas.

Triangulation is preferred for hills and undulating areas, since it is easy to establish stations at reasonable distances apart, with intervisibility. In plane and crowded areas, it is not suitable as the intervisibility of stations is affected.

The main disadvantage of triangulation is the accumulation of error in the lengths and directions of lines, sincebothofthem,forsuccessivelines,dependuponthecomputationsforthoseoftheprecedingline,which necessitates a number of check bases.

In triangulation, the entire area to be surveyed is covered with a framework of triangles. If the length and direction ofone side, and all three angles ofa triangle are measured precisely, the lengths and directions of the remaining two sides of the triangle can be computed. The length of the first line, which is measured precisely is known as ***base line***. The other two computed sides are used as new base lines for two other trianglesinterconnectedwiththefirsttriangle.Byextendingthisprocess,i.e.,themeasurementofthefurther interconnectedtrianglesandusingthecomputedsides,achainornetworkoftrianglescanbespreadoverthe entirearea. Theapexofthetrianglesso located witharelativelygreater accuracyprovide horizontalcontrol ofthe survey. Thus, triangulation may be defined asa systemofmultiplying ground controlsonthe earth’s surface. Asacheck,thelengthofoneofthesidesofthelasttriangleisalsomeasuredandcomparedwiththe computed one this side is known as check base.

TOTALSTATION

A total station, also known as electronic tacheometer, is an optical instrument. It is a combination of an electronictheodolite formeasuringhorizontalandverticalangles,anelectromagneticdistancemeasurement (EDM) device for measurement ofslope distances and on-board software to convert the raw observed data tothreedimensionalcoordinates.Atotalstationmaydeterminetheactualpositions(X,Y,andZornorthing, easting and elevation) of surveyed points, or the position of the instrument from known points, in absolute terms.

The various featuresofa TCAversiontotalstation are shown in Fig. 1. The totalstationsystemoffers more functionalityandgreaterflexibilityforawidervarietyofsurveyapplications.Thelargedisplayispositioned under the telescope to give the user access to much more information at a glance. The keyboard, with its functionkeys, iseasilyunderstoodandpermitsconvenient input.Removabledatastorage,the large battery capacityandon-boardapplicationprogramsensurethateveryavailablefacilityiscontainedinoneunit.Some ofthetotalstationsystemsalsooffertheexternalconnectionofexternaldataloggers,computersorbatteries.

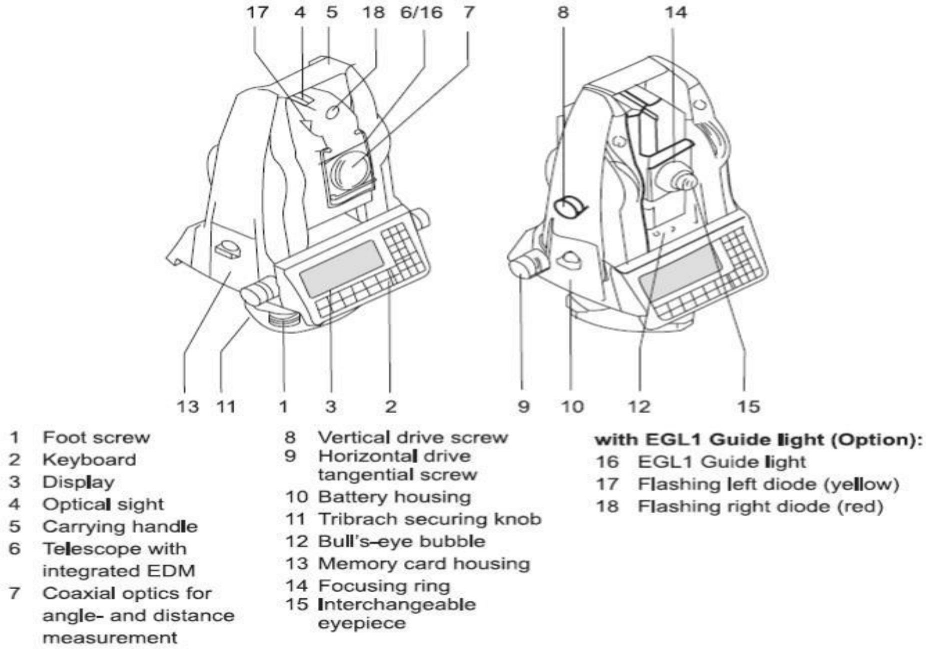


Fig:1Featuresoftotalstation

Distance Measurement

Theelectromagneticdistance measuringdevice, measuresthedistance fromthe instrumenttoitstarget.The EDMsendsoutaninfraredbeamwhichisreflectedbacktotheunit,andthe unit usestime measurementsto calculate the distance travelled by the beam. Generally, a total station measures a slope distance, and the microprocessor uses the vertical angle recorded by the theodolite along the line of sight to calculate the horizontal distance. In addition, the height distance between the trunnion axis and the prism centre is also calculated and displayed.

All the total stations use coaxial optics in which the EDM transmitter and receiver are combined with the theodolite telescope. **Three modes are usually available for distance measurement:**

1. ***StandardMode:***It hasaresolutionof1mmand ameasurement timeof1 to 2 s.
2. ***Preciseortimemode:***Ithasaresolutionof1butameasurementtimeof3to4s.Thisismoreaccurate than the standard mode, since the instrument refines the arithmetic mean value by making repeated measurements.
3. ***TrackingorFastMode:***Thedistance measurementisrepeatedautomaticallyatintervalsofless than 1. Normally, this mode has a resolution of10 mm. The range ofa totalstation is typically1 to 3 km toasingleprismassuminggoodvisibility.Theprecisionofatypicaltotalstationis5mm.Thedistance readings are automatically corrected for atmospheric effects such as pressure and temperature.

Ingeneral, threedistance measurement functionalitiesareavailablewiththe totalstationsystem, the firstbeingthedistancemeasurementwithareflector(IR-Mode),secondlythedistancemeasurement without a reflector (RL-Mode) and the third being the distance measurement - long range.

ACCESSORIES

1. **Track light or Lumi Guide:** It is a visible light which enables a pole-mounted prismto be set directly onthe lineofsight. Thedevice flashesthreecolour lights. Iftheprismistothe left ofthe lineofsight, a green light flash. Ifthe prism is tothe right, ared light is seen. And if the prism is on line, a white light flashes, the frequency of which doubles when it strikes the prism, confirming that the prism is in the correct position.
2. **Getronics Unicom:** It is a communication system which allows speech to be transmitted from the instrument to the prism. This consists of a small microphone onthe controlpanelwhich is activated by pressing a key and a receiver with small loudspeaker mounted on the prism pole.
3. **Retroreflector:** Aspecialformofreflector knownascorner cubeprism, whichispole mounted, isused asatarget.Theseareconstructedfromglasscubesorblocks,andtheyreturnabeamalong apathexactly parallel to the incident path over a range of angles of incidence of about 20° to the normal of the front face ofthe prism. As a result, the alignment is not criticaland is quicklyset when making observations. Associatedwithallreflectingprismsisaprismconstant.Thisisthedistancebetweentheeffectivecentre of the prism and the plumbing and pivot point of the prism. The effective centre of a prism is normally well behind the physical centre or vertex. A typical prism constant value is - 30 or - 40 mm.

SETTING-UPANDORIENTINGATOTAL STATION

Theprocessofsettinguptotalstationconsistsofcentring itoverthestation, levellingandorientation. Since thetwoprocesses, centringand levelling, influenceeachother,theprocess isoftrialanderror.Thestep-by- step procedure for setting up is as follows.

Centring

The tripod is placed over the station and its three legs are spread. It is ensured that tripod is at suitable height sothatthesurveyorcanworkconvenientlywhen thetotalstationistightenedover itstop. Oneofits legs is placed firmly in the ground and the other two legs are moved radially in or out so asto bring approximately centre of the total station over the station mark. With the laser beam emitted by the total station ensure the centring has been achieved. The laser plummet is located in the standing axis ofthe instrument (Fig. 5). Ifnot, slide the instrument over the tripod by loosening it, bythe use oftightening screw provided with the tripod plate to achieve exact centring.

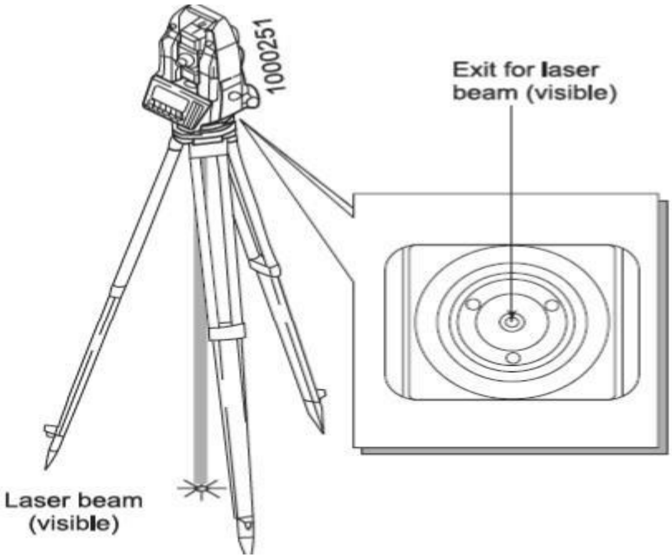


Fig:5Centringwithlaserplummet

LEVELLING-UP

ApproximateLevelling-Up

The total station has to be set approximately over the ground reference point using the optical or laser plummet primarily and then it has to be ensured that the tripod legs are firmly placed into the ground. The totalstationplate has then to be levelled using the tripod legs as shown inFig. 6. Now, anyone ofthe three legs is moved circumferentially, keeping the other two legs fixed, to bring the bull's eye bubble (Fig. 6) centralwhichisprovidedoverthetribrach.Inthisprocess,thecentringwillgetdisturbed.Anumberoftrials may be required to achieve levelling.

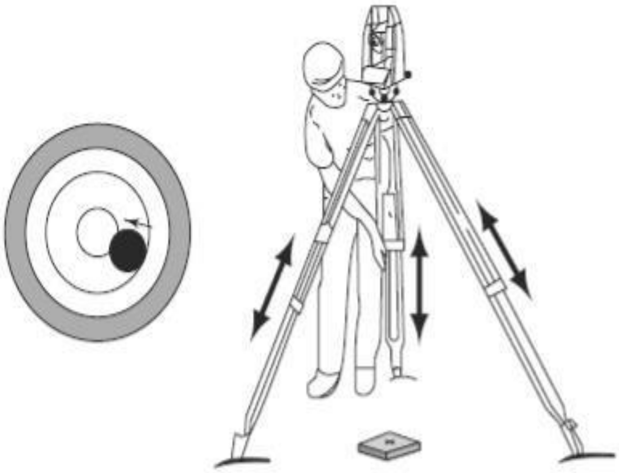


Fig:6 ApproximateLevellingup pfatotalstation

Precisionlevelling-up

Oncethetotalstationisapproximatelylevelledandplacedover thestationpoint onthetripod,thesystemis fine-tuned through the levelling using the electronic bubble with the help of the foot-screws. The steps are portrayed in Fig.7.

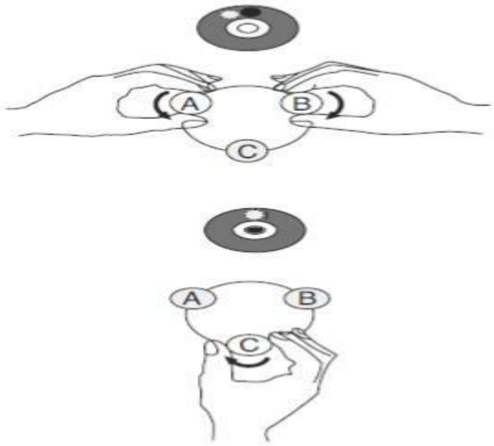


Fig:7Precision levelling-up

Orientation

After the instrument has been switched on, the instrument modeland software modelare brieflydisplayed. The horizontal circle can be oriented and application programs started. For this first, the coordinates of the stationpointareset,andthepointnumberofstationpointisentered.Thecoordinatesofthestationisentered directlyoris importedfromdatafileofthememorycard. Thestationdataconsistsofpoint number, easting, northing, stationheight and instrument height.Orientation impliesfixing line ofsight ina particular known direction w.r.t. which measurement of angles or bearings are done.

Fortraversingbymeasurementofanglesthereferencepoint isbacksighted, andtheHzdirectionissetto0° 00 00, or a known value is entered. The total station is ready for making the measurements of angles. The totalstationcanalso beoriented innorthdirectionwithanattachment consistingofa magnetic needle. This facility is available in some of the makes of totalstation.

GPSSURVEYING TECHNIQUES

ThepositionofstationaryormovingobjectcanbedeterminedthroughGPS.Whenthepositionofastationary or moving object is determined with respect to a well-defined coordinate (x, y, z) by using a single GPS receiver and by making observations to four or more satellites, it is called point positioning or absolute positioning (Fig. 9.10). However, if the coordinates of an unknown point are determined with respect to a known point it is called ***relative positioning.***

In other words, relative positioning aims at determination of the vector between the two points, by observations to four or more satellites bytwo receivers placed at the two points simultaneously(Fig. 9.11). Incasetheobjecttobepositionedisstationary,itistermedasstaticpositioning,whileiftheobjectismoving, it iscalledkinematicpositioning.GPSsurveyingimpliestheprecisemeasurementofthevectorbetweentwo GPS instruments. ***The GPS surveys may be classed as static surveys the traditional static surveying; dynamic surveys the rapid-static surveying, pseudo-static surveying, and kinematic surveying.***

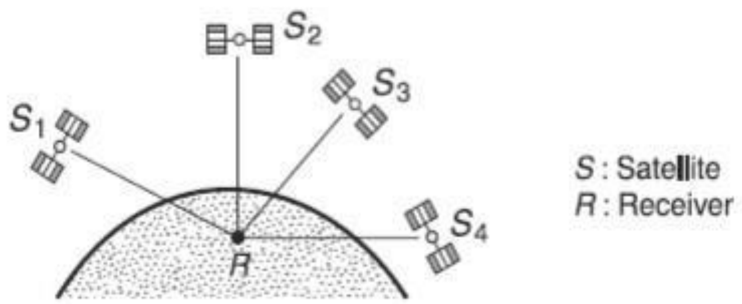


Fig. 9.10 *Point positioning*

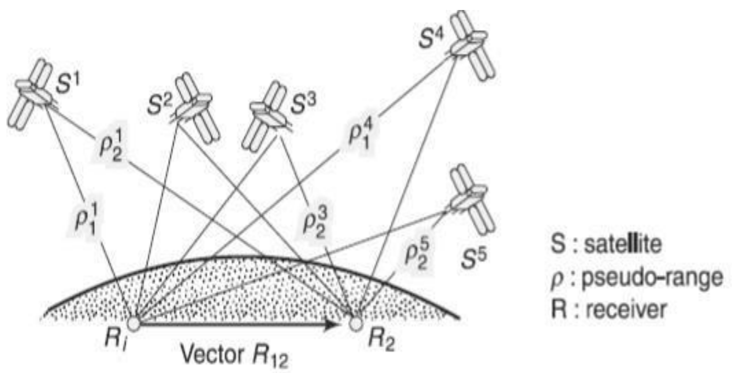


Fig. 9.11 *Relative positioning*

USESANDAPPLICATIONSOFGPS

GPSisacomplexsystemthatcanbeusedinmanyways.Forbasicpointpositioningusinggeodeticreceivers with a computer and post-processing software accuracy at the centimetre level is achievable. Although the accuracyisimportant,somesurveyorsfeelthatthemainadvantageofGPSisthatitcanbeusedinany weather conditionsdayornight.ThisenablesGPS surveying to be carried outover extended periodsat any timeoftheyearwithoutrestrictions suchasrain, fogandpoorvisibility. Anotheradvantagewhensurveying with GPS is that intervisibility between stations or points surveyed is not necessary.

Thisallowscontrolstationsto beplaced conveniently. Further, withdifferential GPS accuracyof100 mfor navigationpurposes, 1 mfor mapping and a few millimetres for geodetic positioning is possible. However, the high cost of GPS surveying has restricted the realisation of the full potential of GPS till date. Added to this, there are difficulties in defining heights above datums such as mean sea level and with real time data processingandcontrol.Despitethesedrawbacks,GPShasbeenverysuccessfullyusedinsurveyingandother fields.

GPS is a toolthat willprovide the word a new “internationalstandard” for defining locations and distances anditwouldallownationstomonitorandusenaturalresourcesmoreefficientlythaneverbefore.Thegeneral overallusesofGPSarenumerous.GPSitself,orincombinationwithGeographicInformationSystem(GIS), orotherspatiallyrelateddatabaseshasemergedasanew,dynamic,spatiallyrelatedinformationutility.This utility can process both spatial data and relational data, as well as it is capable of the real-time processing requiredfornavigationandrouting.TheinitialconceptionofGPSwasmilitarypositioning, weaponsaiming and navigationsystem. It wasto replace TRANSIT and other navigationsystemsand to provide worldwide weather-independent guidance for military use. But because of its potential GPS will soon be part of the overall utilityof technology. Some of the uses and applications global, regionaland local are as follows.

GPSACCURACY

AccuracyachievablewithGPSdependsonseveralconditions,e.g.,singleor multi-receiveroperation;single ordualfrequencydata;receiver noise level Selective Availabilityonoroff(S/Ahas beenput offsince May 2nd2000);P-codeavailableornot;staticorkinematicpositioning;real-timeorpost-processedresultsextent ofdatamodelling;accuracyoforbits;andfiducialconcept(fiducialpointsmeanspreviouslywell-determined coordinates). The ultimate accuracy of GPS is determined by the sum of several sources of error already described in the previous section. The contribution ofeach source may varydepending on atmospheric and equipment conditions. In addition, the accuracy of GPS can purposefully be degraded by Selective Availability.

FIELDPROCEDURESFORATOTALSTATIONINTOPOGRAPHIC SURVEYS

Totalstationscanbeused inanytypeofpreliminarysurvey,controlsurvey,orlayout survey.Theyaremost suitable for topographic surveys in which the surveyor can find the X, Y, Z (easting, northing, elevation) positions ofa large number ofpoints (about 2to 3 times ofthose using conventional techniques) per day.

InitialDataEntry

Theinitialdata entrycouldbe allorsome ofthefollowing:

1. Projectdescription
2. Dataandcrew
3. Temperature
4. Pressure
5. Prismconstant
6. Curvatureandrefractionsetting
7. Sea-levelcorrection
8. Numberofmeasurement repetitions
9. ChoiceofFace1 andFace2 positions
10. Automaticpointnumberincrementation
11. Choiceofunits

SURVEYSTATIONDESCRIPTORS

Each survey station or point must be described with respect to surveying activity, station identification and other attribute data. Generally, the total stations prompt the data entry and then automatically assign appropriatelabels. Point descriptiondatacanbeenteredasalpha(forexample, backsight asBS)ornumeric (for example, backsight as 20) codes.

SURVEYSTATIONENTRIES

1. CodeSay20 (BS),30(IS),40 (FS)
2. Heightofinstrument
3. Stationnumber(say)110
4. Stationidentificationcode
5. Coordinatesofoccupiedstation
6. Coordinatesofbacksightstation

SIGHTEDPOINTENTRIES

1. Operationcode

2. Heightofprism

3. Stationnumber:120(BS)

4. Stationidentificationcode

Procedure

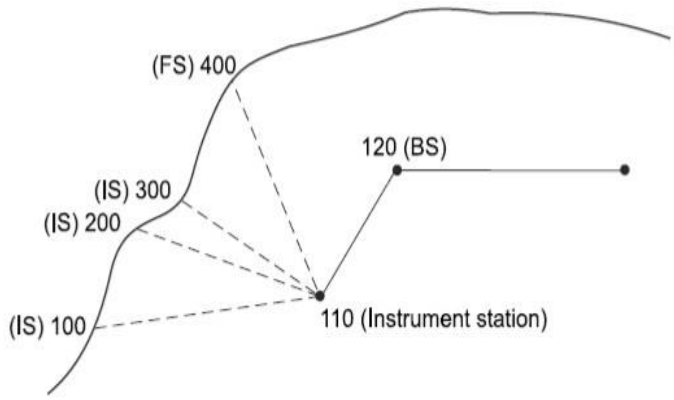


Fig.8.

The total station is mounted on tripod, centred and levelled. The initial data and occupied station data are entered.

(i) Whentheinstrumentissetupandturnedonitsetsitselftobepointingtozerodegrees(north) whenpower is first supplied. The totalstation is then reset to zero degree when it is actuallypointing north.

(ii) Ifbatterydiesduringmeasurements,theinstrumentmustberesettozerodegrees.

2. Sightatdesiredstation, say120;pressthezero-setbuttontosetthehorizontalcircleatzero.

3. Enter code 20 (BS). The prism is mounted on a pole of known height. The reflection point of the prism gets aligned with the centreofthe pole. Since the instrument aims at the prism, it calculates the position of theprismandnotthatofgroundpoint.Thegroundpointislocatedbysubtractingtheheightofthepole.This necessitates that the pole is held upright while making observations. Measure and enter the height of the prism.

4. Presstheappropriate measure buttons,e.g.,slopedistance,etc.

5. Press the record button after each measurement. In the automatic mode, all the three X, Y, and Z measurements are made after pressing just one button.

6. Afterthestationmeasurementshavebeenrecorded, thedatarecorderonboardwillprompt forthestation point number (e.g., 120), and the station identification code.

7. Fornextsights, repeatsteps4-7usingappropriate data.

8. When all the topographic details in the area of the occupied station (110) have been recorded, the total station is moved to the next traverse station and the process is repeated.

9. Download the data to a computer, where it is stored into a format that is compatible with the computerprogram that is to process the data.
10. Ifthe topographic data are for a closed traverse, the traverse closure is calculated, and then alladjusted values of X, Y, Z are computed.
11. Fromthedatastoredincoordinatefiles,thedatarequiredforplottingbydigitalplottersisassembled,and the survey can be quickly plotted at any desired scale.

GPSERROR SOURCES

Iftheworldwaslikealaboratorywithperfectlaboratoryconditions,basicGPSwouldbealotmoreaccurate. Unfortunately,itisnotso,andhasplentyofopportunitiesforaradio-basedsystemthatspanstheentireplanet togetfouled up. Inaccuracies inGPSsignalscome fromthevarietyofsources(Fig9.7), likesatelliteclocks, imperfectorbitsandespeciallyfromthesignal'stripthroughtheearth'satmosphere.Sincetheseinaccuracies are variable, it is hard to predict what they will be in order to correct for them. Although, these errors are small, but to get the kind of accuracies some criticalpositioning jobs require, all the errors, no matter how minor, areto be minimised. What is needed is a wayto measure the actualerrors astheyhappen. The error sources can be classified into three groups, namely satellite-related errors, propagation-medium related errors, and receiver-related errors. These are known as systematic errors. However, sometimes errors are introduced intentionally known as selective availability

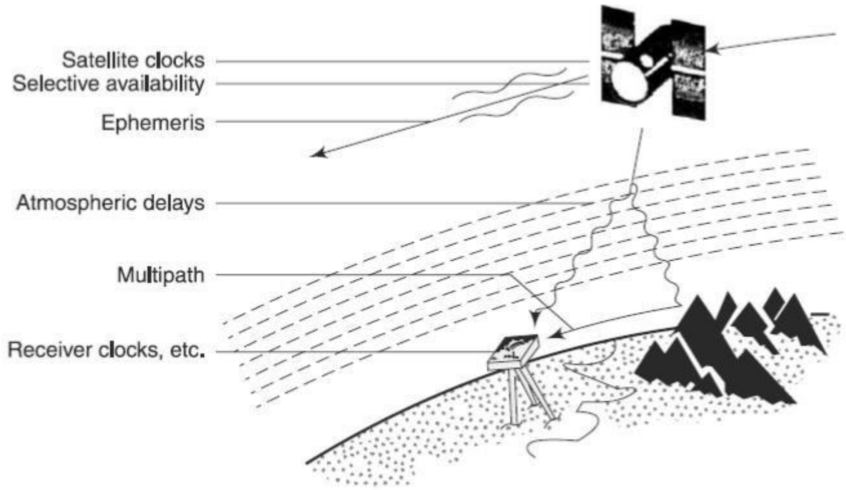


Fig. 9.7 *Error sources*

SystematicErrors

SatelliteError:

GPS satellites are equipped with veryaccurate atomic clocks. But as good as these clocks are, theyare not perfect.Slightinaccuraciesintheirtimekeepingultimatelyleadtoinaccuraciesinourpositionmeasurements. The satellite's position in space is also important equally because it is the starting point for all of the GPS calculations. GPS satellites are placed into very high orbits and so are relatively free from the perturbing effects of the earth's upper atmosphere,

butevenso theystilldriftslightlyfromtheirpredictedorbitscontributing totheerrors.

1. Signal propagation errors: GPS satellites transmit their timing information byradio, and that is another sourceoferror because radio signals in the earth's atmosphere (ionosphere and troposphere) do notbehave aspredictablydesired.Itisassumedthatradiosignalstravelatthespeedoflight,whichispresumablya constant. However, the speed of light is not constant. It is only constant in vacuum. In the realworld, light (or radio) slows downdepending onwhat it is travelling through. As a GPS signalcomes downthroughthe charged particles in the ionosphere and then through the water vapor in the troposphere, it gets delayed a little. Sincecalculationofdistanceassumesaconstant speedoflight,thisdelay results into amiscalculation ofthesatellite'sdistance,whichinturntranslatesintoanerrorinposition.Goodreceiversaddinacorrection factor for a typical trip through the earth's atmosphere, which helps, but since the atmosphere varies from pointtopointandmomenttomoment,nocorrectionfactororatmosphericmodelcanaccuratelycompensate for the delays that actually occur.

2. Receiver errors: The receivers are also not perfect. They can introduce their own errors which usually stem from their clocks or internal noise, multipath and antenna face centre variation.

*(a) Multipath:*AstheGPS signalarrivesatthesurfaceoftheearthit mayget reflectedbylocalobstructions andgetstothereceiver'santennaviamorethanonepath.Thisformoferroriscalledmultipatherrorbecause, in a sense, the signal is getting to the antenna by multiple paths. First, the antenna receives the direct signal it being the fastest, and then the reflected signals arrive a little later (Fig. 9.8). These delayed signals can interfere with the direct signal giving noisy results. Secondary effects are reflections at the satellite during signal transmission.

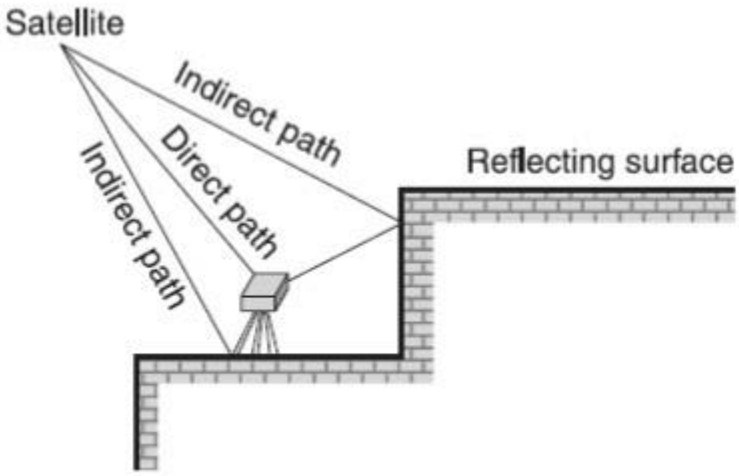


Fig. 9.8 *Multipath effect*

The influence ofthe multipath, however, canbe estimated byusing a combination ofL1 and L2 codes, and carrier-phase measurement.The principle is based onthe fact that troposphere, clock errors, and relativistic effects influencecodeand carrierphasesbythesameamount.This is nottrueforionosphericrefractionand multipath which are frequency dependent. Taking ionospheric-free code ranges and carrier phases, and forming corresponding differences, all aforementioned effects, except for multipath, cancel out. The most effective counter measure to multipath is to avoid sites where it could be a problem.

Theeliminationofmultipathsignalsisalsopossiblebyselectinganantennathattakesadvantageofthesignal polarisation.GPSsignalsareright-handedcircularlypolarised,whereasthereflectedsignalsare left-handed polarised. Areductionofmultipatheffect mayalso be achieved bydigitalfiltering, wideband antennas, and radio frequency absorbent antenna ground planes. The absorbent antenna ground plane reduces the interferenceofsatellitesignalswith lowor even negative elevationangleswhichoccur incaseofmultipath.

(b) Antennaphase:centreoffsetandvariation:Thephasecentreoftheantennaisthepointtowhichtheradio signal measurement is referred and generally is not identical with the geometric antenna centre. The offset depends onthe elevation, the azimuth, and the intensityofthe satellite signaland is different forL1 and L2 codes. Also, the true antenna phase centre may be different from the manufacturer indicated centre. This antennaoffsetmaysimplyarisefrominaccurateproductionseries.Further,theantennaphasecentrecanvary with respect to the incoming satellite signals. The variation is systematic and may be investigated by test series.

Systematic effectscanbe eliminated byappropriate combinationsofthe observables. Differencing between receiverseliminatessatellite-specific biases,and differencing betweensatelliteseliminatesreceiver-specific biases. Thus, double differenced pseudoranges are, to a high degree, free of systematic errors originating fromthesatellitesandfromthereceivers.Withrespecttorefraction,thisisonlytrueforshortbaselinewhere the measured ranges at both end points are affected equally. In addition, ionoshperic refraction can be virtually eliminated by an adequate combination of dual frequency data.

Multipathiscausedbymultiplereflectionofthesignal.Theinterference betweenthedirectandthereflected signalislargelynotrandom;however,itmayalsoappearasanoise.Asimilareffectiscalled***imaging,***where a reflecting obstacle generates an image ofthe real antenna whichdistortsthe antenna pattern. Botheffects, multipathand imaging, canbeconsiderablyreducedbyselecting sitesprotectedfromreflections(buildings, vehicles, trees, etc.) and by appropriate antenna design. It should be noted that multipath is frequency dependent. Therefore, carrier phases are less affected than code ranges where multipath can amount to the meter level. The random noise mainly contains the actual observation noise plus random constituents of multipath (especially for kinematic applications).

Themeasurementnoise,anestimationofthesatellitebiases,andthecontributionsfromthewavepropagation are combined in the User Equivalent Range Error (UERE). This UERE is transmitted via the navigation message.IncombinationwithDOPfactor,UEREallowsforanestimationoftheachievablepointpositioning precision.

GPS SURVEYING

Thissystemconsistsofthree segments:

thespacesegment,

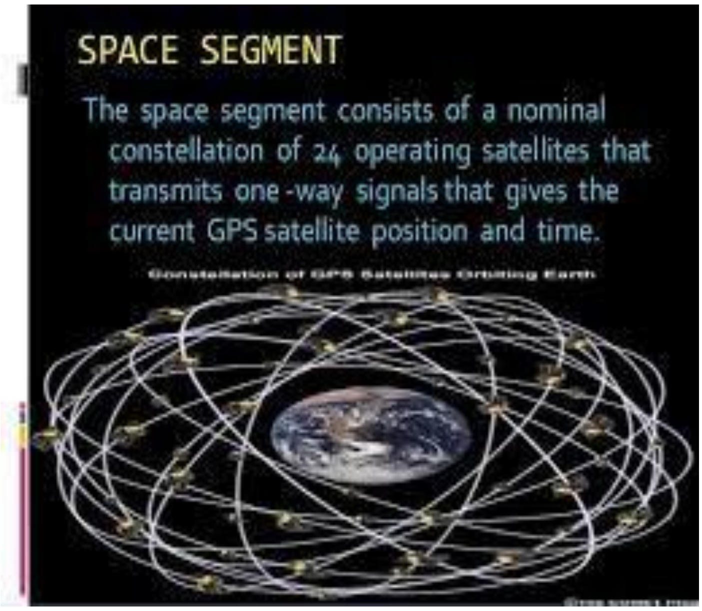
thecontrolsegment,and the

user segments.

Theairforcedevelops,maintains,andoperatesthespaceandcontrolsegments.The GPScontrol segment consists of a global network of ground facilities that track the GPS satellites, monitor their transmissions, perform analyses, and send commands and data to the constellation.

Space Segment:

TheSpaceSegmentcontains24satellites,in12-hournear-circularorbitsataltitudeofabout20000km,with inclination oforbit 55°. The constellation ensures at least 4 satellites in view fromany point onthe earth at any time for 3-D positioning and navigation on world-wide basis. The three-axis controlled, earth-pointing satellites continuously transmit navigation and system data comprising predicted satellite ephemeris, clock error etc., on dual frequency L1 and L2 bands.



ControlSegment

This has a Master ControlStation(MCS), few Monitor Stations (MSs) and anUp Load Station(ULS). The MSsaretransportablescheltersswithreceiversandcomputers;alllocatedinU.S.A.,whichpassivelytrack satellites, accumulating ranging data fromnavigation signals. This is transferred to MCS for processing by computer,toprovide best estimatesofsatelliteposition,velocityandclockdrift relativetosystemtime.The data thus processed generates refined information of gravity field influencing the satellite motion, solar pressure parameters, position, clock bias and electronic delay characteristics of ground stations and other observable system influences. Future navigation messages are generated from this and loaded into satellite memory once a day via ULS which has a parabolic antenna, a transmitter and a computer. Thus, role of ControlSegment is: - To estimate satellite [space vehicle (SV)] ephemeridesand atomic clock behaviour. - To predict SV positions and clock drifts. - To upload this data to SVs.

The control segment consists of one Master Control Station (MCS) and five monitoring stations. The five monitorstationsareusedtotrackthe flight pathsofthesatellite,whichlocatedat Hawaii, Kwajalein,Diego Garcia,AscensionIslandandColoradoSprings,Colorado.ThistrackinginformationissenttotheAirForce SpaceCommand'smastercontrolstationatSchrieverAirForceBaseinColoradoSprings,whichisoperated bythe2ndSpaceOperationsSquadron(2SOPS)oftheUnitedStatesAir Force(USAF).Generally,theMCS will compare the tracking data from the monitor stations and calculates the satellite orbit and clock parameters.

UserSegment

The user equipment consists ofan antenna, a receiver, a data-processor with software and a control/display unit. The GPS receiver measures the pseudo range, phase and other data using navigation signals from minimum4satellitesandcomputesthe3-Dposition, velocityandsystemtime. Thepositionis ingeocentric coordinates in the basic reference coordinate system: World Geodetic reference System 1984 (WGS 84), whichare converted and displayed as geographic, UTM, grid, or anyother type ofcoordinates. Corrections likedelayduetoionosphericandtroposphericrefraction,clockerrors,etc.are alsocomputedandappliedby the user equipment / processing software.

Theuser’sGPS receiver istheuser segment (US)oftheGPS.Ingeneral, GPS receiversarecomposedofan antenna, tunedtothe frequenciestransmittedbythesatellites, receiver-processors,andahighly-stableclock (crystaloscillator). Inaddition, theymight include adisplayforproviding locationandspeed informationto theuser.Areceiverisoftendescribedbyitsnumberofchannels,signifyinghow manysatellitesitcanmonitor simultaneously.Originallyitislimitedtofourorfivechannels,butithastypicallyincreasedoveryears2007. Now, the receivers can have between 12 and 20 channels.