# MOHAMED SATHAK A.J.COLLEGE OF ENGINEERING, SIRUSERI 603103

# CE8351 - SURVEYING

## II CIVIL

**R.B. RAKESH**

**DEPARTMENT OF CIVIL ENGINEERING**

**2020 -2021**

**CE8351 SURVEYING L T P C 3 0 0 3**

**UNIT I FUNDAMENTALS OF CONVENTIONAL SURVEYING AND LEVELLING 9**

Classifications and basic principles of surveying - Equipment and accessories for ranging and chaining - Methods of ranging - Compass - Types of Compass - Basic Principles- Bearing – Types - True Bearing - Magnetic Bearing - Levelling- Principles and theory of Levelling – Datum-
- Bench Marks – Temporary and Permanent Adjustments- Methods of Levelling- Booking – Reduction - Sources of errors in Levelling - Curvature and refraction.

**UNIT II THEODOLITE AND TACHEOMETRIC SURVEYING 9**

Horizontal and vertical angle measurements - Temporary and permanent adjustments - Heights and distances - Tacheometer - Stadia Constants - Analytic Lens -Tangential and Stadia Tacheometry surveying - Contour – Contouring – Characteristics of contours – Methods of contouring – Tacheometric contouring - Contour gradient – Uses of contour plan and map

**UNIT III CONTROL SURVEYING AND ADJUSTMENT 9**

Horizontal and vertical control – Methods – specifications – triangulation- baseline – satellite stations – reduction to centre- trigonometrical levelling – single and reciprocal observations – traversing – Gale’s table. - Errors Sources- precautions and corrections – classification of errors true and most probable values - weighed observations – method of equal shifts – principle of least squares - normal equation – correlates- level nets- adjustment of simple triangulation networks.

**UNIT IV ADVANCED TOPICS IN SURVEYING 9**

Hydrographic Surveying – Tides – MSL – Sounding methods – Three point problem – Strength of fix – astronomical Surveying – Field observations and determination of Azimuth by altitude and hour angle methods –.Astronomical terms and definitions - Motion of sun and stars - Celestial coordinate systems - different time systems - Nautical Almanac - Apparent altitude and corrections - Field observations and determination of time, longitude, latitude and azimuth by altitude and hour angle method

**UNIT V MODERN SURVEYING 9**

Total Station : Advantages - Fundamental quantities measured - Parts and accessories - working principle - On board calculations - Field procedure - Errors and Good practices in using Total Station GPS Surveying : Different segments - space, control and user segments - satellite configuration - signal structure - Orbit determination and representation - Anti Spoofing and Selective Availability - Task of control segment - Hand Held and Geodetic receivers - data processing - Traversing and triangulation.

**TOTAL : 45 PERIODS**

## UNIT-1

## FUNDAMENTALS OF CONVENTIONAL SURVEYING AND LEVELLING

### DEFINITION OF SURVEYING

- Surveying is the Art of determining the relative position on above or beneath the Surface of the earth by means of direct or indirect measurements of distance,direction andelevation.
- It also includes the art of establishing points by predetermined angular & LinearMeasurements.

### CLASSIFICATION OF SURVEYING

### i)PLANE SURVEYING

- Plane Surveying is defined as the divison of Surveying in which all the survey works are carried based on the assumption that,the surface of earth is a plane and curvature of the earth isIgnored.
- In Dealing with the plane Surveying,plane geometry and Trignometry are onlyrequired.
- The Surveys having an area of about 260km2 may only be treated as plane surveys.

**USES:**

- Plane Surveys which generally include the area upto 260km2 are carried out for engineering projects,on large scales to determine relative positions of individual features on the earthssurface.
- Plane Surveys are used to prepare the layout for highways,canals,railways,construction of various featuresetc.

### ii) GEODETIC SURVEYING:

- The Surveys in which curvaqture of the earth is taken into account and higher degree of accuracy required is called geodeticsurveying.

**USES:**

- Geodetic Surveys carried out with higher degree of accuracy to provide the spaced control points on the earthsurface.
- It Requires advanced instruments.In India Surveys carried out by the department of survey of india under the control and direction of surveyor general ofIndia.

**SURVEYING**

**PlaneSurveying** **Geodetic Surveying**

**CLASSIFICATION BASED ON**

| PURPOSEOFSURVEYING | NATUREOFFIELD | METHODS ADOPTED | INSTRUMENTUSED |
|---|---|---|---|
| i.EngineeringSurveys | 1.EngineeringSurveys | 1.Triangulation | 1.Chain Surveying |
| ii.MilitaryorDefence | a.Topographical | 2.Traversing | 2.Compass Surveying |
| iii.MineSurveys | b.CadastralSurvey | 3.Levelling | 3.Plane Table Surveying |
| iv.Geologicalor | c.CitySurvey | 4.Tacheometry | 4.Levelling Surveying |
| GeographicalSurveys | 2.MarineSurvey& | | 5.TheodoliteSurveying |
| v. ArchaeologicalSurveys | 3.AstronomicalSurvey | | 6.Total Station |
| vi.Route and LocationSurveys | | | |

**CLASSIFICATION BASED ON PURPOSE OF SURVEYING**

**i) ENGINEERINGSURVEYS**

- The Determination of quantities or to afford sufficient data for the designing of engineering works such as roads and reservoirs,or those connected with sewage disposal or watersupply

**II) MILITARY OR DEFENCESURVEY**

- This is used for determining points of strategicimportance.

**III) MINESURVEY**

- This is used for the exploring mineralwealth

**OBJECTS OF THE SURVEY**

- To Calculate The Distances Between Various Points And To Calculate The Levels Of Various Points
- To check out the alignment of various engineeringstructures.
- To calculate the areas and volumes,involved in the various engineeringprojects.
- To Prepare the plans and maps sections and profile,contoursects.
- To Measure and to determine the relative positions of the various objects of the earthssurface

**EQUIPMENT AND ACCESSORIES FOR CHAINING AND RANGING:**

(i)Chain
(ii)Arrows
(iii) Pegs
(iv)Surveyors' band
(v) Ranging rods and rangingpoles
(vi) Offsetrods
(vii) Laths
(viii) Whites
(ix) Plumb bobsand
(x) Lineranger.

**(i) CHAIN**

The Chain Is Made Up Of Steel Wire Which Is Divided Into Links And Togs (Rings) To Facilitate Folding.

- It Is Sometimes Used As A Unit OfMeasurement
- It Has Brass Handles At Both Ends For Easy Handling. The Link Is 0.2m Or 200mm InDiameter.
- The Length Is 20m Or30m.

**(ii) ARROWS:**

- Arrows are made of steel wire of diameter 4mm and their ends are bent into a circle where red cloth is tied to facilitate visibility. They are used for showing points on theground.

**iii) PEGS**

- Pegs are made of wood 40mm square by 50cm long and are used for permanently markingpositions duringsurvey

**iv) SURVEYORS'BAND**

- The surveyor's band is made of a steel strip which is rolled into a metal frame with a winding handle. It is 30m, 50m or 100m long. Is used in projects where more accuracy measurement isrequired.

**(v) RANGING RODS AND RANGINGPOLES:**

- A ranging rod is a surveying instrument used for marking the position of stations and for sightings of those stations as well as forranging
- Ranging poles are used to mark areas and to set out straight lines on the field. They are also used to mark points which must be seen from a distance, in which case a flag may be attached to improve the visibility.

**(vi) OFFSET RODS**

- These rods are also similar to ranging rods and they are 3 m long. They are made up of hard wood and are provided with iron shoe at oneend.
- A hook or a notch is provided at other end. At height of eye, two narrow slits at right angles to each other are also provided for using it for setting rightangles.

**(vii) LATHS**

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

**(viii) WHITES**

- Whites are the pieces of sharpened thick sticks cut from the nearest place in the field. One end of the stick is sharpened and the other end is split. White papers are inserted in the split to improve the visibility. Whites are also used for the same purpose aslaths.

**(IX) PLUMBBOBS:**

- In measuring horizontal distances along sloping ground plumb bobs are used to transfer the position to ground.They are also used to check the verticality of rangingpoles.

**(X) LINERANGER:**

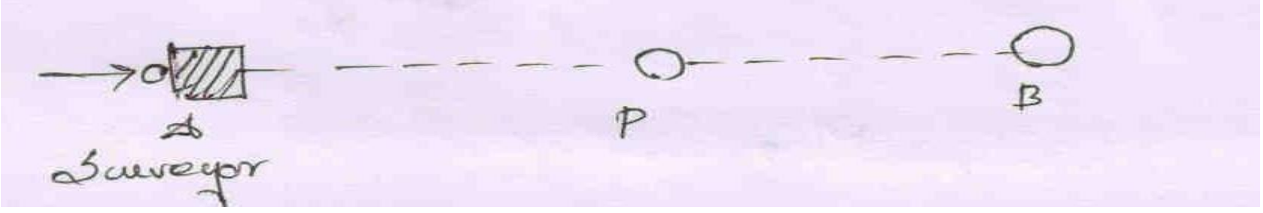
- It is an optical instrument used for locating a point on a line and hence useful for ranging. It consists of two isosceless prisms placed one over the other and fixed in an instrument withhandle.

**METHODS OF RANGING**

i)Direct Ranging
ii)Indirect Ranging

**i)DIRECT RANGING:**

- Direct Ranging is done when the two ends of the survey lines areintervisible.



**ii) INDIRECTRANGING**

- It is done when both the ends of the survey line are not intervisible either due to Long distance betweenthem.


**COMPASS SURVEYING**

- Compass surveying is a type of surveying in which the directions of surveying lines are determined with a magnetic compass, and the length of the surveying lines are measured with a tape or chain or laser rangefinder.

i)Prismatic Compass
ii)SurveyorCompass

**i)PRISMATIC COMPASS**

- A prismatic compass is a navigation and surveying instrumentwhichis extensively used to find out the bearing of the traversing and included angles between them, waypoints (an endpoint of the course) anddirection.

**ii) SURVEYORCOMPASS**

- Surveyor's compass consists of a circular brass box containing a magnetic needle which swings freely over a brass circle which is divided into 360 degrees. The horizontal angle is measured using a pair of sights located on north – south axis of the compass. They are usually mounted over a tripod and leveled using a ball and socketmechanism.

**BASIC PRINCIPLE OF COMPASSSURVEY**

- The Principle of Compass Survey is Traversing; which involves aseriesof connected lines the magnetic bearing of the lines are measured by prismatic compass and the distance (lengths) of the are measured bychain.

**BEARING**

- The Bearing of a line is the Horizontal Angle which it makes with a reference line(meridian) depending upon theMeridian.

**TYPES OF BEARING**

i. True Bearing
ii. Magnetic Bearing
iii. ArbitraryBearing

**i) TRUEBEARING**

- True Bearing of a line is the horizontal angle which it makes with the true meridian through one of the extremities of theline.

**ii) MAGNETIC BEARING**

- The Magnetic Bearing of a line is the horizontal angle which it makes with the magnetic meridian passing through one of the extremities of theline.

**iii) ARBITRARYBEARING**

- Arbitrary Bearing of a line is the horizontal angle which it makes with any arbitrary meridian passing through one of theExtremities.

**LEVELLING**

Levelling is a branch of surveying, the object of which is:

- To Find The Elevations Of Given Points With Respect To A Given Or Assumed Datum,And
- To Establish Points At A Given Or AssumedDatum.

**BASIC PRINCIPLE OF LEVELING**

- The fundamental principle of leveling lies in finding out the separation of level lines passing through a point of known elevation (B.M.) and that through an unknown point (whose elevation is required to bedetermined).

**METHODS OF LEVELLING**

**i)BAROMETRIC LEVELLING**

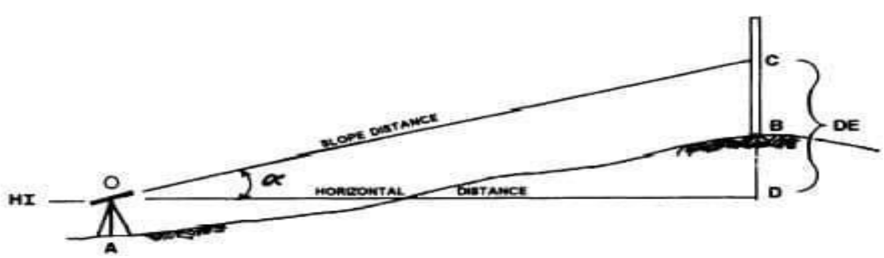
- Barometric Leveling. Barometer is an instrument used to measure atmosphere at any altitude. So, in this method of leveling, atmospheric pressure at two different points is observed, based on which the vertical difference between two points isdetermined.

**ii) DIRECTLEVELLING**

- It is the most commonly used method of leveling. In this method, measurements are observed directly from levelinginstrument.

**iii) TRIGONOMETRICLEVELING**

- The process of leveling in which the elevation of point or the difference between points is measured from the observed horizontal distances and vertical angles in the field is called trigonometricleveling.




**SOURCES OF ERRORS IN LEVELLING**

There are following types of Errors in Leveling :-

1. InstrumentalErrors
2. CollimationError

3. Error due to Curvature &Refraction
4. OtherErrors

**1.INSTRUMENTAL ERRORS & CORRECTION**

1. Collimationerror
   - Correction: Check before use and equalisesights.
2. Under sensitivebubble.
3. Errors in staffgraduation
   - Correction:Check
4. Loose tripodhead.
5. Telescope not parallel to bubbletube
   - Correction: Permanentadjustment.
6. Telescope not at right angles to the verticalaxis
   - Correction: Permanentadjustment

**2.COLLIMATIONERROR**

- Collimation error occurs when the collimation axis is not truly horizontal when the instrument is level. The effect is illustrated in the sketch below, where the collimation axis is tilted with respect to the horizontal by an anglea.

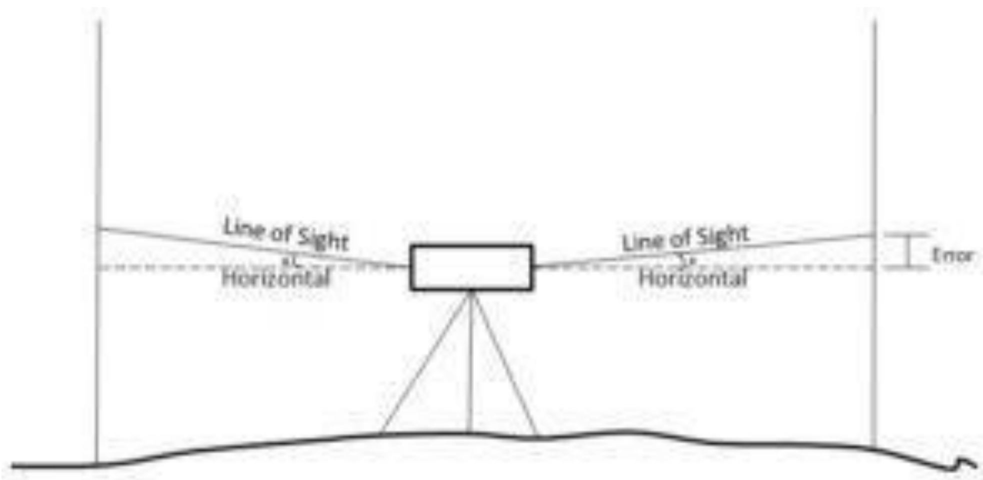



**Fig: Collimation Error**

**3.CURVATURE OF THEEARTH:**

The earth appears to "**fall away**" with distance. The curved shape of the earth means that the level surface through the telescope will depart from the horizontal plane through the telescope as the line of sight proceeds to the horizon.

This effect makes actual level rod readings too large by:

$$(C - r) = 0.0206D^2$$

where D is the sight distance in thousands of feet

## PART - A (2 marks)

**1. Describe the principle of surveying. (AUC Apr/May 2011) (AUC Nov/Dec2011)**

The fundamental principles upon which the surveying is being carried out are

- Working from whole topart.
- After deciding the position of any point, its reference must be kept from at least two permanent objects or stations whose positions have already been welldefined.

**2.What is the purpose of an optical square? (AUC Apr/May 2011) (AUC May/June 2012)**

It is more accurate than the cross staff and it can be used for locating objects situated at larger distances. It is small and compact hand instrument and works on the principle of reflection.

**3. What do you mean byreciprocalranging? (AUC Apr/May2010)**

When the end stations are not intervisible due to high ground or a hill or if the ends are too long. In such cases, intermediate points can be fixed on the survey line by a process known as Reciprocal ranging or Indirect ranging.

**4. What do you mean by scalein surveying? (AUC Nov/Dec2011)**

Scale is a fixed ratio that every distance on the plan bears with corresponding distance on the ground. For example: 1cm = 10m.

**5. Defineconditionedtriangles. (AUC Nov/Dec2010)**

The accuracy of a triangulation system, in which any error in angular measurement has a minimum effect upon the computed lengths, is known as well-conditioned triangle.

**6. Explain the range ofreciprocalranging. (AUC May/June2013)**

The vision ranging and line ranger can be adopted only when the end stations are inter-visible. The line of sight between two stations is obstructed by natural or man-made objects or not clearly visible. Under such conditions, indirect or reciprocal ranging is applicable.

**7. What do you mean byplanesurveying? (AUC May/June2013)**

Plane surveying is a process of surveying in which the portion of the earth being surveyed is considered a plane. In this training manual, we used in plane surveying rather than those used in geodetic surveying.

**8. What is meant bygeodeticsurveying? (AUC Nov/Dec 2012)**

Geodetic surveying is a process of surveying in which the shape and size of the earth are considered. The methods used in geodetic surveying are beyond the scope of this training manual.

**9.Explain the Methods OfRanging**.

i) DirectRanging

ii) IndirectRanging

**i) DIRECT RANGING:**

- Direct Ranging is done when the two ends of the survey lines areintervisible.

**ii) INDIRECTRANGING:**

- It is done when both the ends of the survey line are not intervisible either due to Long distance betweenthem.

**10.Define True Meridian. (AUC Nov/Dec 2012) (AUC Nov/Dec2010)**

- True Meridian is defined as the line Joining the Geographical North and South Pole.
- True Meridian at various Places are notEqual

**11.What is MagneticMeridian? (AUC May/Jun2012)**

- Magnetic Meridian is defined as the Longitudinal axis indicated by the freely Suspended,properly balanced MagneticNeedle.
- It Does not coincide with the true Meridian except in certain places during theyear

**12.What are the types of corrections tobeapplied? (AUC Nov/Dec 2014)**

- Correction forLength.
- Correction forTemperature.
- Correction forPull.
- Correction forSag.
- Correction forSlope.

**What Is TwoPointProblem? (AUCMay/Jun 2013)**

Two Point Problem is defined as the process of locating the plane table on the sheet by sighting two well defined Points And its locations are already plotted on the Paper.

**14. Define ThreePointProblem? (AUC May/Jun2013)**

Two Point Problem is defined as the process of locating the plane table on the sheetby sighting two well defined Points And its locations are already plotted on the Paper.

**15. Distinguish Between AngleAndBearing. (AUC May/Jun2012)**

- An Angle is defined as the deviation of one straight line with respect to the other one.
- Bearing is defined as the angle or Inclination of a survey Line with respect to the north - SouthDirection.

**16. What are the Sources Of LocalAttraction?**

- Magnetic Materials such As magenetic Rocks,iron Ores,Electrical cables etc..are sources of LocalAttraction.

**17. Name the different ways of classification ofSurveying.**

Classification Of Survey is based on

**i.** Purpose ofSurveying
**ii.** Nature of thefield
**iii.** Methodsemployed
**iv.** InstrumentsUsed

**18. How do you fix a point from control points(or a SurveyLine)?**

The position of a third point can be located from control points by anyone of the following Ways.
1.Two linear measurement
2.Two AngularMeasurement
3.One linear measurement & one AngularMeasurement.

**19. Write the equation for correction oftemperature**

Temperature correction

$$Ct = \alpha(Tm-To)L$$

α -coefficient of thermal expansion
Tm -mean temperature during measurements
To -normal temperature at stanrardlization
L-measured length of the line.

**20. What circumstances in which reciprocal ranging is used ?(or) When do yourequire ranging? (or) Explain the use of reciprocalranging?**

i. Reciprocal ranging is the method of indirect ranging and it is adopted whenthe
ii. Two end stations are move to raisedgrounds

**21. In a chain how will you set out a rightangle?**

i) Cross-staff is the instrument used to locate the intersection point of a particular offset on achain line.
ii)Optical Squares are also like cross-staves used for setting out the right angles inchange-surveying.

**22. What are the Instruments Used for chainSurveying?**

1.Chain
2.Tape
3.Ranging Rods
4.Offset Rods
5.Plumb Bob
6. Pegs
7.Cross-staff

8.Optical Square
9.Arrows

**23. Write a difference between a map &Plan.**

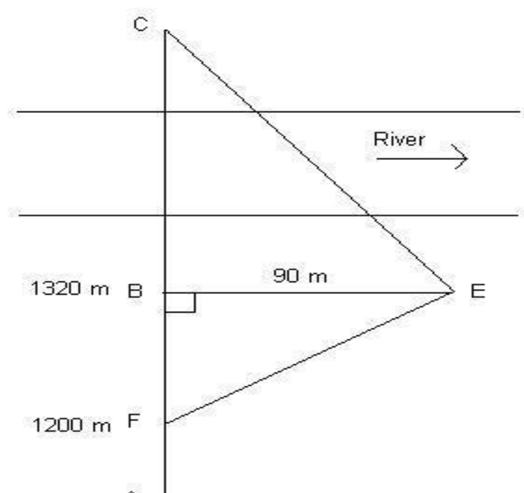
| S.No. | Factor | Map | Plan |
|---|---|---|---|
| 1. | Sclae | Maps are the Drawing with Small Scale | Plans are the drawing with Large Scale |
| 2. | Details | A map generally deals about the Geographical Details | A Plan deals with the details of the engineering Structures. |

**24. List out various Classification ofsurveying.**

i. ChainSurveying.
ii. CompassSurveying.
iii. Plane tableSurveying.
iv. Theodolite LevelSurveying.
v. Tacheometric Surveying.
vi. Total stationSurveying.

## PART-B

**1.A survey line ABC crossing a river angles cuts its banks at B and C. To determine the width BC of the river. The following operation was carried out. A point E was established on the perpendicular BE such that angle CEF is a right angle where F is a point on the survey line.If the chainage of F and B are respectively 1200 m and 1320 m and the distance EB is 90 m.Calculate the width of the river and also the chainageofC. (AUC Apr/May2011)**




BF = Chainage of B – Chainage of F

= 1320 – 1200

BF = 120 m

From Δ EBF,

tan BEF=120/90 = 1.33

$\angle BEC = \angle CEF - \angle BEF$

$= 90^{O} - 53^{O} 3'$

$\angle BEC = 36^{O}57'$

From Δ BEC,

$\tan (36^{O} 57') = CB/BE = CB/90$

$CB = 90 \times \tan (36^{O} 57')$

CB = 67.69 m

The width of the river, CB = 67.69 m

Chainage of C = chainage of B + width of the river

= 1320 + 67.69

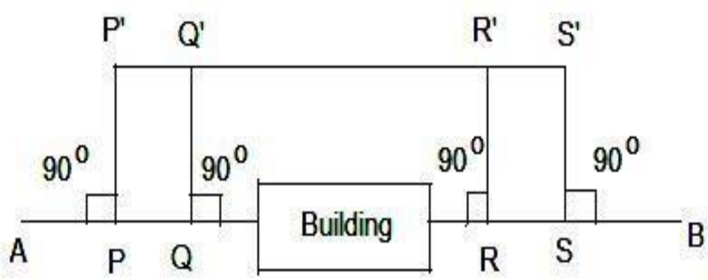
Chainage of C = 1387.69 m

**2.Explain the methods of chaining while there are obstacles such as building or river. (AUC Nov/Dec 2011) (AUC May/June 2012) (AUC Apr/May2010).**

In this case it is required to prolong the chain line beyond the obstacle and to find the distance across it. In this case the typical obstacle is a building. One of the following two methods may be adopted.

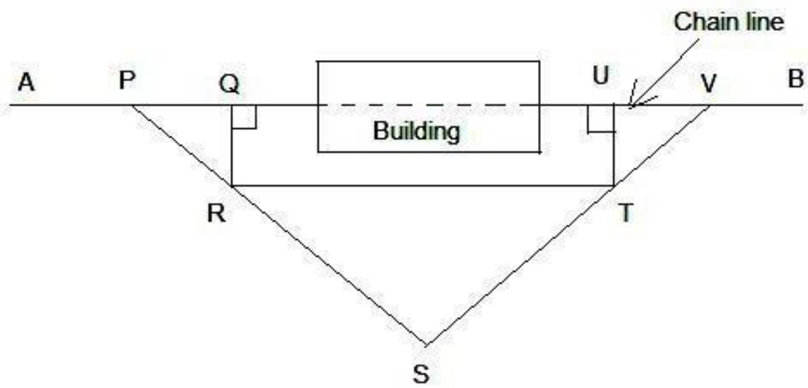
**FIRST METHOD:**

On one side of the chain line AB, two points P and Q are selected. Perpendiculars of equal length PP' and QQ' are erected. The line P'Q' is extended till the building is passed. On the extended line, two points R and S are selected. The perpendicular at R and S are so erectedsuchthat RR' = SS' = QQ' = PP'. then the points P', Q', R' and S' will lie on the sameline.Then Q'R = QR and the distance Q'R' is measured to set QR, then the line isextended.




**SECOND METHOD:**

This method is also equally applicable for this condition. Two points P and Q on the chain line AB are selected on the one side of the chain line. A perpendicular QR is erected at Q such that QR = PR. Points P and R are jointed and extended upto S. A perpendicular SV is set at S such that PS = SV. On the line SV a point T is marked such that ST = SR. with V as centre and radius equal to QR cut an arc such that PQ = QR = VT = UT. Then U and V are on the chain line AB. The distance RT is measured. Thus the obstructed length, QU = RT.




**3.Determine the sag correction for a 30 m steel tape under a pull of 80 N in 3 bays of 10 m each.The area of the cross section of the tape is 8 mm$^2$ and the unit weight of steel may be taken as 77 kN/m$^3$. (AUC Nov/Dec2011)**

**Solution:**

**Given:**

L = 30 m; n = 3; P = 80 N; Area = 8 mm$^2$ = 8 x 10$^{-6}$ m$^2$; $\gamma$ = 77 kN/m$^3$

Total weight of tape = 77 x 10$^3$ x 8 x 10$^{-6}$ x 10 = 6.16 N

$$C_s = LW^2 / 24n^2P^2$$

$$= 10 \times (6.16)^2 / 24 \times (1)^2 \times (80)^2$$

$$= 0.00247 \text{ m}$$

$C_s$ = 3 x 0.00247 = **0.00741 m**

True length = 30 – 0.00741

**True length = 29.993 m**

**4. Explain the field and office work inchainsurveying? (AUC May/June 2013)**

**Field and Officework:**

The practice of surveying actually boils down to fieldwork and office work. TheFieldwork Consists Of Taking Measurements, Collecting Engineering Data, And Testing Materials. TheOffice Work Includes Taking Care Of The Computation And Drawing The Necessary Information For The Purpose Of TheSurvey.

**Field Work:**

- Field work is of primary importance in all types of surveys. To be a skilled surveyor, you must spend a certain amount of time in the field to acquire neededexperience.
- The study of this training manual will enable you to understand the underlying theory of surveying, the instruments and their uses, and the surveyingmethods.
- However, a high degree of proficiency in actual surveying, as in other professions, depends largely upon the duration, extent, and variation of your actualexperience.

- You should develop the habit of STUDYING the problem thoroughly before going into the field, you should know exactly what is to be done; how you will do it; why youprefer a certain approach over other possible solutions; and what instruments and materials you will need to accomplish theproject.
- It is essential that you develop SPEED and CONSISTENT ACCURACY in all your fieldwork. This means that you will need practice in handling the instruments, taking observations and keeping field notes, and planning systematicmoves.
- It is important that you also develop the habit of CORRECTNESS. You should not accept any measurement as correct without verification. Verification, as much as possible, should be different from the original method used inmeasurement.
- The precision of measurement must be consistent with the accepted standard for a particular purpose of the survey. Fieldwork also includes adjusting the instruments and caring for fieldequipment.
- Do not attempt to adjust any instrument unless you understand the workings or functions of its parts. Adjustment of instruments in the early stages of your career requires close supervision from a seniorEA.

**Office Work:**

- Office work in surveying consists of converting the field measurements into a usable format.The conversion of computed, often mathematical, values may be required immediately to continue the work, or it may be delayed until a series of field measurements iscompleted.
- Although these operations are performed in the field during lapses between measurements, they can also be considered office work. Such operations are normally done to savetime.
- Special equipment, such as calculators, conversion tables, and some drafting equipment is used in most office work. In office work, converting field measurements (also called reducing) involves the process of computing, adjusting, and applying a standard rule to numericalvalues.

**5.Explain how you will conduct chain survey to measure a land parcel in agriculturefield. (AUC May/June2013)**

- Using chaining and ranging the distance between two points can be measured.The instruments required are chain, arrows, ranging rods, pegs andhammers.

**Procedures:**

- First mark a straight line of a standard length on a flat firm ground. The two end points A and B are selected on a survey line which is to bemeasured.

- A ranging rod is erected at the point B, while the surveyor stands with another rod at point A. A rod is established at a point in line with AB at a distance not greater than one chain length fromA.
- The surveyor at A then signals the assistant to move transverse to the chain line till he is line with A and B. Similarly other intermediate points can beestablished.
- Then by using chain, the distance is measured. To find the pacing length, we should walk along the chain line and it is found from pacinglength.

Pacing length = Distance between the points/No of steps

The distance between two points = (No of arrow x Nominal length +Fractional length) m

- The distance between two points can be calculated and also same procedure is used to find the other side of the line. The finally land parcel of agricultural field ismeasured

## UNIT II

## THEODOLITE AND TACHEOMETRIC SURVEYING

### THEODOLITE

- A theodolite is an instrument which is used primarily to measure angles, both horizontal and vertical. It is also used for many other subsidiary work during surveying such as setting up of intermediate points between inter visible points, establishment of inter visible points, prolonging a line, laying out traverse etc.
- A modern theodolite consists of a movable telescope mounted within two perpendicular axes the horizontal or trunnion axis, and the vertical axis. When the telescope is pointed at a target object, the angle of each of these axes can be measured with greatprecision.

### TYPES OF THEODOLITE

There are different types of theodolite available. It may be classified into three broad categories.

- Vernier or TransitTheodolite
- DigitalTheodolite
- TotalStation

### TRANSIT THEODOLITE

- A Transit Theodolite Is One In Which The Telescope Can Be Revolved Through AComplete Revolution About Its Horizontal Axis In A VerticalPlane.

### DIGITAL THEODOLITE

- Digital theodolite is a modern engineering instrument for measuring both horizontal and vertical angles, It is a key tool in surveying and engineeringwork.
- The theodolite consists of a telescope movable within two perpendicular axes- the horizontal axis, and the vertical axis.When the telescope is pointed at a desired object, the angle of each of these axes can be measured with greatprecision.

### TOTAL STATION

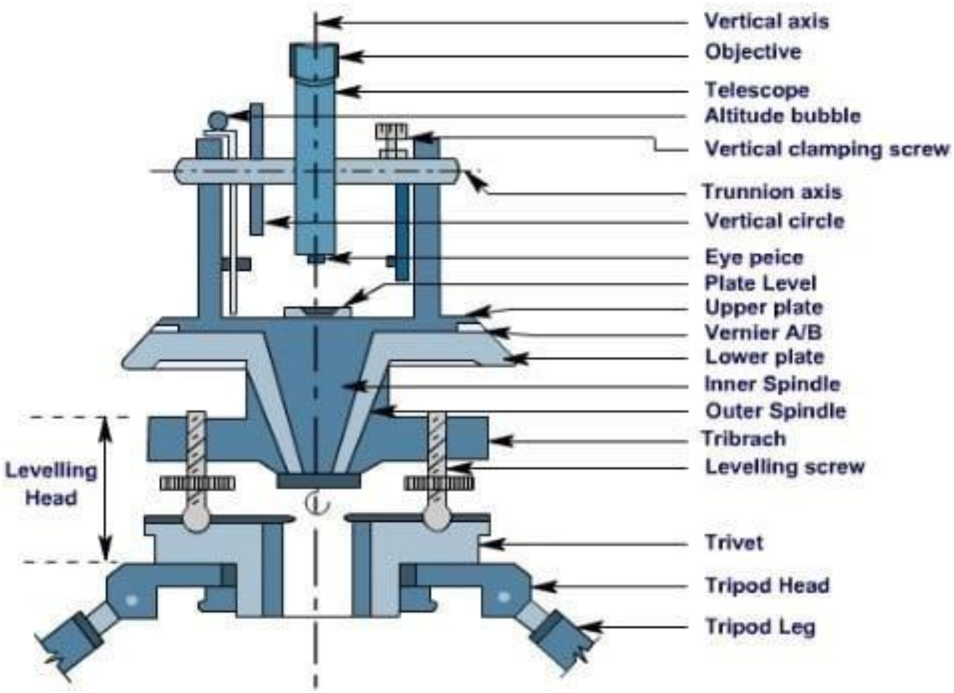
- Atotal station or TST (total station theodolite) is an electronic/optical instrument used for surveyingandbuildingconstruction.

- The total station is an electronic theodolite(transit) integrated with anelectronic distance measurement(EDM) to read slope distances from the instrument to a particular point, and an on-board computer to collect data and perform advanced coordinate basedcalculations.

**DIFFERENT PARTS OF THEODOLITE**

Each type of theodolite is peculiar in its construction and mode of operation. However, inherent fundamentals of all are same. In this course, the details will be considered for vernier type theodolite which is most popular and is being widely used. The salient parts of a vernier theodolite have been discussed below Figure

- LevelingHead
- ShiftingHead
- LowerPlate
- UpperPlate
- PlateLevels
- Standard (or aFrame)
- VernierFrame
- Telescope
- VerticalCircle
- Altitude Bubble
- Screws
- TripodStand




**METHODS OF HORIZONTAL ANGLE MEASUREMENT :**

- GeneralMethod
- RepititionMethod
- ReiterationMethod

**PERMANENT ADJUSTMENTS OF THEODOLITE :**

- The permanent adjustments are made to establish the relationship between the fundamental lines of the theodolite and , once made , they last for a long time. They are essential for the accuracy of observations.
- The permanent adjustments in case of a transit theodolites are**:-**

**i) Adjustment of Horizontal PlateLevels.**

- The axis of the plate levels must be perpendicular to the verticalaxis.

**ii) CollimationAdjustment.**

- The line of collimation should coincide with the axis of the telescope and the axis ofthe objective slide and should be at right angles to the horizontalaxis.

**iii) Horizontal axisadjustment.**

- The horizontal axis must be perpendicular to the verticalaxis.

**iv) Adjustment of Telescope Level or the Altitude Level PlateLevels.**

- Theaxisof the telescope levels or the altitude level must be parallel to the line of collimation.

**v) Vertical Circle IndexAdjustment.**

- The vertical circle vernier must read zero when the line of collimation ishorizontal.

## TEMPORARY ADJUSTMENTS OF THEODOLITE

- The temporary adjustments are made at each set up of the instrument before we start taking observations with the instrument. There are three temporary adjustments of atheodolite:-

  i)Centering.
  ii)Levelling.
  iii)Focussing.

## STADIA CONSTANT

- The distance is chosen so that there is a fixed, integer ratio between the distance observed between the marks and the distance from the telescope to the measuring device observed. This is known as the **stadia constant** or **stadia** interval factor. For example, a typical **stadia** mark pair are set so that the ratio is100.

## ANALYTIC LENS

- It is a special convex lens, fitted in between the object glass and eyepiece, at a fixed distance from the object glass, inside the telescope of a tacheometer. The function of the anallactic lens is to reduce the stadia constant tozero.
- Thus, when tacheometer is fitted with anallactic lens, the distance measured between instrument station and staff position (for line of sight perpendicular to the staff intercept) becomes directly proportional to the staff intercept. Anallactic lens is provided in external focusing type telescopesonly

## TACHEOMETER

- A tacheometer is similar to an ordinary transit theodolite fitted with stadia wires in addition to the centralcross-hairs.
- As accuracy and speed are necessary, the telescope fitted with a tacheometer must fulfill additional requirements. Also, the vertical circle should be morerefined.
- The telescope of the tacheometer is usually longer than that of the Ordinary theodolite and has a higher power ofmagnification.
- The object glass is of greater diameter, and the lens system is of better quality. The magnification power should not be less than20-25.

- The effective aperture should not be less than 3.5-4.5 cm in diameter facilitating the obtaining of a brightimage.
- The multiplying constant of the instrument (f/I) is generally kept as 100. Sometimes an additional pair cross-hairs is provided such that the multiplying constant (f/I) is50.

**TACHEOMETRY SURVEY**

- Tachometry is a branch of angular surveying in which A horizontal & vertical distance is of points are obtain by optical means as suppose to ordinary slow process of measure by tape chain.
- This methods is very rapid & convenient. All though the accuracy of tachometry is low it is best adopted in obstructed such as steep & broken ground stretches of water etc whichmake drawn agedifficult.
- They primary object of tachometry is the preparation of contour maps are plans required with both horizontal & vertical measurements also accuracy improvement it provides at check an distance measure withtape.

At the instruments a normally transit theodalite pitted with stadia diaphragm is generally used for tachometry survey. A stadia diaphragmessentially consist of one stadia hair above on the other an equal distance below the horizontal cross hair. Telescope is used in stadia surveying are of 3 types :-

a. Simple external focusingtelescope
b. External focusinganalytic
c. Internal focusingtelescope

**METHODS OF TACHOMETRIC SURVEY:**

(1) Stadia system
(2) Tangentialsystem

**(1) STADIA SYSTEM OF TACHEOMETRY**

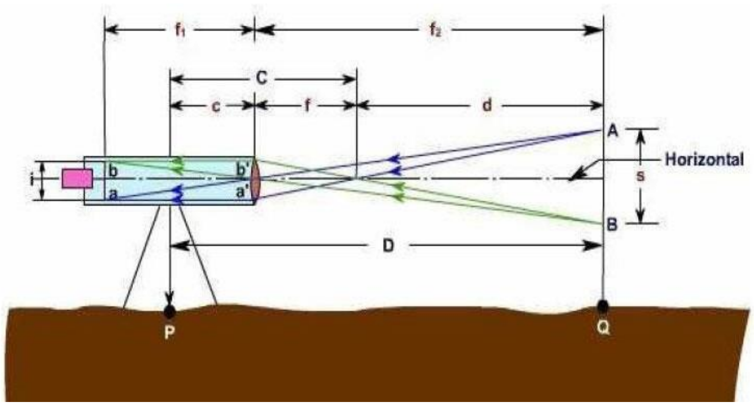
Inthestadiasystem,thehorizontaldistancetothestaffStationfromtheinstrumentstationandtheelevation of the staff station concerning the line of sight of the instrument is obtained with only one observation from the instrumentStation.

In the stadia method, there are mainly two systems of surveying.

(1) fixed hair methodand,
(2) movable hairmethod.

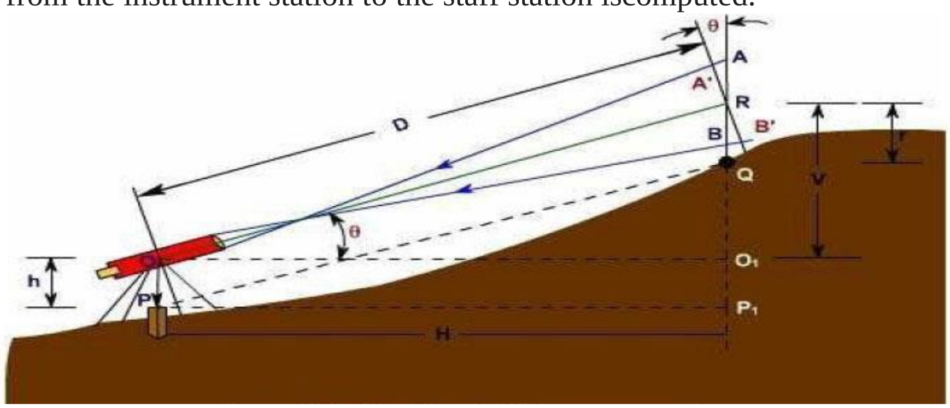
**(i) FIXED HAIRMETHOD:**

- In the fixed hair method of tacheometric surveying, the instrument employed for taking observations consist of a telescope fitted with two additional horizontal cross hairs one above and the other below the centralhair.
- These are placed equidistant from the central hair and are called stadiahairs.
- When a staff is viewed through the telescope, the stadia hairs are seen to intercept a certain length of the staff and this varies directly with the distance between the instrument and the stations.As the distance between the stadia hair is fixed, this method is called the “fixed hairmethod.”




**(ii) MOVABLE HAIR METHOD**

- In the movable Hair method of tacheometric surveying, the instrument used for taking observations consist of a telescope fitted with stadia hairs which can be moved and fixed at any distance from the central hair (within the limits of thediaphragm).
- The staff used with this instrument consists of two targets (marks) at a fixed distance apart (say 3.4 mm).
- The Stadia interval which is variable for the different positions of the staff is measured, and the horizontal distance from the instrument station to the staff station iscomputed.




**(2) TANGENTIAL SYSTEM OF TACHEOMETRICSURVEYING:**

- In this system of tacheometric surveying, two observations will be necessary from theinstrument station to the staff station to determine the horizontal distance and the difference in the elevation between the line of collimation and the staffstation.

- The only advantage of this method is that this survey can be conducted withordinary transit theodolite. As the ordinary transit theodolite are cheaper than the intricate and more refined tacheometer, so, the survey will be more economical.
- So, far as the reduction of field notes, distances and elevations are concerned there is not much difference between these twoSystems.
- But this system is considered inferior to the stadia system due to the following reasons and is very seldom usednowadays.
- This involves measurement of two vertical angles, and the instrument may get disturbed betweenthe two observations. The speed is reduced due to more number of observations and the changes in the atmospheric conditions will affect the readingsconsiderably.
- The staff used in this method is similar to the one employed in the movable hair method of stadia surveying. The distance between the targets or vanes may be 3-4m.

**CONTOUR**

A Contour Line May Be Defined As “An Imaginary Line Passing Through Points Of Equal Reduced Levels”. Acontour Line May Also Be Defined “As The Intersection Of A Level Surface With The Surface Of The Earth”. Thus,Contour Lines On A Plan Illustrates The Topography Of The Area.

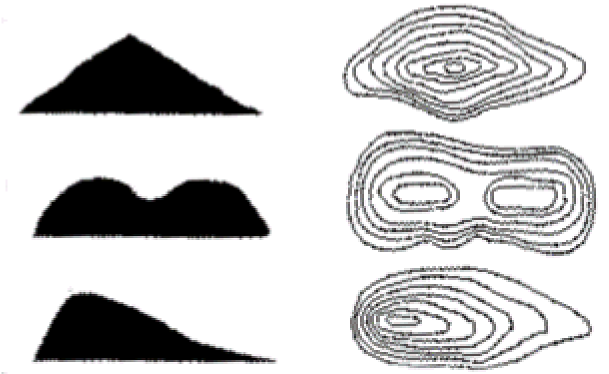
**CONTOUR INTERVAL**

The vertical distance between consecutive contours is termed as contour interval. Generally the contour intervals are taken in the range of 1 to 15 m. The contour interval is inversely proportional to the scale of the map. When we have less time to complete a survey for a large area contour interval is kept larger.

**METHODS OF CONTOURING**

1. **DIRECTMETHOD:**
   - In this method a series of points are located on the ground having same elevation. For a particular contour value the staff man is directed to move right or left until the required reading is obtained, this method is time consuming but it gives accurateresult.
2. **INDIRECTMETHOD**:
   - In Block Contouring the given area is divided into number of grids with a known interval and the staff reading is taken on the respective grid points to find the R.L values, by the method of interpolation the contour is plotted. In Radial contouring the same method is adopted but the R.L values are found on the radial lines running from the center point. This method is normally preferred on hillyareas.

**CHARACTERISTICS OF CONTOUR LINES:**



i. Steep slopes - contours are closelyspaced
ii. Gentle slopes - contours are less closelyspaced
iii. Valleys - contours form a V-shape pointing up the hill - these V's are always an indication of a drainage path which could also be a stream orriver
iv. Ridges - contours form a V-shape pointing down thehill
v. Summits - contours formingcircles
vi. Depressions - are indicated by circular contour with lines radiating to the center
vii. If the middle value is higher in a contour it means it is anelevation
viii. If the middle value is lesser in an contour it means it is andepression

**CONTOUR GRADIENT**

- An imaginary line on the surface of the earth having a constant inclination with the horizontal (slope) is called contourgradient.
- The inclination of a contour gradientis generally given either as rising gradient or falling gradient, and is expressed as ratio of the vertical height to a specified horizontaldistance.

**CONTOUR PLAN**

- A plan drawn to a suitable scale showing surface contours or calculated contours of coal seams tobe developed. These plans are important during the planning stage of aproject.

**CONTOUR MAP**

- A map showing elevations and surface configuration by means of contourlines

USES OF CONTOUR MAPS

- Determination of intervisibility between two points.
- Drawing ofsections.
- Measurement of drainage area is another use of contourmaps
- Measurement ofearthwork.
- Calculation of reservoircapacity.

# UNIT-3

## CONTROL SURVEYING ANDADJUSTMENTS

### CONTROL SURVEYING

- Horizontal and vertical control are developed to create a framework around which other surveys can be adjusted. These control surveys are used for accurate mapping projects in the construction of underground utility systems, roadways, power lines, tunnels, and many other high precisionprojects.

### HORIZONTAL CONTROLS & ITS METHODS

- The horizontal control consists of reference marks of known plan position, from which salient points of designed structures may be set out. For large structures primary and secondary control points are used. The primary control points are triangulation stations. The secondary control points are reference to the primary controlstations.
- Reference grids are used for accurate setting out of works of large magnitude. The following typesof reference grids areused:

- **SURVEY GRID**
- **SITEGRID**
- **STRUCTURALGRID**
- **SECONDARY GRID**

### SURVEY GRID

- Survey grid is one which is drawn on a survey plan, from the original traverse. Original traverse stations form the control points of thegrid.

### SITE GRID

- The site grid used by the designer is the one with the help of which actual setting out is done. As far as possible the site grid should be actually the survey grid. All the design points are related in terms of site gridcoordinates.

### STRUCTURAL GRID

- The structural grid is used when the structural components of the building are large in numbers and are so positioned that these components cannot be set out from the site grid with sufficient accuracy. The structural grid is set out from the site gridpoints.

### SECONDARY GRID

- The secondary grid isestablished inside the structure, to establish internal details of the building, which are otherwise not visible directly from the structuralgrid.

### VERTICAL CONTROL & ITS METHODS:

The vertical control consists of establishment of reference marks of known height relative to some special datum. All levels at the site are normally reduced to the nearby bench mark, usually known as master bench mark.

The setting of points in the vertical direction is usually done with the help of following rods:

1. Boning rods andtravelers
2. SightRails
3. Slope rails or batterboards
4. Profileboards

**1.BONING RODS ANDTRAVELERS**

- A boning rod consist of an upright pole having a horizontal board at its top, forming a 'T 'shapedrod.
- Boning rods are made in set of three, and many consist of three 'T' shaped rods, each of equal size and shape, or two rods identical to each other and a third one consisting of longer rod with a detachable or movable 'T' piece. The third one is called traveling rod ortraveler.

**2.SIGHTRAILS:**

- A sight rail consist of horizontal cross piece nailed to a single upright or pair of uprights driven into the ground.
- The upper edge of the cross piece is set to a convenient height above the required plane of the structure, and should be above the ground to enable a man to conveniently align his eyes with the upperedge.
- A stepped sight rail or double sight rail is used in highly undulating or fallingground.

**3.SLOPE RAILS OR BATTERBOARDS:**

- These are used for controlling the side slopes in embankment and in cuttings. These consist of two vertical poles with a sloping board nailed near theirtop.
- The slope rails define a plane parallel to the proposed slope of the embankment, but at suitable vertical distance above it. Travelers are used to control the slope during fillingoperation.

**4.PROFILEBOARDS:**

- These are similar to sight rails, but are used to define the corners, or sides of a building. A profile board is erected near each cornerpeg.
- Each unit of profile board consists of two verticals, one horizontal board and two cross boards. Nails or saw cuts are placed at the top of the profile boards to define the width of foundation and the line of the outside of thewall.

**TRIANGULATION SURVEYING**

- Triangulation surveying is the tracing and measurement of a series or network of triangles to determine distances and relative positions of points spread over an area, by measuring the length of one side of each triangle and deducing its angles and length of other two sides by observation from thisbaseline.

**CLASSIFICATION OF TRIANGULATION SYSTEM:**

- The basis of the classification of triangulation figures is the accuracy with which the length and azimuth of a line of the triangulation are determined. Triangulation systems of different accuracies depend on the extent and the purpose of the survey. The accepted grades of triangulationare:

1. First order or PrimaryTriangulation

2. Second order or SecondaryTriangulation

3. Third order or TertiaryTriangulation

**1. FIRST ORDER OR PRIMARYTRIANGULATION:**

- The first order triangulation is of the highest order and is employed either to determine the earth's figure or to furnish the most precise control points to which secondary triangulation may be connected.
- The primary triangulation system embraces the vast area (usually the whole of the country). Every precaution is taken in making linear and angular measurements and in performing the reductions. The following are the general specifications of the primarytriangulation:

| | |
|---|---|
| 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 kilometers |
| 4. Length of the sides of triangles | : 30 to 150 kilometers |
| 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 kilometers |
| 8. Probable error or computed distance | : 1 in 60,000 to 1 in 250,000 |
| 9. Probable error in astronomic azimuth | : 0.5 seconds |

**2. SECONDARY ORDER OR SECONDARYTRIANGULATION**

The secondary triangulation consists of a number of points fixed within the framework of primary triangulation. The stations are fixed at close intervals so that the sizes of the triangles formed are smaller than the primary triangulation. The instruments and methods used are not of the same utmost refinement. The general specifications of the secondary 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 kilometers

8. Probable error or computed distance : 1 in 20,000 to 1 in 50,000

9. Probable error in astronomic azimuth : 2.0 sec

**3 THIRD ORDER OR TERTIARY TRIANGULATION:**

- The third-order triangulation consists of a number of points fixed within the framework of secondary triangulation, and forms the immediate control for detailed engineering andother surveys. The sizes of the triangles are small and instrument with moderate precision may be used. The specifications for a third-order triangulation are as follows:

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 kilometers
8. Probable error or computed distance : 1 in 5,000 to 1 in 20,000
9. Probable error in astronomic Azimuth: 5 sec.

**BASE LINE.**

- In surveying,a baseline is a line between two points on the earth's surface and the direction and distance between them. In a triangulation network, at least one baseline needs to be measured to calculate the size of the triangles bytrigonometry

**FACTORS TO BE CONSIDERED WHILE SELECTING BASE LINE.**

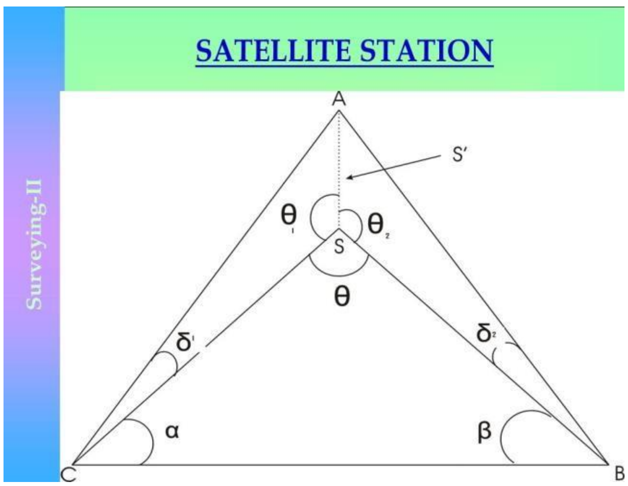
- The measurement of baseline forms the most important part of the triangulation operations. The base line is laid down with great accuracy of measurement and alignment as it forms the basis for the computations of triangulationsystem.
- The length of the base line depends upon the grades of the triangulation. Apart from main base line, several other check bases are also measured at some suitable intervals. In India, ten bases were used, the lengths of the nine bases vary from 6.4 to 7.8 miles and that of the tenth base is 1.7miles.

- Selection of Site for Base Line. Since the accuracy in the measurement of the base line depends upon the site conditions, the following points should be taken into consideration while selecting thesite:

1. The site should be fairly level. If, however, the ground is sloping, the slope should be uniform and gentle. Undulating ground should, if possible beavoided.
2. The site should be free from obstructions throughout the whole of the length. The line clearing should be cheap in both labour andcompensation.
3. The extremities of the base should be intervisible at groundlevel.
4. The ground should be reasonably firm and smooth. Water gaps should be few, and if possible not wider than the length of the long wire ortape.
5. The site should suit extension to primary triangulation. This is an important factor since the error in extension is likely to exceed the error inmeasurement.

**SATELLITE STATION**

- A Satellite Station Is Used When The Instrument Cannot Be Set Up At The Main Station.The Distance Of The Satellite From Its Station Is Usually Very Small As Compared To The Length Of The Sides Of The Triangulation.
- In order to secure well condition triangle or better intervisibility objects such as church tops,plag poles or towers etc.are sometime selected as triangulationstations.
- If the instrument is impossible to set up over that point a subsidiary station known as a satellite station or false station is selected as near as possible to the mainstation.
- Observations aremade to the other stations with the same precicion from the satellitestation.



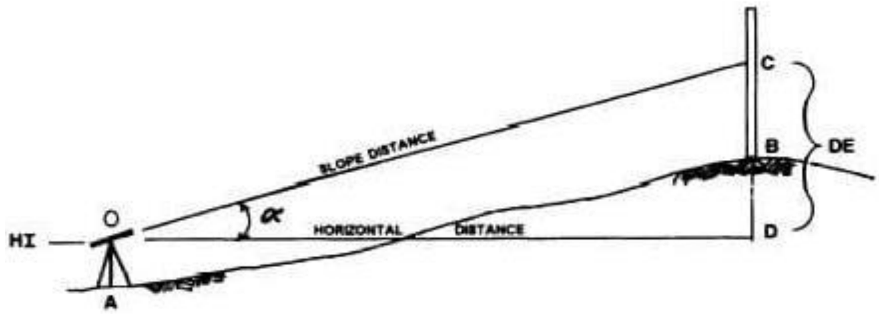

**REDUCTION TO CENTER**

- The angles are then corrected and reduced to what they would be from the truestation.
- The operation applying to this correction due to the eccentricity of the station is generally known as reduction tocenter
- Distance between true station and satellite station is determined by method of trigonometricleveling.

**TRIGONOMETRIC LEVELING**

- The process of leveling in which the elevation of point or the difference between points is measured from the observed horizontal distances and vertical angles in the field is called trigonometricleveling
- Trigonometric Leveling is the branch of Surveying in which we find out the vertical distance between two points by taking the vertical angular observations and the knowndistances.

- The known distances are either assumed to be horizontal or the geodetic lengths at the mean sea level(MSL). The distances are measured directly(as in the plane surveying) or they are computed as in the geodeticsurveying.
- The trigonometric Leveling can be done in twoways:
  **(1) Observations taken for the height anddistances**
  **(2) Geodetic Observations**




**(1) OBSERVATIONS TAKEN FOR THE HEIGHT AND DISTANCES**:
- In this way, we can measure the horizontal distance between the given points if it isaccessible.
- We take the observation of the vertical angles and then compute the distances using them. If the distances are large enough then we have to provide the correction for the curvature and refraction and that we provide to the linearly to the distances that we havecomputed.

**(2) GEODETIC OBSERVATIONS:**
- In the second way, i.e geodetic observations, the distances between the two points are geodetic distances and the principles of the plane surveying are not applicable here. The corrections for the curvature and refraction are applied directly to the anglesdirectly.

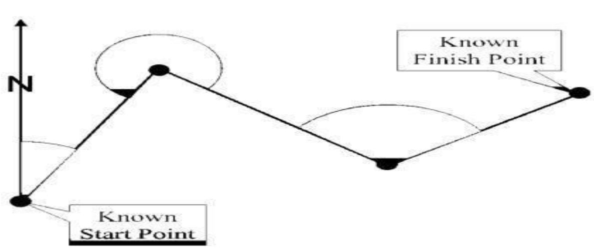
**TRAVERSING**

- Traverse Is A Method In The Field Of Surveying To Establish Control Networks. It Is Also UsedIn Geodesy.
- Traverse networks Involve Placing Survey Stations Along A Line Or Path Of Travel, AndThen Using The Previously Surveyed Points As A Base For Observing The NextPoint.

There Are Two Types Of Traverses:

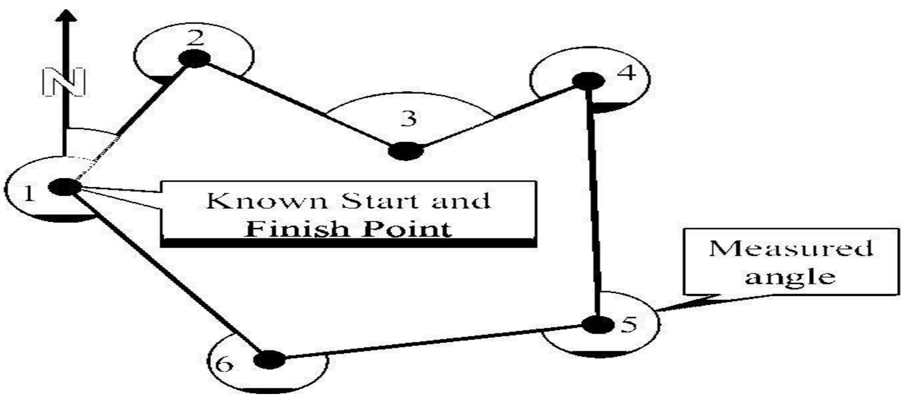
- OpenTraverse
- ClosedTraverse.

**OPEN TRAVERSE**
- An open traverse originates at a point of known position and terminates at a point of unknown position.



## CLOSED TRAVERSE

- A closed traverse originates and terminates at points of known positions. When closedtraverse originates and terminates at the same point, it is called the closed-looptraverse.




## METHODS TRAVERSING

- There are four methods by which the direction of the survey lines are determined are asfollow.

17. By the chainangle
18. By the free or loose needlemethod
19. By the fast needlemethod
20. By the measurement of angles between the successivelines.

### 1.BY THE CHAINANGLE

- In this method, the entire work is done with a chain/tape only and The angle between the successive lines is measured with thechain.

- Angles fixed by the measurements are known as chainangle.

### 2.BY THE FREE OR LOOSE NEEDLEMETHOD

- In this method, an angular instrument such as compass or theodolite, is set up at each of the successive stations and the bearing of each lines is taken with reference to the magnetic meridian and not with reference to the adjacentlines.

### 3. BY THE FAST NEEDLE METHOD

- In this method, a theodolite is used to determine the bearing of each line. The bearing of first line is measured with the magnetic meridians and the bearing of the successive lines are found from the deflection angle or from the includedangle.

**4.BY THE MEASUREMENT OF ANGLES BETWEEN THE SUCCESSIVELINES:**

In this method, a theodolite is used for measurement of angles. The horizontal angles measured in a traverse may be

- Included anglesor
- Deflection angles (between the successivelines)

This is the most accurate method and is generally used for large surveys and accurate work.

**GALE'S TABLE**

- Traverse computations are usually done in a tabular form is called Gale'stable

**CHARACTERISTICS:**

- The sum of all the observed interior angles is found which should be equal to (2n-4) right angle.
- If exterior angles are measured then the sum should be equal to (2n+4) rightangles.

**Use Of Gale's Table**

- The sum of latitudes (∑L) and departures (∑D) arefound.
- Necessary corrections are done in closed traverse such that ∑L = O and ∑D =O.
- The independent coordinates of the lines are obtained from corrected consecutive coordinates.
- The coordinates are positive and the entire traverse lie in the firstquadrant.

**SOURCES OF ERROR IN MEASUREMENT**

1. Instrumentalerrors
2. Personalerrors
3. Naturalerrors

**1.Instrumentalerrors**

- Error may arise due to imperfection or faulty adjustment of the instrument with which measurement is beingtaken.

**For example:**

- A tape may be too long or an angle measuring instrument maybe out of adjustment. Such errors are known as Instrumentalerros.

**Personal Error**

- Error may also arise due to want of perfection of human sight in observing and of touch in manipulatinginstruments.

**For example:**

- An error maybe taking the level readings or reading an angle on a circle of theodolite. Such errors are known as Personalerrors.

**Natural errors**

- Errors may also be due to variations in natural phenomena such as temperature, humidity, wind, refraction and magnetic declination. If it is not properly observed while taking measurements, the results will beincorrected.

**TERMS USED FOR ERRORS IN SURVEYING**

**TRUE VALUE OF A QUANTITY**

- The value of a quantity which is absolutely free from any error is called the true value. It can neverbe found out and the true value of a quantity is indeterminate.

**MOST PROBABLE VALUE OF A QUANTITY**

- Most probable value of a given quantity from the given available set of observation is the onefor which the sum of the squares of the residual errors is aminimum.
- The most probable value of a quantity is one which is most likely to be true value than any other values. This is most likely to be free, but not likely to be absolutely free, from errors. In case of direct observations of equal weight, the most probable value is the arithmetic mean. In case of direct observations of unequal weights, the most probable value is the weights; the most probable value is the weighted arithmeticmean.

**WEIGHT OF AN OBSERVATION**

- The weight of an observation is a number giving an indication ofits precision and trust worthiness, when making a comparison between several quantities of differentworth.
- If a certain observation of weight 4 it means that it is 4 times as much reliable as an observation of weight 1. When two quantities (or) observations are assumed to be equally reliable, the observed values are said to be of equal weight (or) of unitweight
- The weight of an observation is a factor depending on the importance attached to the observation. It actually give an indication of the precision and trustworthiness of the observation when making a comparison between several quantities of differentworth.

**PRINCIPLES OF LEAST SQUARES**

- The least squares principle states that the SRF should be constructed (with the constant and slope values) so that the sum of the squared distance between the observed values of your dependent variable and the values estimated from your SRF is minimized (the smallest possiblevalue).
- It is found from the probability equation that the most probable values of a seriesof errors arising from observations of equal weight are those for which the sum of the squares is a minimum.
- The fundamental law of least squares is derived from this. According to the principle of least squares, the most probable value of an observed quantity available from a given set of observations is the one for which the sum of the squares of the residual errors is aminimum.
- When a quantity is being deduced from a series of observations, the residual errors will be thedifference between the adopted value and the several observedvalues,
- Let $V1$, $V2$, $V3$ etc. be theobserved

**NORMAL EQUATION**

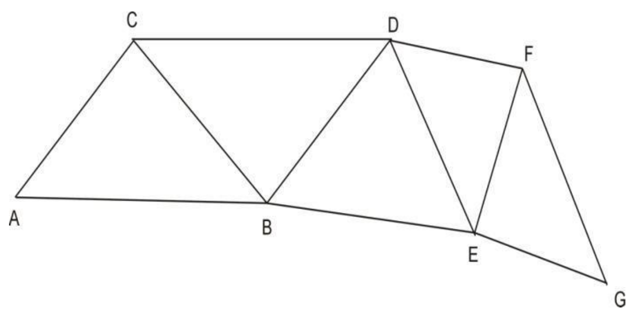
- Is The One Which Is Formed By The Multiplying Each Equation By The CoefficientOf
- The Unknown, Whose Normal Equation Is To Be Formed Out By Adding The Equation ThusFormed

**ADJUSTMENTS OF SIMPLE TRIANGULATION NETWORKS.**

- Single angle
- Stationadjustment
- Figure adjustment

**STATION ADJUSTMENT**

- Sum Of The Angles About A Station Should Be 360°. If Not, Find The Difference And Adjust The Difference Equally To All The Angles Algebraically To Make Their Sum Equal To 360°. Suppose; For A Station**B.**




| **Angles** | **Observed Value** | **Correction** | **Corrected Value** |
|---|---|---|---|
| **∟1** | **---** | **-12''** | **---** |
| **∟2** | **---** | **-12''** | **---** |
| **∟3** | **---** | **-12''** | **---** |
| **∟4** | **---** | **-12''** | **---** |
| | **$\sum = 360^o\ 00'\ 48''$** | | **$\sum = 360^o\ 00'\ 00''$** |

**FIGURE ADJUSTMENT**

- The determination of most probable values of angles involved in any geometrical figure so as to fulfill the geometrical conditions is called the figure adjustment. All cases of figure adjustment necessarily involve one or more conditional equations. The geometrical figures used in a triangulation systemare:
  - (a). Triangles.
  - (b). Quadrilaterals.
  - (c). Polygons with centralstations.

**UNIT 4**

**ADVANCED TOPICS IN SURVEYING**

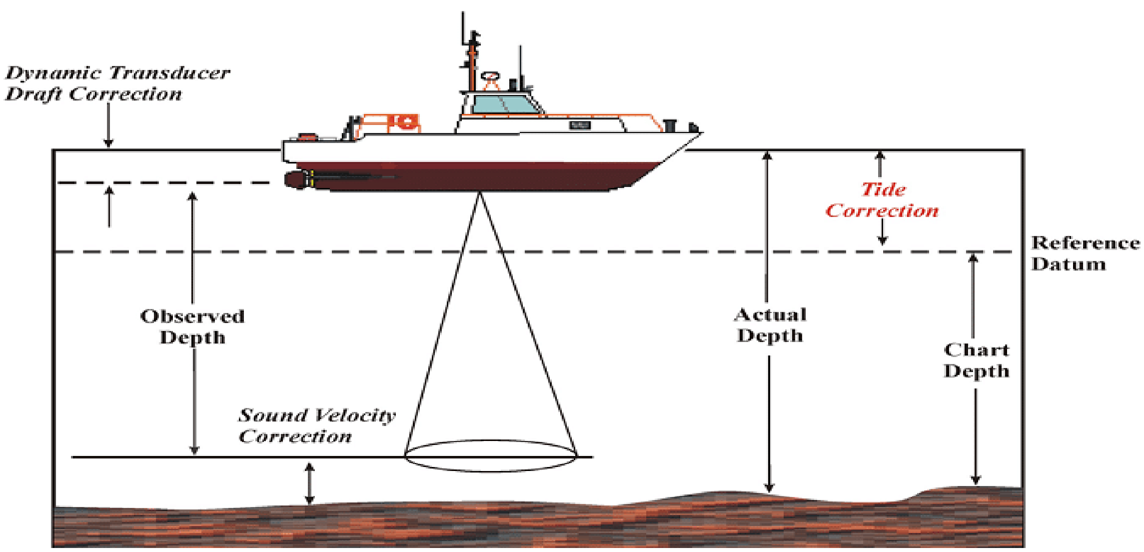
## HYDROGRAPHIC SURVEYING

- Hydrographic survey is the science of measurement and description of features which affect maritime navigation, marine construction, dredging, offshore oil exploration/offshore oil drilling and related activities.
- Hydrographic surveying or bathymetric surveying is the survey of physical features present underwater. It is the science of measuring all factors beneath water that affect all the marine activities like dredging, marine constructions, offshore drillingetc.

- Hydrographic surveying is mainly conducted under authority concerns. It is mainly carried out by means of sensors, sounding or electronic sensor system for shallowwater.
- The information obtained from hydrographic surveying is required to bring up nautical charts which involves,

i. Available depths
ii. ImprovedChannels
iii. Breakwaters
iv. Piers
v. The aids to navigation harborfacility

- These survey also take part in necessary data collection relating to construction and developments of port facilities, such as pier construction. This help in finding the loss in capacity due to silt and many uncertainties.




**Applications of Hydrographic Surveying**

Following are the applications of hydrographic surveying:

- Dock and HarborEngineering
- Irrigation
- RiverWorks
- Landreclamation
- WaterPower
- FloodControl
- Sewage Disposal

**Uses of Hydrographic Surveying**

Uses of hydrographic surveying are given below:

1. Depth of the bed can bedetermined
2. Shore lines can bedetermined
3. Navigation ChartPreparation
4. Locate sewer fall by measuring directcurrents
5. Locating mean sealevel
6. Scouring, silting and irregularities of the bed can beidentified
7. Tidemeasurement
8. River and stream dischargemeasurement
9. Massive structures like bridges, dams harbors areplanned

**Preliminary Steps in Hydrographic Surveying**

- The method starts by locating special control points along the shore line. The sounding method is employed to determine the depth at various points by means of stationaryboats.
- Sounding locations can be either made from boat to the control points or by fixing a point in theboat and taking sounding from the control point. Before this procedure certain preliminary steps have to be made:

1. Reconnaissance
2. Locate HorizontalControl
3. Locate verticalControl

**1.Reconnaissance**

- As every project require a start-up plan to complete it effectively and economically, reconnaissance has to be undergone. A complete reconnaissance of whole survey area to choose the best way of performing thesurvey.

- This would facilitate satisfactory completion of the survey in accordance with the requirements and specifications governing such work. Aerial photographs would help thisstudy.

**2.Locating HorizontalControl**

- The horizontal control is necessary to locate all features of the land and marine in true relative positions. Hence a series of lines whose lengths and azimuths are determined by means ofeither triangulation or any othermethods.
- Tachometric and plane table survey can be conducted in order to undergo rough works. No rules are kept for establishing horizontal control as topography, vegetation, type, size of topography affectthe rules.

- But in general a rules can be kept for type of controlsay:

- It is advisable to run traverses along each shore, connecting each other by frequent tie lines –If water body > 1kmwide
- It is advisable to run transverse line only along one of the banks -If water body isnarrow
- Triangulation system -If shorelines filled byvegetation
- Large network of triangulation system for large lakes and ocean shorelines

- A combined triangulation and traversing is shown in figure1.

**3.LOCATING VERTICALCONTROL**

- Before sounding establishment of vertical control is essential to determined. Numerous benchmarks are placed in order to serve as vertical control. Setting and checking the levels of the gauges areuses ofbenchmarks

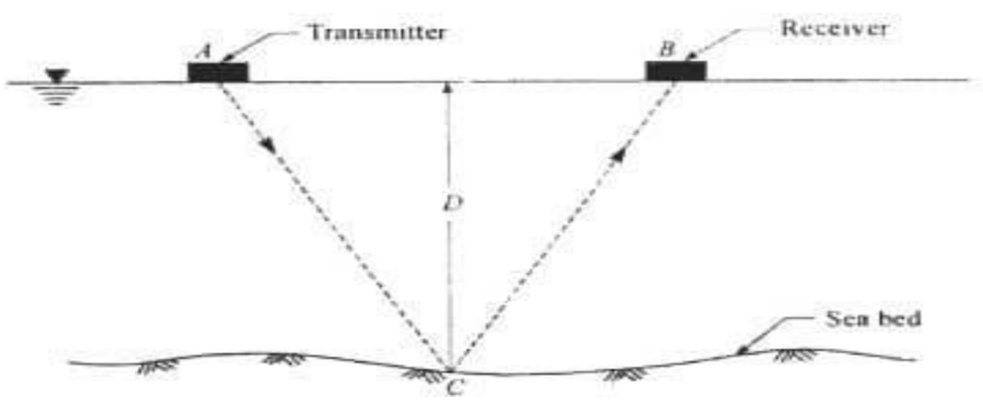



**Fig. 1: Combined Triangulation and Traversing in Hydrographic Survey**

## SOUNDING IN HYDROGRAPHIC SURVEY

- The process of determining depth below water surface is called as sounding. The step before undergoing sounding is determining the mean sealevel.
- If the reduced level of any point of a water body is determined by subtracting the sounding frommean sea level, hence it is analogous tolevelling.

### Methods of Locating Soundings in Hydrographic Surveying

- The soundings are located by the observations made from the boat or from the shore or fromboth.
- There are four methods are there to locate the soundingsby:

1. Conning the surveyvessel
2. Observations with theodolite orsextant
3. Theodolite angles and EDM distances from theshore
4. Microwave systems

## SOUNDING BY CONNING THE SURVEY VESSEL

- In this method, conning means keeping the boat at known course. This method is suitable for rivers, open sea up to 5 km off shore. The markers are fixed on the shore called as ranges along whichvessel or boat is run. This method is again sub divided into two types as follows.
- Location by crossrope
- Location by range and timeinterval

### *Location by Cross Rope*

- In this method, a wire or rope with markings or tags at known distances is stretched across the channel. The starting point of rope at the shore is marked as reference point. Then using boat, the sounding at different distances of wire are determined by weightedpole.
- This method is more accurate. This is most suitable for rivers, narrow lakes and for harbors. This is also suitable for knowing the amount of material removed bydredging.

### *Location by Range and Time Interval*

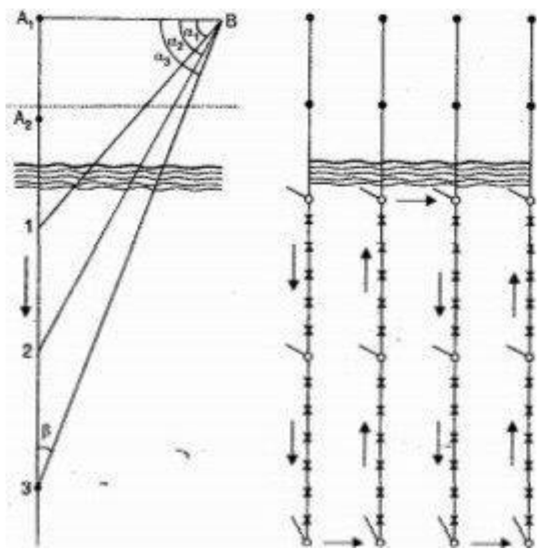
- In this method, the boat is positioned in range with two signals provided on the shore. Then, the boat is rowed at constant speed and time required to reach the instant of sounding is measured which givesthe distance of total point along the range. This method is more suitable for less width channels or rivers. It is not so muchaccurate.

## SOUNDING BY OBSERVATIONS WITH THEODOLITE OR SEXTANT

- Theodolite or sextant is used to measure angles in surveying. In this method, the sounding islocated by measuring angles. Here also, there are a lot of subdivided methods are there to locate sounding. They are
- By range and one angle from theshore
- By range and one angle from theboat
- By two angles form theshore
- By two angles from theboat
- By one angle from the shore and one angle fromboat
- By intersectingangles
- Bytachometry

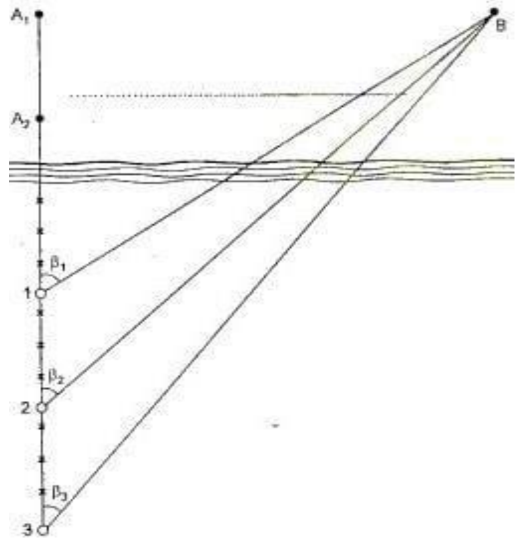
***By Range and One Angle from the Shore***

- In this method, boat is kept in range line with the help of two signals on the shore. The boat is moved and the point where sounding is measuring is observed by the theodolite or sextant and angle is noted. Using this angle, we can fix the point in therange.
- Likewise, all other soundings are observed from different stations. The angle should bemore than 30 degrees otherwise fix should be poor.so, whenever the angle is less than $30^o$, new instrument station is selected. This method is so accurate and easy for plotting the sounding details.



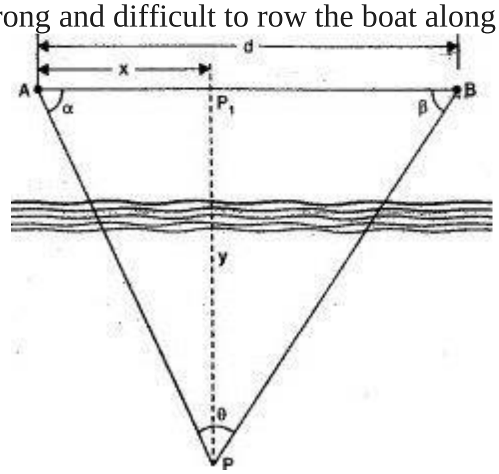
***By Range and One Angle from the Boat***

- This method is similar to the above method, but in this case, the angular measurements are taken from the boat to different stations positioned on the shore. This is also havingsimilar accuracy to the abovemethod.
- But, there are some advantages in this method as compared with above method. Angle measured from the shore from different stations is difficult when compared to angleobserved from the boat to allstations.
- So, the surveyor in this case has better control over the operations. Check can be made by measuring second angle towards some other signal on the shore for importantfixes.



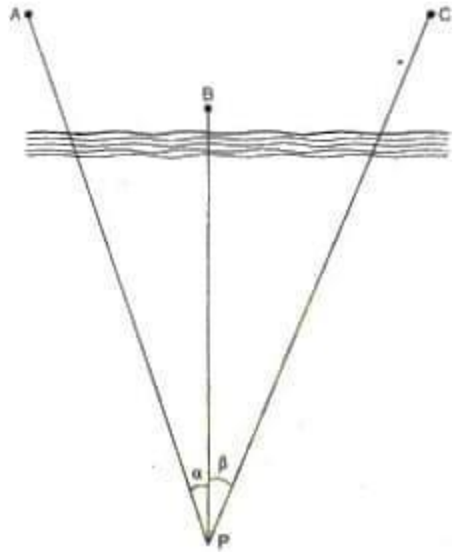

***By Two Angles form the Shore***

- In this method, two instrument stations are fixed on the shore with proper distance. Two instruments and two instrument men are required to do this job. From the two instrument stations, angular observations are made and a point is located where sounding is measured.
- If the angle made by instrument is less than $30^0$ then new instrument station is selected. Inthis case, primary setting out and erecting range signals are eliminated. This method is useful when water currents are strong and difficult to row the boat along range line.



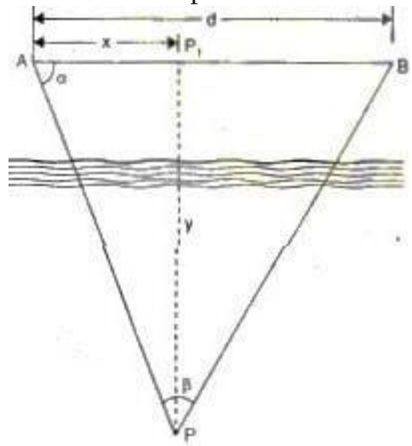

***By Two Angles from the Boat***

- In this method, three constant points on the shore are selected. Using three-point problem, boat is positioned in range line and angles are observed from the boat to two of the three known positions.
- The known positions may be light house, church spire, etc. like objects on the shore. If fixed positions are not available, then go for shore signals or rangingrods.




***By One Angle from the Shore and One Angle from Boat***

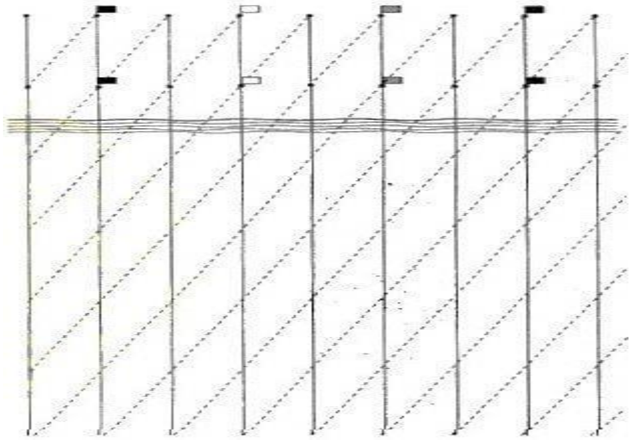
- This method also requires two instruments and two men to operate. This is the combination of above two methods. In this method, two instrument points are located on the shore and instrument is placed only at one point. Other instrument is placed in theboat.
- The first angle is measured from the first point on the shore to boat and from the boat second angle is measured from boat to second point. At that instant sounding ismeasured.




***By Intersecting Angles***

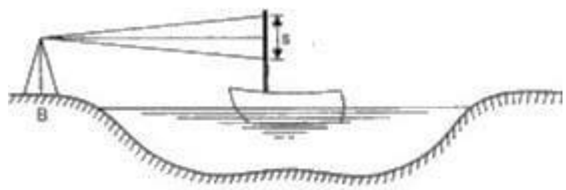
- In this method, sounding is determined periodically atsame points. This method is essentially used for harbors, reservoirs etc. to know the amount of silting or scouring happened at that points.
- Number of signals are erected on the shore and a boat is rowed perpendicular to the shore and measure the sounding at a point where inclined line of signal intersect the line of signal next to it as shown infig.

- Flag rods are erected at sounding points to avoid confusion for the next round ofmeasuring.

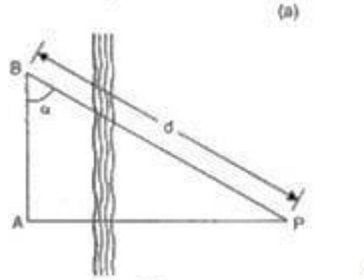


*By Tachometry*

- In this method, tachometer is placed on the shore and staff is placed on a boat. The staff intercept “s” is known by tachometer from this the distance between boat and instrument is known. This method is suitable when water is stable and sounding location is nearer to the shore.




(a)




(b)

**SOUNDING BY THEODOLITE ANGLES AND EDM DISTANCES FROM SHORE**

- In this method, EDM and Theodolite are placed on the shore in fixed positions. From thisset up, the reflector placed on the boat is targeted and point of sounding islocated.
- This method is more accurate when the water is still. This is one of the modern methods of fixing soundingvessel.

**SOUNDING BY MICROWAVE SYSTEMS**

- In this method, a device called Tellurometer is used which contains three units’ namely master unit, remote unit and master antenna. Master unit is fixed to the boat and other two units are located on the shore at two shorestations.

- The distances are measured from boat to the shore stations using micro waves produced by tellurometer. Now from all these known distances the antenna produces the two sets ofrange information. Tellurometer isuseful for distances up to 100km from theshore.

**The specific need for sounding are**

1. Preparation of navigation charts that is an all-time information for future purpose also
2. Material that to be dredged has to be determined early to facilitate easy movement in project without any confusion
3. Material dredging should also accompany where filling has to be done. Material dumping is alsomeasured
4. Design of backwaters, sea wells require detailed information that is obtained fromsounding

**EQUIPMENT FOR SOUNDING**

The essential equipment used for undergoing sounding are

1. Shore signals andbuoys
2. SoundingEquipment
3. Instruments for measuringangles

**1. SHORE SIGNAL ANDBUOYS**

- These are required to mark the range lines. A line perpendicular to shore line obtained by line joining 2 or 3 signals in a straight line constitute the range line along which sounding has to be performed. Angular observations can also be made from sounding boats by this method. To make it visible from considerable distance in the sea it is made highlyconspicuous.
- A float made of light wood or air tight vessel which is weighted at bottom kept vertical by anchoring with guywires are called buoys. In order to accommodate a flag a hole is drilled. Under water deep, the range lines are marked by shore signals & thebuoys.

**2. SOUNDINGEQUIPMENT**

The individual units involved are explained one by one:

**A. SOUNDINGBOAT**

A flat bottom of low draft is used to carry out sounding operation. Large size boats with motor are used for sounding in sea. The soundings are taken through wells provided in the boat. A figure depicting sounding boat is shown in fig.2.

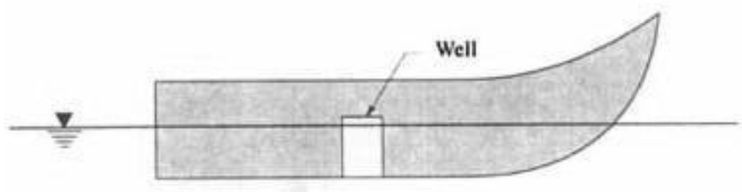


**Fig.2: Sounding Boat**

**B. SOUNDING POLE ORROD**

Rod made of seasoned timber 5 to 10cm diameter and 5 to 8m length. A lead shoe of sufficient weight is connected at bottom to keep it vertical. Graduations are marked from bottom upwards. Hence readings on the rod corresponding to water surface is water depth.

**C. LEAD LINE**

A graduated rope made of chain connected to the lead or sinker of 5 to 10kg, depending on current strength and water depth. Due to deep and swift flowing water variation will be there from true depth hence a correction is required.

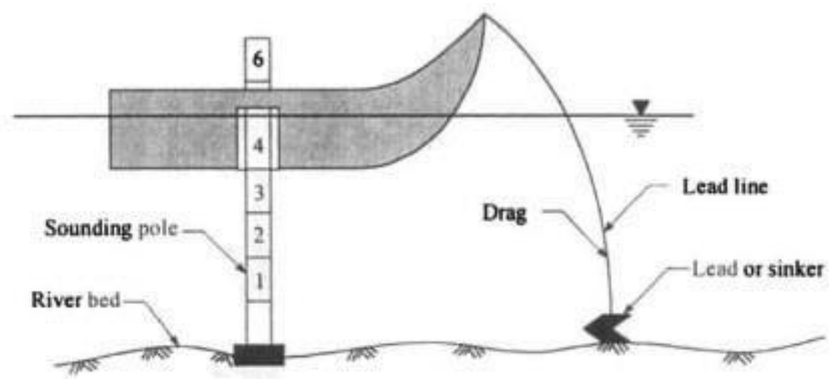



**Fig.3.Sounding Pole and Lead line**

- Other sounding equipment used are Weddell's sounding machine. These are employed when large sounding work has to be undergone. A standard machine to measure maximum of 30 to 40m is designed that are bolted over the well of the soundingboat.

- Another equipment used is fathometer which is an echo-sounding instrument used to determine ocean depth directly. Recording time of travel by sound waves is the principle employed. Here the time of travel from a point on the surface of the water to the bottom of the ocean and back isrecorded.
- Knowing the velocity of sound waves the depth can be calculated as shown infig.4.

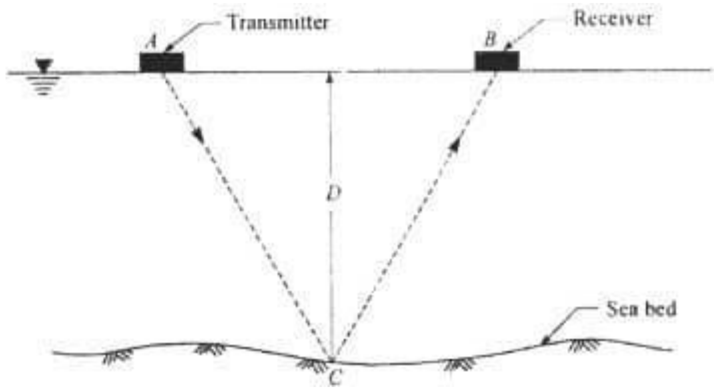



**Fig.4: Echo Sounding in Hydrographic Survey**

**TIDES**

- All celestial bodies exert a gravitational force on each other. These forces ofattraction between earth and other celestial bodies (mainly moon and sun) cause periodical variations in the level of a water surface, commonly known as tides.
- There are several theories about the tides but none adequately explains all the phenomenon of tides. However, the commonly used theory is after Newton, and is known as the equilibriumtheory.
- According to this theory, aforce of attraction exists between two celestial bodies, acting in the straight line joining the centre of masses of the two bodies, and the magnitude of this force is proportional to the product of the masses of the bodies and is inversely proportional to the square of the distance betweenthem.
- We shall apply this theory to the tides produced on earth due to the force of attraction between earth and moon. However, the following assumptions are made in the equilibrium theory:

1. The earth is covered all round by an ocean of uniformdepth.
2. The ocean is capable of assuming instantaneously the equilibrium , required by thetide producing forces. This is possible if weneglect
   - (i) Inertia OfWater,
   - (ii) Viscosity OfWater,
   - (Iii) Force Of Attraction Between Parts OfItself.

**TYPES OF TIDES:**

**i)LUNAR TIDE**

- Lunar Tide, also known as moon tide, is the tide caused in the sea due to the gravitational attraction caused by the moon. A tide is generally defined as the rise and fall in the level of the sea with respect to the land. ... The tides produced due to gravitational attraction caused by the sun are called solartides.

**ii)THE SOLAR TIDES**

- The phenomenon of production of tides due to force of attraction between earth and sun is similar to the lunartides.
- Thus, there will be superior solar tide and an inferior or anti-solar tide. However, sun is at a large distance from the earth and hence the tide producing force due to sun is muchless.
- Solar tide = 0.458 Lunartide.
- Combined effect : Spring and neaptides
- Solar tide = 0.458 Lunartide.

- Above equation shows that the solar tide force is less than half the lunar tide force. However, their combined effect is important, specially at the new moon when both the sun and moon have the same celestial longitude, they cross a meridian at the sameinstant.

**MEAN SEA LEVEL**

- For all important surveys, the datum selected is the mean sea level at a certainplace.
- The mean sea level may be defined as the mean level of the sea, obtained by taking the mean of all the height of the tide, as measured at hourly intervals over some stated period covering a whole number of complete tides.
- The mean sea level, defined above shows appreciable variations from day to day, from month to month and from year toyear.
- Hence the period for which observations should be taken depends upon the purpose for which levels are required.
- The daily changes in the level of sea may be more. The monthly changes are more orless periodic. The mean sea level in particular month may be low while it may be high in some other moths.
- Mean sea level may also show appreciable variations in its annual values. Dueto
- variations in the annual values and due to greater accuracy needed in moderngeodetic
- levelling, it is essential to base the mean sea level on observations extending over a period of about 19 years.
- During this period, the moon's nodes complete one entire revolution. The height of mean sea level so determined is referred to the datum of tide gauge at which the observations aretaken.
- The point or place at which these observations are taken is known as a tidal station. If the observations are taken on two stations, situated say at a distance of 200 to 500 kms on an open coast, one of the station is called primary tidal station while the other is called secondary tidalstation.
- Both the stations may then be connected by a line oflevel.

**ASTRONOMICAL SURVEYING**

- An astronomical survey is a general map or image of a region of the sky which lacks a specific observational target. Alternatively, an astronomical survey may comprise a set of many images or spectra of objects which share a common type orfeature.

**ASTRONOMICAL TERMS AND DEFINITIONS**

- To observe the positions / direction and movement of the celestial bodies, an imaginary sphere of infinite radius is conceptualized having its centre at the centre of the earth. The stars are studdedover the inner surface of the sphere and the earth is represented as a point at thecentre.

**CELESTIAL SPHERE :**

- An imaginary sphere of infinite radius with the earth at its centre and other celestial bodies studdedon its inside surface is known as celestialsphere.

**GREAT CIRCLE (G.C) :**

- Theimaginarylineofintersectionofan infiniteplane,passing throughthecentreoftheearthandthe circumference of the celestial sphere is known as greatcircle.

**ZENITH (Z) :**

- If a plumb line through an observer is extended upward, the imaginary point at which it appears to intersect the celestial sphere is known as Zenith. The imaginary point at which it appears to intersect downward in the celestial sphere is known as Nadir(N).

**VERTICAL CIRCLE :**

- Great circle passing through zenith and nadir is known as verticalcircle.

**HORIZON:**

Great circle perpendicular to the line joining the Zenith and Nadir is known as horizon.

**POLES :**

- If the axis of rotation of the earth is imagined to be extended infinitely in both directions, the points at which it meets the celestial sphere are known aspoles.
- The point of intersection in the northern hemisphere is known as north celestial pole and that in the southern hemisphere as south celestialpole.

**EQUATOR :**

- The line of intersection of an infinite plane passing through the centre of the earth and perpendicular to the line joining celestial poles with the celestialsphere.

**HOUR CIRCLE :**

- Great circle passing through celestial poles is known as hour circle, also known as declinationcircle.

**MERIDIAN :**

- The hour circle passing through observer's zenith and nadir is known as (observer's) meridian.It represents the North-South direction at observerstation.

**ALTITUDE (H) :**

- The altitude of a celestial body is the angular distance measured along a vertical circle passing through the body. It is considered positive if the angle measured is above horizon and below horizon, itis considered as negative.

**AZIMUTH (A) :**

- The azimuth of a celestial body is the angular distance measured along the horizon from the observer's meridian to the foot of the vertical circle passing through the celestial body.

**THREE POINT PROBLEM**

- In this method, three well defined points, having locations already being plotted on thedrawing are involved. These are used to find and subsequently plot the location of the plane tablestation.

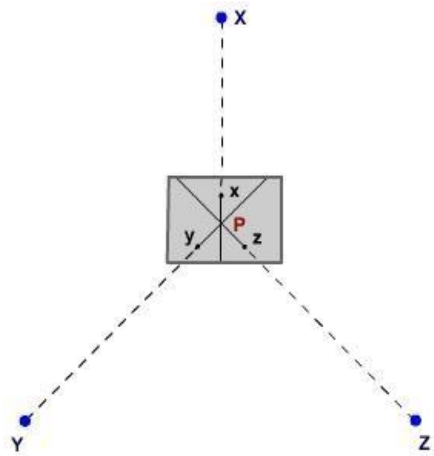



Figure 35.1 Principle of Three-point problem solution

- The method is based on the fact that, in a correctly oriented plane table, resectors through well defined points get intersected at a point which represents the location of the plane table station on thedrawing

There are several methods for solution of the three point problem:

(i) trial and Errormethod,
(ii) mechanical method,
iii)Graphical method,
(iv)Analytical methodand
(v) geometrical construction method.

Of these, the trial and error method is easy, quick and accurate. It is commonly used in practice and hence, has been discussed in detail.

- In three point problem, if the orientation of the plane table is not proper, the intersection of the resectors through the three points will not meet at a point but will form a triangle, known as triangle of error(Figure).
- The size of the triangle of error depends upon the amount of angular error in theorientation.

- The trial and error method of three point problem, also knon as Lehman's method minimises the triangle of error to a point iteratively. The iterative operation consist of drawing of resectors from known points through their plotted position and the adjustment of orientation of the planetable.
- Theestimationoflocationoftheplanetabledependsonitspositionrelativeto thewelldefined points considered for this purpose. Depending on their relative positions, three cases may arise:

(i) The position of plane table is inside the greattriangle;

(ii) The position of plane table is outside the greattriangle;

(iii) The position of plane table lies on or near the circumference of the greatcircle.

In case of (iii), the solution of the three-point problem becomes indeterminate or unstable. But for the cases (i) and (ii), Lehmann,s rules are used to estimate the location of plane table.

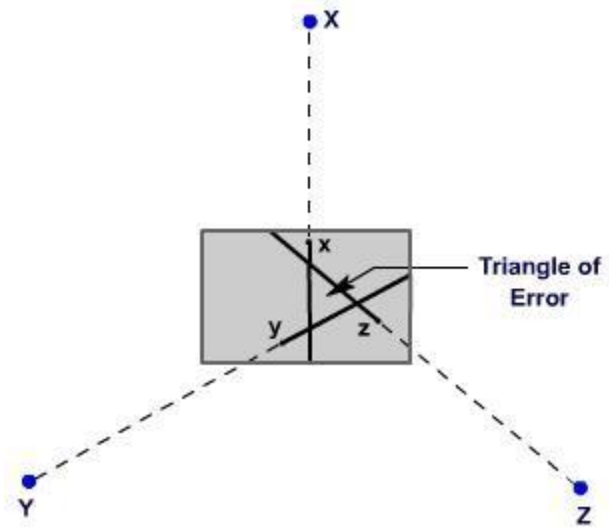



Figure 35.2 Triangle of Error

**STEPS FOR THREE POINT PROBLEM**

- LetX,Y,andZrepresentthegroundlocationofthewelldefinedobjectswhoseplottedpositionsare x, y, and z, respectively. Let P be the plane table station whose plotted position, say p, is to be determined.

(i) Select a plane table position inside the great triangle XYZ and set up the table over P and orient it by judgment so that apparent line xy is approximately parallel to the imaginary sideXY.

(ii) Pivoting the alidade on x, y, and z bisect the signals placed at X, Y, and Z in turn and draw rays. If the orientation of the table is correct, the three rays will meet at one point which is the desired location of p on the sheet. If not, the rays will form a triangle oferror

(iii) Choose a point p' inside the triangle of error such that its perpendicular distances from each ray is in proportion to the respective distances of P from the three ground objects. For selection of location of p', Lehmann's rules (1) and (3) need to beapplied.

(iv) Align the alidade along p' x (assuming X to be the farthest station) rotate the table till flag at X is bisected, and clamp thetable.

(v) Pivoting the alidade on x, y, and z repeat the process as in step (ii) above. If the estimation of p as p' is correct, the three rays will intersect at a point otherwise again a triangle of error will be formed but ofsmaller size and within the previous triangle of error. .

(vi) Estimate again the location of p' in the new triangle of error applying the rules, (i) and (iii), andrepeat the steps (iv) and(v).

(vii) The method is repeated till all the three rays intersect at a point. The point of intersection is the required location p of the plane-table stationP.

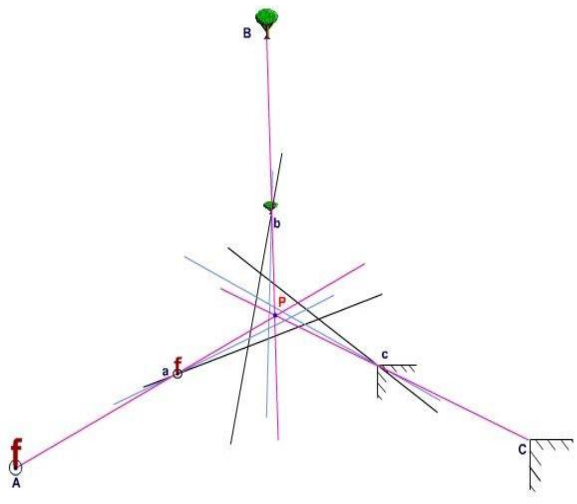



Figure 35.6 Solution of Three Point Problem

**STRENGTH OF FIX**

- The accuracy with which a plane table station can be located through three point problem is known as itsfix.
- The degree of accuracy of solution of the three point problem is designated as its strength i.e., ifthe accuracy is high, the fix is termed as strong and for low accuracy, fix is called aspoor.
- Theaccuracyoffixdependsontherelativepositionsoftheplotted pointsand that oflocationofthe plane table station. Thus, the choice of plotted objects and location of table should be made to get a strongfix.

The strength of fix is good if

- the location of station is chosen within the great triangle formed by joining the three well defined objectsX;
- the middle object is nearer to the position of the plane table than other twoobjects;
- of the two interior angles subtended by the three objects at the plane table stations, one is small andthe other is large. However, the objects subtending small angle should be widely separated to eachother.

The strength of fix is poor if

- The location of the plane table is on or near the circumference of the great circle.
- Both the interior angles, subtended, by well defined objects, at the plane table stations, aresmall.

Figure 35.7provides a pictorial representation of the quality of strength of figure with reference to the location of the three chosenobjects

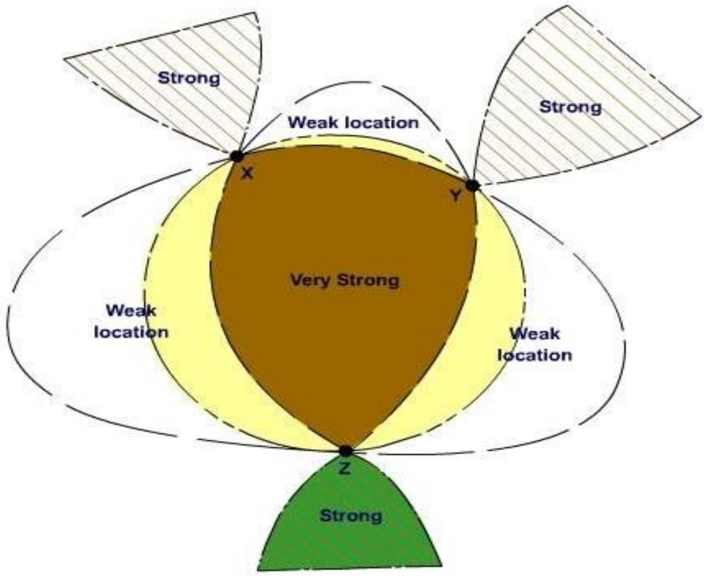



Figure 35.7 Qualitative presentation of strength of fix or figure

**CELESTIAL COORDINATE SYSTEM**

- In astronomy, a celestial coordinate system is a system for specifying positions of celestial objects:satellites,planets,stars,galaxies, and soon.
- Coordinate systemscan specify an object's position inthree-dimensional spaceorplotmerely its direction on acelestial sphere, if the object's distance is unknown ortrivial.
- The coordinate systems are implemented in either sphericalorrectangular coordinates. Spherical coordinates, projected on thecelestial sphere, are analogous to thegeographic coordinate systemused on the surface ofEarth.
- These differ in their choice of fundamental plane, which divides the celestial sphere into two equalhemispheresalong agreatcircle.
- Rectangular coordinates, in appropriate units, are simply the cartesian equivalent of thespherical coordinates, with the same fundamental (x, y) plane and primary (x-axis) direction. Each coordinate system is named after its choice of fundamental plane.

**AZIMUTH OF A LINE**

- Azimuth of a line is its horizontal angle measured clockwise from geographic or true meridian.
- For field observation, the most stable and retraceable reference is geographic north. Geographic north is based on the direction of gravity (vertical) and axis of rotation of theearth.
- A direction determined from celestial observations results in astronomic (Geographic) north reference meridian and is known as geographic or truemeridian.
- The azimuth of a line is determined from the azimuth of a celestial body. For this, the horizontal angle between the line and the line of sight to the celestial body is required to be observed during astronomic observation along with other celestialobservation.

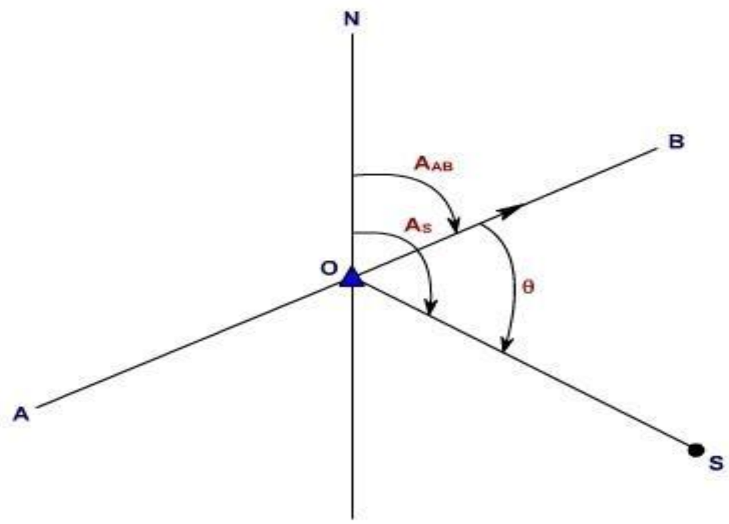



Figure 26.1 Relation between azimuths of a line with that of a celestial body

Let AB be the line whose azimuth ($A_{AB}$) is required to be determined (Figure 26.1). Let O be the station for celestial observations. Let S be the celestial body whose azimuth ($A_s$) is determined from the astronomical observation taken at O. The horizontal angle from the line AB to the line of sight to celestial body (at the station O) is observed to be q° clockwise. The azimuth of the line, AB can be computed from

$A_{AB} = A_S - q°$ (clockwise).

If $A_{AB}$ computes to be negative, 360° is added to normalize the azimuth.

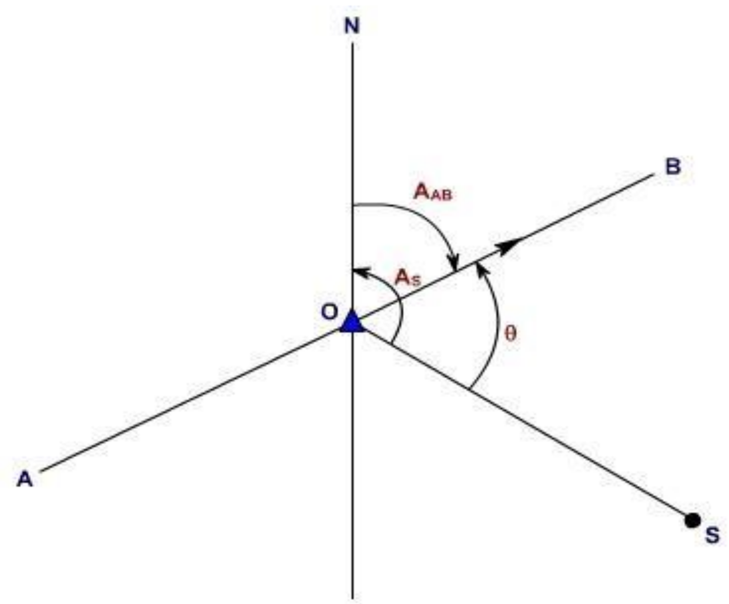



Figure 26.2 Determination of azimuth of a line from azimuth of a celestial body

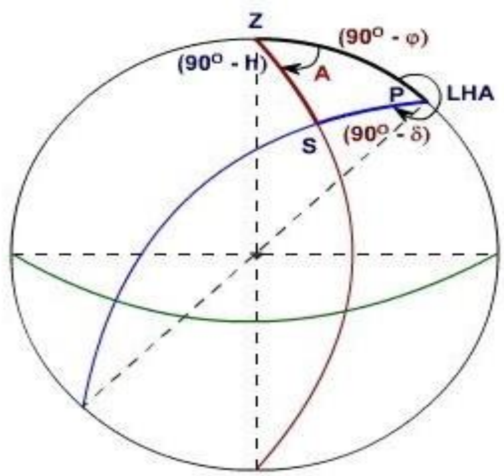
- In order to compute the azimuth of a line with proper sign, it is better to draw the known parameters. The diagram itself provides the azimuth of the line with proper sign. For example, inFigure 26.2, first a line of sight to celestial body, OS isdrawn.
- Then, the azimuth of the celestial body, $A_S$ is considered in counter-clockwise from the line OS and obtained the true north direction i.e, the line ON. Similarly, the horizontal angle q° is represented in counter clockwise (since the angle from the line to the celestial body is measured clockwise) direction from OS to obtain the relative position of the line. The angle NOB represents the azimuth of the line AB.

## DETERMINATION OF AZIMUTH OF A CELESTIAL BODY

- In field astronomy, a celestial body provides the reference direction. So, from the geographic location (latitude and longitude) of the station, ephemeris data of celestial body and either time or altitude of the same celestial body, the azimuth of the celestial body is computed by solving astronomical triangle.
- If time is used, the procedure is known as the hour-angle method. Likewise, if altitude is measured, the procedure is termed as the altitudemethod.
- The basic difference between these two methods is that the altitude method requires observation of approximate time and an accurate vertical angle of the celestial body, whereas the hour angle method requires observation of accuratetime.
- Recent developments of time receivers and accurate timepieces, particularly digital watches with split -time features, and time modules for calculators, the hour-angle method is more accurate, faster. It requires shorter training forproficiency.
- It has fewer restrictions on time of day and geographic location and thus is more versatile. The method is applicable to the sun, Polaris, and other stars. Consequently, the hour-angle method is emphasized, and its use by surveyors isencouraged.

## HOUR ANGLE METHOD

- In this method, precise time is being noted when the considered celestial body is being bisected. The observed time is used to derive the hour angle and declination of the celestial body at the instant of observation.
- The geographic position (latitude and longitude) of the observation station is required to be known a priori for the hour angle method. Usually, these values are readily obtained from available maps. However, to achieve better accuracy, latitude and longitude must be more accurately determined specially during observations for celestial bodies close to the equator--e.g., the sun-than for bodies near the pole--e.g.,Polaris.



- The declination of the celestial bodies at the instant of observation is required to be known for computation of azimuth of the celestial body. It is available in star almanac at the 0, 6, 12 and 18 hours of UTI of each day ( Greenwich date). Thus, the declination at the instant of observation (of celestial body) is determined by linear interpolation for corresponding the UT1 time of observation. However, since the declination of the sun varies rapidly, its interpolation is done using therelation:
- Declination, d = Decl $0^h$ + (Decl $24^h$ - Decl $0^h$) $\left(\frac{UTI}{24}\right)$ - (0.0000395) (Decl $0^h$) sin (7.5 UT1)-------------(Equation 26.2)
- The hour-angle of the celestial body is being derived using the GHA (available in star almanac with reference to Greenwich date) and the longitude of the observation station. For observations in the Western Hemisphere, if UTI is greater than local time, the Greenwich date is the same as local date and if UTI is less than local time, Greenwich date is the local date plus oneday.
- For the Eastern Hemisphere, if UTI is less than local time (24-hr basis), Greenwich date is the same as local date and if UTI is greater than local time, Greenwich date is local date minus one day. The hour angle of the celestial body at the observation station is theLHA.
- Thus, it is the LHA at UTI time of observation which is necessary to compute the azimuth of a celestial body. Hence, as can be seen fromFigure 26.3, the equation for the LHAis

LHA = GHA - W l(westlongitude) -------------------- (Equation26.3)

Or LHA = GHA + E l(eastlongitude) -------------------- (Equation26.4)

  - LHA should be normalized to between 0° and 360° by adding or subtracting 360°, ifnecessary.
- The Greenwich hour angle (GHA) of celestial bodies-the sun, Polaris, and selected stars-is tabulated in star almanac from 0 hr to 24 hr at an interval of 6 hours of UTI time of each day (Greenwich date). Thus, to find GHA at the time of observation linear interpolation is required to be performed.
- The GHA can also be derived by making use of the equation of time E (apparent time minus mean time) by using therelation:

GHA = 180° +15E ---------------------------- Equation(26.5)

- where E is in decimal hours. In those cases where E is listed as mean time minus apparent time, the algebraic sign of E should bereversed.
- Once the parameters (declination and Hour angle of the celestial body, latitude of the observation station) required to compute the azimuth of the celestial body are available, the computation of azimuth of the celestial body is carried out using the relations of astronomicaltriangle

# UNIT 5

# MODERN SURVEYING

## TOTAL STATION

- **Total station** is a surveyingequipment combination ofElectromagnetic Distance Measuring Instrumentand electronictheodolite.
- It is also integrated with microprocessor, electronic data collector and storage system. The instrument can be used to measure horizontal and vertical angles as well as sloping distance of object to the instrument.

## BASIC PRINCIPLE

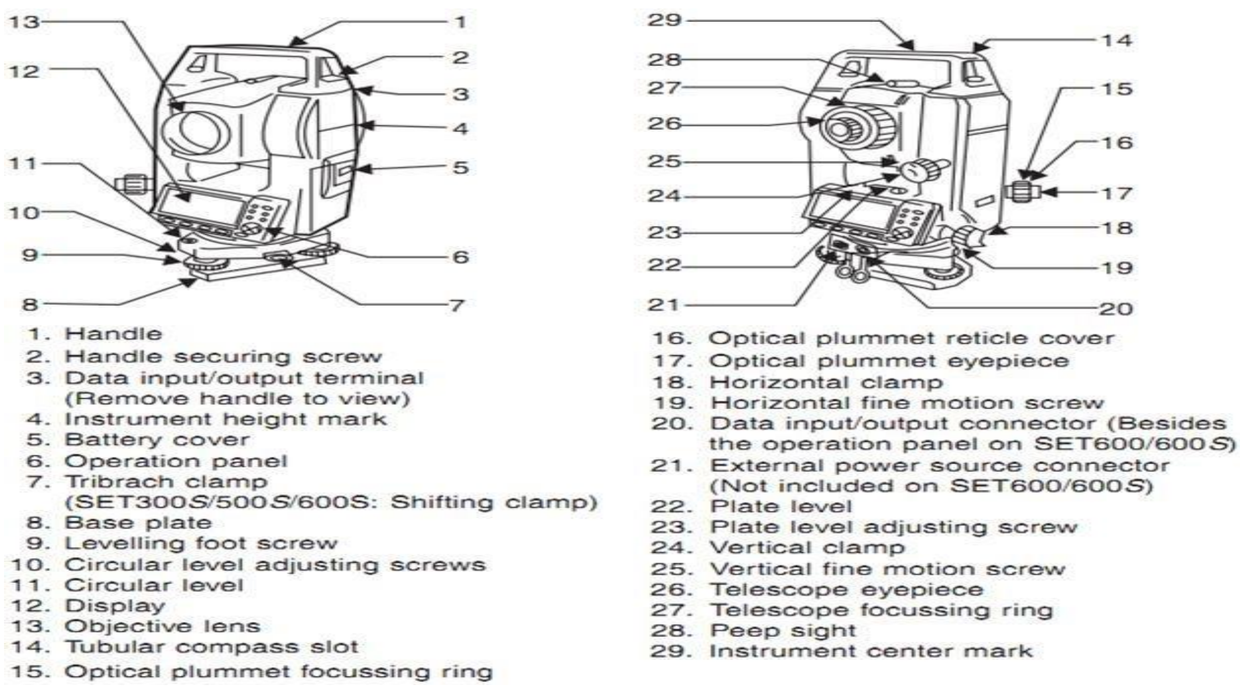
- Although taping and theodolites are used regularly on site – total stations are also used

- extensively in surveying, civil engineering and construction because they can measureboth
- distances andangles.

**CAPABILITY OF A TOTAL STATION**

Microprocessor unit in total station processes the data collected to compute:

i. Average of multiple anglesmeasured.
ii. Average of multiple distancemeasured.
iii. Horizontaldistance.
iv. Distance between any twopoints.
v. Elevation of objectsand
vi. All the three coordinates of the observedpoints.




IMPORTANT OPERATIONS OF TOTAL STATION

DISTANCE MEASUREMENT

- Electronicdistancemeasuring(EDM)instrumentisamajorpartoftotalstation.Itsrangevariesfrom 2.8 km to 4.2 km.
- The accuracy of measurement varies from 5 mm to 10 mm per km measurement. They are used with automatic target recognizer. The distance measured is always sloping distance from instrument to the object.

ANGLE MEASUREMENTS

- The electronic theodolite part of total station is used for measuring vertical and horizontal angle. For measurement of horizontal angles any convenient direction may be taken as referencedirection.
- For vertical angle measurement vertical upward (zenith) direction is taken as reference direction. The accuracy of angle measurement varies from 2 to 6seconds.

DATA PROCESSING

- This instrument is provided with an inbuilt microprocessor. The microprocessor averages multiple observations.
- Withthehelpofslopedistanceandverticalandhorizontalanglesmeasured,whenheightofaxisof instrument and targets are supplied, the microprocessor computes the horizontal distance and X, Y, Z coordinates.
- Theprocessoriscapableofapplyingtemperatureandpressurecorrectionstothemeasurements,if atmospheric temperature and pressures are supplied.

DISPLAY

- Electronic display unit is capable of displaying various values when respective keys arepressed.
- The system is capable of displaying horizontal distance, vertical distance, horizontal and vertical angles, difference in elevations of two observed points and all the three coordinates of the observed points.

ELECTRONIC BOOK

- Each point data canbe stored in an electronic note book (like compactdisc).
- The capacity of electronic note book varies from 2000 points to 4000 points data. Surveyor can unload the data stored in note book to computer and reuse the notebook.

**USES OF TOTAL STATION**

- The total station instrument is mounted on a tripod and is levelled by operating levelling screws. Within a small range instrument is capable of adjusting itself to the level position. Then vertical and horizontal reference directions are indexed using onboardkeys.
- It is possible to set required units for distance, temperature and pressure (FPS or SI). Surveyor can select measurement mode like fine, coarse, single orrepeated.
- When target is sighted, horizontal and vertical angles as well as sloping distances are measured and by pressing appropriate keys they are recorded along with point number. Heights of instrument and targets can be keyed in after measuring them with tapes. Then processor computes various information about the point and displays onscreen.
- This information is also stored in the electronic notebook. At the end of the day or wheneverelectronic note book is full, the information stored is downloaded to computers.
- The point data downloaded to the computer can be used for further processing. There are software like auto civil and auto plotter clubbed with AutoCad which can be used for plotting contours at any specified interval and for plotting cross-section along any specifiedline.

**ADVANTAGES OF USING TOTAL STATIONS**

The following are some of the major advantages of using total station over the conventional surveying instruments:

Field work is carried out veryfast.

Accuracy of measurement ishigh.

Manual errors involved in reading and recording are eliminated.

Calculation of coordinates is very fast and accurate. Even corrections for temperature and pressure are automatically made.

Computers can be employed for map making and plotting contour and cross-sections. Contour intervals and scales can be changed in no time.

However, surveyor should check the working condition of the instruments before using. For this standard points may be located near survey office and before taking out instrument for field work, its working is checked by observing those standard points from the specified instrument station.

**OPERATIONS INVOLVED WHILE USING TOTAL STATIONS :**

1. Establishing the siteDatum:

   a) Selecting the siteDatum

   b) EstablishingNorth

2. Setting up the Totalstation:

   a) Placing and leveling Tripod onDatum

   b) Placing and leveling the Gun onTripod

   c) Linking the data connector toGun

3. Data collector options andsetting

   a) Mainmenu

   b) Basic settings

4. Creating and Operating Jobfiles:

   a) Creating a new Jobfile

   b) Opening an existingfile

5. Shootingpoints

   a) Identifying the important points toshoot

   b) shootingpoints

   c) Shooting additionalpoints

   d) Noting the specialfeatures

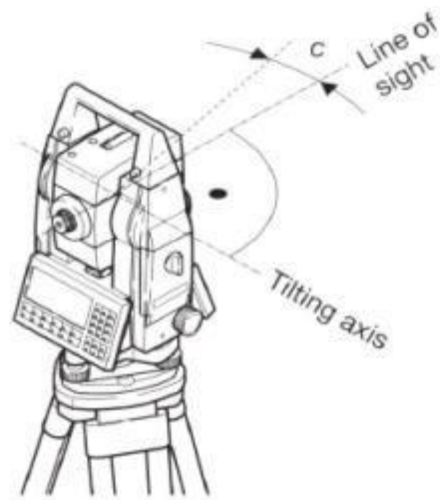
6. Post Processing – Data downloading,conversion

7. Plotting/Mapgeneration.

**TOTAL STATION ERRORS**

**1. HORIZONTAL COLLIMATION OR LINE OF SIGHTERROR**

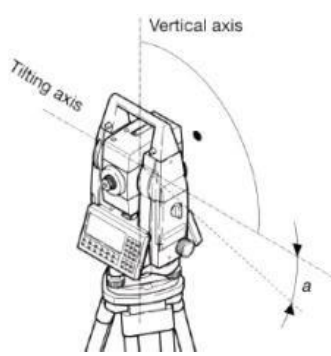
- Horizontal collimation or line of sight error is when the line of sight is not perpendicular to the tilting axis of the instrument. This is an axialerror.
- Line of sight error effects the horizontal angle readings and increases with steep sightings. The error can be overcome or eliminated by observing on twofaces.
- For single face measurements, an on-board calibration function is used to determine the deviation (c)of actual line of sight and deviated line of sight. The on-board software then apply a correction for each measured horizontal angles readingautomatically.




- The catch is here if the deviation of line of sight from actual line of sight exceeds more than a desired value, the instrument must be send to service centre or manufacturer for manualcalibration.

**2. TILTING AXIS ERROR OR TILTERROR**

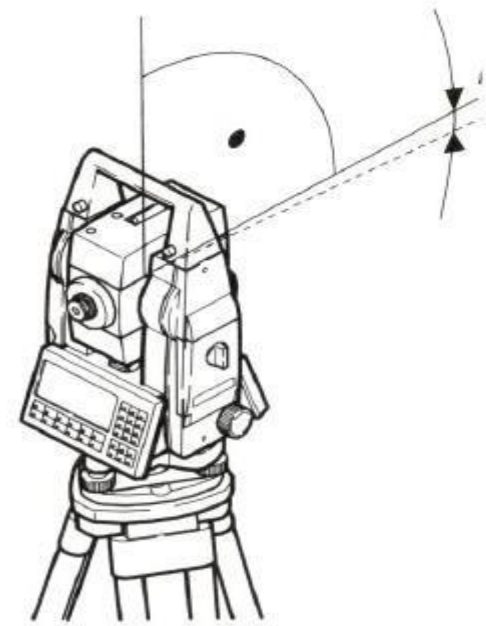
- Tilting axis or tilt error is the error when the axis to the total station is not perpendicular to the vertical axis or plumb line. The error effect on horizontal readings when the instrument is tilted (steep sightings) but have no effect on sightings taken when the instrument ishorizontal.
- Like horizontal collimation error the tilting error can be eliminated by two face measurement. Another method is to apply the measured tilting error at the time of calibration process for allreadings.




- If the tilt error is more than the specified error for instrument, must be send to calibrationlab.

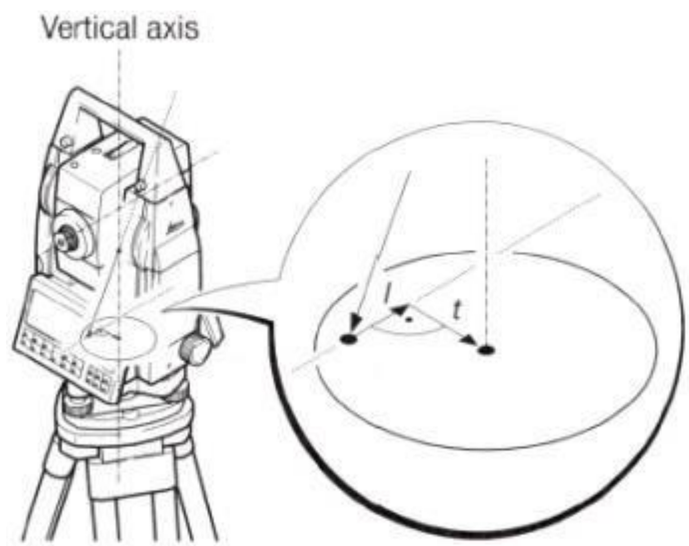
**3. VERTICAL COLLIMATION ERROR OR VERTICAL INDEXERROR**

- If the horizontal base line of angle from 0° to 180° in the vertical circle does not coincide withthe vertical axis of instrument. This zero point error is present in all vertical circle readings and like the horizontal collimation error, it is eliminated by taking FL and FR readings or by determining i.



**4. COMPENSATOR INDEXERROR**

- This error is caused by not leveling the total station correctly and carefully. This error can't be eliminated by taking two face (face left and face right) readings unlike the horizontal collimation error.
- If the instrument is fitted with a compensator it will measure residual tilts of the instrument and will apply corrections to the horizontal and vertical angles for these.




- However all compensators will have a longitudinal error l and traverse error t known as zero point errors. These are averaged using face left and face right readings but for single face readings must be determined by the calibration function of the totalstation.

**SEGMENTS OF GPS**

- For better understanding of GPS, we normally consider three major segmentsviz.

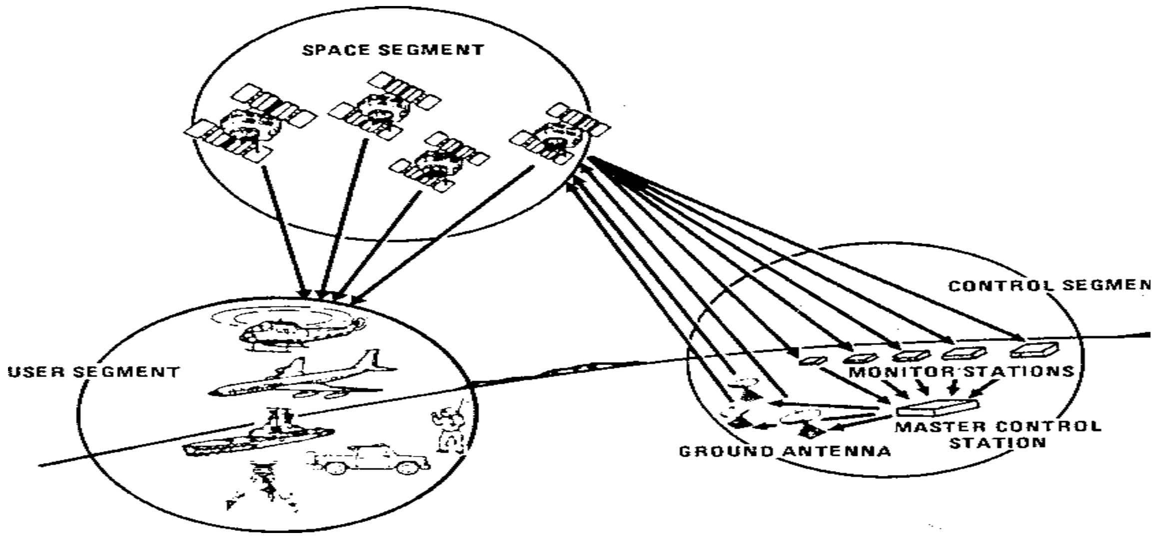
space segment, Control segment and User segment. Space segment deals with GPS satellites systems, Control segment describes ground based time and orbit control prediction and in User segment various types of existing GPS receiver and its application is dealt .

- Table 2 gives a brief account of the function and of various segments along with input andoutput information.

Table 2. Functions of various segments of GPS

| SEGMENT | INPUT | FUNCTION | OUTPUT |
|---|---|---|---|
| Output | Navigation message | Generate and Transmit code | P-Code<br>C/A Code<br>L1,L2 |
| Control | P-Code<br>Observations Time | Produce GPS time predict ephemeris | Navigation message |
| User | Code observation<br>Carrier phase observation | Navigation solution<br>Surveying | Position velocity time |

- GLONASS (Global Navigation & Surveying System) a similar system to GPS is being developed by former Soviet Union and it is considered to be a valuable complementary system to GPS for futureApplication.




**SPACE SEGMENT**

- Space segment will consist 21 GPS satellites with an addition of 3 active spares.These satellites are placed in almost six circular orbits with an inclination of 55degree.
- Orbital height of these satellites is about 20,200 kmcorresponding to about 26,600 km from the semi majoraxis.
- Orbital period is exactly 12 hours of sidereal time and this provides repeated satellite configuration every day advanced by four minutes with respect to universaltime.

**SATELLITE CONFIGURATIONS**

- The satellite configuration specifies the GlobalProtect LSVPN configuration settings to deploy tothe connecting satellites. You must define at least one satelliteconfiguration.

**OBSERVATION PRINCIPLE AND SIGNAL STRUCTURE**

- NAVSTAR GPS is a one-way ranging system i.e. signals are only transmitted by the satellite. Signal travel time between the satellite and the receiver is observed and the range distance is calculated through the knowledge of signal propagationvelocity.
- One way ranging means that a clock reading at the transmitted antenna is compared with a clock reading at the receiver antenna. But since the two clocks are not strictly synchronized, theobserved

signal travel time is biased with systematic synchronization error.

- Biased ranges are known as pseudoranges. Simultaneous observations of four pseudorangesare necessary to determine X, Y, Z coordinates of user antenna and clockbias.
- Real time positioning through GPS signals is possible by modulating carrier frequencywith Pseudorandom Noise (PRN)codes.
- These are sequence of binary values (zeros and ones or +1 and -1) having random character but identifiabledistinctly. ThuspseudorangesarederivedfromtraveltimeofanidentifiedPRNsignal code.
- Two different codes viz. P-code and C/A code are in use. P means precision or protected and C/A means clear/acquisition or coarseacquisition.

| | |
|---|---|
| Atomic Clock (G, Rb) fundamental frequency | 10.23. MHz |
| L1 Carrier Signal | 154 X 10.23 MHz |
| L1 Frequency | 1575.42 MHz |
| L1 Wave length | 19.05 Cm |
| L2 Carrier Signal | 120 X 10.23 MHz |
| L2 Frequency | 1227.60 MHz |
| L2 Wave Length | 24.45 Cm |
| P-Code Frequency (Chipping Rate) | 10.23 MHz (Mbps) |
| P-CodeWavelength | 29.31 M |
| P-Code Period | 267 days : 7 |
| C/A-Code Frequency (Chipping Rate) | 1.023/MHz(Mbps) |
| C/A-Code Wavelength | 293.1 M |
| C/A-Code Cycle Length | 1Milisecond |
| Data Signal Frequency | 50 bps |
| Data Signal Cycle Length | 30 Seconds |

## STRUCTURE OF THE GPS NAVIGATION DATA

- Structure of GPS navigation data (message) is shown in F The user has to decode the data signal to get access to the navigationdata.
- For on line navigation purposes, the internal processor within the receiver does the decoding. Most of the manufacturers of GPS receiver provide decoding software for post processing purposes. With a bit rate of 50 bps and a cycle time of 30 seconds, the total information content of a navigationdata set is 1500bits.
- The complete data frame is subdivided into five subframes of sixsecond duration comprising 300 bits of information. Each subframe contains the data words of 30 bits each. Six of these are control bits. The first two words of each subframe are the Telemetry Work (TLM) and the C/A-P-Code Hand over Work (HOW). The TLM work contains a synchronization pattern, which facilitates the access to the navigation data. Since GPS is a military navigation system of US, a limited access to the total system accuracy is made available to the civilianusers.

## CONTROL SEGMENT

- Control segment is the vital link in GPS technology.Main functions of the control segment.
    - Monitoring and controlling the satellite systemcontinuously
    - Determine GPSsystemtime
    - Predict the satellite ephemeris and the behavior of each satelliteclock.
    - Update periodically the navigation message for each particularsatellite.

## USER SEGMENT

- Appropriate GPS receivers are required to receive signal from GPS satellites forthe purpose of navigation or positioning. Since, GPS is still in its development phase,many rapid advancements have completely eliminated bulky first generation user equipments and now miniature powerful models are frequently appearing in the market.

**ORBIT DETERMINATION:**

- Orbit Determination is the process to estimate the position and velocity (state vector) of a satellite at a specific epoch based on models of the forces acting on the satellite, integration of satellite orbital motion equations and measurements to thesatellites.
- Orbit Determination (OD) is generally divided into twocategories:
  i. preliminary orbitdetermination
  **ii.** precise orbit determination(POD**).**

**RELIMINARY ORBIT DETERMINATION**

- Preliminary Orbit Determination is a geometric method to estimate orbit elements from a minimal set of observations before the orbit is known from othersources.
- Traditionally, and still typically used, ground-based satellite observations of angles, distance or velocity measurements, which depend on the satellite's motion with respect to the centre of theEarth.

**PRECISE ORBIT DETERMINATION**

- Precise Orbit Determination is a dynamic, or combined geometric and dynamic method, a process completed with two distinct procedures: orbit integration and orbitimprovement.
- Orbit integration yields a nominal orbit trajectory, while orbit improvement estimates the epoch state with all the measurements collected over the data arc in a batch estimation process.

**ORBIT REPRESENTATION**

- Orbit Representation is a means of representing a satellite orbit as a continuous trajectory with discrete observation data at the time ofinterest.
- The simplest orbit representation is the "osculating Keplerian elements" method, which describes an orbit as anellipse.
- The most typical example is the satellite almanacs published by NASA for almost all spacecrafts in orbit. Figure 1.1 illustrates the concepts of the Keplerian elements with respect to the earth-centred inertial coordinatesystem.

ANTI-SPOOFING

- The function of anti-spoofing (AS) of the GPS system is designed for an anti potential spoofer (or jammer). A spoofer generates a signal that mimics the GPS signal and attempts to cause the receiverto track the wrongsignal.
- When the AS mode of operation is activated, the P code will be replaced with a secure Y code available only to authorised users, and the unauthorised receiver becomes a single L1 frequency receiver. AS had been tested frequently since 1 August 1992 and formally activated at 00:00 UT on 31 January 1994 and now is in continuous operation on all Block II and latersatellites.
- The broadcasted ionospheric model (in the navigation message) may be used to overcome the problem of absence of the dual-frequencies, which are originally implemented for eliminating the ionospheric effects. Of course, the method of using the ionospheric model cannot be as accurate as the method of using dual-frequencies data, and consequently the precision is degraded. Carrier phase smoothed C/A code may be used to replace the absence of the Pcode

**SELECTIVE AVAILABILITY**

- Selective availability (SA) is a degradation of the GPS signal with the objective to deny full position and velocity accuracy to unauthorised users by dithering the satellite clock and manipulating the ephemerides.

- In case SA is on, the fundament frequency of the satellite clock is dithered, so that the GPS measurements are affected. The broadcast ephemerides are manipulated so that the computed orbit will have slow variations. Several levels of SA effects are possible. The SA is enabled on Block IIand later satellites (Graas and Braasch1996).
- **The authorised users may recover the** un-degraded data and exploit the full system potential. For doing so they must possess a key that allows them to decrypt correction data transmitted in the navigation message (Georgiadou and Daucet1990).
- For high-precision users, IGS precise orbit and forecast orbit data may be used. Using known positions (or monitor stations), the range corrections can be computed. Differential GPS may also eliminate at least a part of the SA effects. SA has been switched off since May2000.

**HANDHELD GPS RECEIVERS**

- Handheld GPS receivers are used for absolute positioning or for relative positioning usingDGPS-services or WAAS/EGNOS signals. The positioning is realized using codepseudo-ranges.
- Moreover it is well known that some of the handheld receivers use phase-smoothed code for positioning. This means that the phase signal is available and may be used for Precise Differential GPS (PDGPS)positioning.

**GEODETIC RECEIVER**

- Geodetic GPS receivers have the advantage of providing additionally the P-codeobservations, with a noise level smaller than the noise on the C/Acode.
- The accuracy requirements of geodetic receivers are usually about 1-5 cm (or evenbetter).

## UNIT II

## THEODOLITE SURVEYING AND TACHEOMETRIC SURVEYING

**Horizontal and vertical angle measurements - Temporary and permanent adjustments - Heights and distances Tacheometer - Stadia Constants - Analytic Lens -Tangential and Stadia Tacheometry surveying - Contour – Contouring – Characteristics of contours – Methods of contouring – Tacheometric contouring - Contour gradient – Uses of contour plan and map**

### THEODOLITE

The measurement of horizontal and vertical angles and it is the most precise instrument designed for points on line, prolonging survey lines, establishing grades, determining difference in elevation, setting out curves etc.

Theodolite may be classified as

Transit theodolite.   Non-transit theodolite.

**Transit theodolite**

It is the one in which the line of sight can be reversed by revolving the telescope through 180º in the vertical plane.

**Non-transit theodolite**

It may be either plain theodolite or Y-theodolite in which the telescope cannot be transited.

### THE ESSENTIALS OF THE TRANSIT THEODOLITE

The telescope is an integral part of the theodolite and is mounted on a spindle known as horizontal axis or Trunnion axis

The telescope may be internal focusing type or external focusing type.

The Vertical Circle

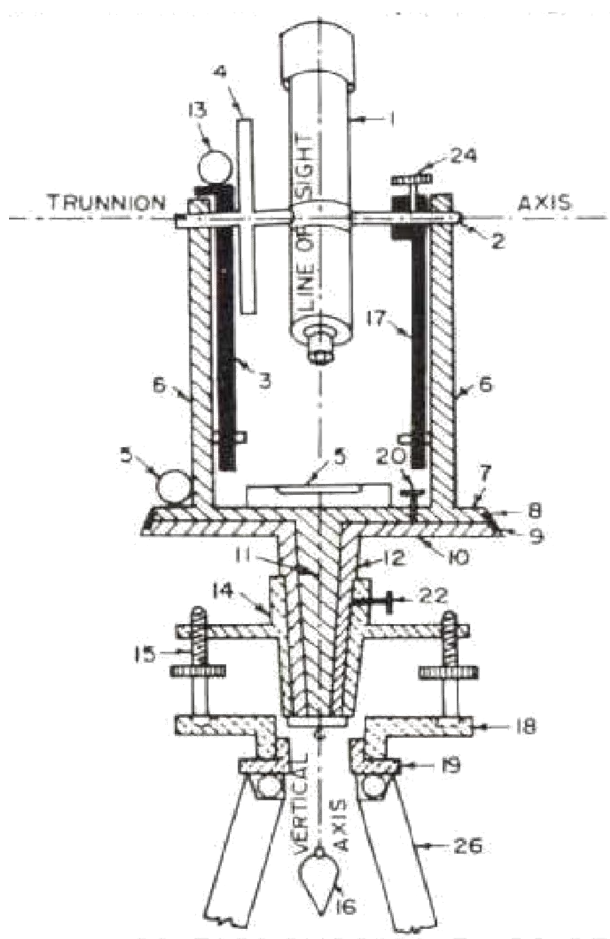
The vertical circle is a circular graduated arc attached to the Trunnion axis of thete1escope consequently the graduated arc rotates with the telescope when the latter is turned about the horizontal axis.

By means of vertical circle clamp and its corresponding slow motion or tangent screw the telescope can be set accurately at any desired position in vertical plane.

The Index Frame (or T-Frame or Vernier Frame)

The index frame is a -1shaped frame consisting of a vertical leg known as clipping arm and a horizontal bar known as vernier arm or index arm.

At the two extremities of the index arm are fitted two verniers to read the vertical circle.




The index arm is centered on the Trunnion axis in front of the vertical circle and remains fixed

When the telescope is moved in the vertical plane, the vertical circle moves relative to the verniers with the help of which reading can be taken.

For adjustment purposes, however, the index arm can be rotated slightly with the help of a clip screw fitted to the clipping arm at its lower end.

Glass magnifiers are placed in front of each vernier to magnify the reading. A long sensitive bubble tube, sometimes known as the altitude bubble is placed on the top of the index frame.

**The Standards (or A-Frame)**

Two standards resembling the letter A are mounted on the upper plates The Trunnion axis of the telescope is supported on these.

The –T frame and the arm of vertical circle clamp are also attached to the A frame.

The Leveling Head,

The leveling head usually consists of two parallel triangular plates known as tribrach plates.

The upper tribrach has three arms each carrying a levelling screw The lower tribrach plate or foot plate has a circular hole through which a plumb bob may be suspended.

In some instruments, for levelling screws are provided between two parallel plates.

A levelling head has three distinctive functions:

(a) To support the main part of the instrument.

(b) To attach the theodolite to the tripod.

(c) To provide a mean for levelling the theodolite.

**The Two Spindles**

The inner spindle or axis is solid and conical and fits into the outer spindle which is hollow and ground conical in the interior.

The inner spindle is also called the upper axis since it carries the vernier or upper plate

The outer spindle carries the scale or lower plate and is, therefore, also, known as the lower axis.

Both the axes have a common axis which forms the vertical axis of the instrument.

The Lower Plate (or Scale Plate)

The lower plate is attached to the outer spindle.

The lower plate carries a horizontal circle at its leveled edge and is, therefore, also known as the scale plate.

The lower plate carries a lower clamp screw and a corresponding slow motion or tangent screw with the help of which it can be fixed accurately in any desired position.

When the clamp is tightened, the lower plate is fixed to the upper tribrach of the levelling head.

On turning the tangent screw, the lower plate can be rotated slightly.

The Upper Plate (or Vernier Plate)

The upper plate or vernier plate is attached to the inner axis and carries two verniers with magnifiers at two extremities diametrically opposite.

The upper plate supports the standards it carries an upper clamp screw and a corresponding tangent screw for purpose of accurately fixing it to the lower plate.

On clamping the upper and unclamping the lower clamp, the instrument can rotate on its outer axis without any relative motion between the two plates.

If, however, the lower clamp is clamped and upper clamp unclamped, the upper plate and the instrument can rotate on the inner axis with a relative motion between the vernier and the scale.

For using any tangent screw, its corresponding clamp screw must be tightened.

The Plate Levels

The upper plate carries two plate levels placed at right angles to each other.

One of the plate levels is kept parallel to the Trunnion axis. In some theodolite only one plate level is provided. The plate level can be centered with the help of foot screws

**Tripod**

When in use, the theodolite is supported on a tripod which consists of three solid or framed legs.

At the lower ends, the legs are provided with pointed steel shoes. The tripod head carries at its upper surface an external screw to which the foot plate of the leveling head can be screwed.

**The Plumb bob**

A plumb bob is suspended from the hook fitted to the bottom of the inner axis to centre the instrument exactly over the station mark.

**The Compass**

Theodolite is provided with a compass which can be either tubular type or trough type. A trough compass consists of a long narrow rectangular box along the longitudinal axis of which is provided a needle balanced upon a steel pivot.

Small flat curve scales of only a few degrees are provided on each side of the trough.

**Striding Level**

Some theodolite is fitted with a striding level. It is used to test the horizontality of the transit axis or Trunnion axis.

**DEFINITIONS**

**The vertical axis**

The vertical axis is the axis about which the instrument can be rotated in a horizontal plane. This is the axis about which the lower and upper plates rotate.

**The horizontal axis.**

The horizontal or Trunnion axis is the axis about which the telescope and the vertical circle rotate in vertical plane.

**The line of sight or line of collimation**

It is the line passing through the intersection of the horizontal and vertical cross- hairs and the optical centre of the object glass and its continuation.

**The axis of level tube**

The axis of the level tube or the bubble line is a straight line tangential to the longitudinal curve of the level tube at its centre. The axis of the level-tube is horizontal when the bubble is central.

**Centering.**

The process of setting the theodolite exactly over the station mark is known as centering.

**Transiting.**

It is the process of turning the telescope in vertical plane through 1800 about the Trunnion axis. Since the line of sight is reversed in this operation, it is also known as plunging or reversing.

**Swinging the telescope.**

It is the process of turning the telescope in horizontal plane. If the telescope is rotated in clock-wise direction, it is known as right swing., If telescope is rotated in the anti-clockwise direction, it is know as the left swing.

Face left observation. If the face of the vertical circle is to the left of the observer, the observation of the angle (horizontal or vertical) is known as face left observation.

**Face Right Observation**

If the face of the vertical circle is to the right of the observer, the observation is known as face right observation.

**Telescope normal**

A telescope is said to be normal or direct when the face of the vertical circle is to the left and the ―bubble (of the telescope) up.

**Telescope inverted**

A telescope is said to inverted or reversed when of the vertical circle is to the right and the ―bubble down‖.

**Changing face**

It is an operation of bringing the face of the telescope from left to right and vice versa.

**TEMPORARY ADJUSTMENTS**

Temporary adjustments or station adjustments are those which are made at every instrument setting and preparatory to taking observation with the instrument.

The temporary adjustments are:

(1) Setting over the station.

(2) Leveling up

(3) Elimination parallax.

**SETTING UP**

The operation of setting up includes

**Centering**

Centering of the instrument over the station mark by a plumb bob or by optical plummet, and approximate leveling with the help of tripod legs.

Some instruments are provided with shifting head with the help of which accurate centering can be done easily.

By moving the leg radially, the plumb bob is shifted in the direction of the leg while by moving the leg circumferentially or sideways considerable change in the inclination is effected without disturbing the plumb bob. The second movement is, therefore, effective in the approximate levelling of the instrument.

The approximate levelling is done either with reference to a small circular bubble provided on tribrach or is done by eye judgment.

**Levelling up**

After having centered and approximately leveled the instrument, accurate levelling is done with the help of foot screws and with reference to the plate levels.

The purpose of the levelling is to make the vertical axis truly vertical.

The manner of levelling the instrument by the plate levels depends upon whether there are three levelling screws or four levelling screws.

**Three Screw Head.**

Turn the upper plate until the longitudinal axis of the plate level is roughly parallel to a line joining any two of the levelling screws Hold these two levelling screws between the thumb and first finger of each hand and turn them uniformly so that the thumbs move either towards each other or away from each other until the bubble is central.

It should be noted that the bubble will move in the direction of movement of the left thumb

Turn the upper plate through 90 degree, until the axis of the level passes over the position of the third levelling screw C

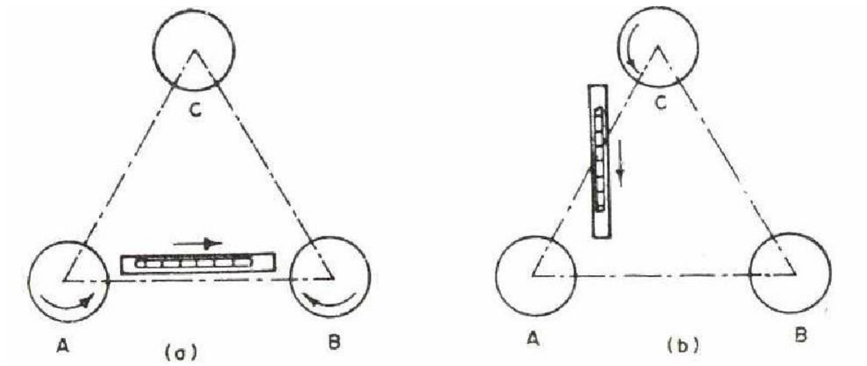
Turn this levelling screw until the bubble is central.

Return the upper plate through 90 degree to its original position and repeat step (2) till the bubble is central.

Turn back again through 90 degree and repeat step (4).

Repeat steps (2) and (4) till the bubble is central in both the positions. Now rotate the instrument through 180 degree.

The bubble should remain in the centre of its run, provided it is in correct adjustment.




**Four Screw Head**

Turn the upper plate until the longitudinal axis of the plate level is roughly parallel to the line joining two diagonally opposite screws (such as D and B)

Bring the bubble central exactly in the same manner as described in step (2) above.

Turn the upper plate through 90 degree until the spirit level axis is parallel to the other two diagonally opposite screws (such as A and C)

Centre the bubble as before.

Repeat the above steps till the bubble is central in both the positions. Turn through 180 degree to check the permanent adjustment, as for the three screw instrument.

**ELIMINATION OF PARALLAX**

Parallax is a condition arising when the image formed by the objective is not in the plane of the cross-hairs.

Unless parallax is eliminated, accurate sighting is impossible.

Parallax can be eliminated in two steps:

By Focusing the Eye-Piece

To focus the eye-piece for distinct vision of the cross-hairs, point the telescope towards the sky or hold a sheet of white paper in front of the objective and move eye-piece in or out till the cross-hairs are seen sharp and distinct.

In some telescopes, graduations are provided at the eye-piece end so that one can always remember the particular graduation position to suit his eyes..

By Focusing the objective.

The telescope is now directed to wards the object to be sighted and the focusing screw is turned till the image appears clear and sharp.

The image so formed is in the plane of cross-hairs.

**MEASUREMENT OF HORIZONTAL ANGLES**

To measure the horizontal angle PQR

Set up the instrument at Q and level it accurately.


Release all clamps. Turn the upper and lower plates in opposite directions till the zero of one of the vernier (say A) are against the zero of the scale and the vertical circle is to the left.

Clamp both the plates together by upper clamp and lower clamp and bring the two zeros into exact coincidence by turning the upper tangent screw.

Take both vernier readings.

The reading on vernier B will be 180 degree, if there is no instrumental error. Loose the lower clamp and turn the instrument towards the signal at F. Since both the plates are clamped together, the instrument will rotate about the outer axis.

Bisect point F accurately by using lower tangent screw. Check the readings of verniers A and B.

There should be no change in the previous reading.

Unclamp the upper clamp and rotate the instrument clockwise about the inner axis to bisect the point R.

Clamp the upper clamp and bisect R accurately by using upper tangent screw. Read both verniers. The reading of vernier A gives the angle PQR directly while the vernier B gives by deducting 180 degree.

While entering the reading, the full reading of vernier A (i.e., degrees, minutes and seconds) should be entered

While only minutes and seconds of the vernier B are entered.

The mean of the two such vernier readings gives angle with one face.

Change the face by transiting the telescope and repeat the whole process.

**REPETITION METHOD**

It is used to measure a horizontal angle to a finer degree of accuracy than that obtainable with the least count of the vernier.

An angle is measured two or more times by allowing the vernier to remain clamped each time at the end of each measurement instead of setting it back at zero when sighting at the previous station.

Thus an angle reading is mechanically added several times depending upon the number of repetitions.

The average horizontal angle is then obtained by dividing the final reading by the number of repetitions.

To measure the angle PQR

Set the instrument at Q and level it. With the help of upper clamp and tangent screw, set 00 reading on vernier A.

Note the reading of vernier B.

Loose the lower clamp and direct the telescope towards the point P. Clamp the lower clamp and bisect point P accurately by lower tangent screw.

Unclamp the upper clamp and turn the instrument clockwise about the inner axis towards R. Clamp the upper clamp and bisect R accurately with the upper tangent screw.

Note the reading of verniers A and B to get the approximate value of the angle PQR.

Unclamp the lower clamp and turn the telescope clockwise to sight P again.

Bisect P accurately by using the lower tangent screw.

Unclamp the upper clamp, turn the telescope clockwise and sight R. Bisect R accurately by upper tangent screw.

Repeat the process until the angle is repeated the required number of times. The average angle with face left will be equal to final reading divided by three.

Change face and make three more repetitions as described above.

Find the average angle with face right, by dividing the final reading by three. The average horizontal angle is then obtained by taking the average of the two angles obtained with face left and face right. „Sets" by Method of Repetition for High Precision

**First Method**

Keeping the telescope normal throughout, measure the angle clockwise by 6 repetitions. Obtain the first value of the angle by dividing the final reading by 6.

Invert the telescope and measure the angle counter-clockwise by 6 repetitions - Obtain the second value of the angle by dividing the final reading by 6.

Take the mean of the first and second values to get the average value of the angle by first set.

Take as many sets in this way as may be desired. For first order work, five or six sets are usually required.

The final value of the angle will be obtained by taking the mean of the values obtained by different sets.

**Second Method**

Measure the angle clockwise by six repetitions, the first three with the telescope normal and the last three with the telescope inverted. Find the first value of the angle by dividing the final by six.

Without altering the reading obtained in the sixth repetition, measure the explement of the angle clockwise by six repetitions, the first three with telescope inverted and the last three with telescope normal.

Take the reading which should theoretically by equal to zero (or the initial value). If not, note the error and distribute half the error to the first value of the angle. The result is the corrected value of the angle by the first set

Take as many sets as are desired and find the average angle.

For more accurate work, the initial reading at the beginning of each set may not be set to zero but to two different values.

The following errors are eliminated by method of repetition:

Errors due to eccentricity of verniers and centers are eliminated by taking both vernier readings.

Errors due to in adjustments of line of collimation and the Trunnion axis are eliminated by taking both face readings.

The error due to inaccurate graduations is eliminated by taking the readings at different pans of the circle.

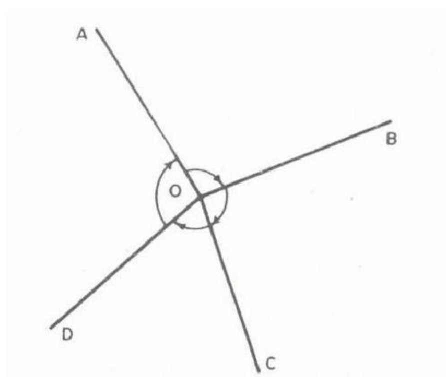
Errors due to inaccurate bisection of the object, eccentric centering etc., may be to some extent counter-balanced in different observations.

It should be noted, however, that in repeating angles, operations such as sighting and clamping are multiplied and hence opportunities for error are multiplied.

The limit of precision in, the measurement of an angle is ordinarily reached after the fifth or sixth repetition.

Errors due to slip, displacement of station signals, and want of verticality of the vertical axis etc., are not eliminated since they are all cumulative.

**DIRECTION METHOD (OR REITERATION METHOD)**



To measure the angles AOB, BOC, COD etc., by reiteration, proceed as follows

Set the instrument over 0 and level it. Set one vernier to zero and bisect point A (or any other reference object) accurately.

Loose the upper clamp and turn the telescope clockwise to point B. Bisect B accurately using the upper tangent screw. Read both the verniers. The mean of the vernier readings will give the angles AOB.

Similarly, bisect successively, C, D, etc., thus closing the circle. Read both the verniers at each bisection. Since the graduated circle remains in a fixed position throughout the entire process, each included angle is obtained by taking the difference between two consecutive readings.

On final sight to A, the reading of the vernier should be the same as the original setting. It not, note the reading and find the error due to slips etc., and if the error is small, distribute it equally to all angles. If large, repeat the procedure and take a fresh set of readings. Repeat steps 2 to 4 with the other face.

The procedure for each set is as follows

Set zero reading on one vernier and take a back sight on A. Measure clockwise the angles AOB, BOC, COD, DOA, etc., exactly in the same manner as explained above and close the horizon. Do not distribute the error.

Reverse the telescope, unclamp the lower clamp and back Sigh on A. Take reading and foresight on D, C, B and A, in counter Clockwise direction and measure angles AOD, DOC, COB and BOA..

Two values of each of the angles are obtained.

The mean of the two is taken as the average value of each of the uncorrected angles. The sum of all the average angles so found should be 360°.

In the case of discrepancy, the error (if small) may be distributed equally to all the angles.

The values so obtained are the corrected values for the first set. Several such sets may be taken by setting the initial angle on the vernier to different values.

**MEASUREMENT OF VERTICAL ANGLES**

Vertical angle is the angle which the inclined line of sight to an object makes with the horizontal.

It may be an angle of elevation or angle of depression depending upon whether the object is above or below the horizontal plane passing through the Trunnion axis of the instrument.

**The procedure is as follows**

Level the instrument with reference to the plate level.

Keep the altitude level parallel to any two foot screws and bring the bubble central.

Rotate the telescope through 90° till the altitude bubble is on the third screw.

Bring the bubble to the centre with the third food screw. Repeat the procedure till the bubble is central in both the positions.

Loose the vertical circle clamp and rotate the telescope in vertical plane to sight the object. Use vertical circle tangent screw for accurate bisection.

Read both verniers (i.e. C and D) of vertical circle.

The mean of the two gives the vertical circle. Similar observation may be made with another face. The average of the two will give the required angle.

## OPERATIONS WITH THEODOLITE

### TO MEASURE MAGNETICBEARING OF A LINE

To measure the magnetic bearing of a line, the theodolite should be provided with either a tubular compass or trough compass.

**Procedure**

Set the instrument at P and level it accurately. Set accurately the vernier A to zero. Loose the lower clamp.

Release the needle of the compass. Rotate the instrument about its outer axis till the magnetic needle roughly points to north. Clamp the lower clamp.

Using the lower tangent screw, bring the needle exactly against the mark so that it is in magnetic meridian.

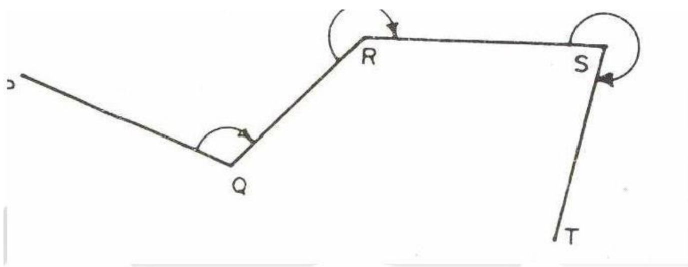
The line of sight will also be in the magnetic meridian that upper clamp and point the telescope towards Q. Bisect Q accurately using the upper tangent screw. Read verniers A and B.

Change the face and repeat steps 2, 3 and 4. The average of the two will give thecorrect bearing of the line PQ.

## TO MEASURE DIRECT ANGLES

Direct angles are the angles measured clockwise from the preceding line to the following (i.e. next) line.

To measure the angle PQR




Set the theodolite at Q and level it accurately. With face left, set the reading on vernier A to zero.

Unclamp the lower clamp and direct the telescope to P. Bisect it accurately using the lower tangent screw.

Unclamp the upper clamp and swing telescope clockwise and sight R. Bisect R accurately using the upper tangent screw. Read both verniers.

Plunge the telescope, unclamp the tower clamp and take back sight on P.Reading on the vernier will be the same as in step

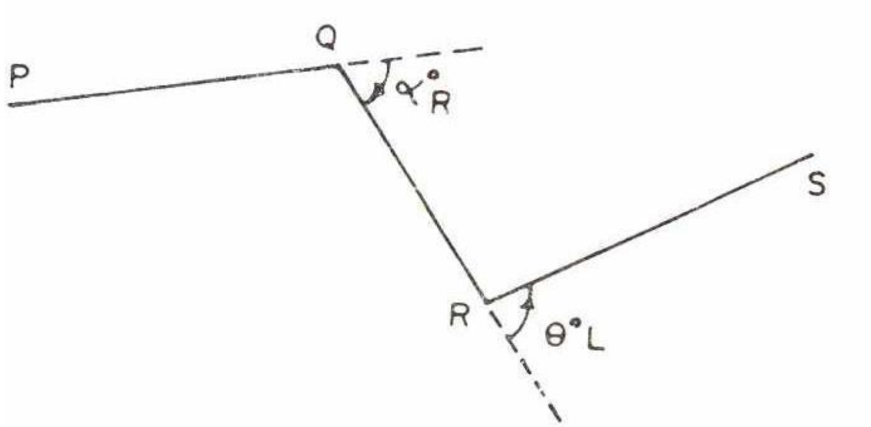
Unclamp the upper clamp and bisect R again. Read the verniers. The reading will be equal to twice the angle. LPQR will then be obtained by dividing the final reading by two.

## TO MEASURE DEFLECTION ANGLES

It is the angle which a survey line makes with the prolongation of the proceeding line. It is designated as Right (R) or Left (L) according as it is measured to the clockwise or to anti-clockwise from the prolongation of the previous line.

Set the instrument at Q and level it.

With both plates clamped at 0°, take back sight on P.




Plunge the telescope. Thus the line of sight is in the direction PQ produced when the reading on vernier A is 0°.

Unclamp the upper clamp and turn the telescope clockwise to take the foresight on R. Read both the verniers.

Unclamp the lower clamp and turn the telescope to sight P again. The vernier still read the same reading as in (4). Plunge the telescope.

Unclamp the upper clamp and turn the telescope to sight R. Read both verniers.

Since the deflection angle is doubled by taking both face readings, one-half of the final reading gives the deflection angle at Q.

## SOURCES OF ERROR IN THEODOLITE WORK

Instrumental,

Personal and

Natural.

## INSTRUMENTAL ERRORS

The instrumental errors are due to imperfect adjustment of the instrument, structural defects in the instrument, imperfections due to wear. Error due to imperfect adjustment of plate levels

If the upper and lower plates are not horizontal when the bubbles in the plate levels are centred, the vertical axis of the instrument will not be truly vertical

The horizontal angles will be measured in an inclined plane and not in horizontal plane. The vertical angles measured will also be incorrect.

The error may be serious in observing the points the difference in elevation of which is considerable.

The error can be eliminated only by careful leveling with respect to the altitude bubble if it is in adjustment. The errors cannot be eliminated by double sighting.

Error due to the vertical axis to horizontal axis not being perpendicular

If the horizontal axis is not perpendicular to the vertical axis, the line of sight will move in an inclined plane when the telescope is raised or lowered.

$$\tan e = \frac{P_1 P_2}{AP_1} = \frac{PP_1 \tan \beta}{AP_1} = \tan \alpha_1 \tan \beta$$

Thus, the horizontal and vertical angles measured will be incorrect.

Let P and Q be the two points to be observed, P and Q being their projection on a horizontal trace

Let the line of sight AP makes an angle 1 with horizontal.

When the telescope is lowered after sighting P, it will move in an inclined plane APP2 and not in the vertical plane APPI. The horizontal angle measured will now be with reference to AP2 and not with AP1 If is the instrumental error and e is the resulting error, we get Since e and will be usually small, we get

Error due to non-parallelism of the axis of telescope level and line of collimation

If the line of sight is not parallel to the axis of telescope level, the measured vertical angles will be incorrect since the zero line of the vertical verniers will not be a true line of reference.

It will also be a source of error when the transit is used as a level. The error can be eliminated by taking both face observations.

Error due to imperfect adjustment of the vertical circle vernier

If the vertical circle verniers do not read zero when the line of sight is horizontal, the vertical angles measured will be incorrect.

The error is known as the index error and can be eliminated either by applying index correction or by taking both face observations.

Error due to imperfect graduations

The error due to defective graduations in the measurement of an angle may be eliminated by taking the mean of the several readings distributed over different portions of the graduated circle.

Error due to eccentricity of verniers

The error is introduced when the zeros of the vernier are not at the ends of the same diameter.

Thus, the difference between the two vernier readings will not be 180°, but there will be a constant difference of other than 180.

The error can be eliminated by reading both the verniers and taking the mean of the two.

Manipulating wrong tangent screw

The error is introduced by using the upper tangent screw while taking the back sight or by using the lower tangent screw while taking a foresight.

The error due to the former can be easily detected by checking the vernier reading after the back sight point is sighted, but the error due to the latter cannot be detected.

It should always be remembered to use lower tangent screw while taking a back sighting and to use upper tangent screw while taking the foresight reading.

Errors in sighting and reading

Inaccurate Bisection Of Points Observed

The observed angles will be incorrect if the station mark is not bisected accurately due to some obstacles etc.

Care should be always be taken to intersect the lowest point of a ranging rod or a narrow placed at the station mark if the latter is not distinctly visible.

The error varies inversely as the length of the line of sight.

If the ranging rod put at the station mark is not held vertical, the error e is given By

$$\tan e = \frac{\text{Error in verticality}}{\text{Length of sight}}$$

**Parallax**

Due to parallax, accurate bisection is not possible. The error can be eliminated by focusing the eye-piece and objective.

Mistakes

Mistakes insetting the vernier, taking the reading and wrong booking of the readings.

**NATURAL ERRORS**

Sources of natural errors are

Unequal atmospheric refraction due to high temperature.

Unequal expansion of parts of telescope and circles due to temperature changes. Unequal settlement of tripod. ( Wind producing vibrations)

**PERMANENT ADJUSTMENTS OF THEODOLITE**

The permanent adjustments of a transit are as follows

Adjustment of plate level Adjustment of line of sight Adjustment of the horizontal axis

Adjustment of altitude bubble and vertical index frame.

**ADJUSTMENT OF PLATE LEVEL- Desired Relation**

The axis of the plate bubble should be perpendicular to the vertical axis when the bubble is central.

Object

The object of the adjustment is to make the vertical axis truly vertical; to ensure that, once the instrument is leveled up, the bubble will remain central for all directions of sighting

**Necessity**

Once the requirement is accomplished the horizontal circle and also the horizontal axis of the telescope will be truly horizontal, provided both of these are perpendicular to the vertical axis.

**Test**

Set the instrument on firm ground. Level the instrument in the two positions at right angles to each other 3 in temporary adjustment.

When the telescope is on the third foot screw, swing it through 180 degree. If the bubble remains central, adjustment is correct.

**Adjustment**

If not, level the instrument with to the altitude bubble till it remains central in two position right angles to each other.

Swing the telescope through 180 degree. If the bubble moves from its centre, bring it back halfway with the levelling screw with the clip screw.

Repeat till the altitude bubble remains central in all positions. The vertical axis is now truly vertical.

Centralize the plate levels(s) of the horizontal plate with capstan headed screw.

**ADJUSTMENT OF LINE OF SIGHT**

Desired Relation

The line of sight should coincide with the optical axis of the telescope.

Object

The object of the adjustment is to place the intersection of the cross-hair in the optical axis.

Thus, both horizontal as well as vertical hair is to be adjusted.

**Necessity**

Horizontal hair

The adjustment is of importance only in the case of external focusing telescope in which the direction of line of sight will change while focusing if the horizontal hair does not intersect the vertical hair in the same point in which the optical axis docs.

**Vertical hair**

If the adjustment is accomplished, the line of collimation will be perpendicular to the horizontal axis and hence the line of sight will sweep out a plane when the telescope is plunged. Test for horizontality and verticality of hairs,

To see this, level the instrument carefully, suspend a plumb to at some distance and sight it through the telescope by careful using.

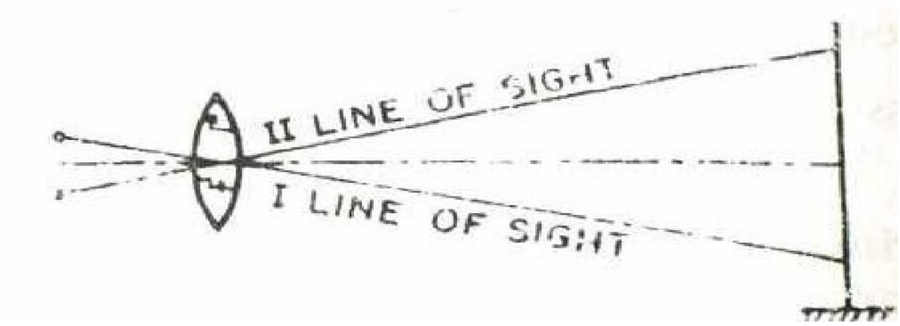
If the image of the plumb bob string is parallel to the vertical hair, the latter is vertical.

If not, loose the capstan screws the diaphragm and rotates it till the vertical hair coincides with the image of the string.

**Adjustment of Horizontal Hair**

Level the instrument carefully with all clamps fixed. Take a reading on a staff placed some distance apart

Unclamp the lower clamp, transit the telescope and set it through 180 degree. See the same reading on the vertical circle and see the staff. If the same reading is obtained, the horizontal hair is in adjustment.




**Adjustment**

If not, adjust the horizontal hair by top and bottom capstan screws of the diaphragm until the reading on the staff is the mean of the two.

Repeat the test till the adjustment is corrected

Adjustment of Vertical hair

Set the instrument on a level ground) so that a length of about 100 in is available to either side of it. Level it.

**TACHEOMETER**

Tacheometric is a branch of surveying in which horizontal and vertical distances are determined by taking angular observation with an instrument known as a tachometer. Tacheometric surveying is adopted in rough in rough and difficult terrain where direct leveling and chaining are either not possible or very tedious. The accuracy attained is such that under favorable conditions the error will not exceed 1/100. and if the purpose of a survey does not require accuracy, the method is unexcelled. Tacheometric survey also can be used for Railways, Roadways, and reservoirs etc. Though not very accurate. Tacheometric surveying is very rapid, and a reasonable contour map can be prepared for investigation works within a short time on the basis of such survey. Uses of Tachometry Tachometry is used for preparation of topographic map where both horizontal and vertical distances are required to be measured; survey work in difficult terrain where direct methods of measurements are inconvenient; reconnaissance survey for highways and railways etc; Establishment of secondary control points.

### INSTRUMENTS USED IN TACHOMETRIC SURVEYING

An ordinary transits theodolite fitted with a stadia diaphragm is generally used for tacheometric surveying. The stadia diaphragm essentially consists of one stadia hair above and the other an equal distance below the horizontal cross hair, the stadia hair being mounted in the same ring and in the same vertical plane as the horizontal and vertical cross-hair. The telescope used in stadia surveying are three kinds, The Simple external focusing telescope. The external focusing anal lactic telescope (porro's telescope). The internal focusing telescope.

### STADIA TACHOMETRIC

The first type is known as stadia theodolite, while the second type is known as tacheometer. The tacheometer has the advantage over the first and third type due to fact that the additive constant of the instrument is zero. The instruments employed in tachometry are the engineer's transit and the leveling rod or stadia rod, the theodolite and the subtense bar, the self-reducing theodolite and the leveling rod, the distance wedge and the horizontal distance rod, and the reduction tacheometer and the horizontal distance rod.

### Features of tacheometer or Characteristic of tacheometer

The multiple constant (f/i) should have a normal value of 100 and the error contained in this value should not exceed 1 in 1000. The axial horizontal lines should be exactly midway between the other two lines. The telescope should be fitted with an anallatic lens to make the additive constant (f + d) exactly to zero. The telescope should be truly analectic. The telescope should be powerful having a magnification of 20 to 30 diameters. The Aperture of the object should be 35 to 45 mm in diameter. Levelling and Stadia Staff Rod For short distances, ordinary leveling staves are used. The leveling staff normally 4m long, and it can be folded with here parts.

The graduations are so marked that a minimum reading of 0.005 or 0.001m can be taken. Different systems of Tacheometric Measurement

The various systems of tacheometric survey may be classified as follows,

### THE STADIA METHOD

i. Fixed Hair Method and ii. Movable Hair Method 2 The Tangential System Measurements by means of special instruments. The principle is common to all system is to calculate the horizontal distance between two points A and B their deference in elevation, by observing 1) the angle at the instrument at A subtended by known short distance a long a staff kept at B and 2) the vertical angle to B from A.

**Stadia systems**

In this systems staff intercepts, at a pair of stadia hairs present at diaphragm, are considered. The stadia system consists of two methods: a) Fixed-hair method and b) Movable-hair method.

## FIXED-HAIR METHOD

In this method, stadia hairs are kept at fixed interval and the staff interval or intercept (corresponding to the stadia hairs) on the leveling staff varies. Staff intercept depends upon the distance between the instrument station and the staff.

## MOVABLE- HAIR METHOD

In this method, the staff interval is kept constant by changing the distance between the stadia hairs. Targets on the staff are fixed at a known interval and the stadia hairs are adjusted to bisect the upper target at the upper hair and the lower target at the lower hair. Instruments used in this method are required to have provision for the measurement of the variable interval between the stadia hairs. As it is inconvenient to measure the stadia interval accurately, the movable hair method is rarely used.

## Tangential method

In this method, readings at two different points on a staff are taken against the horizontal cross hair and corresponding vertical angles are noted.

## Subtense bar method.

In this method, a bar of fixed length, called a subtense bar is placed in horizontal position. The angle subtended by two target points, corresponding to a fixed distance on the subtense bar, at the instrument station is measured. The horizontal distance between the subtense bar and the instrument is computed from the known distance between the targets and the measured horizontal angle.

## Principles of Stadia Method

A tacheometer is temporarily adjusted on the station P with horizontal line of sight. Let a and b be the lower and the upper stadia hairs of the instrument and their actual vertical separation be designated as i. Let f be the focal length of the objective lens of the tacheometer and c be horizontal distance between the optical centre of the objective lens and the vertical axis of the instrument. Let the objective lens is focused to a staff held vertically at Q, say at horizontal distance D from the instrument station.

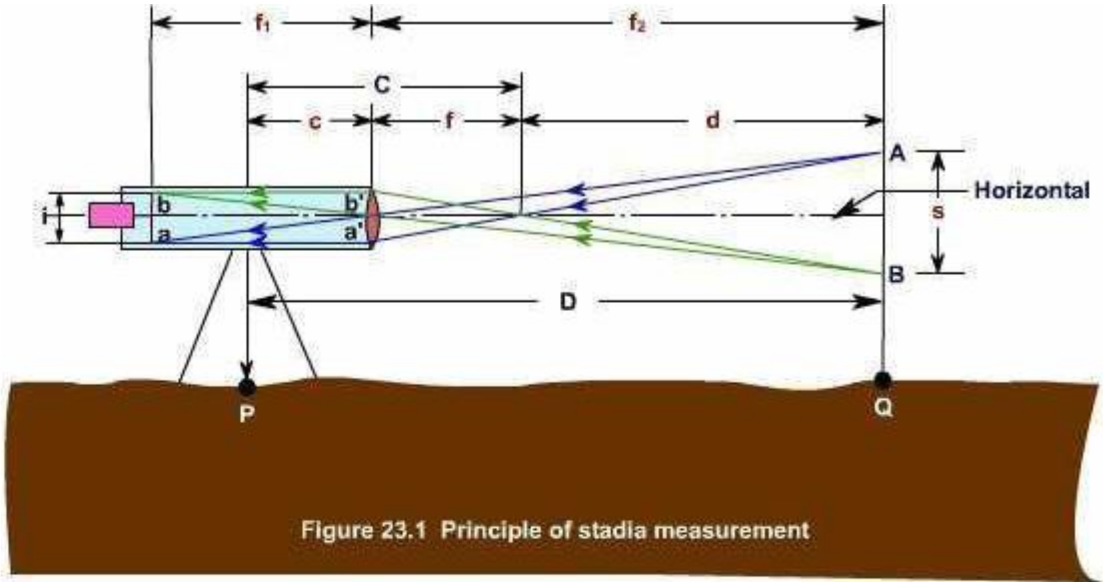
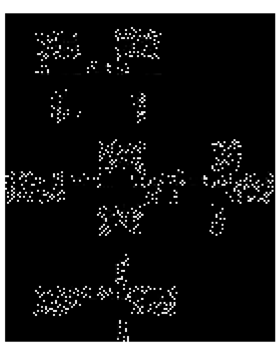



Figure 23.1 Principle of stadia measurement

By the laws of optics, the images of readings at A and B of the staff will appear along the stadia hairs at a and b respectively. Let the staff interval i.e., the difference between the readings at A and B be designated by s. Similar triangle between the object and image will form with vertex at the focus of the objective lens (F). Let the horizontal distance of the staff from F be d. Then, from the similar Ds ABF and a' b' F,



as a' b' = ab = i. The ratio (f / i) is a constant orf a particular instrument and is known as stadia interval factor, also instrument constant. It is denoted by K and thus d = K.s --------------------- Equation (1)

The horizontal distance (D) between the center of the instrument and the station point (Q) at which the staff is held is d + f + c. If C is substituted for (f + c), then the horizontal distance D from the center of the instrument to the staff is given by the equation

D = Ks + C ---------------------- Equation (2)

The distance C is called the stadia constant. Equation (23.2) is known as the stadia equation for a line of sight perpendicular to the staff intercept.

**ANALYTIC LENS:**

**THEORY OF ANALLATIC LENS:**

An additional convex lens, called an anallatic lens, is provided in the external focussing telescope between the eye — piece and the object — glass at a fixed distance from the later, to eliminate the additive constant, (f+d), from the distance formula:

The anallatic lens is seldom placed in the internal focussing telescope since the value of the additive constant is only a few centimeters and can be neglected. The disadvantage of the anallatic a lens is the reduction in brilliancy of the image due to increase observation of light.

The value of the additive constant, (δ+d) can be made equal to zero by bringing the apex (G) of the taceometric angle AGB (Fig. 10. 4) into exact coincidence with the centre on\f the insrument.

Let, S = the staff intercept AB.

i = the length b a of the image of AB i.e. the actual stadia interval when the anallatic lens is interposed.

i = the length ba of the image of AB when no anallatic lens was provided.

O = the optical centre of the object – glass.

O = the optical centre of the anallatic lens

e = the distance between the optical centre of the object glass and the anallatic lens.

f = to cel length of object glass.

f’ = focal length of the anallatic lens.

F = Principle focus of the anallatic lens.

G = the centre of the instrument.

d = the distance from the centre of the object — glass top the vertical axis of the instrument.

D – the distance from the vertical axis of the instrument to the staff.

$f_1$and $f_2$ = the conjugate focal length of the object —glass.
k = the distance from the optical centre of the object glass to the actual image b a.

(k— e) and ($f_2$ —e) = the conjugate focal length of the anallatic lens.
The ray of light from A and B are refracted by the object — glass to meet at F. The anallatic lens is so placed that F is its principal focus. Thus ray of light would become parallel to the axis of the telescope after passing through the anallatic lens and give actual image b a of the staff intercept AB.

The negative sign is used in (ii) since b 'a' and ba are on the same side of the anallatic lens.

Now the conditions that D should be proportional to S requires that the 2nd and 3$^{rd}$ terms in (v) are equal to zero so that
In this condition, the apex G of the tacheomeric angle AGB exactly coincides with the instruments

**TACHOMETRY SURVEY:-**

Tachometry is a branch of angular surveying in which A horizontal & vertical distance is of points are obtain by optical means as suppose to ordinary slow process of measure by tape chain. This method is very rapid & convenient. All though the accuracy of tachometry is low it is best adopted in obstructed such as steep & broken ground stretches of water etc which make drawn age difficult. They primary object of tachometry is the preparation of contour maps are plans required with both horizontal & vertical measurements also accuracy improvement it provides at check an distance measure with tape.

At the instruments a normally transit theodolite pitted with stadia diaphragm is generally used for tachometry survey. A stadia diaphragm essentially consists of one stadia hair above on the other an equal distance below the horizontal cross hair. Telescope is used in stadia surveying are of 3 types :-

(i) Simple external focusing telescope

(ii) External focusing analytic

(iii) Internal focusing telescope

Different system of Tachometry measurements:-

(i) Fixed hair method (or) stadia method

(ii) Movable hair method (or) substance

**Fixed hair method:-**

In method observation are made with stadia diaphragm having stadia wires at fixed (a) constant distance occur. They reading an the staff corresponding to all three wires are taken. The staff intercepts that is the different of reading corresponding to top & bottom stadia wires will depend on the distance of the staff from the instrument when the staff intercept is more than the length of the staff only ½ interne of real. For inclined said reading may be taken by keeping the staff either vertical a normal to the line of site.

**Sub tense method:-**

This method is similar to fixed hair method except the stadia internal is varying table arrangement is may to --- distance between the stadia hair so as to said them against the two targets on the staff kept at a point and observation this in this case the staff intercept that is the distance between the two forgets is kept fixed while the stadia interval that is the distance between the stadia hair is carrying as is the case of fixed hair method inclined site they out show be taken the tangential method. They stadia being taken against the horizontal hair as against any two point on the staff on their corresponding vertical angles are measured. This measurement of vertical angles tube for one single observation. Staff in theodolite normal mean by perpendicular. Least count of staff 0.005m.

**Held staff vertical:-**

Horizontal D = MS $\cos^2\theta + \cos\theta$

Vertical V = MS $\sin^2\theta / 2 + C \sin\theta$

H = height of instruct

R = observe staff reading

S = staff intercept

This is top – bottom hair radia

O = angle made by line of site with horizontal

Reduced level of θ = R.L of p + h-v-r

RL of p + h + v – r = RL of O

Staff normal to lined site:

Find the elevation & horizontal distance of point Q view from A is than angle of 30° above horizontal with staff intercept with staff held vertically occur 3.855 m & C.H reading 1.930 m elevation of point A is 10m above MSL take multiplying as 100 assume h to be 1 m

RL of A = 10.000 m

RL of Q = RL of A + height of instrument + v – r

V = MS $\sin^2\theta / 2 + c \sin\theta$

= 100 x 3.855 $\sin^2 30° / 2$

= 48.19m

reduce level of Q = 10.000 + 1.000 + 48.190 – 1.930

= 57.260m

Horizontal distance:

$D = MS\cos^2\theta + \cos\theta$

$= 100 \times 3.855 \cos^2 30° / 2$

Reduce level Q= 57.260m

2. Find vertical & horizontal distance between point A & B. If the instrument located 0.8m above A clip of 22° to a staff held normal to the line of site at B. The staff read 1.650, 2.150, and 2.650

Assume m = 100

RL of $\theta$ = RL of A + h – v- r $\cos\theta$

S = 1m

R = 2.150        r $\cos\theta$ = 1.993

$V = (ms + c)\sin\theta$

$= (100 \times 1)\sin 22° = 37.461$m

RL of $\theta$ = 0 + 0.8 – 37.461 – 1.993

= 38.654m

Horizontal distance D

$D = (ms + c)\cos\theta + r\sin\theta$

$= (100)\cos 22° + 0.805$

D = 93.51m

3. To determine distance between two points with base of one point is axiable and instrument station in the same vertical plane as the elevated object.

$$D = \frac{d\tan^2\theta}{\tan\theta 1 - \tan\theta 2}$$

$h = D \tan\theta 1$

Instrument with two different Axis :-

$$D = \frac{(d + s \cot \theta 2)\tan\theta 2}{\tan\theta 1 - \tan \theta 2}$$

$$h = \frac{(d + s \cot \theta 2)\sin\theta 1 \sin\theta 2}{\sin(\theta 1 - \theta 2)}$$

$h2 = h1 + s$

use positive sign with s cot θ2 when instrument a axis a Q is lower & negative sign when it's height the instrument axis at P.

1. An instrument was setup at station P and the angle of elevated to an objective was 9°30' the same object was focus from a point 4m away the first one angle was 11° 150' the staff reading s from a B.M having elevation 2650.38m are 1.310m and 1.815m respectively. Find the RL of Q =

$\theta 1 = 9°30'$ $\theta 2 = 11°15'$

$$D = \frac{(d - s \cot \theta 2)\tan\theta 2}{\tan\theta 1 - \tan \theta 2}$$

$$= \frac{(4 - 0.535\ (d\ 9°30')\tan 9°30'}{\tan 11°15' - \tan 9°30'} = \frac{0.164}{0.0315}$$

D = 5.21m = -204.13m

$$h2 = \frac{(4 - 0.505 \times 5.98)\sin 11°15' \quad \sin 9°30'}{\sin(11°15' - 9°30')}$$

= 39.68m

h2 = h1 + s = 39.68 + 0.505

= 39.175m

RL of stadia = R.L of $D^2 + S^2$

= 2650.38 + 1.310 – 39.175

= 2612.52m

2. An instrument was setup a P and the angle of elevation to a volume 4m above the focus of the staff held at Q was 9°30‘. A horizontal distance Pl was know 2000m determine the RL of staff station Q given RL of instrument axis was 2650.38m

$0.0673\ d^2$

$0.0673 \times 2000^2$

Ccr = 0.673 x 2

= 0.27m

V = D tanθ

= 2000 x tan 9°30‘ = 334.69m

RL of Q = RL of instrument axis + Ccr + V – h

= 2650.38 + 0.27 + 334.69 – 4

= 2981.34m

3. A instrument was setup a P and angle of depression to a plane 2m above the fast of the staff held at Q was 5° 36‘ H. d between P & Q was 3000m determine RL of staff station Q given the staff readings as a B.M of elevation 436.050m was 2.865m

Cn = $0.0673 \times 3^2$ = 0.6057

V = 3000 x tan 5°36‘ = 294.15m

RL of Q = BM + cn – V – n + instrument

= 436.050 + 2.865 – 0.6057 – 294.15

= 142.16m

Measurement of horizontal angles :-

(i) Direct method

(ii) Method of Repetition

(iii) Method of Reiteration

Precaution to be taken theodalite observation :-

(i) Turn the theodalite by the standards and not by using telescope ensuring slow & smooth movement.

(ii) Done force the foot screws & tangent screws to heart.

(iii) Clamp vertical axis tightly while observing the horizontal angles.

Sources of errors in theodalite :-

(i) Instrumental Error

(ii) Personal Error

(iii) National Error

Contouring in surveying is the determination of elevation of various points on the ground and fixing these points of same horizontal positions in the contour map.

To exercise vertical control leveling work is carried out and simultaneously to exercise horizontal control chain survey or compass survey or plane table survey is to be carried out.

If the theodolite is used, both horizontal and vertical controls can be achieved from the same instrument. Based on the instruments used one can classify the contouring in different groups.

**Characteristics of Contour Maps**
The contours maps have the following characteristics:

1. Contour lines must close, not necessarily in the limits of the plan.

2. Widely spaced contour indicates flat surface.

3. Closely spaced contour indicates steep ground.

4. Equally spaced contour indicates uniform slope.

5. Irregular contours indicate uneven surface.

6. Approximately concentric closed contours with decreasing values towards centre (Fig. 1) indicate a pond.

7. Approximately concentric closed contours with increasing values towards centre indicate hills.

8. Contour lines with U-shape with convexity towards lower ground indicate ridge (Fig. 2).

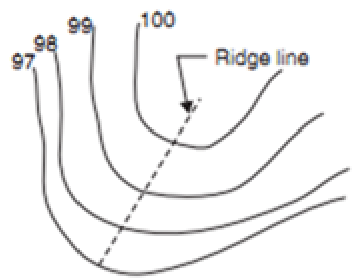



**Fig. 2**

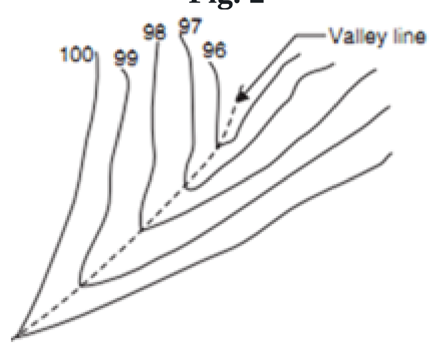



**Fig. 3**

9. Contour lines with V-shaped with convexity towards higher ground indicate valley (Fig.3).

10. Contour lines generally do not meet or intersect each other.

11. If contour lines are meeting in some portion, it shows existence of a vertical cliff (Fig. 4).

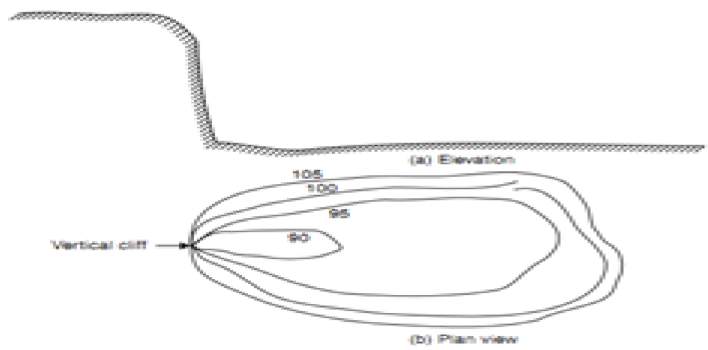



**Fig. 4**

12. If contour lines cross each other, it shows existence of overhanging cliffs or a cave (Fig. 5).

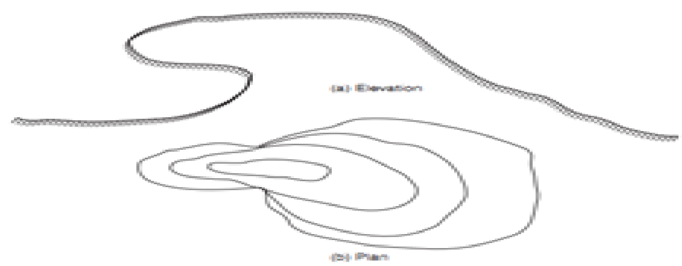



**Fig. 5**

**Uses of Contour Maps**

Contour maps are extremely useful for various engineering works:

1. A civil engineer studies the contours and finds out the nature of the ground to identify. Suitable site for the project works to be taken up.

2. By drawing the section in the plan, it is possible to find out profile of the ground along that line. It helps in finding out depth of cutting and filling, if formation level of road/railway is decided.

3. Intervisibility of any two points can be found by drawing profile of the ground along that line.

4. The routes of the railway, road, canal or sewer lines can be decided so as to minimize and balance earthworks.

5. Catchment area and hence quantity of water flow at any point of nalla or river can be found. This study is very important in locating bunds, dams and also to find out flood levels.

6. From the contours, it is possible to determine the capacity of a reservoir.

**Methods of Contour Surveying**

There are two methods of contour surveying:

1. Direct method
2. Indirect method

**Direct Method of Contouring**

It consists in finding vertical and horizontal controls of the points which lie on the selected contour line.

For vertical control levelling instrument is commonly used. A level is set on a commanding position in the area after taking fly levels from the nearby bench mark. The plane of collimation/height of instrument is found and the required staff reading for a contour line is calculated.

The instrument man asks staff man to move up and down in the area till the required staff reading is found. A surveyor establishes the horizontal control of that point using his instruments.

After that instrument man directs the staff man to another point where the same staff reading can be found. It is followed by establishing horizontal control.

Thus, several points are established on a contour line on one or two contour lines and suitably noted down. Plane table survey is ideally suited for this work.

After required points are established from the instrument setting, the instrument is shifted to another point to cover more area. The level and survey instrument need not be shifted at the same time. It is better if both are nearby to communicate easily.

For getting speed in levelling some times hand level and Abney levels are also used. This method is slow, tedious but accurate. It is suitable for small areas.

**Indirect Method of Contouring**

In this method, levels are taken at some selected points and their levels are reduced. Thus in this method horizontal control is established first and then the levels of those points found.

After locating the points on the plan, reduced levels are marked and contour lines are interpolated between the selected points.

For selecting points any of the following methods can be used:

1. Method of squares
2. Method of cross-section
3. Radial line method

**Method of Squares**

In this method area is divided into a number of squares and all grid points are marked (Ref. Fig. 1).

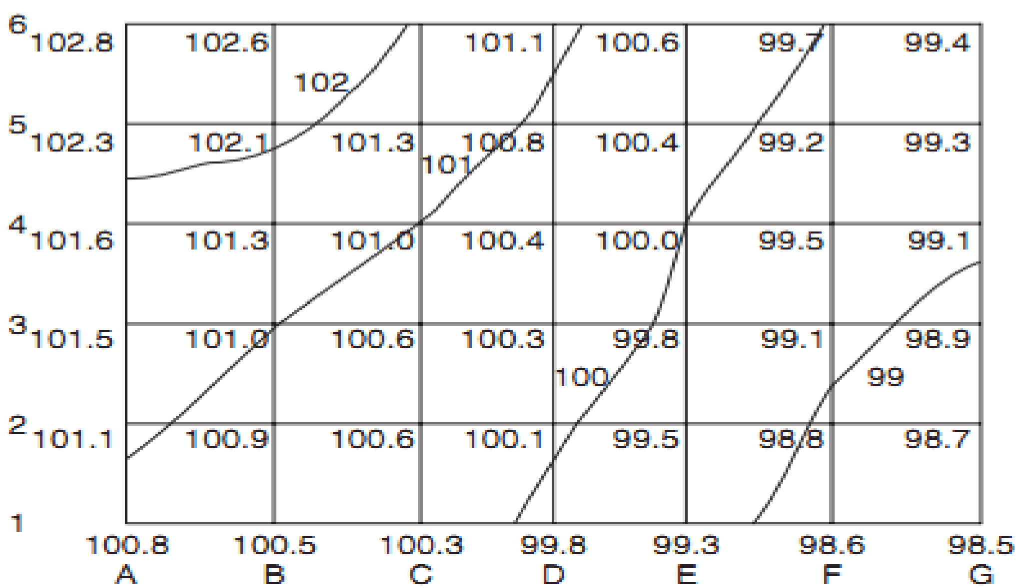


**Fig. 1**

Commonly used size of square varies from 5 m × 5 m to 20 m × 20 m. Levels of all grid points are established by levelling. Then grid square is plotted on the drawing sheet. Reduced levels of grid points marked and contour lines are drawn by interpolation .

**Method of Cross-Section**

In this method cross-sectional points are taken at regular interval. By levelling the reduced level of all those points are established. The points are marked on the drawing sheets, their reduced levels (RL) are marked and contour lines interpolated.

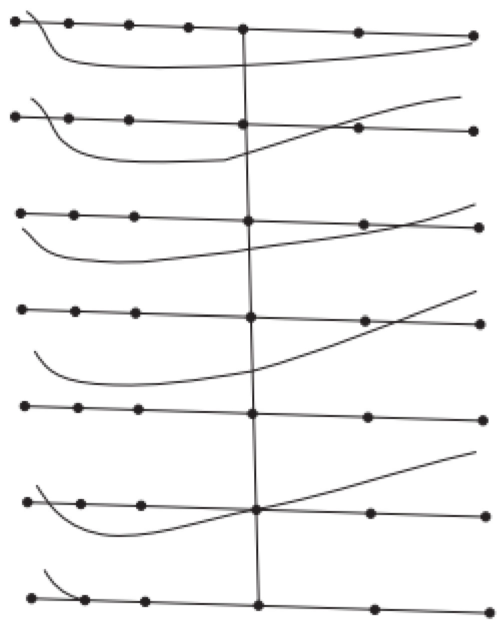


**Fig. 2**

Figure 2 shows a typical planning of this work. The spacing of cross-section depends upon the nature of the ground, scale of the map and the contour interval required. It varies from 20 m to 100 m. Closer intervals are required if ground level varies abruptly.

The cross- sectional line need not be always be at right angles to the main line. This method is ideally suited for road and railway projects.

**Radial Line Method**

[Fig. 3]. In this method several radial lines are taken from a point in the area. The direction of each line is noted. On these lines at selected distances points are marked and levels determined. This method is ideally suited for hilly areas. In this survey theodolite with tacheometry facility is commonly used.

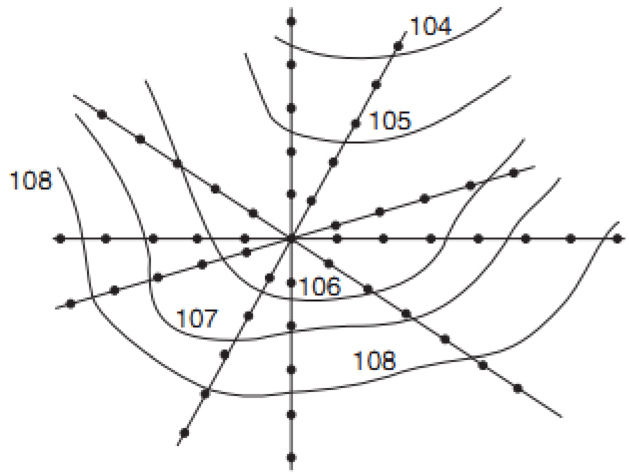



**Fig. 3**

For **interpolating contour points** between the two points any one of the following method may be used:

(a) Estimation

(b) Arithmetic calculation

(c) Mechanical or graphical method.

**Mechanical or graphical method of interpolation** consist in linearly interpolating contour points using tracing sheet:

On a tracing sheet several parallel lines are drawn at regular interval. Every 10th or 5th line is made darker for easy counting. If RL of A is 97.4 and that of B is 99.2 m. Assume the bottom most dark line represents 97 m RL and every parallel line is at 0.2 m intervals. Then hold the second parallel line on A.

Rotate the tracing sheet so that 100.2 the parallel line passes through point B. Then the intersection of dark lines on AB represents the points on 98 m and 99 m contours [Ref. Fig. 4].

Similarly the contour points along any line connecting two neighbouring points may be obtained and the points pricked. This method maintains the accuracy of arithmetic calculations at the same time it is fast.

Similarly the contour points along any line connecting two neighbouring points may be obtained and the points pricked. This method maintains the accuracy of arithmetic calculations at the same time it is fast.

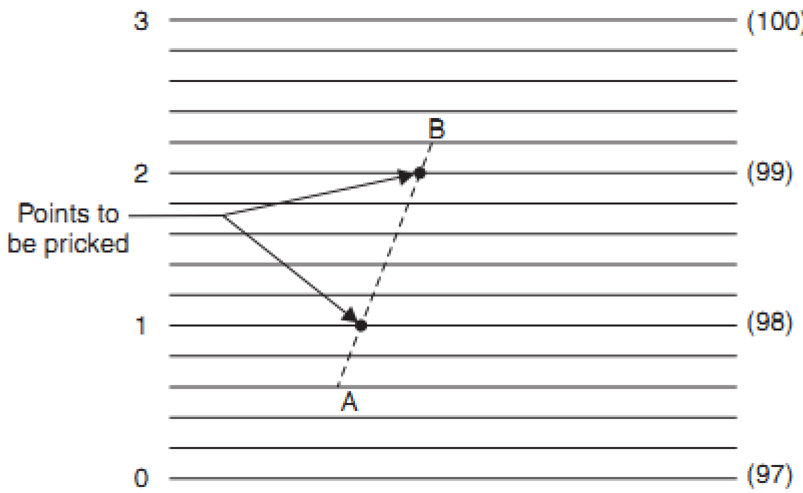



**Fig. 4**

**Drawing Contours**

After locating contour points smooth contour lines are drawn connecting corresponding points on a contour line. French curves may be used for drawing smooth lines. A surveyor should not lose the sight of the characteristic feature on the ground. Every fifth contour line is made thicker for easy readability. On every contour line its elevation is written. If the map size is large, it is written at the ends also.

## TACHEOMETRIC CONTOURING

### By tachometric method

In the case of hilly terrain the tachometric method may be used with advantage.

A tachometer is a theodolite fitted with stadia diaphragm so that staff readings against all the three hairs may be taken.

The staff intercept s is then obtained by taking the difference between the readings against the top and bottom wires.

The line of sight can make any inclination with the horizontal the range of instrument observations.

The horizontal distances need not be measured since the tachometer provides both horizontal as well as vertical control.

Thus if ɵ is the inclination of the line of sight with horizontal the horizontal distance (D) between the instrument axis and the point in which the line of sight against the central wire intersects the staff are given by:

D=K1s cos2ɵ + K2 cos2ɵ V= D tan ɵ

K1 & K2 are instrumental constants.

## Contour Gradient

An imaginary line on the surface of the earth having a constant inclination with the horizontal (slope) is called contour gradient. The inclination of a contour gradient is generally given either as rising gradient or falling gradient, and is expressed as ratio of the vertical height to a specified horizontal distance. If the inclination of a contour gradient is 1 in 50, it means that for every 50 m horizontal distance, there is a rise (or fall) of 1 m.

## Locating contour Gradient on a Map

To locate a rising gradient of 1 in 100 from a point say P situated on 200 m contour on the map having contour interval 5 m at a scale of 1: 5000, draw an arc of radius.

$$= \frac{100 \times 5 \times 100}{5000} = 10 \text{ cm}$$

with radius at P. The arc cuts the 205 m contour at Q. Locate R and S on 210m and 215 m contours taking arcs of radius of 10 mm with centres at R and S, respectively. Join P,Q,R and S. The line P to S represents the contour gradient on the ground having constant slope of 1 in 100.

## Locating contour Gradient on the Ground

To locate a rising gradient of 1 in 100 from the station P, a level is set up at a commanding position and back sight is taken at P. Let the back sight reading be 1.255 m. The staff reading at any point X on the contour gradient can be calculated from its distance from P. For the distance XP of 20 m, the required staff reading would be

$$1.255 - \frac{20}{100} = 1.055\text{m}$$

To locate the point X on the ground, the staff man holds the 20 m-mark of the tape, keeping the zero-mark at P, and moves till the staff reading of 1.055 m is obtained. Likewise, the staff readings for other points at known distance from P, are calculated, and the points are located. If the point Q is on the contour of 105 m, its distance from P would be 500 m in this case. The instruments such as Indian clinometer, theodolite and Ghat tracer may also be used for tracing the contour gradient on the ground.

### Contour Maps and Its Uses

A contour maps consists of contour lines which are imaginary lines connecting points of equal elevation. Such lines are drawn on the plan of an area after establishing reduced levels of several points in the area.

The contour lines in an area are drawn keeping difference in elevation of between two consecutive lines constant. For example, the contour map in fig. 1 shows contours in an area with contour interval of 1 m. On contour lines the level of lines is also written.

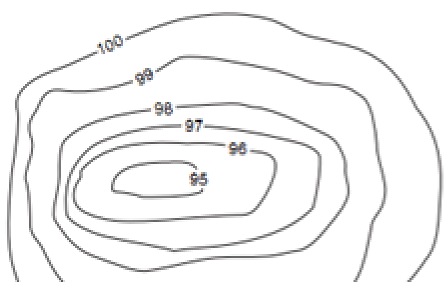



**Fig. 1: Contours**

## UNIT III

## CONTROL SURVEYING

Horizontal and vertical control – Methods – specifications – triangulation - baseline – instruments and accessories – corrections – satellite stations – reduction to centre- trigonometrical levelling – single and reciprocal observations – traversing – Gale’s table.

### 1.1 CONTROL SURVEYS

Control surveys are used to establish highly accurate positions (horizontal control) and elevations (vertical control) of select points or stations. The points are then used to reference other survey observations or measurements.

Typically, a few well-placed horizontal and vertical control stations are established by very precise measurements, using the best equipment and methods available forming what is called the primary control. Additional control stations located across the project area may be added using less precise measurements to form the secondary or "supplemental" control. This type of control, while less accurate, is more readily found, less expensive to establish, and more convenient to use.

For small areas such as a subdivision or even an open-pit mine, usually only one control system, corresponding to the secondary control described above, would be needed. Control surveys can be horizontal and/or vertical, and plane or geodetic surveys. A control survey provides a framework of survey points, whose relative positions, in two or three dimensions, are known to prescribed degrees of accuracy. The areas covered by these points may extend over a whole country and form the basis for the national maps of that country. Alternatively the area may be relatively small, encompassing a construction site for which a large-scale plan is required. Although the areas covered in construction are usually quite small, the accuracy may be required to a very high order. The types of engineering project envisaged are the construction of long tunnels and/or bridges, deformation surveys for dams and reservoirs, three-dimensional tectonic ground

movement for landslide prediction, to name just a few. Hence control networks provide are reference framework of points for:

(1) Topographic mapping and large-scale plan production.

(2) Dimensional control of construction work.

(3) Deformation surveys for all manner of structures, both new and old.

(4) The extension and densification of existing control networks.

The methods of establishing the vertical control, two-dimensional horizontal control will be dealt with here. The methods used for control surveys are:

(1) Traversing.

(2) Triangulation.

(3) Trilateration.

(4) A combination of (2) and (3), sometimes referred to as triangulation.

(5) Satellite position fixing .

(6) Inertial position fixing.

Whilst the above systems establish a network of points, single points may be fixed by intersection and/or resection.

## Sources of error

The sources of error in traversing are:

(1) Errors in the observation of horizontal and vertical angles (angular error).

(2) Errors in the measurement of distance (linear error).

(3) Errors in the accurate centring of the instrument and targets, directly over the survey point (centring error).

## 1.2 HORIZONTAL CONTROL & ITS METHODS

The horizontal control consists of reference marks of known plan position, from which salient points of designed structures may be set out. For large structures primary and secondary control points are used. The primary control points are triangulation stations. The secondary control points are reference to the primary control stations.

**a. Reference Grid**

Reference grids are used for accurate setting out of works of large magnitude. The following types of reference grids are used:

1. Survey Grid
2. Site Grid
3. Structural Grid
4. Secondary Grid

Survey grid is one which is drawn on a survey plan, from the original traverse. Original traverse stations form the control points of the grid. The site grid used by the designer is the one with the help of which actual setting out is done. As far as possible the site grid should be actually the survey grid. All the design points are related in terms of site grid coordinates. The structural grid is used when the structural components of the building are large in numbers and are so positioned that these components cannot be set out from the site grid with sufficient accuracy. The structural grid is set out from the site grid points. The secondary grid is established inside the structure, to establish internal details of the building, which are otherwise not visible directly from the structural grid.

## 1.3 VERTICAL CONTROL & ITS METHODS:

The vertical control consists of establishment of reference marks of known height relative to some special datum. All levels at the site are normally reduced to the near by bench mark, usually known as master bench mark.

The setting of points in the vertical direction is usually done with the help offollowing rods:

1. Boning rods and travelers
2. Sight Rails
3. Slope rails or batter boards
4. Profile board

**a. Boning rods:**

A boning rod consist of an upright pole having a horizontal board at its top, forming a 'T 'shaped rod. Boning rods are made in set of three, and many consist of three 'T' shaped rods, each of equal size and shape, or two rods identical to each other and a third one consisting of longer rod with a detachable or movable 'T' piece. The third one is called traveling rod or traveler.

**b. Sight Rails:**

A sight rail consist of horizontal cross piece nailed to a single upright or pair of uprights driven into the ground. The upper edge of the cross piece is set to a convenient height above the required plane of the structure, and should be above the ground to enable a man to conveniently align his eyes with the upper edge. A stepped sight rail or double sight rail is used in highly undulating or falling ground.

**c. Slope rails or Batter boards:**

These are used for controlling the side slopes in embankment and in cuttings. These consist of two vertical poles with a sloping board nailed near their top. The slope rails define a plane parallel to the proposed slope of the embankment, but at suitable vertical distance above it. Travelers are used to control the slope during filling operation.

**d. Profile boards :**

These are similar to sight rails, but are used to define the corners, or sides of a building. A profile board is erected near each corner peg. Each unit of profile board consists of twoverticals, one horizontal board and two cross boards. Nails or saw cuts are placed at the top of the profile boards to define the width of foundation and the line of the outside of the wall be delivered to the County with the control data upon completion of the work. Computations must be made in accordance with the published standards of the FGCC

**Geodetic Surveying:**

**Definition:**

Geodetic or trigonometrically surveying takes into account the curvature of earth Since very extensive areas and very large distances are involved. In geodic surveying highly refined instruments and methods are used. Geodetic work is undertaken by the state agency e.g. survey of Pakistan undertaken by the state agency.

1. Triangulation
2. Precise leveling

**Object:**

The object of geodetic surveying is to accurately determine the relative position of a sys of widely separated pts (stations) on the surface of earth and also their absolute positions.

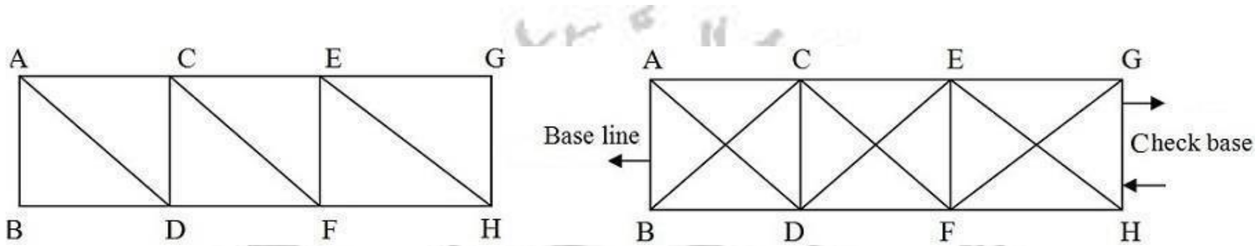
Relative positions are determined in terms of azimuths and length f lines joining them. Absolute positions are determined in terms of latitude and longitudes and elevations above mean sea laves. The methods employed in geodetic surveying are:

1. Triangulation (most accurate but expensive)
2. Precise traverse (inferior and used when triangulation is physically impossible or very expensive) e.g. Densely wooded country.

## 1.4 TRIANGULATION

It is based on the trigonometry proposition that of one side and three angles be computed by the application of since rule. In this method suitable points called triangulation stations are selected and established throughout the area to be surveyed.

The stations may be connected by a series of triangles or a chain of quadrilaterals as shown.




**Baseline:**

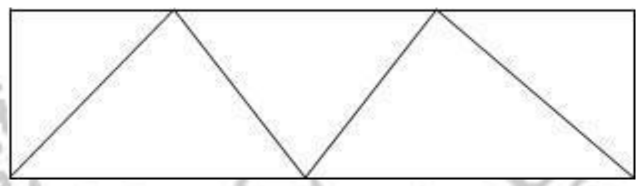
Whose length is measured these stations form the vertices of a series mutually connected, triangles the complete figure being called ‘Triangulation system’. In this system of triangles one line say ‘AB’ and all the angles are measured with greatest care and lengths of all the remaining line in the system are then computed. For checking both the fieldwork and computation anotherline say GH is very accurately measured at the end of the system. The line whose length is actually measured is known as baseline or base and that measured for checking purpose is known as the check base.

**Triangulation Figures:**

The geometric figures used in triangulation system are (i) Triangles (ii) Quadrilaterals (ii) Quadrilaterals, Pentagon, hexagons with centre angle. This arrangement although simple and economical but less accurate since the number of conditions involve in its adjustment is small.

1. Station adjustment ==> sum of angle is 180
2. Figure adjustment ==> sum of angles is 400 grad or 360
3. Quadrilateral; adjust ==> (all the angles are horizontal)

Quadrilaterals pentagons or hexagonal with central stations. For very accurate work a chain of quadrilaterals may be used. There is no station at the intersection of diagonals. This system is most accurate since the number of conditions in its adjustments is much greater. To minimize the effect of small errors in measurement of angles the triangles hold be well shaped or well proportioned i.e. they should not have angle less than 30 or greater than 120.The best shape triangle is equilaterals triangle and best shape quadrilateral is square.



**Well-conditioned triangles**

The accuracy of a triangulation system is greatly affected by the arrangement of triangles in the layout and the magnitude of the angles in individual triangles. The triangles of such a shape, in which any error in angular measurement has a minimum effect upon the computed lengths, is known as *well-conditioned triangle.* In any triangle of a triangulation system, the length of one side is generally obtained from computation of the adjacent triangle. The error in the other two sides if any, will affect the sides of the triangles whose computation is based upon their values.

Due to accumulated errors, entire triangulation system is thus affected thereafter. To ensure that two sides of any triangle are equally affected, these should, therefore, be equal in length. This condition suggests that all the triangles must, therefore, be isoceles. Hence, the best shape of an isoceles triangle is that triangle whose base angles are 56°14′ each. However, from practical considerations, an equilateral triangle may be treated as a wellconditional triangle. In actual practice, the triangles having an angle less than 30° or more than 120° should not be considered.

## 1.5 CLASSIFICATION OF TRIANGULATION

Classification of a triangulation system is based on the accuracy with which the length and angle of a line of the triangulation are determined. The following are the classification based on the order of grades: i) First order or primary triangulation ii) Second order or secondary triangulation iii) Third order or tertiary triangulation

**i) First order or primary triangulation:**

The first order triangulation is of the highest order and is employed either to determine the earth's figure or to furnish the most precise control points to which secondary triangulation may be connected. The primary triangulation system embraces the vast area. Every precaution is taken in making linear and angular measurements and in performing the reductions. The following are the general specifications of the primary triangulation:

Average triangle closure : less than 1 second

Maximum triangle closure : not more than 3 seconds

Length of the base line : 5 to 15 km

Length of the sides of triangles : 30 to 150 km

Actual error of base : 1 in 300000

Probable error of base : 1 in 1000000

Discrepancy between two measures of section: 10 mm km

Probable error of computed distance : 1 in 60000 to 1 in 250000

Probable error in astronomic azimuth : 0.5 seconds

**ii) Second order or secondary triangulation:**

The secondary triangulation consists of a number of points fixed within the framework of primary triangulation. The stations are fixed at close intervals so that the sizes of the triangles formed are smaller than the primary triangulation. The instruments and methods used are not of the same utmost refinement. The general specifications of the secondary triangulation are:

Average triangle closure : 3 seconds

Maximum triangle closure : 8 seconds

Length of the base line : 1.5 to 5 km

Length of the sides of triangles : 8 to 65 km

Actual error of base : 1 in 150,000

Probable error of base : 1 in 500,000

Discrepancy between two measures of section: 20 mm km

Probable error of computed distance : 1 in 20,000 to 1 in 50,000

Probable error in astronomic azimuth : 2 seconds

**iii) Third order or Tertiary triangulation:**

The third order triangulation consists of a number of points fixed within the framework of secondary triangulation and forms the immediate control for detailed engineering and other surveys. The sizes of the triangles are small and instrument with moderate precision may be used. The general specifications of the third order triangulation are:

Average triangle closure : 6 seconds

Maximum triangle closure : 12 seconds

Length of the base line : 0.5 to 3 km

Length of the sides of triangles : 1.5 to 10 km

Actual error of base : 1 in 750,000

Probable error of base : 1 in 250,000

Discrepancy between two measures of section: 25 mm km

Probable error of computed distance : 1 in 5,000 to 1 in 20,000

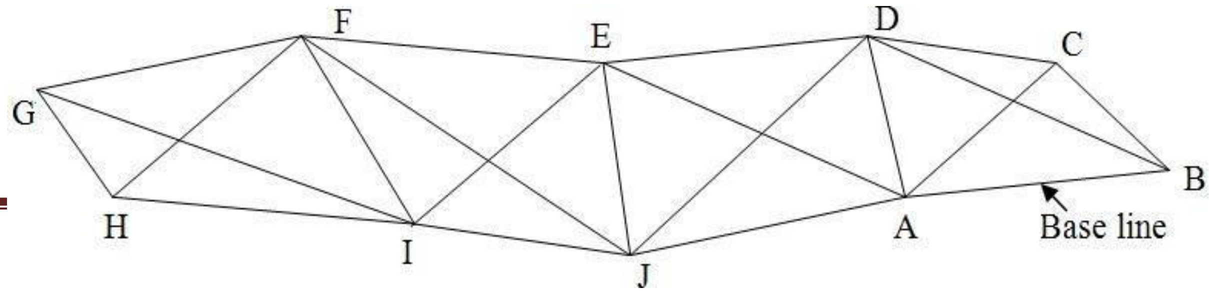
Probable error in astronomic azimuth : 5 seconds

1. The establishment of accurately located control points for surveys of large.
2. The accurate location of indirection work such as:
    - Centre lines, terminal pts shafts for long tunnels
    - Centers lines and abutment for bridges of longs spans.
    - Complex highway interchanges.
3. The establishment of accurately located control pts in connection with aerial surveying.
4. Measurement of deformation of structure such as dams.

**Trilateration:**

Because of the development of highly accurate electronic measuring devices, a triangulation system can be completely observed, computed and adjusted by measuring the lengths o the sides in the network. This procedure is known as trilateration. No horizontal angle need to be measured because the lengths of the sides are sufficient to permit both the horizontal angles and the positions of the stations to be computed.

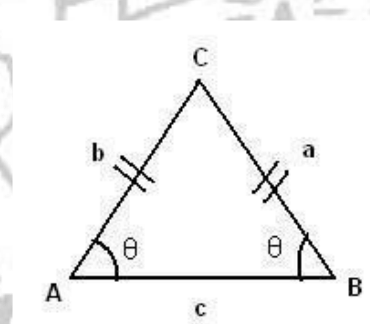
The surveying solution technique of measuring only the side of triangle is called ***triplication***.



**Criterion of strength of a figure with reference to a well conditioned triangle :**

The shape of the triangle formed by the selected triangulation stations should be such that any error in the measurement of the angle shall have a minimum effect upon the length of the calculated sides. Such a triangle is called a well-conditioned triangle.

In a triangle one side may be computed from the computation of adjacent triangle. The error in other two sides may affect the rest of the figure. If the two sides are to be equally accurate, then they should be of equal length, which could be possible only by making the triangle isosceles. In order to find the magnitude of the angle of triangle, let ABC be an isosceles triangle with AB of known length.




The sides BC and CA are to be computed. As the triangle is isosceles $\angle A = \angle B$.

By sine rule, $a = \dfrac{\sin A}{\sin C}\, c$ …………. (1)

Let δA be the error in the measurement of angle.

δa1 be the corresponding error in the side a partially.

Differentiating eqn.(1) with respect to A we get

$= \delta a1$

$$= \frac{\delta a1}{a}$$

$$= \frac{c \cos A.\delta A}{\sin C}$$

$$= \frac{\cos A \delta A}{\sin A}$$

$= \delta A. \cot A$.................................. (2)

Similarly δC be the error in the measurement of C and δa2 be the corresponding error in the side a.

Differentiating eqn.(2) again partially with respect to C, then

$$\delta a = -c\frac{\sin A \cos C}{\sin^2 C}\delta C$$

$$\frac{\delta a2}{a} = \frac{\cos C}{\sin C}\delta C = -\delta C . \cot C$$ ...................... (3)

If $\delta A$ and $\delta C$ are the probable errors in angles, then they are equal to

Then the probable fraction error in the side a

$$= \pm \beta \sqrt{\cot^2 A + \cot^2 C}$$

This is minimum when $\cot^2 A + \cot^2 C$ is minimum.

But C = 180° – A – B = 180° – 2A (since ∠A= ∠

$\cot^2 A + \cot^2 2A$ should be minimum.

Differentiating the above equation with respect to A and equating to zero, we get after reduction

$$4\cos^2 A + 2\cos^2 A - 1 = 0$$

From which A is got 56° 14' approximately.

Hence the best shape of a triangle is an isosceles triangle with base angles 56° 14'.

However for practical consideration 56° 14' = 60°.

For all practical purposes, an equilateral triangle is the most suitable. In general, however, triangles having angles smaller than 30° or greater than 120° should be avoided.

## 1.6 BASELINE MEASUREMENT

Baseline is laid with great accuracy of measurement and alignment as it forms the basis for the computations of triangulation system. The length of the baseline depends on the grades of the triangulation.

**<u>The factors to be considered while selecting base line</u> :**

The measurement of base line forms the most important part of the triangulation operations. The base line is laid down with great accuracy of measurement and alignment as it forms the basis for the computations of triangulation system. The length of the base line depends upon the grades of the triangulation. Apart from main base line, several other check bases are also measured at some suitable intervals. InIndia, ten bases were used, the lengths of the nine bases vary from 6.4 to 7.8 miles and that of the tenth base is 1.7 miles.

**Selection of site for base line:**

The length of the base line to be adopted depends on the magnitude of triangulation work ie., the grade of the triangulation. Apart from main base line additional check bases are also provided at some suitable intervals.

The location of the base line should be such that the site affords accurate measurement . The following factors should be considered in the selection of the location.

- The ground selected should be as plain as possible. However, gentle slope may also be adopted.
- All the main stations of triangulation should be visible from both the ends of the base line.
- It should be possible to build up a network of well-proportioned triangles on the base.
- The site should be free from obstructions throughout the length of the baseline. The expenses involved in clearing the site should be minimum.
- The ground should be reasonably firm and water gaps, if any, should not be wider than the length of a tape.
- The site should be possible for extension to primary triangulation. This is an important aspect, as the error in extension may exceed the error in measurement.

**Forms of Base measuring apparatus:**

There are two forms of base measuring apparatus:

1. Rigid bars

   Before the introduction of invar tapes, rigid bars were used for work of highest precision. The rigid bars may be divided into two classes:

   i) Contact apparatus : It which ends of the bars are brought into successive contacts. Example: The Eimbeck Duplex Apparatus
   ii) Optical apparatus: In which the effective lengths of the bars are engraved on them and observed by microscopes. Example: The Colby apparatus and Woodward Iced Bar apparatus.

2. Flexible apparatus

In recent years, the use of flexible instruments has increased due to the longer length that can be measured at a time without any loss in accuracy. The flexible apparatus consists of a) steel or invar tape, b) steel and brass wires. The flexible apparatus has the following advantages over the rigid bars:

i) Due to the greater length of the flexible apparatus, a wider choice of base sites is available since rough ground with wider water gap can be utilized.
ii) The speed of instrument is quicker, and thus less expensive.
iii) Longer bases can be used and more check bases can be introduced at closer intervals.

## 1.7 EQUIPMENTS FOR BASE LINE MEASUREMENT:

The equipment for base line measurement by flexible apparatus consists of the following:

1. Three standardized tapes: out of the three tapes one is used for field measurement and the other two are used for standardizing the field tape at suitable intervals.

2. Straining devices, marking tripods or stakes, and supporting tripods or staking.

3. A steel tape for spacing the tripods or stakes.

4. Six thermometers, four for measuring the temperature of the field tape and two for standardizing the four thermometers.

5. A sensitive and accurate spring balance

## 1.8 CORRECTION FOR BASELINE

After measuring the length of a baseline, the correct length of the line is computed by applying various applicable corrections, the following corrections and provide expressions for

i) Correction for absolute length
ii) Correction for temperature.
iii) Correction for pull.
iv) Correction for sag.
v) Correction for slope
vi) Correction for alignment
vii) Reduction to sea level
viii) Correction to measurement in vertical plane

### i) Correction for temperature:

If the temperature in the field is more than the temperature at which the tape was standardized, the length of the tape increases measured distance becomes less and the correction is additive. Similarly if the temperature is less, the length of the tape decreases measured distance becomes more and the correction is negative. The temperature correction is given by

$$C = \alpha \, (T_m - T_O) \, L$$

Where α = coefficient of thermal expansion
$T_m$ = mean temperature of tape
$T_O$ = standardized temperature of tape
L = measured length of tape

If however steel and brass wires are used simultaneously as in Jaderin's method, the corrections are given by

$$C_t \ (brass) = \frac{\alpha_b \ (L_S - L_b)}{\alpha_b - \alpha_S}$$

$$C_t \ (steel) = \frac{\alpha_S \ (L_S - L_b)}{\alpha_b - \alpha_S}$$

**ii) Correction for pull or tension:**

If the pull applied during measurement is more than the pull at which the tape was standardized, the length of the tape increases, measured distance becomes less and the

correction is positive. Similarly, if the pull is less, the length of the tape decreases, measured distance becomes more and the correction is negative.

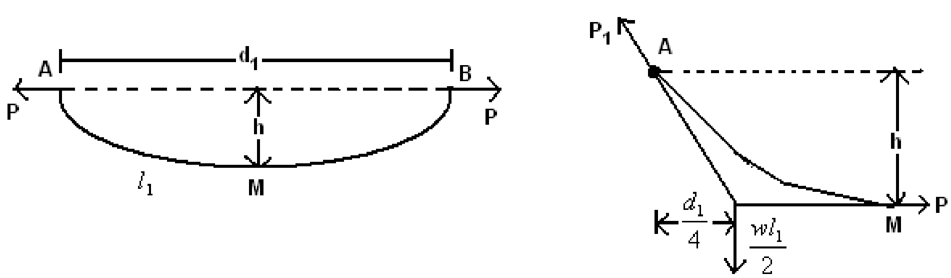
If Cp is the correction for pull, we have

$$C_P = \frac{(P - P_O)\,L}{AE}$$

Where, P = pull applied during measurement (N)
$P_O$ = standard pull (N)
L = measured length (m)
A = cross sectional area of the tape (cm2)
E = young's modulus of elasticity (N / cm2)
The pull is applied in the field should be less than 20 times the weight of the tape.

**iii) Correction for sag:**

When the tape is stretched on supports between two points, it takes the form of a horizontal catenary. The horizontal distance will be less than the distance along the curve. The difference between horizontal distance and the measured length along catenary is called the sag correction. For the purpose of determining the correction, the curve may be assumed to be a parabola.



$$C_S = \frac{l\,(wl)^2}{24n^2\,P^2} = \frac{l\,W^2}{24n^2\,P^2}$$

Where CS = tape correction per tape length

l = total length of the tape

W = total weight of the tape

n = number of equal spans

P = pull applied

If L = total length measured

N = number of whole length tape

Total sag correction = sag correction for any fractional tape length □ S NC

**iv) Correction for absolute length:**

If the absolute length of the tape or wire is not/equal to its nominal or designated length, a correction will have to be applied to the measured length of the line. If the absolute length of the tape is greater than the nominal or the designated length, the measured distance will be too short and the correction will be additive.

Thus, Ca = L.c / l

Where, Ca = correction for absolute length

L = measured length of line

c = correction per tape length

l = designated length of the tape

**Problem: 1**

**A steel tape 20 m long standardized at 55° F with a pull of 98.1 N was used for measuring a baseline. Find the correction per tape length, if the temperature at the time of measurement was 80° F and the pull exerted was 156.96 N. Weight of 1 cubic**

**metre of steel = 77107 N. weight of tape = 7.85 N and E = 2.05 x $10^5$ N/mm$^2$. Coefficient of linear expansion of tape per degree F = 6.2 x $10^{-6}$.**

**Solution:**

L = 20 m; T0 = 55$^o$C; Tm = 80$^o$C; Po = 98.1 N; P = 156.96 N; α = 6.2 x $10^{-6}$;

Weight of steel = 77107 N; Weight of tape = 7.85 N; E = 2.05 x 105 N / mm$^2$

i) **Correction for Temperature:**

Ct = α (Tm - To) L

= 6.2 x $10^{-6}$ (80 – 55) x 20

Ct = 0.0031 m

ii) **Correction for Pull:**

$$C_P = \left(\frac{P - Po}{AE}\right) L$$

$$A = \frac{7.85}{77107\ X\ 20} = 5.1 X 10^{-6}\ m^2 = 5.1\ mm^2$$

$$C_P = \frac{156.96 - 98.1}{5.1\ X\ 2.05\ X\ 10^5}\ X\ 20$$

$$\mathbf{C_P = 0.00112\ m}$$

Here, weight of tape = (Area x 1 x weight of steel) x length

7.85 = (A x 1 x 77107) x 20

**iii) Sag Correction:**

$$C_s = \frac{LW^2}{24n^2P^2} = \frac{20\ X\ 7.85^2}{24\ X\ 1^2\ X\ 156.96^2}$$

$$\mathbf{C_s = 0.00208\ m}$$

Total correction $= C_t + C_P - C_s$

$= 0.0031 + 0.00112 - 0.00208$

**Total correction = 0.00214 m**

True length = Length + correction

$= 20 + 0.00214$

**True length = 20.00214 m**

## Problem :2

**A tape 20 m long of standard length at 29$^0$C was used to measure a line, the m temperature during measurement being 19$^0$C. the measured distance was 882.10 m, the following being the slopes: 2$^o$ 20' for 100 m; 4$^o$ 12' for 150 m; 1$^o$ 06' for 50 m; 7$^o$ 48' for 200 m; 3$^o$ 00' for 300 m; 5$^o$ 10' for 82.10 m. find the true length of the line if the coefficient of expansion is 6.5 x 10$^{-6}$ per degree F.**

## Solution:

$L = 882.10$ m; $T0 = 29^oC = 86^oF$; $Tm = 19^oC = 68^oF$; $\alpha = 6.5 \times 10^{-6}$; **i)**

### i) Correction for Temperature:

$$Ct = \alpha\ (Tm - T0)\ L$$

$$= 6.5 \times 10^{-6}\ (68 - 86) \times 882.10$$

**Ct = - 0.103 m**

**ii) Correction for Slope**

Csl $=\sum l(1-\cos\theta)=$ 100 ( 1–cos 2°10' )+150 ( 1–cos 4°12' )+50 ( 1– cos 1° 06' ) +200 ( 1– cos 7° 48' ) +300 ( 1– cos 3° 00' ) +82.1 ( 1–cos 5°10' )

**Cs = 3.078 m**

Total correction = Ct - Csl

= - 0.103–3.078

**Total correction = - 3.181 m**

True length = Length + correction = 882.1 – 3.181

**True length = 878.919 m**

**Problem: 3**

**A 30 m steel tape was standardized on the flat and was found to be exactly 30 m under no pull at 66$^{o}$ F. It was used in catenary to measure a base of 5 bays. The temperature during the measurement was 92$^{o}$ F and the pull exerted during measurement was 100 N. The area of cross section of the tape was 8mm$^2$. The specific weight of steel is 7.86 kN/m$^2$, α = 0.63 x 10-5 F and E = 2.1 x 10$^5$ N/mm$^2$. Find the true length of the tape.**

**Solution:**

$L = 30$ m; $T_0 = 66^0$ F; $T_m = 92^0$ F; $P_O = 0$; $P = 100$ N; $\alpha = 0.63 \times 10^{-5}$; $A = 8$ mm$^2$;

Weight of steel = 78.6 kN/m$^2$; $E = 2.1 \times 10^5$ N / mm$^2$

**i) Correction for Temperature:**

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

$= 0.63 \times 10^{-5} (92 - 66) \times 30$

$C_t = 0.00491$ **m**

**ii) Correction for Pull:**

$$C_P = \left(\frac{P - Po}{AE}\right) L = \frac{100 - 0}{8 \; X \; 2.1 \; X \; 10^5} \; X \; 30$$

$C_P = 0.00178$ **m**

**iii) Sag Correction:**

$$C_s = \frac{LW^2}{24n^2 P^2}$$

Here each span = $\frac{30}{5} = 6m$

Weight of tape, W = (Area x 1 x weight of steel) x length

$= (8 \times 10^{-6} \times 1 \times 78.6 \times 10^3) \times 6$

$= 3.773$ N

$$C_s = \frac{6 \times 3.773^2}{24 \times 1^2 \times 100^2}$$

$$C_s = 0.000356 \text{ m}$$

**Total sag correction, $C_s = 5 \times 0.000356 = 0.00178$ m**

$$\text{Total correction} = C_t + C_P - C_s$$

$$= 0.00491 + 0.00178 - 0.00178$$

**Total correction = 0.00491 m**

$$\text{True length} = \text{Length} + \text{correction}$$

$$= 30 + 0.00491$$

**True length = 30.00491 m**



**Problem: 4**

**A Base line was measured with a steel tape, which was exactly 30m at 20˚ C, and pull of 6kg and the measured length was 459.242 m. temperature during measurement was 30˚ C and the pull applied was 10 kg. the tape was uniformly supported during the measurement. Find the true length of the line if the cross-sectional area of the tape was 0.02 cm$^2$, the coefficient of expansion per 1˚ C = 0.0000035, and the modulus of elasticity = 2.1 kg/cm$^2$.**

**Solution:**

$C = 0.0000035$

$E = 2.1\ kg/cm^2$

$P = 10\ kg,\ Po = 6\ kg$

$Tm = 30^o\ C,\ To = 20^o\ C$

Measured length = 459. 242 m

**i) Correction for temperature:**

$Ct = L\ \alpha\ (Tm - To)$

$= 30 \times 0.0000035\ (30 - 20)$

$= 1.05 \times 10^{-3}\ m$

**ii) correction for Pull**:

$$C_P = \left(\frac{P - Po}{AE}\right)L$$

$$= \frac{10 - 6}{0.02 \times 2.1 \times 10^2} \times 2$$

$= 2.857 \times 10^{-3}\ m$

Total correction = + 0.00105 + 0.002857

$= 3.907 \times 10^{-3}\ m$

True length = 459.242 + 0.003907 = 459.2459 m

**Problem:5**

**A nominal distance of 30m was set out with a 30m steel tape from a mark on the top of one peg to a mark on the top of one peg to a mark on the top of another, the tape being in catenary under a pull of 100N and at a mean temperature of 70° F . The top of one peg was 0.25 m below the top of the other. The top of the higher peg was 460m above**

**the mean sea level. Calculate the exact horizontal distance between the marks on the two pegs and reduce it to mean sea level, if the tape was standardized at a temperature of 60° F in catenary under a pull of i) 80N ii) 120N iii) 100N**

**Take radius of earth = 6370 km**
**Density of tape = 7.86 g/cm3**
**Section of tape = 0.08 sq.cm**
**Co-efficient of expansion = 6x10-6 per°F**
**Young's modulus = 2 x $10^7$ N/$cm^2$**

**Solution:**

**i) Correction for slope:**

$$C = \frac{h^2}{2L}$$

$= \frac{(0.25)^2}{2 \times 30}$ = 0.0010 m (subtractive)

**ii) correction for temperatue:**

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

= 30 x 6 x 10-6 (70 – 60)

= 0.0018 m (additive)

**iii) correction for pull:**

$$C_P = \left(\frac{P - Po}{AE}\right) L$$

a) **when Po = 80 N,** Pullcorrection = $\frac{(100 - 80)30}{0.08 \times 2 \times 10^7}$ = 0.0004 m (additive)

b) **When Po = 120 N,** Pull correction = (100 – 120) 30 = 0.0004 m (subtraction)

$0.08 \times 2 \times 10^7$

c) **When Po = 100 N,** Pull correction = 0

**iv) Sag correction**

$$C_s = \frac{LW^2}{24n^2P^2}$$

Now mass of tape per metre run = (0.08 x 1 x 100) x (7.86 / 1000) kg = 0.06288 kg/m

Weight of tape per metre run = 0.06288 x 9.81 = 0.6169 N/m

Total weight of tape = 0.6169 x 30 = 18.51 N

a) **When Po = 80 N**

Sag correction = $\frac{30 \times (18.51)^2}{24\,(80)^2} - \frac{30 \times (18.51)^2}{24\,(100)^2}$ = 0.0669 – 0.04283 = 0.02407 (additive)

b) **When Po = 120 N**

Sag correction = $\frac{30 \times (18.51)^2}{24\,(120)^2} - \frac{30 \times (18.51)^2}{24\,(100)^2}$ = 0.02974 – 0.04283 = 0.0131 (subtractive)

c) **When Po = 100 N**

Sag correction is zero

**Final correction:**

a) Total correction = – 0.0010 + 0.0018 + 0.0004 + 0.02407 = + 0.02527 m

b) Total correction = – 0.0010 + 0.0018 – 0.0004 – 0.0131 = – 0.02527 m

**c)** Total correction = – 0.0010 + 0.0018 + 0 + 0 = + 0.0008 m

**Problem :4**

**Find the sag correction for 30 m steel tape under a pull of 80 N in three equal spans of 10 m each. Mass of one cubic cm of steel = 7.86 g/cm$^3$. Area of cross section of the tape = 0.10 sq.cm.**

**Solution:**

**Sag Correction:**

$$C_s = \frac{LW^2}{24n^2P^2}$$

Here each span = $\frac{30}{3} = 10\,m$

Weight of tape, W = (Area x 1 x weight of steel) x length

= (0.10 x 1 x 7.86 x 10 $^{-3}$) x 30 x 100

= 2.358 kg

$$C_s = \frac{30 \ X\ 100 \ X\ 2.358^2}{24 \ X\ 3^2 \ X\ (80/9.81)^2}$$

**Cs = 1.16 cm**

**1.9 INTERVISIBILITY OF TRIANGULATION STATION**

If the distance between stations is more or difference in elevation is less, the intervisibility has to be checked by calculations. It may be necessary to raise both the

instrument and the signal in order to overcome the curvature of the earth and the intervening obstructions. The following three conditions may decide the height of the instrument and the signal.

a) Distance between stations
b) Elevation of stations
c) Intervening ground

**i) Distance between stations:**

Considering the condition of no intervening ground the distance of visible horizonfrom a station of known elevation above datum is given by

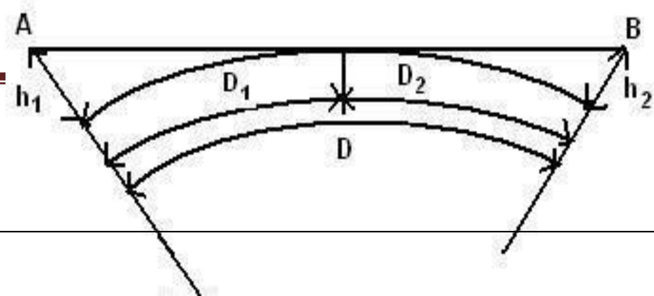
$$h = (1-2m)\,\frac{D^2}{2R}$$

Where h = height of the station above datum
D = distance to the visible horizon
R = mean radius of the earth
m = mean coefficient of refraction
m = 0.07 for sights over land
= 0.08 for sights over sea Taking D and R in kilometers with m = 0.07

$h = 0.06728\ D^2$, in which h is in metres.

**ii) Elevations of stations:**

If there is no obstruction due to intervening ground, the elevation of a station at a distance may be calculated, when it may be visible from another station of known elevation. Then



$$h = (1-2m)^2 2R$$

Substituting m = 0.07 and R in km.

$h = .06728D^2$

h1 = 0.0672 8 $D1^2$(metres)

where h1 = known elevation of station A above datum

D1 = distance from A to the point of tangency D = known distance between A and B

$$D1 = \sqrt{\frac{h1}{0.06728}}$$

$$D1 = 3.853\sqrt{h1}$$

Knowing D1, D2 = D – D1

Where D2 = distance from B to the point of tangency

Knowing D2, h2 the required elevation of B above datum may be calculated from

h2 = 0.06728 $D1^2$(metres)

If the actual ground level of B is known, it can be known whether it is necessary to elevate the station B above the ground. If found to be necessary the required height of tower can be calculated.

It is a point to be noted that the line of sight should not graze the surface at the point of tangency but should be atleast 2 to 3 m above.

**iii) Intervening ground:**

In general during the reconnaissance itself the elevations and positions of peaks in the intervening ground between the proposed stations should be determined.

A comparison of the elevations of stations should be made to the elevation of the proposed line of sight. If the line of sight is clear off the obstruction then the work is proceeded. If not the problem can be solved based on the principle discussed in the previous sections.

**Problem: 5**

**Two stations P and Q are 81 km apart. They are situated on either side of a sea. The instrument axis at P is 39 m above MSL. The elevation of Q is 207 m above MSL. Calculate the minimum height of the signal at Q. The coefficient of refraction is 0.08 and the mean radius of earth is 6370km. (10)**

**Solution:**

There is no intervening ground.

Hence the height of the signal at Q, h = $(1 - 2m)\dfrac{D^2}{2R}$

$$= (1 - (2 \; X \; 0.08)) \; X \; \frac{81^2}{2 \; X \; 6370}$$

Minimum height of signal, **h = 0.43 m**

## 1.13 TRIGNOMETRICAL LEVELLING

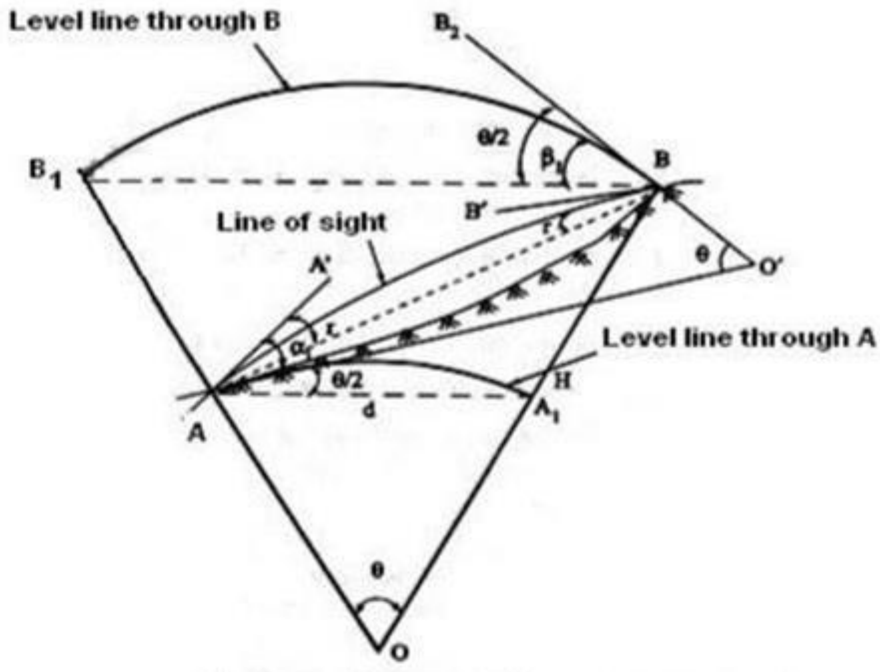


Fig. Terrestrial refraction

**Determination of Difference in Elevation:**

The difference in elevation between the two points P and Q can be found out by two methods:

a) **By Single observation**
b) **By reciprocal observation**

**a) Difference in elevation by single observation:**

in this case, the observations are made from only one station. The following corrections will have to be applied

1. Correction for curvature

2. Correction for refraction and
3. Correction for axis signal

Since the sign of these corrections will depend upon the sign of the angle observed, we shall consider the following cases:

i) When the observed angle is the angle of elevation
ii) When the observed angle is the angle of depression.

**i) For angle of elevation:**

**α = observed angle of elevation to Q**

**α1 = observed angle corrected for axis signal = (α - δ1)**

**d = horizontal distance**

**H= difference in elevation between Q and P**

$$= \frac{d \sin \left\{ \alpha_1 + (1 - 2m) \dfrac{d}{2R \sin 1''} \right\}}{\cos \left\{ \alpha_1 + (1 - m) \dfrac{d}{R \sin 1''} \right\}}$$

**ii) For angle of depression:**

**β = observed angle of depression to P**

**β1 = observed angle to be corrected for axis signal = β + δ2**

**H = difference in elevation between Q and P**

$$= d \sin \left\{ \beta_1 - (1 - 2m) \frac{d}{\phantom{x}} \right\}$$

$$\frac{2R \sin 1''}{\cos\left\{ \beta_1 - (1 - m) - \frac{d}{R \sin 1''} \right\}}$$

**Problem: 10**

**Find the difference in levels of the points A and B and the R.L of B from the following data:**

**Horizontal distance between A and B = 5625.389 m**

**Angle of depression from A and B = 1°28'34"**

**Height of signals of B = 3.886 m**

**Height of instrument at A = 1.497 m**

**Coefficient of refraction = 0.07**

**Rsin1" = 30.876 m. R.L of A = 1265. 85 m**

**Solution:**

d = 5625.389 m; β = - 1° 28' 34''; ; h = 1.497 m; S = 3.886 m;

; R sin 1" = 30.88 m; m = 0.07; RL of A = 1265.85 m

Axis signal correction at A = $\delta = \frac{s - h}{D \sin 1''}$

= 3.886 – 1.497 / 5625.389 sin1"

= 87.596 seconds = 0° 1' 27.6"

$\beta 1 = \beta + \delta = 1^o\ 28'\ 34'' + 0^o\ 1'\ 27.6''$

$= 1^o\ 30'\ 1.6''$

$$\Theta = \frac{d}{R \sin 1''} = \frac{5625.389}{} = \frac{182.169}{30.88''} \text{ seconds} = 0^o\ 3'\ 2.17''$$

$\Theta / 2 = 182.169 / 2 = 0^o\ 1'\ 31.08''$

$r = m\,\Theta = 0.07 \times 0^o\ 1'\ 31.08''$

$= 0^o\ 0'\ 6.38''$

The difference in elevation (H) is given by

$$= \frac{d \sin(\beta 1 + m\theta - \theta/2)}{\cos(\beta 1 + m\theta - \theta)}$$

$$= \frac{5625.389 \sin(1^o\ 28'\ 34'' + 0^o\ 0'\ 6.38'' - 0^o\ 1'\ 31.08'')}{\cos(1^o\ 28'\ 34'' + 0^o\ 0'\ 6.38'' - 0^o\ 1'\ 31.08'')}$$

$= 142.58 / 0.999$

$= 142.625$ m

R.L of B = R.L of A + H = 1265. 85 + 142.625

= 1408.475 m

## 1.14 TRAVERSING:

A series of intervisible points at which angles are measured, and between which distances are measured.Traversenetworks have many advantages, including:

- Less reconnaissance and organization needed;
- While in other systems, which may require the survey to be performed along a rigid polygon shape, the traverse can change to any shape and thus can accommodate a greatdeal of different terrains;
- Only a few observations need to be taken at each station, whereas in other surveynetworks a great deal of angular and linear observations need to be made and considered;
- Traverse networks are free of the strength of figure considerations that happen intriangular systems;
- Scale error does not add up as the traverse is performed. Azimuth swing errors can alsobe reduced by increasing the distance between stations.
- The traverse is more accurate than triangulateration (a combined function of the triangulationand trilateration practice).

**Traverse Types :**

1.A closed polygonal traverse starts and finishes on the same known point.

2.A closed link traverse joins two known points.

3.An open traverse starts on a known point and finishes on an unknown point.

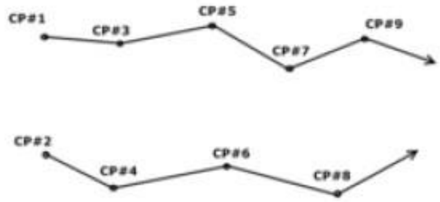



Diagram of an open traverse

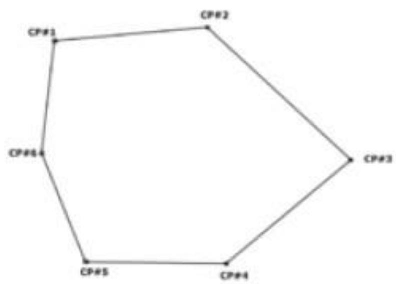


Diagram of a closed traverse

## 1.15 GALE'S TRAVERSE TABLE

Traverse computation are usually done in a tabular form. One such form is Gale's traverse table and is widely used because of its simplicity.

The columns of Gale's table are filled as illustrated below:

1. In the case of theodolite traversing, the included angles are adjusted to satisfy the geometrical conditions, i.e., the sum of the included angles should be $(2n \pm 4) \times 90°$, where $n$ is the number of sides of the closed traverse. The plus sign is used when the angles are exterior angles, and the minus sign when they are interior angles.
   In the case of compass traversing, the observed bearings are adjusted for local attraction.
2. From the observed bearing of a line, e.g., line $AB$ in Table 5.1, the whole circle bearings of all other lines are calculated and then these bearings are reduced to those in the quadrantal system.
3. From the lengths and computed reduced bearings of the lines, the consecutive coordinates, i.e., latitudes and departures are worked out.
4. A check is done to find out whether the algebraic sum of latitudes and the algebraic sum of departures are zero. If not, a correction is applied using the transit rule.
   In the case of a 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 selected that the entire traverse lies in the north-east quadrant. This is done to facilitate plotting of the traverse on a sheet with the left-hand bottom corner of the sheet as the origin.

**Example 5.13** The lengths, bearings and included angles of a closed traverse *ABCDA*, as observed with a transit theodolite, are given below. Prepare a Gale's traverse table and plot the traverse.

| Line | Length | Included angle | W.C.B. |
|---|---|---|---|
| *AB* | 255 m | $\angle A = 93°18'16''$ | 140°42′ |
| *BC* | 656 m | $\angle B = 74°16'25''$ | |
| *CD* | 120 m | $\angle C = 123°42'00''$ | |
| *DA* | 668 m | $\angle D = 68°41'16''$ | |

**Solution** *Corrected Included Angles*

Sum of the observed included angles of the traverse

$$= 93°18'16'' + 74°16'24'' + 123°42'00'' + 68°41'16''$$

$$= 359°57'56''$$

Theoretical sum of included angles = $(2n - 4) \times 90° = (2 \times 4 - 4) \times 90° = 360°$

$$\text{Correction} = 360° - 359°57'56'' = 2'4''$$

A correction of $(2'4''/4) = 31''$ should be applied to each included angle.

Hence, corrected angles are:

$$25x + 11y + 21z - 100 = 0$$

Normal equation of z is $25x + 11y + 21z - 100 = 0$

**a)** The normal equation of an unknown quantity is formed by multiplying each equation by the algebraic co-efficient of that quanity in that equation and the weight of that equation, adding the result.

$3x + 3y + z - 4 = 0$ .......................................................(1)

$x + 2y + 2z - 6 = 0$ .................................................. (2)

$5x + y + 4z - 21 = 0$ ................................................. (3)

**Normal equation of x:**

In equation 1,2 and 3 the coefficients of x and weight of respective equation are (3 x 2), (1 x 3) and (5 x 1) respectively

Hence

$$18x + 18y + 6z - 24 = 0$$

$$3x + 6y + 6z - 18 = 0$$

$$25x + 5y + 20z - 105 = 0$$

$$46x + 29y + 32z - 147 = 0$$

Normal equation of x is $46x + 29y + 32z - 147 = 0$

**Normal equation of y:**

In equation 1,2 and 3 the coefficients of y and weight of respective equation are (3 x 2), (2 x 3) and (1 x 1) respectively

Hence

$$18x + 18y + 6z - 24 = 0$$

$$6x + 12y + 12z - 36 = 0$$

$$5x + y + 4z - 21 = 0$$

$$29x + 31y + 22z - 81 = 0$$

Normal equation of y is $29x + 31y + 22z - 81 = 0$

**Normal equation of z:**

In equation 1,2 and 3 the coefficients of z and weight of respective equation are (1 x 2), (2 x 3) and (4 x 1) respectively

Hence

$$6x + 6y + 2z - 8 = 0$$

$$6x + 12y + 12z - 36 = 0$$

$$20x + 4y + 16z - 84 = 0$$

$$32x + 22y + 30z - 128 = 0$$

Normal equation of y is $32x + 22y + 30z - 128 = 0$

## 3.1 ELECTRO-MAGNETIC DISTANCE MEASUREMENT (EDM)

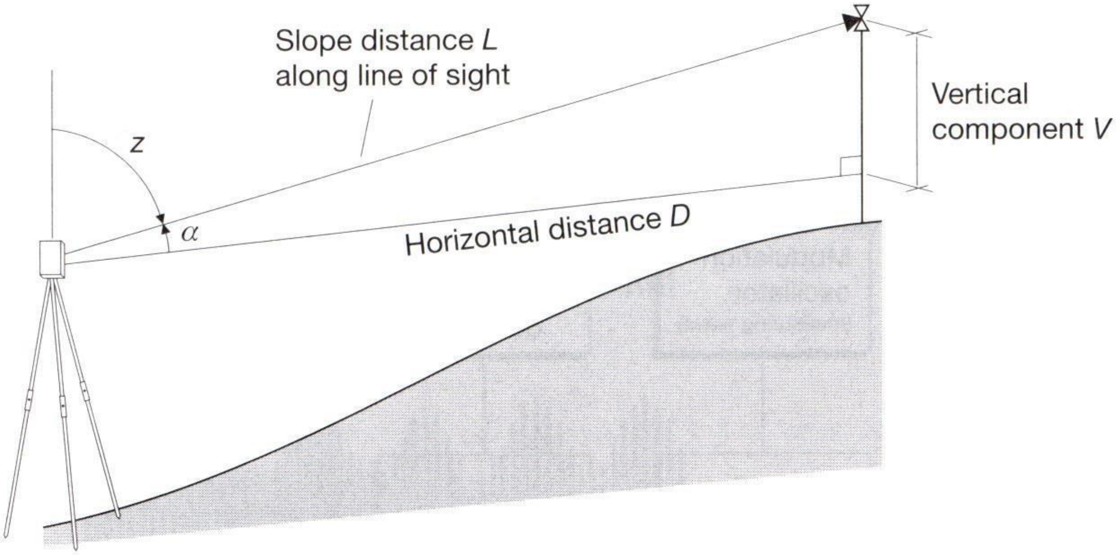
**Introduction:**

Electro-magneticdistancemeasurementisageneralterm usedcollectivelyin the measurementofdistances applyingelectronicmethods.BasicallytheEDM methodis based on generation, propagation, reflection and subsequent reception of electromagneticwaves.

**Fundamentals of EDM:**

Every EDM equipment should perform the following functions:

-  Generation of carrier wave and measuring wave frequencies.
-  Modulation and demodulation of the carrier wave.
-  Measurement of the phase difference between the transmitted and received measuring waves.
-  Display, in some form, the result of this measurement.




**Principle of working of the instrument:**

The basic principle of EDM instrument is the determination of time required for electro-magnetic waves to travel between two stations. here the velocity of electro-magnetic wave is the basis for computation of the distance.

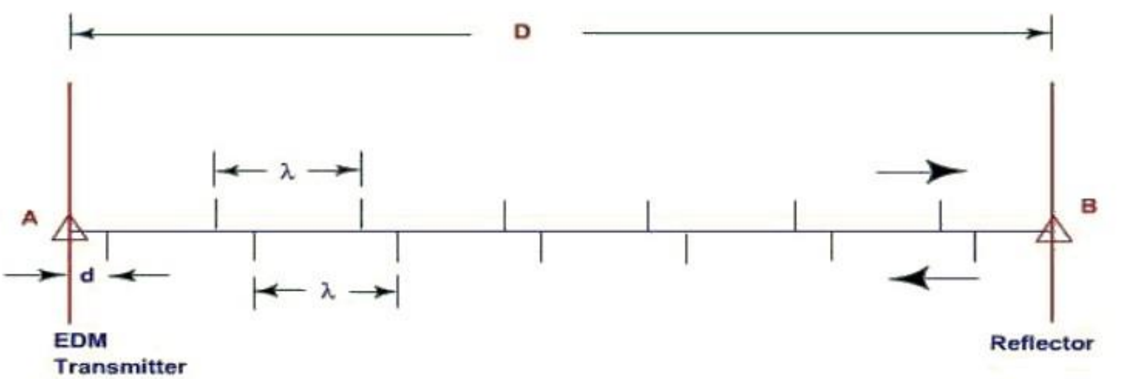



Figure 10.3 Measurement of distance using EDM

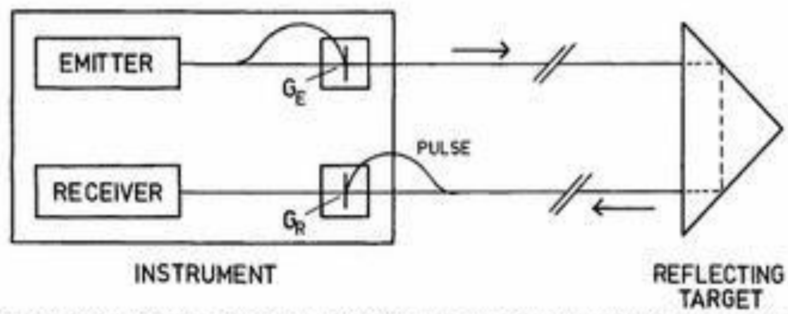


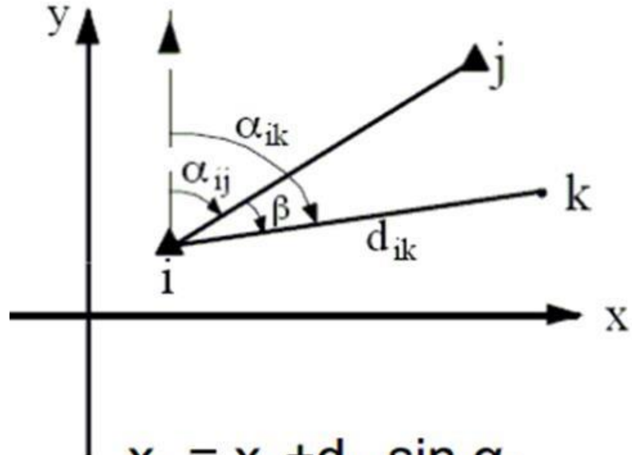

**Fig. 3.1.** Principle of a pulse distance meter. Timing starts and stops when the light pulse passes the emitter gate GE and the receiver gate GR, respectively

**Distance = velocity x time**

$$2d = c\Delta t'$$
$$= c(t_R - t_E) \ ,$$

where $d$ = distance between instrument and target
$c$ = velocity of light in the medium
$\Delta t'$ = flight time of pulse
$t_E$ = time of departure of pulse, timed by gate $G_E$
$t_R$ = time of arrival of returning pulse, timed by gate $G_R$.

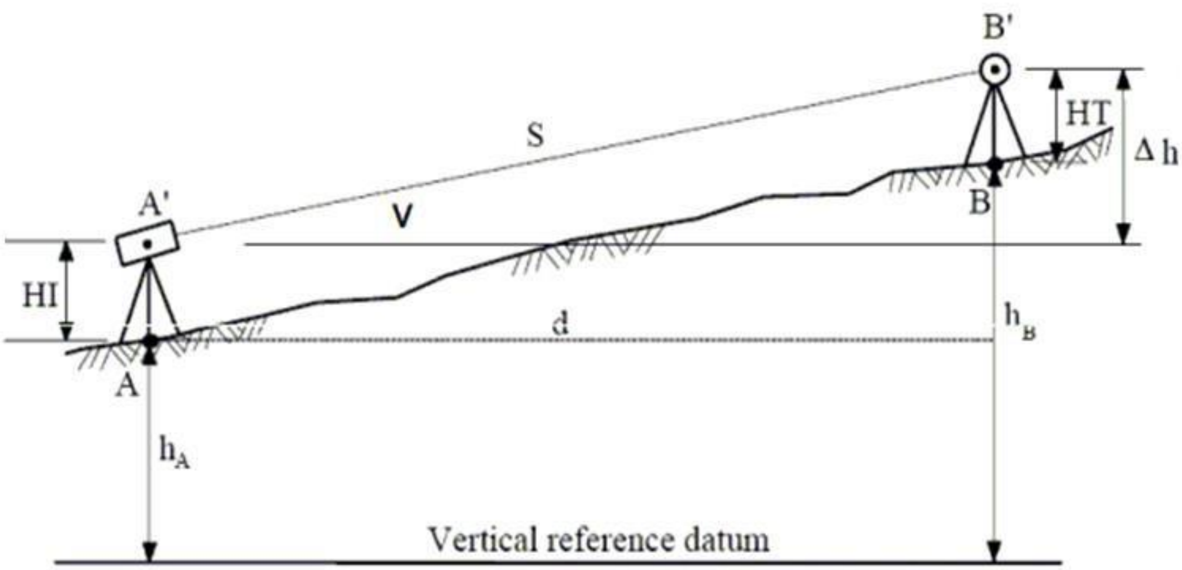
**Horizontal Component (x,y):**




$$x_k = x_i + d_{ik} \sin \alpha_{ik}$$

$$y_k = y_i + d_{ik} \cos \alpha_{ik}$$

**Vertical coordinate (z):**




$$h_B = h_A + HI + S \sin v - HI$$

## 3.15 COMPARISON BETWEEN ELECTRO OPTICAL AND MICROWAVE SYSTEMS

| S.no | Description | Electro optical | Microwave |
|---|---|---|---|
| 1. | Accuracy of instrument | a) Short range types 0.1 km to 5 km<br>5 mm + 5 ppm to 5 mm + 0.1 ppm<br>b) Long range type upto 70 km<br>5 mm + 0.1 ppm | For distance upto 150 km and depending on atmospheric refractive index<br>22 mm +5 ppm to 1 mm + 1 ppm |
| 2. | Operation principle | Set up EDM at one end of the line being measured and a reflector at the other end of the line. | Master unit transmits a series of modulated radio waves to remote antenna in the remote instrument. |
| | | EDM sends a modulated beam of light to the reflector | Remote instrument interprets these signals and sends them back to the antenna to the master unit. |
| | | Reflectors acts like a mirror and returns the light pulse back to EDM. | Master unit measure the time required for the radi waves to make the round trip. |
| | | EDM register readings that are converted into linear distance between the EDM and the reflector | Distance is computed on the velocity of the radio waves. |
| | | Requires on e operator. | Requires one operator at each end of the line. |

| | | | |
|---|---|---|---|
| 3. | Advantages | Less susceptible to atmospheric conditions | Can penetrate fog and rain. |
| | | Less expensive: only a single transmitter neded. | Longer range. |
| | | | Transmitter at both ends allow voice communication. |
| 4. | Diasdvantage | Shorter range | Atmospheric effects are greater. |
| | | | Susceptible to ground reflected signals |
| | | | More expensive: requires two transmitters. |

# UNIT 4

# ADVANCED TOPICS IN SURVEYING

**Hydrographic Surveying – Tides – MSL – Sounding methods – Three point problem – Strength of fix – astronomical Surveying – Field observations and determination of Azimuth by altitude and hour angle methods –.Astronomical terms and definitions - Motion of sun and stars - Celestial coordinate systems - different time systems - Nautical Almanac - Apparent altitude and corrections - Field observations and determination of time, longitude, latitude and azimuth by altitude and hour angle method**

## HYDROGRAPHIC SURVEYING

- Hydrographic survey is the science of measurement and description of features which affect maritime navigation, marine construction, dredging, offshore oil exploration/offshore oil drilling and related activities.
- Hydrographic surveying or bathymetric surveying is the survey of physical features present underwater. It is the science of measuring all factors beneath water that affect all the marine activities like dredging, marine constructions, offshore drilling etc.
- Hydrographic surveying is mainly conducted under authority concerns. It is mainly carried out by means of sensors, sounding or electronic sensor system for shallow water.
- The information obtained from hydrographic surveying is required to bring up nautical charts which involves,

i. Available depths

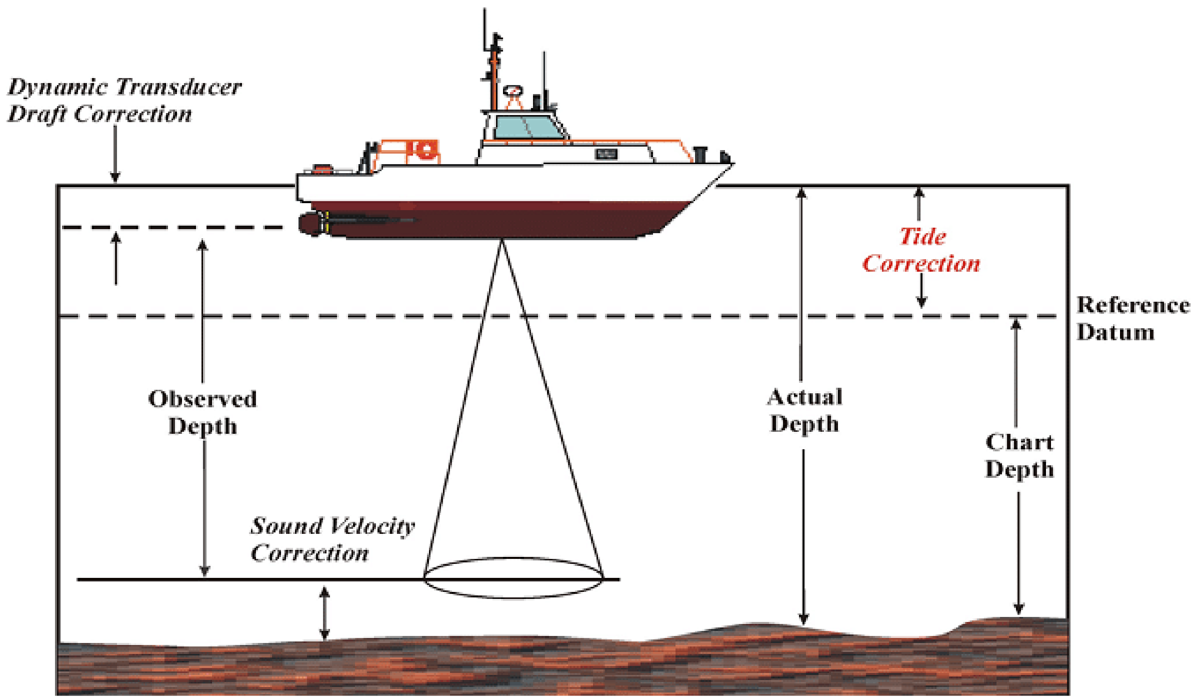
ii. Improved Channels

iii. Breakwaters

iv. Piers

v. The aids to navigation harbor facility

- These survey also take part in necessary data collection relating to construction and developments of port facilities, such as pier construction. This help in finding the loss in capacity due to silt and many uncertainties.




**Applications of Hydrographic Surveying**

Following are the applications of hydrographic surveying:

- ✓ Dock and Harbor Engineering
- ✓ Irrigation
- ✓ River Works
- ✓ Land reclamation
- ✓ Water Power
- ✓ Flood Control
- ✓ Sewage Disposal

**Uses of Hydrographic Surveying**

Uses of hydrographic surveying are given below:

1. Depth of the bed can be determined
2. Shore lines can be determined
3. Navigation Chart Preparation
4. Locate sewer fall by measuring direct currents
5. Locating mean sea level
6. Scouring, silting and irregularities of the bed can be identified

7. Tide measurement

8. River and stream discharge measurement

9. Massive structures like bridges, dams harbors are planned

**Preliminary Steps in Hydrographic Surveying**

- ✓ The method starts by locating special control points along the shore line. The sounding method is employed to determine the depth at various points by means of stationary boats.
- ✓ Sounding locations can be either made from boat to the control points or by fixing a point in the boat and taking sounding from the control point. Before this procedure certain preliminary steps have to be made:

1.Reconnaissance

2.Locate Horizontal Control

3.Locate vertical Control

**1.Reconnaissance**

• As every project require a start-up plan to complete it effectively and economically, reconnaissance has to be undergone. A complete reconnaissance of whole survey area to choose the best way of performing the survey.

This would facilitate satisfactory completion of the survey in accordance with the requirements and specifications governing such work. Aerial photographs would help this study.

**2.Locating Horizontal Control**

- ✓ • The horizontal control is necessary to locate all features of the land and marine in true relative
- ✓ positions. Hence a series of lines whose lengths and azimuths are determined by means of either
- ✓ triangulation or any other methods.
- ✓ • Tachometric and plane table survey can be conducted in order to undergo rough works. No rules are kept for establishing horizontal control as topography, vegetation, type, size of topography affect the rules.
- ✓ • But in general a rules can be kept for type of control say:
- ✓ • It is advisable to run traverses along each shore, connecting each other by frequent tie lines –If water body > 1km wide

- ✓ It is advisable to run transverse line only along one of the banks -If water body is narrow
- ✓ Triangulation system -If shorelines filled by vegetation
- ✓ Large network of triangulation system for large lakes and ocean shore lines
- ✓ A combined triangulation and traversing is shown in figure 1.

**3.LOCATING VERTICAL CONTROL**

• Before sounding establishment of vertical control is essential to determined. Numerous benchmarks are placed in order to serve as vertical control. Setting and checking the levels of the gauges are uses of benchmarks.

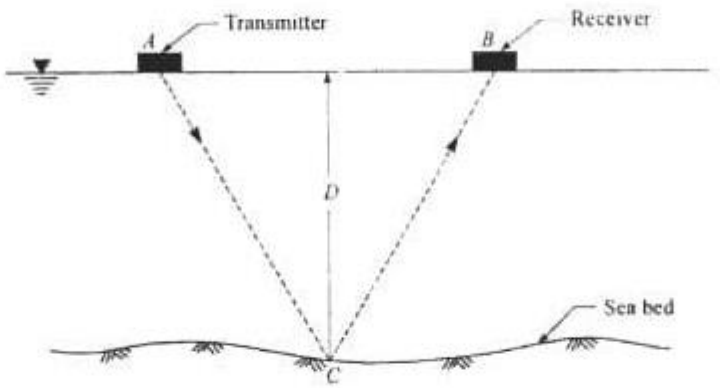



**Fig. 1: Combined Triangulation and Traversing in Hydrographic Survey**

**SOUNDING METHODS**

**SOUNDING IN HYDROGRAPHIC SURVEY**

• The process of determining depth below water surface is called as sounding. The step before undergoing sounding is determining the mean sea level.

• If the reduced level of any point of a water body is determined by subtracting the sounding from mean
sea level, hence it is analogous to levelling.

**Methods of Locating Soundings in Hydrographic Surveying**

• The soundings are located by the observations made from the boat or from the shore or from both.

• There are four methods are there to locate the soundings by:

1. Conning the survey vessel

2. Observations with theodolite or sextant

3. Theodolite angles and EDM distances from the shore

4. Microwave systems

**SOUNDING BY CONNING THE SURVEY VESSEL**

- In this method, conning means keeping the boat at known course. This method is suitable for rivers, open sea up to 5 km off shore. The markers are fixed on the shore called as ranges along which vessel or boat is run. This method is again sub divided into two types as follows.
- Location by cross rope
- Location by range and time interval

***Location by Cross Rope***

- In this method, a wire or rope with markings or tags at known distances is stretched across the channel. The starting point of rope at the shore is marked as reference point. Then using boat, the sounding at different distances of wire are determined by weighted pole.
- This method is more accurate. This is most suitable for rivers, narrow lakes and for harbors. This is also suitable for knowing the amount of material removed by dredging.

***Location by Range and Time Interval***

- In this method, the boat is positioned in range with two signals provided on the shore. Then, the boat is rowed at constant speed and time required to reach the instant of sounding is measured which gives the distance of total point along the range. This method is more suitable for less width channels or rivers. It is not so much accurate.

**SOUNDING BY OBSERVATIONS WITH THEODOLITE OR SEXTANT**

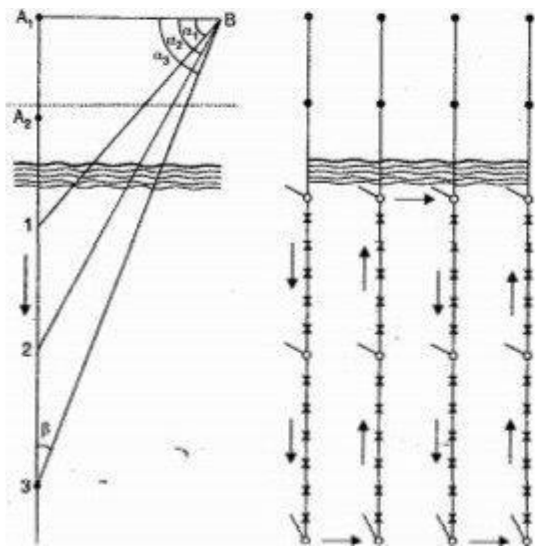
Theodolite or sextant is used to measure angles in surveying. In this method, the sounding is located by measuring angles. Here also, there are a lot of subdivided methods are there to locate sounding. They are

- By range and one angle from the shore
- By range and one angle from the boat
- By two angles form the shore
- By two angles from the boat
- By one angle from the shore and one angle from boat
- By intersecting angles
- By tachometry

***By Range and One Angle from the Shore***

• In this method, boat is kept in range line with the help of two signals on the shore. The boat is moved and the point where sounding is measuring is observed by the theodolite or sextant and angle is noted. Using this angle, we can fix the point in the range.

• Likewise, all other soundings are observed from different stations. The angle should be more than 30 degrees otherwise fix should be poor.so, whenever the angle is less than $30^{o}$, new instrument station is selected. This method is so accurate and easy for plotting the sounding details.

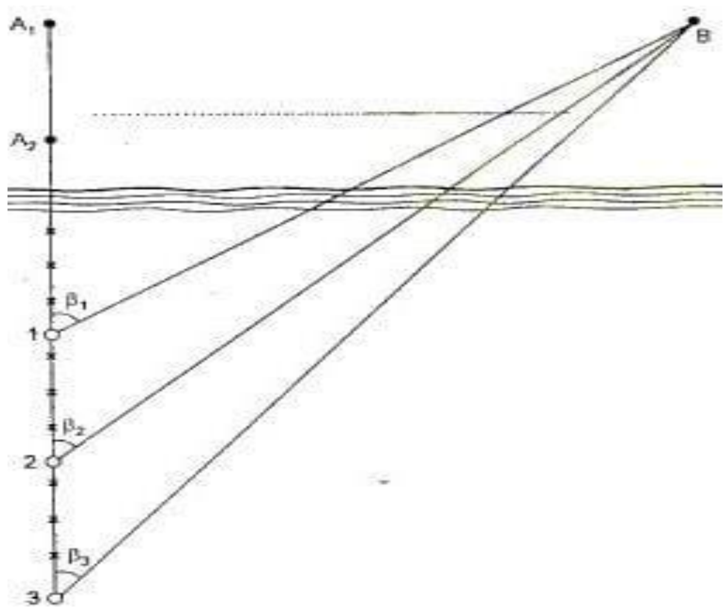



***By Range and One Angle from the Boat***

• This method is similar to the above method, but in this case, the angular measurements are taken from the boat to different stations positioned on the shore. This is also having similar accuracy to the above method.

• But, there are some advantages in this method as compared with above method. Angle measured from the shore from different stations is difficult when compared to angle observed from the boat to all stations.

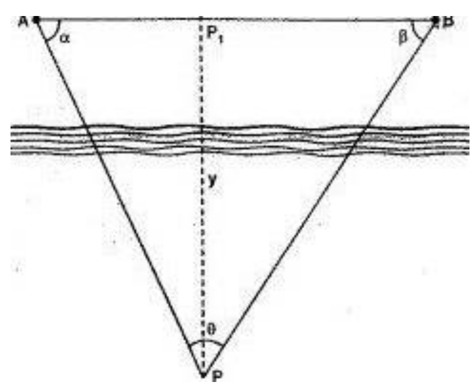
• So, the surveyor in this case has better control over the operations. Check can be made by measuring second angle towards some other signal on the shore for important fixes.




***By Two Angles form the Shore***

• In this method, two instrument stations are fixed on the shore with proper distance. Two instruments and two instrument men are required to do this job. From the two instrument stations, angular observations are made and a point is located where sounding is measured.

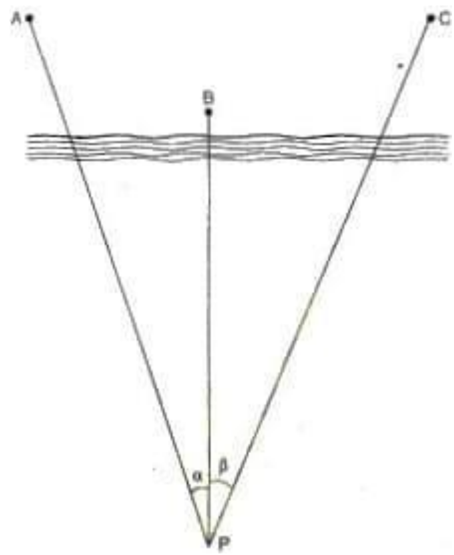
• If the angle made by instrument is less than $30^0$ then new instrument station is selected. In this case, primary setting out and erecting range signals are eliminated. This method is useful when water currents are strong and difficult to row the boat along range line.




***By Two Angles from the Boat***

• In this method, three constant points on the shore are selected. Using three-point problem, boat is positioned in range line and angles are observed from the boat to two of the three known positions.

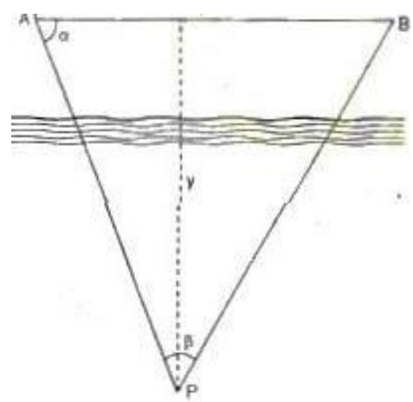
• The known positions may be light house, church spire, etc. like objects on the shore. If fixed positions are not available, then go for shore signals or ranging rods.




***By One Angle from the Shore and One Angle from Boat***

• This method also requires two instruments and two men to operate. This is the combination of above two methods. In this method, two instrument points are located on the shore and instrument is placed only at one point. Other instrument is placed in the boat.

• The first angle is measured from the first point on the shore to boat and from the boat second angle is measured from boat to second point. At that instant sounding is measured.

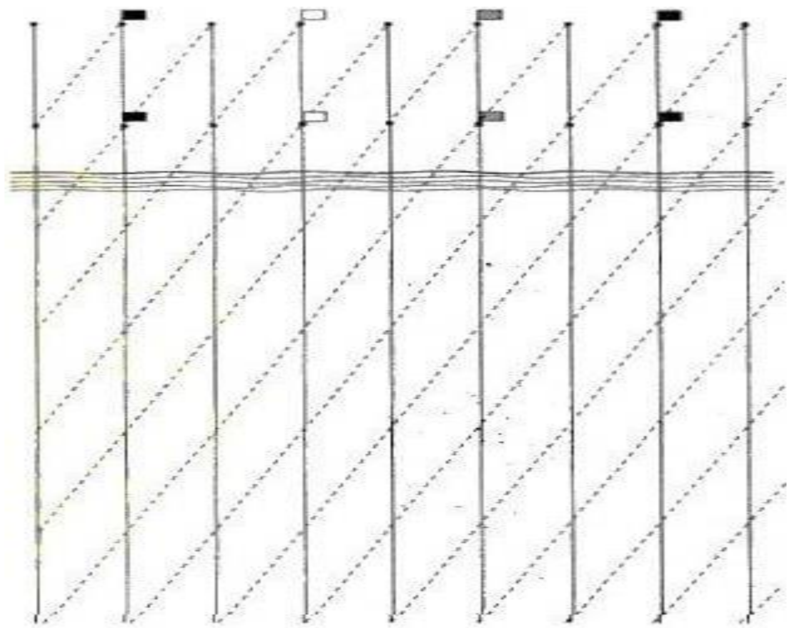



***By Intersecting Angles***

• In this method, sounding is determined periodically at same points. This method is essentially used for harbors, reservoirs etc. to know the amount of silting or scouring happened at that points.

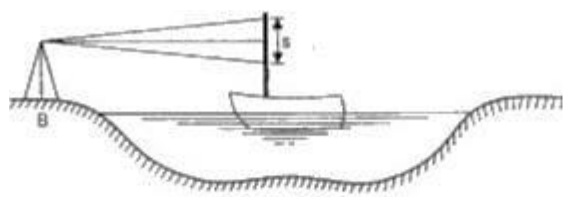
• Number of signals are erected on the shore and a boat is rowed perpendicular to the shore and measure the sounding at a point where inclined line of signal intersect the line of signal next to it as shown in fig.

• Flag rods are erected at sounding points to avoid confusion for the next round of measuring.

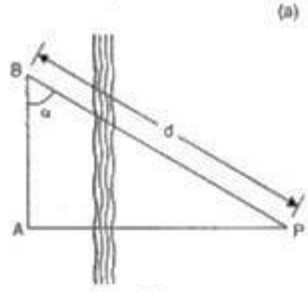


***By Tachometry***

• In this method, tachometer is placed on the shore and staff is placed on a boat. The staff intercept “s” is known by tachometer from this the distance between boat and instrument is known. This method is suitable when water is stable and sounding location is nearer to the shore.




(a)




(b)

**SOUNDING BY THEODOLITE ANGLES AND EDM DISTANCES FROM SHORE**

• In this method, EDM and Theodolite are placed on the shore in fixed positions. From this set up, the reflector placed on the boat is targeted and point of sounding is located.

• This method is more accurate when the water is still. This is one of the modern methods of fixing sounding vessel.

**SOUNDING BY MICROWAVE SYSTEMS**

• In this method, a device called Tellurometer is used which contains three units' namely master unit, remote unit and master antenna. Master unit is fixed to the boat and other two units are located on the shore at two shore stations.

• The distances are measured from boat to the shore stations using micro waves produced by tellurometer. Now from all these known distances the antenna produces the two sets of range information. Tellurometer is useful for distances up to 100km from the shore.

**The specific need for sounding are**

1. Preparation of navigation charts that is an all-time information for future purpose also
2. Material that to be dredged has to be determined early to facilitate easy movement in project without any confusion.
3. Material dredging should also accompany where filling has to be done. Material dumping is also measured
4. Design of backwaters, sea wells require detailed information that is obtained from sounding

**EQUIPMENT FOR SOUNDING**

The essential equipment used for undergoing sounding are

1. Shore signals and buoys
2. Sounding Equipment
3. Instruments for measuring angles

**1. SHORE SIGNAL AND BUOYS**

• These are required to mark the range lines. A line perpendicular to shore line obtained by line joining

2 or 3 signals in a straight line constitute the range line along which sounding has to be performed.

Angular observations can also be made from sounding boats by this method. To make it visible from

considerable distance in the sea it is made highly conspicuous.

• A float made of light wood or air tight vessel which is weighted at bottom kept vertical by anchoring

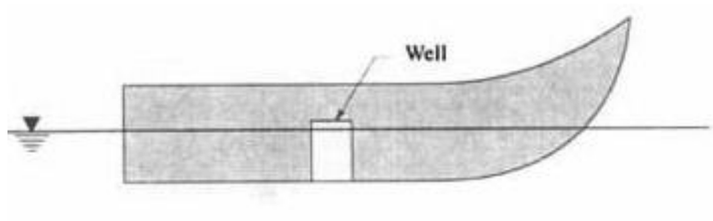
with guywires are called buoys. In order to accommodate a flag a hole is drilled. Under water deep, the
range lines are marked by shore signals & the buoys.

**2. SOUNDING EQUIPMENT**

The individual units involved are explained one by one:

**A. SOUNDING BOAT**

A flat bottom of low draft is used to carry out sounding operation. Large size boats with motor are used for sounding in sea. The soundings are taken through wells provided in the boat. A figure depicting sounding boat is shown in fig.2.




**B. SOUNDING POLE OR ROD**

Rod made of seasoned timber 5 to 10cm diameter and 5 to 8m length. A lead shoe of sufficient weight is connected at bottom to keep it vertical. Graduations are marked from bottom upwards. Hence readings on the rod corresponding to water surface is water depth.

**C. LEAD LINE**

A graduated rope made of chain connected to the lead or sinker of 5 to 10kg, depending on current strength and water depth. Due to deep and swift flowing water variation will be there from true depth hence a correction is required.

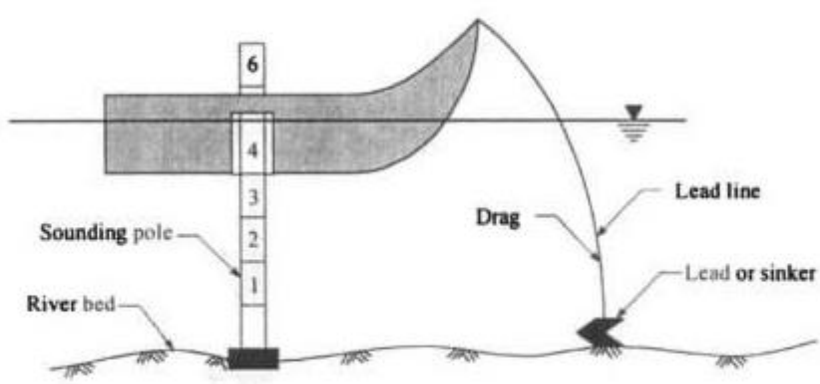



**Fig.3.Sounding Pole and Lead line**

• Other sounding equipment used are Weddell's sounding machine. These are employed when large sounding work has to be undergone. A standard machine to measure maximum of 30 to 40m is designed that are bolted over the well of the sounding boat.

• Another equipment used is fathometer which is an echo-sounding instrument used to determine ocean depth directly. Recording time of travel by sound waves is the principle employed. Here the time of travel from a point on the surface of the water to the bottom of the ocean and back is recorded.

• Knowing the velocity of sound waves the depth can be calculated as shown in fig.4.

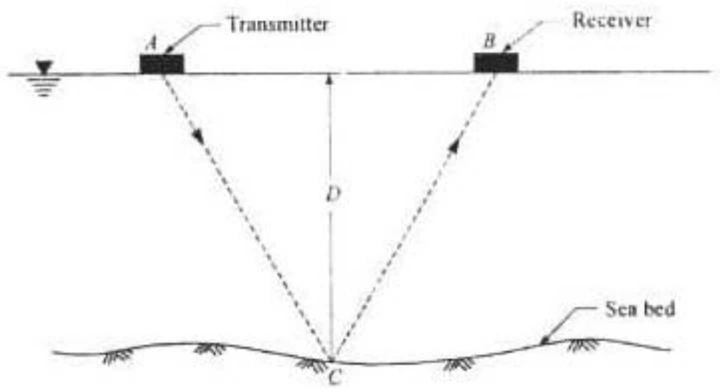



**Fig.4: Echo Sounding in Hydrographic Survey**

**TIDES**

• All celestial bodies exert a gravitational force on each other. These forces of attraction between earth and other celestial bodies (mainly moon and sun) cause periodical variations in the level of a water surface, commonly known as tides.

• There are several theories about the tides but none adequately explains all the phenomenon of tides.

However, the commonly used theory is after Newton, and is known as the equilibrium theory.

• According to this theory, aforce of attraction exists between two celestial bodies, acting in the straight line joining the centre of masses of the two bodies, and the magnitude of this force is proportional to the product of the masses of the bodies and is inversely proportional to the square of the distance between them.

• We shall apply this theory to the tides produced on earth due to the force of attraction between earth and moon. However, the following assumptions are made in the equilibrium theory :

1. The earth is covered all round by an ocean of uniform depth.

2. The ocean is capable of assuming instantaneously the equilibrium , required by the tide producing forces. This is possible if we neglect

(i) Inertia Of Water,

(ii) Viscosity Of Water,

(Iii) Force Of Attraction Between Parts Of Itself.

**TYPES OF TIDES:**

**i)LUNAR TIDE**

• Lunar Tide, also known as moon tide, is the tide caused in the sea due to the gravitational attraction caused by the moon. A tide is generally defined as the rise and fall in the level of the sea with respect to the land.The tides produced due to gravitational attraction caused by the sun are called solar tides.

ii)**THE SOLAR TIDES**

• The phenomenon of production of tides due to force of attraction between earth and sun is similar to the lunar tides.

• Thus, there will be superior solar tide and an inferior or anti-solar tide. However, sun is at a large distance from the earth and hence the tide producing force due to sun is much less.

• Solar tide = 0.458 Lunar tide.

• Combined effect : Spring and neap tides

• Solar tide = 0.458 Lunar tide.

• Above equation shows that the solar tide force is less than half the lunar tide force. However, their combined effect is important, specially at the new moon when both the sun and moon have the same celestial longitude, they cross a meridian at the same instant.

**MEAN SEA LEVEL**

• For all important surveys, the datum selected is the mean sea level at a certain place.

• The mean sea level may be defined as the mean level of the sea, obtained by taking the mean of all the height of the tide, as measured at hourly intervals over some stated period covering a whole number of complete tides.

• The mean sea level, defined above shows appreciable variations from day to day, from month to month and from year to year.

• Hence the period for which observations should be taken depends upon the purpose for which levels are required.

• The daily changes in the level of sea may be more. The monthly changes are more or less periodic. The mean sea level in particular month may be low while it may be high in some other moths.

• Mean sea level may also show appreciable variations in its annual values. Due to

• variations in the annual values and due to greater accuracy needed in modern geodetic

• levelling, it is essential to base the mean sea level on observations extending over a period of about 19 years.

• During this period, the moon's nodes complete one entire revolution. The height of mean sea level so determined is referred to the datum of tide gauge at which the observations are taken.

• The point or place at which these observations are taken is known as a tidal station. If the observations are taken on two stations, situated say at a distance of 200 to 500 kms on an open coast, one of the station is called primary tidal station while the other is called secondary tidal station.

• Both the stations may then be connected by a line of level.

**THREE POINT PROBLEM**

• In this method, three well defined points, having locations already being plotted on the drawing are involved. These are used to find and subsequently plot the location of the plane table station.

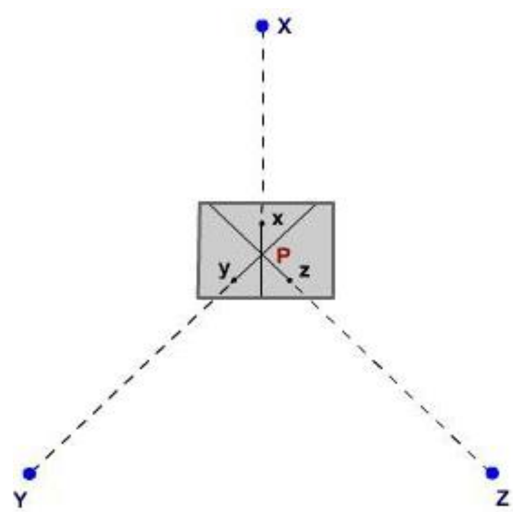



Figure 35.1 Principle of Three-point problem solution

• The method is based on the fact that, in a correctly oriented plane table, resectors through well defined points get intersected at a point which represents the location of the plane table station on the drawing .There are several methods for solution of the three point problem:

(i) trial and Error method,

(ii) mechanical method,

iii)Graphical method,

(iv)Analytical method and

(v) geometrical construction method.

Of these, the trial and error method is easy, quick and accurate. It is commonly used in practice andhence, has been discussed in detail.

• In three point problem, if the orientation of the plane table is not proper, the intersection of the resectors through the three points will not meet at a point but will form a triangle, known as triangleof error (Figure).

• The size of the triangle of error depends upon the amount of angular error in the orientation.

• The trial and error method of three point problem, also knon as Lehman's method minimises thetriangle of error to a point iteratively. The iterative operation consist of drawing of resectors fromknown points through their plotted position and the adjustment of orientation of the plane table.

• The estimation of location of the plane table depends on its position relative to the well defined points

considered for this purpose. Depending on their relative positions, three cases may arise :

(i) The position of plane table is inside the great triangle;

(ii) The position of plane table is outside the great triangle;

(iii) The position of plane table lies on or near the circumference of the great circle.

In case of (iii), the solution of the three-point problem becomes indeterminate or unstable. But for the cases (i) and (ii), Lehmann,s rules are used to estimate the location of plane table.

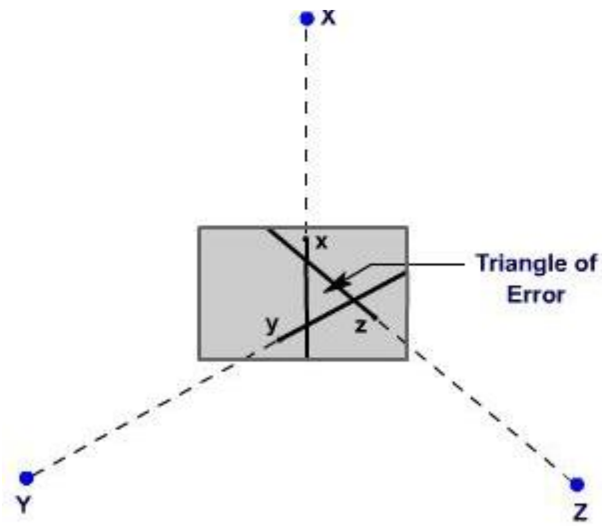



Figure 35.2 Triangle of Error

**STEPS FOR THREE POINT PROBLEM**

• Let X, Y, and Z represent the ground location of the well defined objects whose plotted positions are x, y, and z, respectively. Let P be the plane table station whose plotted position, say p, is to be determined.

(i) Select a plane table position inside the great triangle XYZ and set up the table over P and orient it by judgment so that apparent line xy is approximately parallel to the imaginary side XY.

(ii) Pivoting the alidade on x, y, and z bisect the signals placed at X, Y, and Z in turn and draw rays. If the orientation of the table is correct, the three rays will meet at one point which is the desired location of p on the sheet. If not, the rays will form a triangle of error

(iii)Choose a point p' inside the triangle of error such that its perpendicular distances from each ray is in proportion to the respective distances of P from the three ground objects. For selection of location of p', Lehmann's rules (1) and (3) need to be applied.

(iv) Align the alidade along p' x (assuming X to be the farthest station) rotate the table till flag at X is bisected, and clamp the table.

(v) Pivoting the alidade on x, y, and z repeat the process as in step (ii) above. If the estimation of p as p' is correct, the three rays will intersect at a point otherwise again a triangle of error will be formed but of smaller size and within the previous triangle of error. .

(vi) Estimate again the location of p' in the new triangle of error applying the rules, (i) and (iii), and repeat the steps (iv) and (v).

(vii) The method is repeated till all the three rays intersect at a point. The point of intersection is the required location p of the plane-table station P.

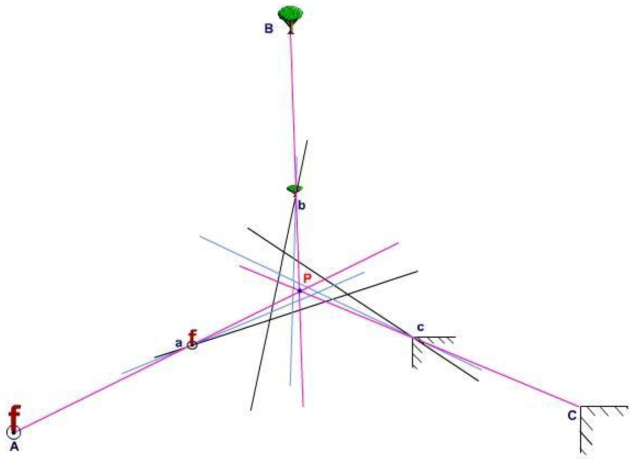



Figure 35.6 Solution of Three Point Problem

**STRENGTH OF FIX**

• The accuracy with which a plane table station can be located through three point problem is known as its fix.

• The degree of accuracy of solution of the three point problem is designated as its strength i.e., if the accuracy is high, the fix is termed as strong and for low accuracy, fix is called as poor.

• The accuracy of fix depends on the relative positions of the plotted points and that of location of theplane table station. Thus, the choice of plotted objects and location of table should be made to get astrong fix.The strength of fix is good if

• the location of station is chosen within the great triangle formed by joining the three well definedobjects X;the middle object is nearer to the position of the plane table than other two objects; of the two interior angles subtended by the three objects at the plane table stations, one is small and the other is large. However, the objects subtending small angle should be widely separated to each other. The strength of fix is poor if

• The location of the plane table is on or near the circumference of the great circle.

• Both the interior angles, subtended, by well defined objects, at the plane table stations, are small.

Figure 35.7 provides a pictorial representation of the quality of strength of figure with reference to the location of the three chosen objects.

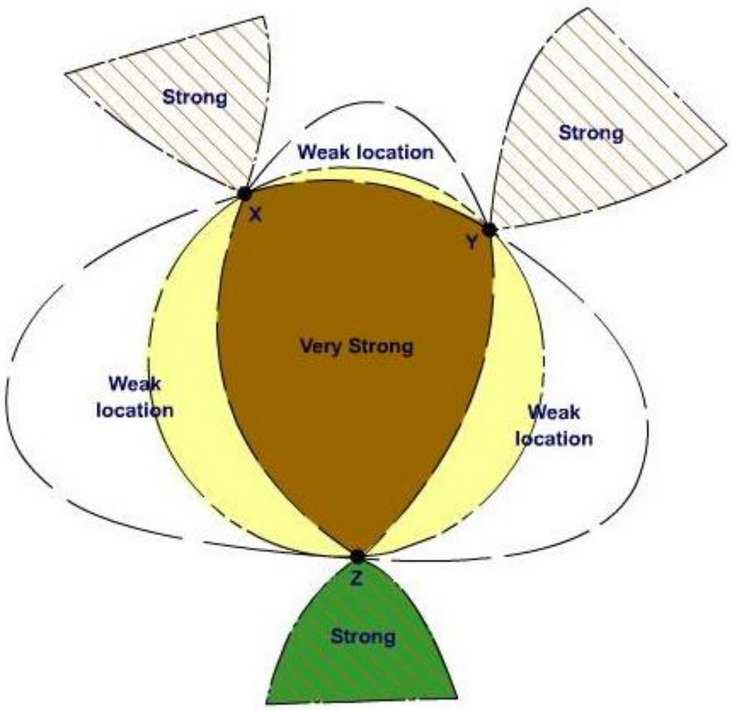


Figure 35.7 Qualitative presentation of strength of fix or figure

**ASTRONOMICAL SURVEYING**

An astronomical survey is a general map or image of a region of the sky which lacks a specific observational target. Alternatively, an astronomical survey may comprise a set of many images or spectra of objects which share a common type or feature.

## ASTRONOMICAL TERMS AND DEFINITIONS

• To observe the positions / direction and movement of the celestial bodies, an imaginary sphere of infinite radius is conceptualized having its centre at the centre of the earth. The stars are studded over the inner surface of the sphere and the earth is represented as a point at the centre.

## CELESTIAL SPHERE :

• An imaginary sphere of infinite radius with the earth at its centre and other celestial bodies studded on its inside surface is known as celestial sphere.

## GREAT CIRCLE (G.C) :

• The imaginary line of intersection of an infinite plane, passing through the centre of the earth and the circumference of the celestial sphere is known as great circle.

**ZENITH (Z) :**

• If a plumb line through an observer is extended upward, the imaginary point at which it appears to intersect the celestial sphere is known as Zenith. The imaginary point at which it appears to intersect downward in the celestial sphere is known as Nadir (N).

**VERTICAL CIRCLE :**

• Great circle passing through zenith and nadir is known as vertical circle.

**HORIZON:**

Great circle perpendicular to the line joining the Zenith and Nadir is known as horizon.

**POLES :**

• If the axis of rotation of the earth is imagined to be extended infinitely in both directions, the points at which it meets the celestial sphere are known as poles.

• The point of intersection in the northern hemisphere is known as north celestial pole and that in the southern hemisphere as south celestial pole.

**EQUATOR :**

• The line of intersection of an infinite plane passing through the centre of the earth and perpendicular to the line joining celestial poles with the celestial sphere.

**HOUR CIRCLE :**

• Great circle passing through celestial poles is known as hour circle, also known as declination circle.

**MERIDIAN :**

• The hour circle passing through observer's zenith and nadir is known as (observer's) meridian. It represents the North-South direction at observer station.

**ALTITUDE (H) :**

• The altitude of a celestial body is the angular distance measured along a vertical circle passing through the body. It is considered positive if the angle measured is above horizon and below horizon, it is considered as negative.

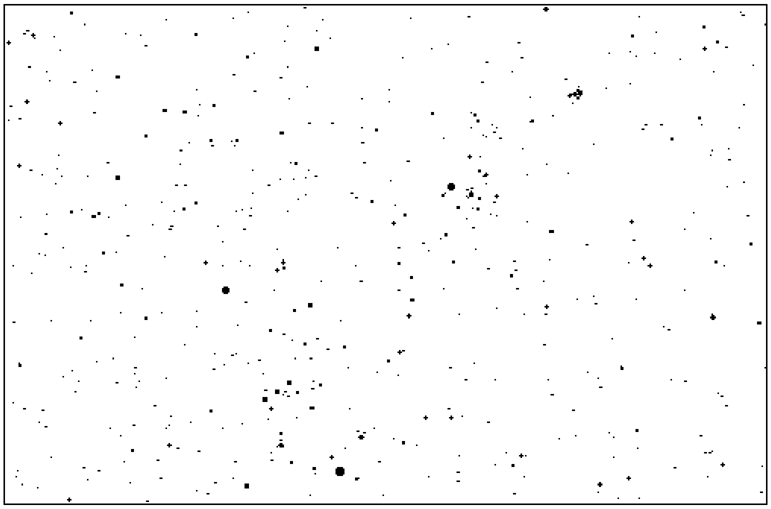
**AZIMUTH (A) :**

• The azimuth of a celestial body is the angular distance measured along the horizon from the observer's meridian to the foot of the vertical circle passing through the celestial body .

**MOTION OF SUN AND STARS**

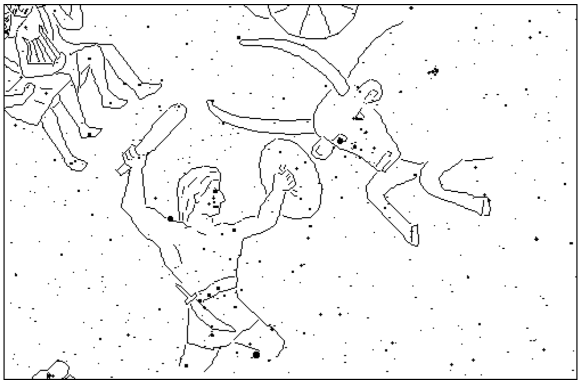
**The Stars**

- The human eye can see about 6000 stars without aid. The picture below covers a field of view of about 70° x 46°, roughly 5% of the entire sky.

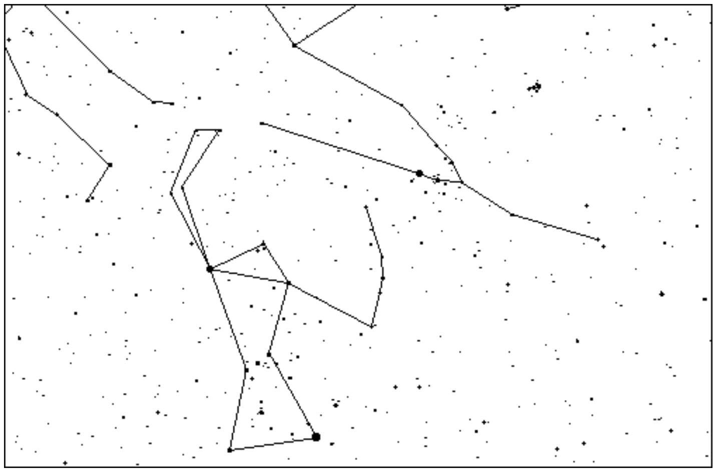


- The stars are grouped into **constellations**. Most of these are the same ones described by the ancient Greeks and Babylonians, although the southern hemisphere has many that were "created" by European explorers a few centuries ago.

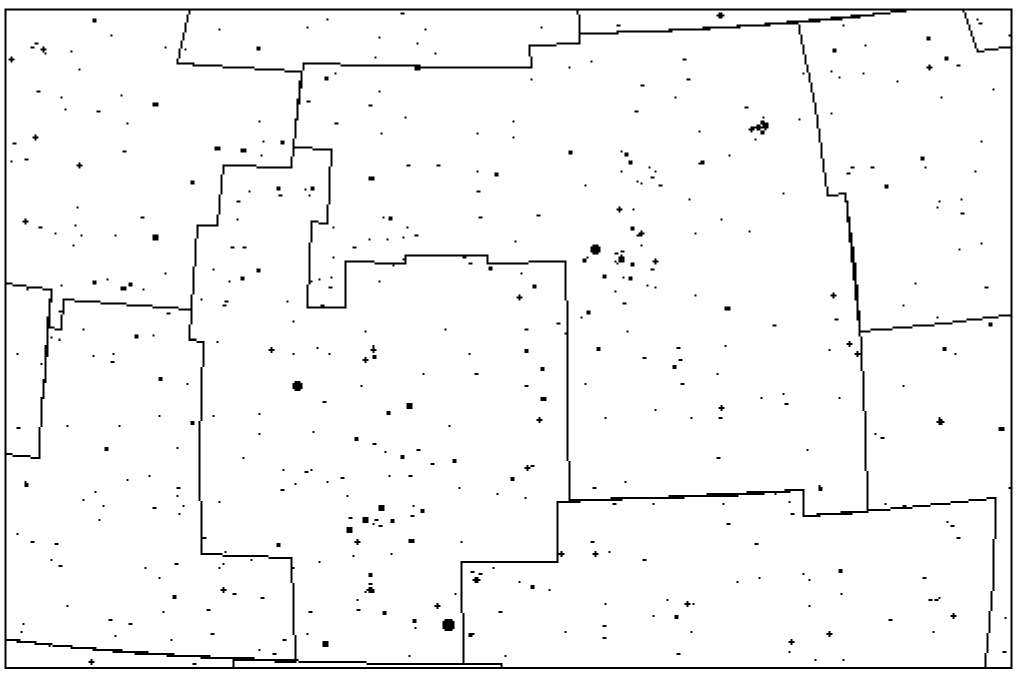
  The ancients thought of the constellations as representing mythical figures such as Orion the Hunter and Taurus the Bull.



- Nowadays we often think of constellations as "stick figures", consisting of lines connecting the major stars.



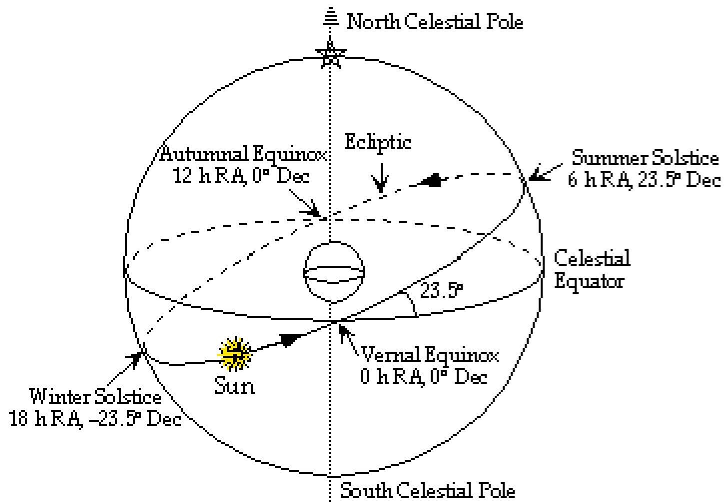
- These figures leave out many stars and other objects, including those that require telescopes to be seen. So, astronomers now think of a constellation as one of 88 *regions* that divide up the sky and completely cover it. Any object can now be said to lie in one constellation or another.



- Many stars have names from Arabic or Greek, e.g. Betelgeuse means "armpit" in Arabic. The stars are also given names such as "Alpha Orionis", using letters from the Greek alphabet followed by the constellation name. After that, numbers are typically used, e.g. "37 Orionis". This ordering is typically (but not always) according to brightness.

**The Motion of the Sun**

- Although the stars are fixed relative to each other, the Sun moves relative to the stars. Once a year, the Sun traces out a circle on the celestial sphere called the **ecliptic**.




  The ecliptic is tilted at an angle of 23.5° with respect to the celestial equator. (The Moon and planets also move near the ecliptic.)

- The Sun crosses the celestial equator at exactly two points, called **equinoxes**, from the Latin for "equal nights" (for reasons we'll see later)

- The equinox where the Sun ascends from the southern to the northern hemisphere is called the **spring** or **vernal equinox** because the Sun is there on March 21. The vernal equinox is chosen to be 0 h R.A.

- The Sun again crosses the celestial equator halfway around, at 12 h R.A. This position is called the **autumnal equinox** because the Sun is there on September 23.

- The positions where the Sun reaches its highest and lowest points are called **solstices**, from the Latin for "the Sun stops" as it changes direction.

- The Sun is highest in the sky (in the northern hemisphere) when it is at 6 h R.A. This position is called the **summer solstice** because the Sun is there on June 21. The Sun then has a declination of +23.5°.

- The Sun is lowest in the sky (in the northern hemisphere) when it is at 18 h R.A. This position is called the **winter solstice** because the Sun is there on December 21.

The Sun then has a declination of -23.5°.

**DIFFERENT TIME SYSTEM (Universal Time: UTC, GMT, UT1)**

To measure time independently of one's location on the Earth, we use so-called "Universal Time". There are a number of variations on the theme of universal time, each appropriate in its own context:

- **UT0** is universal time as measured using the daily motions of stars or extragalactic radio sources. It is rarely used because it is not truly universal as it does not take into account changes in the displacement of the earth's poles ("polar motion"), and so different observatories will measure different UT0 intervals, even though they are looking at the same objects.
- **UT1** is UT0 corrected for polar motion, and so is the same everywhere on earth. It changes slowly over time because of the steady slowdown in the earth's rotation rate, and the irregular jitter in the rotation rate due to motions in its fluid interior which amount to about +/-3 milliseconds per day. **UT1 is effectively the same as Greenwich Mean Time (GMT) as it was defined before 1961**. Older scientific papers will often refer to "UT" or "GMT" interchangeably. This is not correct today.
- **UTC** is Coordinated Universal Time (a compromise acronym between the english CUT and the french TUC). UTC is measured using atomic clocks, and is the basis of all civil timekeeping on Earth. It is kept to within +/-0.9 seconds of UT1 by adding leap seconds to UTC every few years. **All civil atomic clocks and computer time services provide time in UTC.**

**International Atomic Clock Time (TAI)**

The SI unit of time is the **second**, defined as 9,192,631,770 cycles of a hyperfine transition in the ground state of $^{133}$Cs. Thus the "second" is defined as "time measured by atomic clocks." This in turn defines the **International Atomic Time** system or TAI (Temps Atomique Internationale), which is based on statistical measurements of a large number of atomic clocks in the US and Europe. TAI, however, is not the ultimate in time systems in a strict rigorous sense. There are others that could provide better metrics for time measurment, but they have not proven practical to implement. In general, for times most astronomers will encounter (i.e., measurements orders of magnitude less exacting than pulsar timing observations), TAI is effectively (if not precisely) the "ultimate" time system.

UTC is measured by correcting TAI by an integer number of leap seconds to account for the changes in the Earth's rotation rate. Thus TAI is essentially "UTC without the leap seconds". The last leap second was inserted on 2005 December 31, so that since that date the offset between UTC and TAI is 33 seconds, hence:

UTC = TAI - 33s

That is, TAI is currently "ahead of" UTC by 33 seconds.

UTC was introduced in 1961. Between 1961 and 1971, UTC attempted to track changes in the Earth's rotation rate by introducing "elastic seconds", wherein the carrier frequency and tick duration of time signal radio broadcasts was adjusted annually to estimate how fast the Earth would rotate during that year. The accumulated discrepacies between this best guess and the actual measurements were corrected for in small steps of 50 to 100 msec that were applied annually. This system was a codification of schemes which had already been in use by various national laboratories and their radio broadcast time signal services for a number of years. Thus, UTC is "Coordinated" in the sense that the correction steps were determined and implemented by international agreement, instead of locally.

In 1972, an arguably simplified method was introduced whereby while leap seconds would be introduced more or less annually to take into account changes in the rotation rate of the Earth. This method is the one described above. Some in the IAU argued at the time that it should no longer be called "UTC" because of the considerable change in the rotation-correction algorithm, but in the end UTC it remains to this day. Thus far, leap seconds have always been positive, but there is no reason negative leap seconds won't be required in the future as the change in the Earth's rotation is not monotonic.

**Julian Dates (JD & MJD)**

The problem with conventional civil calendar dates is that calendrical conventions are unwieldy for measuring long intervals of time. To count the total number of days or hours that has elapsed between two observations, for example, requires that you keep track of how many days there are in each intervening month, accounting for leap years, etc. To circumvent these convoluted calculations, astronomers have adopted an absolute day count system based on the Julian calendar (the calendar in use before the Gregorian Calendar Reform of 1582 - we astronomers just love the "classics").

The **Julian Day** number (**JD**) is the count of the number days that have elapsed since Greenwich Mean Noon on 1 January -4712 (4713 BC) in the Julian Proleptic Calendar. Julian Days start at noon, unlike UTC Gregorian Calendar days which start at midnight. The somewhat unusual starting date derives from the Julian Period of 7980 Julian Years of 365.25 days each. The Julian Period is the time interval between coincidences of the 28-year Solar Cycle, the 19-year Lunar Cycle, and the 15-year Roman Indiction (a tax cycle). The starting date is the last time all three cycles were coincident. This system provides astronomers with a way to measure secular time differences over long time spans without having to be concerned with getting the vagaries of the calendar correct.

An alternative to JD is the **Modified Julian Date (MJD)**, an abbreviated, 5-digit version of the Julian Date defined as:

MJD = JD - 2400000.5

where 0.5 days is subtracted so as to have MJD start a midnight (i.e., aligning it with the civil time reckoning convention), and the 2400000 is used to reduce the 7-digit day number of JD to a more tractable 5 digits. This definition of MJD is officially recognized by the IAU, the ITU (International Telecommunications Union), and the CCIR (Consultative Committee for Radio).

In practice, both JD and MJD should be referenced to UTC, and in most astronomical research papers where it is not stated explicitly it is usually a safe assumption that UTC is implicit. However, there is no explicit mandate from the IAU (or anyone else) to define JD and MJD in terms of UTC, so it is a good idea to state your time system explicitly when using it in research publications or in data sets made available to others.

One often encounters a 4-digit version of the Modified Julian Date computed relative to JD2450000.0 instead of the internationally mandated JD2400000.5. This non-standard practice has been formally deprecated by the IAU and other international time-keeping organizations, and should not be used. Similarly there are other, even more idiosyncratic definitions of MJD in use, and you should not use any of those, either.

### Heliocentric Julian Date

The **Heliocentric Julian Date (HJD)** is the Julian Date adjusted to the center of the Sun. It depends on the JD of the observation and the celestial coordinates (usually RA and DEC) of the object. HJD takes into account the light-travel time for an event coming from a particular location on the sky to be observed at the center of the Sun. The intent of HJD is to make a first-order accounting of the periodic "paralactic" time shift due to where the Earth was in its orbit when an event was observed. This is important for long-term secular studies where timing data will extend over either a large fraction of a year or many years. The correction can amount to as much as about 16 minutes for observations taken 6 months apart.

A more accurate way to express this is relative to the dynamical center-of-mass ("barycenter") of the Solar System, since the Sun also moves relative to the barycenter. In fact, use of HJD is formally deprecated by the IAU. Despite this, HJD continues to be common in the literature, while one rarely encounters a Barycentric Julian Date or its equivalent outside of fairly specialized papers. Why? One practical reason is that the relevant time systems for use in a Barycentric correction are not simply related to UTC and related terrestrial time systems. The calculation is non-trivial, and it does not help matters that the IAU has not followed up its deprecation of HJD with a set of standardized, portable, robust, and readily available algorithms for accomplishing the conversion. It also does not help that the relevant literature is nearly impenetrable to all but specialists, reflecting both the considerable technical complexities of the problem and the deep philosophical and political divisions prevailing among those working on time systems.

## GPS Time

Arguably the most convenient precision time reference available for astronomers (or almost anything else) is from the **Global Positioning System** or **GPS**. The Navstar Global Positioning System (GPS) is a satellite-based global radio-navigation system developed, deployed, and operated by the United States Department of Defense. GPS consists of a constellation of 24 or more satellites that orbit the Earth every 12 hours in six orbital planes (nominally 4 spacecraft per orbit) spaced 60 degrees apart and inclined 55 degrees relative to the equator. Between 5 and 8 GPS satellites are visible from every point on the Earth at any given moment.

GPS offers two modes:

- ✓ The **Standard Positioning Service** (SPS) provides positional accuracy (95%) of 100m horizontally and 156 meters vertically, and time accuracy (95%) of 340 nanoseconds, depending upon the receiver.
- ✓ The **Precise Positioning Service** (PPS) provides a positional accuracy (95%) of 22 meters horizontally and 27.7 meters vertically, and time accuracy (95%) of 200 nanoseconds. PPS used to be restricted to military users via "Selective Availability" encryption that globally degraded the time signals from the GPS constellation.

On 2000 May 1 then President William Clinton signed an executive order turning off Selective Availablility for good. This gives civilian users (including astronomers) access to the full precision of the GPS system, making the GPS system the most accurate globally-accessible time clock available to date.

**Differential GPS** (DGPS) methods are employed when higher positional precision is required. Applications of DGPS include safety-critical navigation (ships and aircraft), precision surveying, and geodynamics work. If you can wait to make multiple measurements, sub-meter (centimeter or even millimeter) precision is achievable using linked receivers and sophisticated statistical analysis methods (the OSU  is the local expert on DGPS methods).

However, care must be taken when using GPS as an astronomical time reference. Unlike UTC, GPS is **NOT** adjusted for leap seconds. GPS time was last synchronized with UTC on 1980 January 6, and as of 2006 January 01, GPS is ahead of UTC by 14 seconds. Each GPS satellite transmits the UTC correction parameters as part of the navigational data stream, and most good GPS receivers include software to make the conversion for you transparently. Care should be taken to verify that you are indeed reading the UTC time and not the GPS time when you query your GPS receiver for "the time"

## Nautical Almanac

A nautical almanac is a publication describing the positions of a selection of celestial bodies for the purpose of enabling navigators to use celestial navigation to determine the position of their ship while at sea.

The Almanac specifies for each whole hour of the year the position on the Earth’s surface (in declination and Greenwich hour angle) at which the sun, moon, planets and first point of Aries is directly overhead. The positions of 57 selected stars are specified relative to the first point of Aries.

**Contents of Nautical Almanac:-**

The Nautical Almanac contains tabulations of the Sun, Moon, navigational planets and stars for use in the determination of position at sea from sextant observations. In addition, it gives times of sunrise, sunset, twilights, moonrise and moonset, phases of the Moon and eclipses of the Sun and Moon for use in the planning of observations. All the necessary interpolation and altitude correction tables are provided as well as pole star tables and diagrams and notes for the identification of stars and planets. Information on standard times for most countries around the world is provided.

**FIELD OBSERVATIONS AND DETERMINATION OF AZIMUTH BY ALTITUDE AND HOUR ANGLE METHODS**

## DETERMINATION OF AZIMUTH OF A CELESTIAL BODY

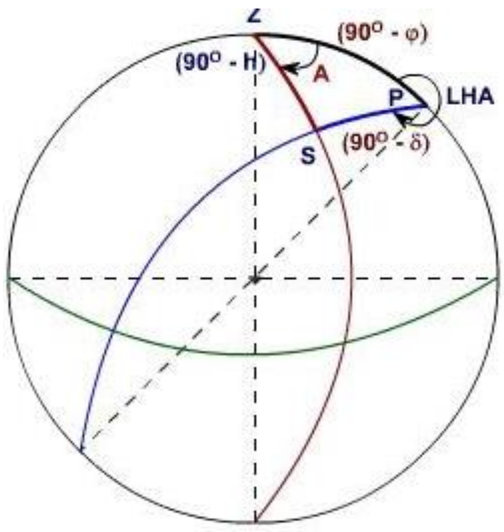
- In field astronomy, a celestial body provides the reference direction. So, from the geographic location (latitude and longitude) of the station, ephemeris data of celestial body and either time or altitude ofthe same celestial body, the azimuth of the celestial body is computed by solving astronomicaltriangle.
- If time is used, the procedure is known as the hour-angle method. Likewise, if altitude is measured, the procedure is termed as the altitude method.
- The basic difference between these two methods is that the altitude method requires observation of approximate time and an accurate vertical angle of the celestial body, whereas the hour angle method requires observation of accurate time.
- Recent developments of time receivers and accurate timepieces, particularly digital watches with split - time features, and time modules for calculators, the hour-angle method is more accurate, faster. It requires shorter training for proficiency.
- It has fewer restrictions on time of day and geographic location and thus is more versatile. The method is applicable to the sun, Polaris, and other stars. Consequently, the hour-angle method is emphasized, and its use by surveyors is encouraged.

**HOUR ANGLE METHOD**

• In this method, precise time is being noted when the considered celestial body is being bisected. The observed time is used to derive the hour angle and declination of the celestial body at the instant of observation.

• The geographic position (latitude and longitude) of the observation station is required to be known a prior for the hour angle method. Usually, these values are readily obtained from available maps.

However, to achieve better accuracy, latitude and longitude must be more accurately determined specially during observations for celestial bodies close to the equator--e.g., the sun-than for bodies near the pole--e.g., Polaris.




• The declination of the celestial bodies at the instant of observation is required to be known for computation of azimuth of the celestial body. It is available in star almanac at the 0, 6, 12 and 18 hours of UTI of each day ( Greenwich date). Thus, the declination at the instant of observation (of celestial body) is determined by linear interpolation for corresponding the UT1 time of observation. However, since the declination of the sun varies rapidly, its interpolation is done using the relation:

• Declination, d = Decl $0_h$ + (Decl $24_h$ - Decl $0_h$) ( ) + (0.0000395) (Decl $0_h$) sin (7.5 UT1) -----------

--(Equation 26.2)

• The hour-angle of the celestial body is being derived using the GHA (available in star almanac with reference to Greenwich date) and the longitude of the observation station. For observations in the Western Hemisphere, if UTI is greater than local time, the Greenwich date is the same as local date and if UTI is less than local time, Greenwich date is the local date plus one day.

• For the Eastern Hemisphere, if UTI is less than local time (24-hr basis), Greenwich date is the same as local date and if UTI is greater than local time, Greenwich date is local date minus one day. The hour angle of the celestial body at the observation station is the LHA.

• Thus, it is the LHA at UTI time of observation which is necessary to compute the azimuth of a celestial body. Hence, as can be seen from Figure 26.3, the equation for the LHA is

LHA = GHA - W l (west longitude) ----------------- (Equation 26.3)

Or LHA = GHA + E l (east longitude) -------------------(Equation 26.4)

• LHA should be normalized to between 0° and 360° by adding or subtracting 360°, if necessary.

• The Greenwich hour angle (GHA) of celestial bodies-the sun, Polaris, and selected stars-is tabulated in star almanac from 0 hr to 24 hr at an interval of 6 hours of UTI time of each day (Greenwich date). Thus, to find GHA at the time of observation linear interpolation is required to be performed.

• The GHA can also be derived by making use of the equation of time E (apparent time minus mean time) by using the relation:

GHA = 180° + 15 E ---------------------------Equation (26.5)

• where E is in decimal hours. In those cases where E is listed as mean time minus apparent time, the algebraic sign of E should be reversed.

• Once the parameters (declination and Hour angle of the celestial body, latitude of the observation station) required to compute the azimuth of the celestial body are available, the computation of azimuth of the celestial body is carried out using the relations of astronomical triangle.

**AZIMUTH OF A LINE**

- ✓ Azimuth of a line is its horizontal angle measured clockwise from geographic or true meridian.
- ✓ For field observation, the most stable and retraceable reference is geographic north. Geographic north is based on the direction of gravity (vertical) and axis of rotation of the earth.

- ✓ A direction determined from celestial observations results in astronomic (Geographic) north reference meridian and is known as geographic or true meridian.
- ✓ The azimuth of a line is determined from the azimuth of a celestial body. For this, the horizontal angle between the line and the line of sight to the celestial body is required to be observed during astronomic observation along with other celestial observation.

**CELESTIAL COORDINATE SYSTEM**

- In astronomy, a celestial coordinate system is a system for specifying positions of celestial objects: satellites, planets, stars, galaxies, and so on.
- Coordinate systems can specify an object's position in three-dimensional space or plot merely its direction on a celestial sphere, if the object's distance is unknown or trivial.
- The coordinate systems are implemented in either spherical or rectangular coordinates. Spherical coordinates, projected on the celestial sphere, are analogous to the geographic coordinate system used on the surface of Earth.
- These differ in their choice of fundamental plane, which divides the celestial sphere into two equal hemispheres along a great circle.
- Rectangular coordinates, in appropriate units, are simply the cartesian equivalent of the spherical coordinates, with the same fundamental (x, y) plane and primary (x-axis) direction. Each coordinate system is named after its choice of fundamental plane.

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# *UNIT V MODERN SURVEYING*

**Total Station : Advantages - Fundamental quantities measured - Parts and accessories - working principle - On board calculations - Field procedure - Errors and Good practices in using Total Station GPS Surveying : Different segments - space, control and user segments - satellite configuration - signal structure - Orbit determination and representation - Anti Spoofing and Selective Availability - Task of control segment - Hand Held and Geodetic receivers - data processing - Traversing and triangulation.**

## TOTAL STATION

- ✓ **Total station** is a surveying equipment combination of Electromagnetic Distance Measuring Instrument and electronic theodolite.
- ✓ It is also integrated with microprocessor, electronic data collector and storage system. The instrument can be used to measure horizontal and vertical angles as well as sloping distance of object to the instrument.

## BASIC PRINCIPLE

Although taping and theodolites are used regularly on site – total stations are also used extensively in surveying, civil engineering and construction because they can measure both distances and angles.

## ADVANTAGES OF USING TOTAL STATIONS

The following are some of the major advantages of using total station over the conventional surveying instruments:

1. Field work is carried out very fast.
2. Accuracy of measurement is high.
3. Manual errors involved in reading and recording are eliminated.
4. Calculation of coordinates is very fast and accurate. Even corrections for temperature and pressure are automatically made.
5. Computers can be employed for map making and plotting contour and cross-sections. Contour intervals and scales can be changed in no time. However, surveyor should check the working condition of the instruments before using. For this standard points may be located near survey office and before taking out instrument for field work, its working is checked by observing those standard points from the specified instrument station.

**FUNDAMENTAL QUANTITIES MEASURED**

**Capability of a total station**

Microprocessor unit in total station processes the data collected to compute:

i. Average of multiple angles measured.

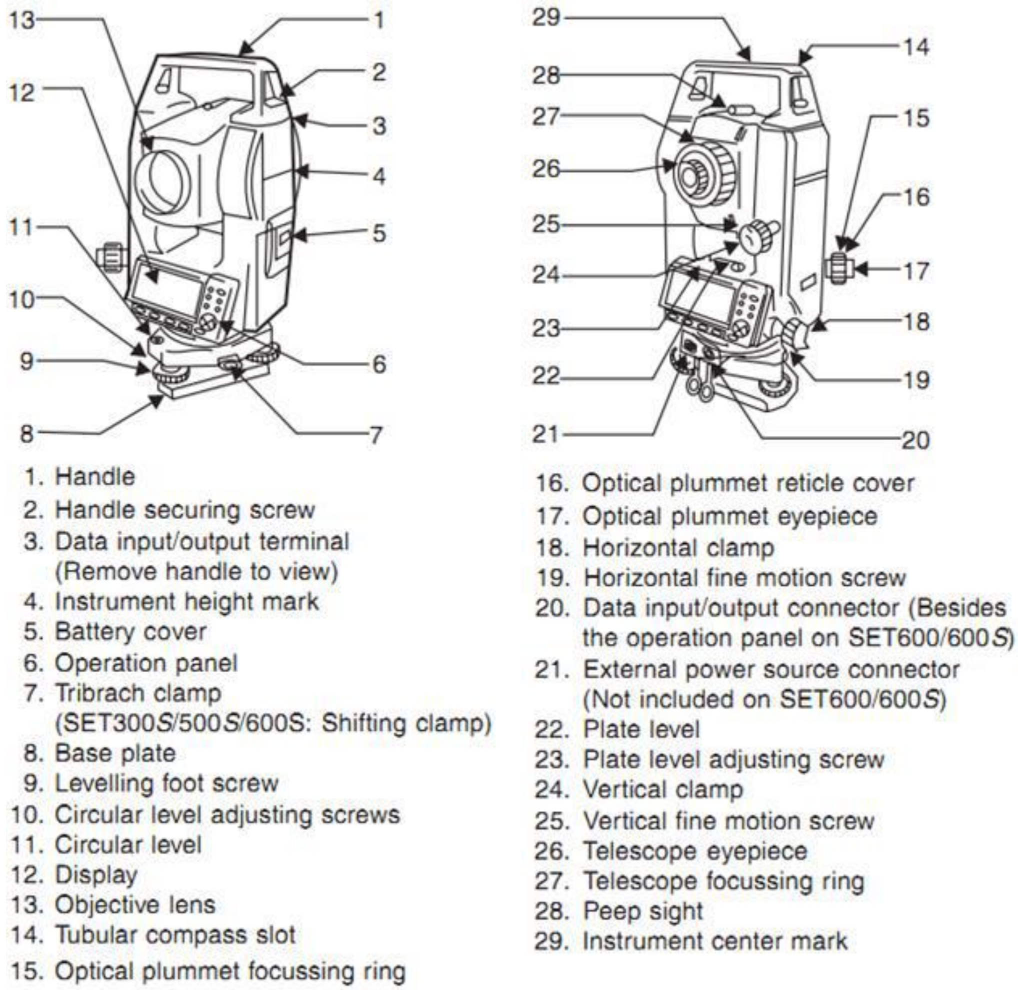
ii. Average of multiple distance measured.

iii. Horizontal distance.

iv. Distance between any two points.

v. Elevation of objects and

vi. All the three coordinates of the observed points.




**IMPORTANT OPERATIONS OF TOTAL STATION DISTANCE MEASUREMENT**

- ✓ Electronic distance measuring (EDM) instrument is a major part of total station. Its range varies from 2.8 km to 4.2 km.

- ✓ The accuracy of measurement varies from 5 mm to 10 mm per km measurement. They are used with automatic target recognizer. The distance measured is always sloping distance from instrument to the object.

**ANGLE MEASUREMENTS**

- ✓ The electronic theodolite part of total station is used for measuring vertical and horizontal angle. For measurement of horizontal angles any convenient direction may be taken as reference direction.
- ✓ For vertical angle measurement vertical upward (zenith) direction is taken as reference direction. The accuracy of angle measurement varies from 2 to 6 seconds.

**DATA PROCESSING**

- ✓ This instrument is provided with an inbuilt microprocessor. The microprocessor averages multiple observations.
- ✓ With the help of slope distance and vertical and horizontal angles measured, when height of axis of instrument and targets are supplied, the microprocessor computes the horizontal distance and X, Y, Z coordinates.
- ✓ The processor is capable of applying temperature and pressure corrections to the measurements, if atmospheric temperature and pressures are supplied.

**DISPLAY**

- ✓ Electronic display unit is capable of displaying various values when respective keys are pressed.
- ✓ The system is capable of displaying horizontal distance, vertical distance, horizontal and vertical angles, difference in elevations of two observed points and all the three coordinates of the observed points.

**ELECTRONIC BOOK**

- ✓ Each point data can be stored in an electronic note book (like compact disc).
- ✓ The capacity of electronic note book varies from 2000 points to 4000 points data. Surveyor can unload the data stored in note book to computer and reuse the note book.

**USES OF TOTAL STATION**

- ✓ The total station instrument is mounted on a tripod and is levelled by operating levelling screws. Within a small range instrument is capable of adjusting itself to the level position. Then vertical and horizontal reference directions are indexed using onboard keys.
- ✓ It is possible to set required units for distance, temperature and pressure (FPS or SI). Surveyor can select measurement mode like fine, coarse, single or repeated.

- ✓ When target is sighted, horizontal and vertical angles as well as sloping distances are measured and by pressing appropriate keys they are recorded along with point number. Heights of instrument and targets can be keyed in after measuring them with tapes. Then processor computes various information about the point and displays on screen.
- ✓ This information is also stored in the electronic notebook. At the end of the day or whenever electronic note book is full, the information stored is downloaded to computers.
- ✓ The point data downloaded to the computer can be used for further processing. There are software like auto civil and auto plotter clubbed with AutoCad which can be used for plotting contours at any specified interval and for plotting cross-section along any specified line.

**OPERATIONS INVOLVED WHILE USING TOTAL STATIONS :**

1. Establishing the site Datum:

a) Selecting the site Datum

b) Establishing North

2. Setting up the Total station:

a) Placing and leveling Tripod on Datum

b) Placing and leveling the Gun on Tripod

c) Linking the data connector to Gun

3. Data collector options and setting

a) Main menu

b) Basic settings

4. Creating and Operating Job files:

a) Creating a new Job file

b) Opening an existing file

5. Shooting points

a) Identifying the important points to shoot

b) shooting points

c) Shooting additional points

d) Noting the special features

6.Post Processing

– Data down loading,conversion

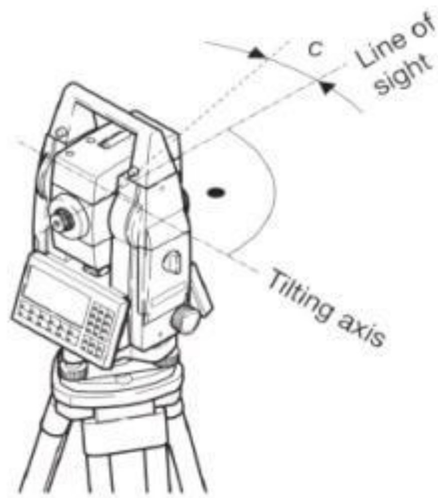
7.Plotting/Map generation.

**ERRORS AND GOOD PRACTICES IN USING TOTAL STATION GPS SURVEYING**

**TOTAL STATION ERRORS**

**1. HORIZONTAL COLLIMATION OR LINE OF SIGHT ERROR**

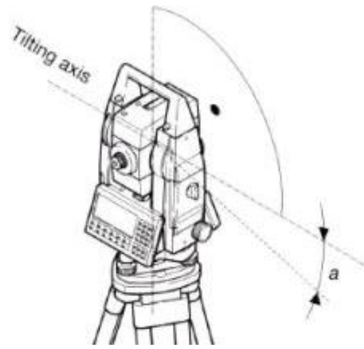
- ✓ Horizontal collimation or line of sight error is when the line of sight is not perpendicular to the tilting axis of the instrument. This is an axial error.
- ✓ Line of sight error effects the horizontal angle readings and increases with steep sightings. The error can be overcome or eliminated by observing on two faces.
- ✓ For single face measurements, an on-board calibration function is used to determine the deviation (c)of actual line of sight and deviated line of sight. The on-board software then apply a correction for each measured horizontal angles reading automatically.




The catch is here if the deviation of line of sight from actual line of sight exceeds more than a desired value, the instrument must be send to service centre or manufacturer for manual calibration.

**2. TILTING AXIS ERROR OR TILT ERROR**

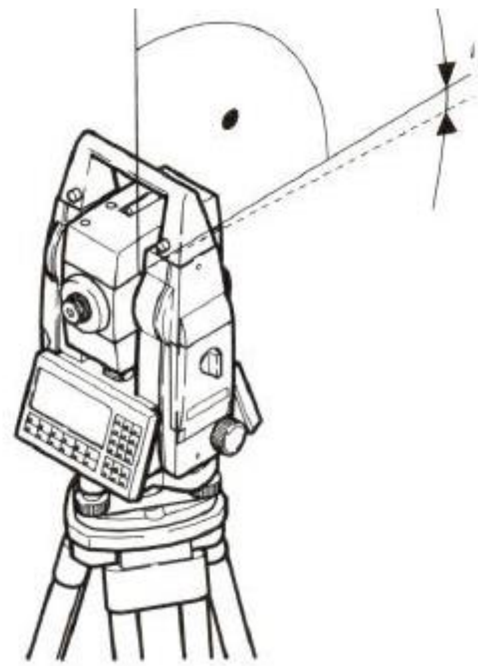
- ✓ Tilting axis or tilt error is the error when the axis to the total station is not perpendicular to the vertical axis or plumb line. The error effect on horizontal readings when the instrument is tilted (steep sightings) but have no effect on sightings taken when the instrument is horizontal.
- ✓ Like horizontal collimation error the tilting error can be eliminated by two face measurement. Another method is to apply the measured tilting error at the time of calibration process for all readings.




If the tilt error is more than the specified error for instrument, must be send to calibration lab.

**3. VERTICAL COLLIMATION ERROR OR VERTICAL INDEX ERROR**

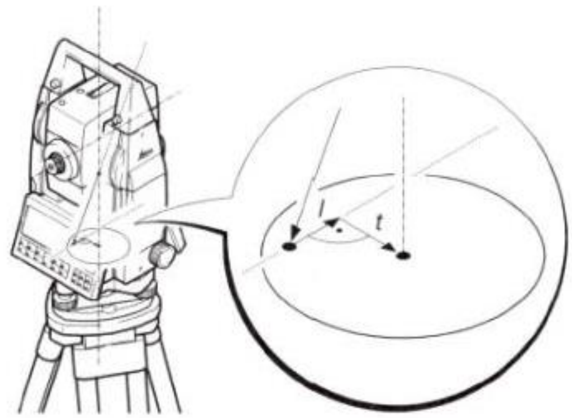
✓ If the horizontal base line of angle from 0° to 180° in the vertical circle does not coincide with the vertical axis of instrument. This zero point error is present in all vertical circle readings and like the horizontal collimation error, it is eliminated by taking FL and FR readings or by determining.



**4. COMPENSATOR INDEX ERROR**

✓ This error is caused by not leveling the total station correctly and carefully. This error can't be eliminated by taking two face (face left and face right) readings unlike the horizontal collimation error.

✓ If the instrument is fitted with a compensator it will measure residual tilts of the instrument and will apply corrections to the horizontal and vertical angles for these.



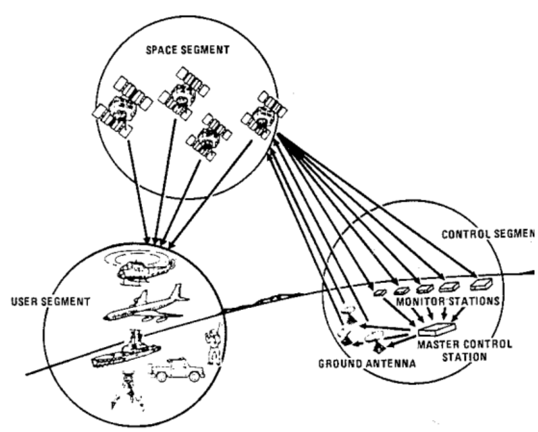

However all compensators will have a longitudinal error l and traverse error t known as zero point errors. These are averaged using face left and face right readings but for single face readings must be determined by the calibration function of the total station.

**DIFFERENT SEGMENTS - SPACE, CONTROL AND USER SEGMENTS**

**SEGMENTS OF GPS**

- ✓ For better understanding of GPS, we normally consider three major segments viz.
- ✓ space segment, Control segment and User segment. Space segment deals with GPS satellites systems, Control segment describes ground based time and orbit control prediction and in User segment various types of existing GPS receiver and its application is dealt .

GLONASS (Global Navigation & Surveying System) a similar system to GPS is being developed by former Soviet Union and it is considered to be a valuable complementary system to GPS for future Application.



**SPACE SEGMENT**

- ✓ Space segment will consist 21 GPS satellites with an addition of 3 active spares. These satellites are placed in almost six circular orbits with an inclination of 55 degree.
- ✓ Orbital height of these satellites is about 20,200 kmcorresponding to about 26,600 km from the semi major axis.
- ✓ Orbital period is exactly 12 hours of sidereal time and this provides repeated satellite configuration every day advanced by four minutes with respect to universal time.

**CONTROL SEGMENT**

Control segment is the vital link in GPS technology.Main functions of the control segment.

- Monitoring and controlling the satellite system continuously
- Determine GPS systemtime
- Predict the satellite ephemeris and the behavior of each satellite clock.
- Update periodically the navigation message for each particular satellite.

**USER SEGMENT**

Appropriate GPS receivers are required to receive signal from GPS satellites for the purpose of navigation or positioning. Since, GPS is still in its development phase,many rapid advancements have completely eliminated bulky first generation user equipments and now miniature powerful models are frequently appearing in the market.

**SATELLITE CONFIGURATIONS**

The satellite configuration specifies the GlobalProtect LSVPN configuration settings to deploy to the connecting satellites. You must define at least one satellite configuration.

**SIGNAL STRUCTURE**

**OBSERVATION PRINCIPLE AND SIGNAL STRUCTURE**

- ✓ NAVSTAR GPS is a one-way ranging system i.e. signals are only transmitted by the satellite. Signal travel time between the satellite and the receiver is observed and the range distance is calculated through the knowledge of signal propagation velocity.
- ✓ One way ranging means that a clock reading at the transmitted antenna is compared with a clock reading at the receiver antenna. But since the two clocks are not strictly synchronized, the observed signal travel time is biased with systematic synchronization error.
- ✓ Biased ranges are known as pseudoranges. Simultaneous observations of four pseudoranges are necessary to determine X, Y, Z coordinates of user antenna and clock bias.
- ✓ Real time positioning through GPS signals is possible by modulating carrier frequency with Pseudorandom Noise (PRN) codes.

- ✓ These are sequence of binary values (zeros and ones or +1 and -1) having random character but identifiable distinctly. Thus pseudoranges are derived from travel time of an identified PRN signal code.
- ✓ Two different codes viz. P-code and C/A code are in use. P means precision or protected and C/A means clear/acquisition or coarse acquisition.

**STRUCTURE OF THE GPS NAVIGATION DATA**

- ✓ Structure of GPS navigation data (message) is shown in F The user has to decode the data signal to get access to the navigation data.
- ✓ For on line navigation purposes, the internal processor within the receiver does the decoding. Most of the manufacturers of GPS receiver provide decoding software for post processing purposes. With a bit rate of 50 bps and a cycle time of 30 seconds, the total information content of a navigationdata set is 1500 bits.
- ✓ The complete data frame is subdivided into five subframes of sixsecond duration comprising 300 bits of information. Each subframe contains the data words of 30 bits each. Six of these are control bits.

The first two words of each subframe are the Telemetry Work (TLM) and the C/A-P-Code Hand over Work (HOW). The TLM work contains a synchronization pattern, which facilitates the access to the navigation data. Since GPS is a military navigation system of US, a limited access to the total system accuracy is made available to the civilian users.

**ORBIT DETERMINATION AND REPRESENTATION**

**ORBIT DETERMINATION:**

• Orbit Determination is the process to estimate the position and velocity (state vector) of a satellite at specific epoch based on models of the forces acting on the satellite, integration of satellite orbital motion equations and measurements to the satellites.

• Orbit Determination (OD) is generally divided into two categories:

i. preliminary orbit determination

ii. precise orbit determination (POD**).**

**PRELIMINARY ORBIT DETERMINATION**

• Preliminary Orbit Determination is a geometric method to estimate orbit elements from a minimal set of observations before the orbit is known from other sources.

• Traditionally, and still typically used, ground-based satellite observations of angles, distance or velocity measurements, which depend on the satellite's motion with respect to the centre of the Earth.

**PRECISE ORBIT DETERMINATION**

• Precise Orbit Determination is a dynamic, or combined geometric and dynamic method, a process completed with two distinct procedures: orbit integration and orbit improvement.

• Orbit integration yields a nominal orbit trajectory, while orbit improvement estimates the epoch state with all the measurements collected over the data arc in a batch estimation process .

**ORBIT REPRESENTATION**

• Orbit Representation is a means of representing a satellite orbit as a continuous trajectory with discrete observation data at the time of interest.

• The simplest orbit representation is the “osculating Keplerian elements” method, which describes an orbit as an ellipse.

• The most typical example is the satellite almanacs published by NASA for almost all spacecrafts in orbit. Figure 1.1 illustrates the concepts of the Keplerian elements with respect to the earth-centred inertial coordinate system.

**ANTI SPOOFING AND SELECTIVE AVAILABILITY**

**ANTI-SPOOFING**

✓ The function of anti-spoofing (AS) of the GPS system is designed for an anti potential spoofer (or jammer). A spoofer generates a signal that mimics the GPS signal and attempts to cause the receiver to track the wrong signal.

✓ When the AS mode of operation is activated, the P code will be replaced with a secure Y code available only to authorised users, and the unauthorised receiver becomes a single L1 frequency receiver. AS had been tested frequently since 1 August 1992 and formally activated at 00:00 UT on 31 January 1994 and now is in continuous operation on all Block II and later satellites.

✓ The broadcasted ionospheric model (in the navigation message) may be used to overcome the problem of absence of the dual-frequencies, which are originally implemented for eliminating the ionospheric effects. Of course, the method of using the ionospheric model cannot be as accurate as the method of using dual-frequencies data, and consequently the precision is degraded. Carrier phase smoothed C/A code may be used to replace the absence of the P code.

**SELECTIVE AVAILABILITY**

✓ Selective availability (SA) is a degradation of the GPS signal with the objective to deny full position and velocity accuracy to unauthorised users by dithering the satellite clock and manipulating the ephemerides.

✓ In case SA is on, the fundament frequency of the satellite clock is dithered, so that the GPS measurements are affected. The broadcast ephemerides are manipulated so that the computed orbit will have slow variations. Several levels of SA effects are possible. The SA is enabled on Block II and later satellites (Graas and Braasch 1996).

- ✓ **The authorised users may recover the** un-degraded data and exploit the full system potential. For doing so they must possess a key that allows them to decrypt correction data transmitted in the navigation message (Georgiadou and Daucet 1990).
- ✓ For high-precision users, IGS precise orbit and forecast orbit data may be used. Using known positions (or monitor stations), the range corrections can be computed. Differential GPS may also eliminate at least a part of the SA effects. SA has been switched off since May 2000.
- ✓

**HANDHELD GPS RECEIVERS**

- ✓ Handheld GPS receivers are used for absolute positioning or for relative positioning using DGPSservices or WAAS/EGNOS signals. The positioning is realized using code pseudo-ranges.
- ✓ Moreover it is well known that some of the handheld receivers use phase-smoothed code for positioning. This means that the phase signal is available and may be used for Precise Differential GPS (PDGPS) positioning.

**GEODETIC RECEIVER**

- ✓ Geodetic GPS receivers have the advantage of providing additionally the P-code observations, with a noise level smaller than the noise on the C/A code.
- ✓ The accuracy requirements of geodetic receivers are usually about 1-5 cm (or even better).