CE3351 SURVEYING AND LEVELLING L T P C 3 0 0 3

OBJECTIVES:

- To introduce the rudiments of plane surveying and geodetic principles to Civil Engineers and to learn the various methods of plane and geodetic surveying to solve the real world problems.
- To introduce the concepts of Control Surveying. To introduce the basics of Astronomical Surveying

UNIT I FUNDAMENTALS OF CONVENTIONAL SURVEYING 9

Definition – Classifications – Basic principles – Equipment and accessories for ranging and chaining – Methods of ranging – Well conditioned triangles – Chain traversing – Compass – Basic principles – Types – Bearing – System and conversions – Sources of errors and Local attraction – Magnetic declination – Dip – compass traversing – Plane table and its accessories – Merits and demerits – Radiation – Intersection – Resection – Plane table traversing.

UNIT II LEVELLING 9

Level line – Horizontal line – Datum – Benchmarks – Levels and staves – Temporary and permanent adjustments – Methods of leveling – Fly leveling – Check leveling – Procedure in leveling – Booking – Reduction – Curvature and refraction – Reciprocal leveling – Precise leveling - Contouring.

UNIT III THEODOLITE SURVEYING 9

Horizontal and vertical angle measurements – Temporary and permanent adjustments – Heights and distances – Tacheometric surveying – Stadia Tacheometry – Tangential Tacheometry – Trigonometric leveling – Single Plane method – Double Plane method.

UNIT IV CONTROL SURVEYING AND ADJUSTMENT 9

Horizontal and vertical control – Methods – Triangulation – Traversing – Gale‘s table – Trilateration – Concepts of measurements and errors – Error propagation and Linearization – Adjustment methods - Least square methods – Angles, lengths and levelling network.

UNIT V MODERN SURVEYING 9

Total Station: Digital Theodolite, EDM, Electronic field book – Advantages – Parts and accessories – Working principle – Observables – Errors - COGO functions – Field procedure and applications.GPS: Advantages – System components – Signal structure – Selective availability and antispoofing receiver components and antenna – Planning and data acquisition – Data processing – Errors inGPS – Field procedure and applications.

TOTAL 45 PERIODS

OUTCOMES:

On completion of the course, the student is expected to

- CO1 Introduce the rudiments of various surveying and its principles.
- CO2 Imparts knowledge in computation of levels of terrain and ground features
- CO3 Imparts concepts of Theodolite Surveying for complex surveying operations
- CO4 Understand the procedure for establishing horizontal and vertical control
- CO5 Imparts the knowledge on modern surveying instruments

TEXTBOOKS:

1. Dr. B. C. Punmia, Ashok K. Jain and Arun K Jain, Surveying Vol. I & II, Lakshmi Publications Pvt Ltd, New Delhi, Sixteenth Edition, 2016.
2. 2. T. P. Kanetkarand S. V. Kulkarni, Surveying and Levelling, Parts 1 & 2, Pune Vidyarthi Griha Prakashan, Pune, 2008.

REFERENCES:

1. R. Subramanian, Surveying and Levelling, Oxford University Press, Second Edition, 2012.
2. James M. Anderson and Edward M. Mikhail, Surveying, Theory and Practice, Seventh Edition, Mc Graw Hill 2001.
3. Bannister and S. Raymond, Surveying, Seventh Edition, Longman 2004.
4. S. K. Roy, Fundamentals of Surveying, Second Edition, Prenticeˆ Hall of India2010.
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CE3351 SURVEYING AND LEVELLING LTPC 3003

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Surveying:-

surveying is defined as the art of determining the relative positions of points on, above or below the earth surface. ie, horizontal, vertical distances, angles & directions. Thus the measurements could be either direct or indirect.

objectives of survey (or) purpose of survey:-

* To prepare plan or map of an area
* To determine the heights of objects in a vertical plane.
* To fix the control points & thus establish the boundaries.
* To prepare navigational chart.
* To set out the engineering works such as roads, building, dam, bridge, railways etc.
* To prepare astronomical charts.

plan: When the area surveyed is small & the scale to which its result plotted is large, then it is known as plan

Map: When the area surveyed is large and the scale to which its result plotted is small then it is called as a map.

Classification of surveying :-

* Primary classification
* Based on purpose of survey
* Based on instrument used
* Based on Frame work/method adopted.
* Based on type of field survey

Primary classification :-

1. Plane surveying
2. Geodetic Surveying

Plane Surveying :-

* The earth surface is assumed as a plane & the curvature of earth is ignored (neglected) is called plane surveying

* plane survey is to be adopted for only in small areas (upto 195 sq. km)
  All Plumb lines are considered parallel.
* In this survey the line connecting two points on the earth is considered as a straight line and the angle b/w any two lines considered as a plane angle. All △les are formed by survey lines are considered as plane triangles.

Uses

* It is used for the layout of highways, railways, canals, construction of bridges, dams, building etc.

* The degree of accuracy is low.

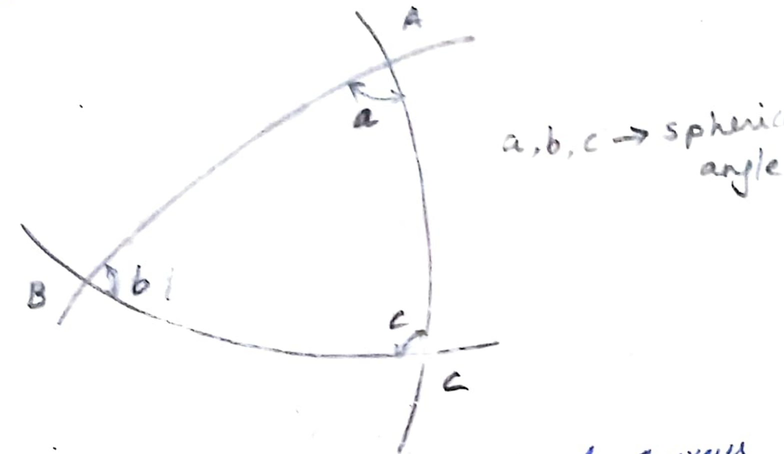
Geodetic Survey :-

* The surface of the earth is considered as a spherical.

* The effect of curvature is in to an account for all measurements is known as geodetic survey.

* The lines connecting any two points on the earth surface is not a straight line but curve.

* Geodetic survey work is include larger magnitude & high degree of precision.
* The angle between any two arcs is treated as spherical angle.



<u>Uses</u> :-

* Engineering surveys, topographical surveys, Cadastral surveys etc.,
* This survey is conducted in India by Survey of India.

b) Based on Nature / type of Field survey :-

1. Land survey → Topographical survey, → city survey, → cadastral survey
2. Marine (or) Hydrographic Survey
3. Astronomical survey.

1. Land survey :-

* To determine the old land lines and its directions and subdividing the land into predetermined shape & sizes.
* calculating the areas & locating their positions.

a) Topographical Survey :-

* It is consist of horizontal & vertical location of certain points by linear and angular measurements.
* To determine the natural features of a country such as rivers, streams, lake, hills etc. artificial features → roads, railways, canal, town, village etc.,

b) City Survey:-

* It is connection with the construction of streets, water supply systems, sewers + other works.

c) Cadastral survey:-

* To the fixing of property lines, the calculation of land area. ~~or~~ ~~data~~
* To fix the boundaries of municipalities + others.

Marine (or) Hydrographic Surveying:-

It is deals with the water bodies like streams, lakes, coastel waters and the shore lines consists in acquiring data to chart the depth of of water bodies.

* The purpose of this survey is to study the nature bed etc.
* To determine water + locating the mean sea level for the purpose of navigation, harbour works, construction etc.,
* To prepare ~~determine~~ the navigational chart etc.,

(To find out the depth of water sound techniques + tide fluctuations)

Astronomical Survey:-

To determination of the absolute location and of any point or the absolute location of the direction of any line on the surface of the earth.

It consists in observations to the heavenly bodies such as the sun or any fixed star, moon etc.,

b) Classification based on object/purpose of survey:-

* Engineering survey
* Millitary survey
* Mine survey
* Geological survey
* Archaeological survey

Engineering survey :-

To collect the datas for the designing and construction of engineering works such as roads, railway, canal, bridge reservoir and connected with the sewage disposal or water supply.

Millitary survey :- (Preparation of map)

Aerial & topographical maps of enemy areas indicating important roads, airport, missile site, early warning & other type of radars, anti-aircraft positions.

Mine survey :- → Exploring mineral wealth below the earth surface

The exploration of mineral deposits & to guide tunneling and other operations associated with mining.

Geological survey :-

This is used for determining the different strata in the earth crust.

Archaeological survey :-

* To prepare map of ancient culture
* To identified the earthquake, land slide, fort, temple etc in ancient culture.

c) classifications based on Instruments used :-

Chain survey
Theodolite survey
Compass survey
Traverse survey
Triangulation survey
Tacheometric survey
plane table survey
Photogrammetric survey
Aerial survey

Chain Survey:-

It is surveyed only linear measurements are made in field.

* In this type of survey is suitable for only in small areas.
* The area is divided into a network of triangles & trapezoids

Compass Survey:-

The direction of survey lines are determined with a compass.

* A chain or tape is used for linear measurements.

Plane table survey:-

* It is a graphical method of survey in which field observations and plotting are done at the same time.

Instruments used for plane table surveys are Drawing board, Tripod, Alidade, trough compass, Spirit level, U-frame, plumb-bob, peg & mallet

Theodolite Survey:

* It is used for measuring both horizontal & vertical angles.

Levelling :-

The relative vertical heights of points are determined by the instruments of dumpy level & levelling Staff.

Tacheometric Survey:

It is a rapid & economical survey by which the horizontal distances & the difference in elevations are determined indirectly using a theodolite & graduated rod.

Triangulation Survey:-

The field is to be surveyed into a network of triangles. A single line ~~is~~ called base line is measured accurately and the length of other lines are computed from the measured angles. $\frac{a}{\sin A} = \frac{b}{\sin B} = \frac{b}{\sin C}$

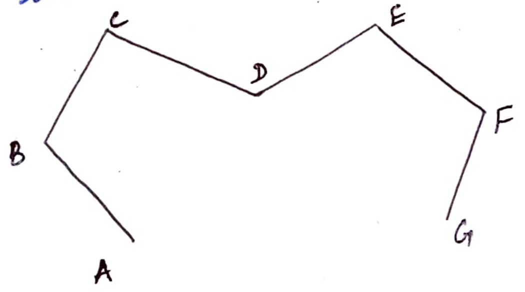
Traverse survey:-

In traverse survey directions of survey lines are fixed by angular measurements and not forming a network of triangles.

A traverse survey is one in which the frame work consist of a series of connected lines, the length are measured chain or tape, and the directions are measured with an angle measuring instrument (compass, theodolite etc.)

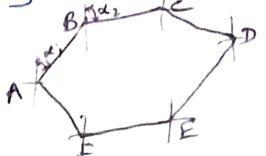
open Traverse:-

A traverse is said to be open when it does not ~~to~~ form a closed polygon.



closed Traverse:-

A closed traverse is one when it returns to the starting point forming a closed polygon.



Photogrammetric survey:-

Features on the surface of earth are located by measurements from photographs.

Electromagnetic Distance Measurement (EDM) survey:-

* This is the electronic method of measuring distances using the propogation, reflection & subsequent reception of either light or radio waves.

Examples of EDM : Tellurometer, geodimeter, distomat.

Total station survey:-

The electronic theodolites combined with EDM and electronic data collectors are called total stations.

A total station reads and records horizontal & vertical angles, together with slopes distances.

The instrument has capabilities of calculating rectangular co-ordinates of the observed points, slope corrections, remote object elevations etc.,

Satellite based survey:-

Remote sensing & global positioning system (GPS) are the satellite based surveys.

* Acquiring data for positioning on land, on the sea, and in place using satellite based navigation system based on the principle of trilateration is known as GPS.

GPS uses :-

* Satellite signals, accurate time & sophisticated algorithms to generate distances in order to triangulate positions.

Remote sensing :-

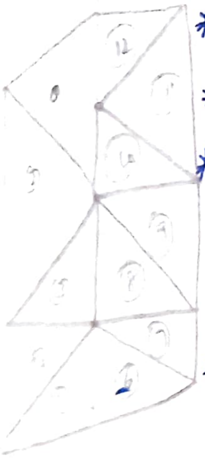
In remote sensing the data about an object is collected by sensors placed on satelites by employing electromagnetic energy as the means of detecting & measurements.

PRINCIPLES OF SURVEYING

Nov/Dec - 2009

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

To work from whole to part :-



* It is the main principle of surveying.
* It is adopted for plane or ~~&~~ Geodetic survey.
* The main idea of ~~this~~ ~~principle~~ ~~is~~ working from whole to part is to localize the errors & prevent their accumulation
* It is very essential to estabilish first a system of control points and fix them with higher precision.
* Minor control points can then be established by less precise methods.

GPS uses :-

* satellite signals, accurate time & sophisticated algorithms to generate distances in order to triangulate positions.

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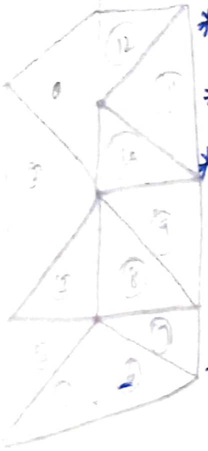

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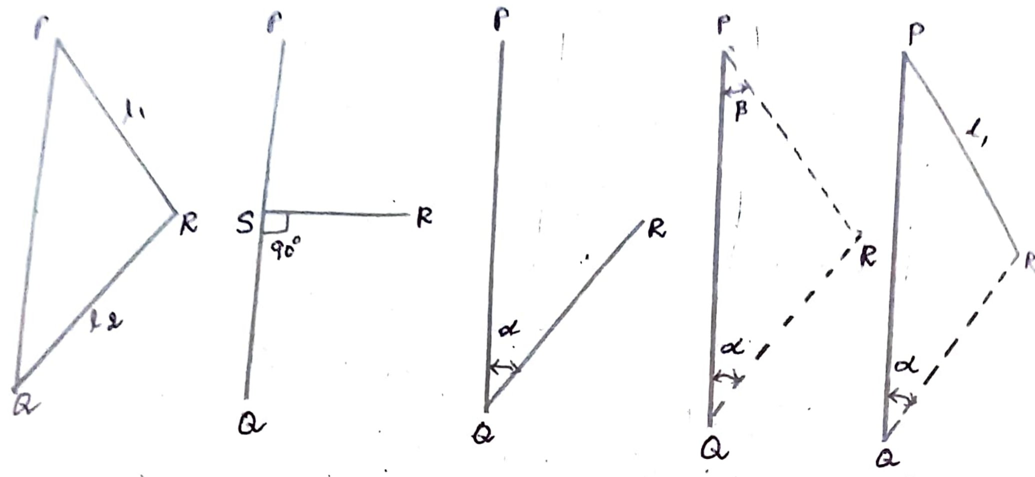
To locate a point by at least two measurements



The relative positions of the points to be surveyed should be located by measurement from at least two points of reference, the positions of which have already been fixed. ~~Let P & Q be the reference points on the ground.~~

* Let the points P & Q ~~can~~ will thus serve as reference points for fixing the relative positions of other points. Any other point, such as R. can be located by any of the following direct methods.

(i) measurement of two distances

(ii) measurement of two angles.

(iii) measurement of one angle & one distance.

(iv) Perpendicular length

Units of Linear measurements

Units of Length :-

10 mm = 1 cm

10 cm = 1 decimetre (dm)

10 deci.metre = 1 m

10 m = 1 decametre (d)

10 decametre = 1 hectometre (hm)

10 hectometre = 1 kilometre (km)

Units of Area :-

$100 \text{ mm}^2 = 1 \text{ cm}^2$

$100 \text{ cm}^2 = 1 \text{ decimetre}^2$

$100 \text{ decimeter}^2 = 1 \text{ m}^2$

$100 \text{ m}^2 = 1 \text{ are}$

$100 \text{ are} = 1 \text{ hectare}$

$100 \text{ hectare} = 1 \text{ km}^2$

$= 1 \times 10^6 \text{ m}^2$

Units of volume :-

$1000\ mm^3 = 1\ cm^3$

$1000\ cm^3 = 1\ \text{decimeter}^3$

$1000\ dm^3 = 1\ m^3$

Units of Angular measurements :-

1 minutes $= 60''$ (sec)

1 degree $(^\circ)$ = 60 minutes

1 right angle $= 90^\circ$
($\pi/2$ radians)

1 right angle = 100 grades (g).

Extras :-

1 Feet = 12 inch

1 inch = 2.54 cm

1 chain = 66 feet

1 km = 1000 m

1 hect $= 10{,}000\ m^2$

1 furlong = 660 feet

1 mile = 8 Furlong.

LINEAR MEASUREMENTS.

The various methods for making linear measurements and their relative merit depends upon the degree of precision required.

* Direct measurements ~~---~~ distance
* Measurements by optical means
* Electromagnetic methods.

Direct measurements :-

The distances are actually measured on the ground with the help of a chain or a tape.

optical methods :-

In the optical methods, observations are taken through a telescope and calculations are done for the distances, such as in tacheometry or triangulation.

Electromagnetic methods :-

The distances are measured with instruments that rely on propogation, reflection & subsequent reception of either radio waves, light waves or infrared waves.

Direct measurements :-

The various methods of measuring the distances directly are as follows.

* Pacing
* Measurement with passometer
* Measurement with Pedometer - each step are registered
* Measurement by odometer & speedometer
* chaining.

Cycle wheel dia = 0.6m

circumference = $2\pi r = \pi \times 0.6 = 1.884$ m

CHAIN SURVEY

* Chain surveying is the simplest and quite useful land surveying

Purposes of chain surveying:

* To find the area of ~~the~~ a given land
* To collect necessary data for preparing a plan
* To obtain necessary data for the description of the boundaries of a land
* To re-establish the boundaries of an area already surveyed.
* To divide the given land into a number of units of required sizes.

Principle of chain surveying:-

* Triangulation → a system of of surveying in which the sides of the various Δ^{les} are computed.
* Traversing →

Triangulation :-

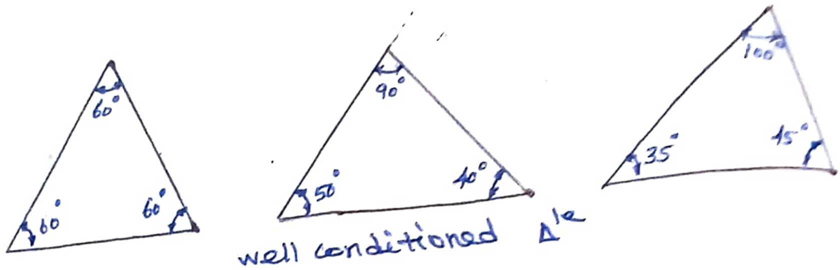
* The ~~vast~~ area is dividied into a network of triangles.
* The sides of triangles are measured directly on the field by chain or tape.

Traversing :-

* In traversing the directions of survey lines are fixed by angular measurements & not forming a network of triangles.

Well conditioned triangles:-

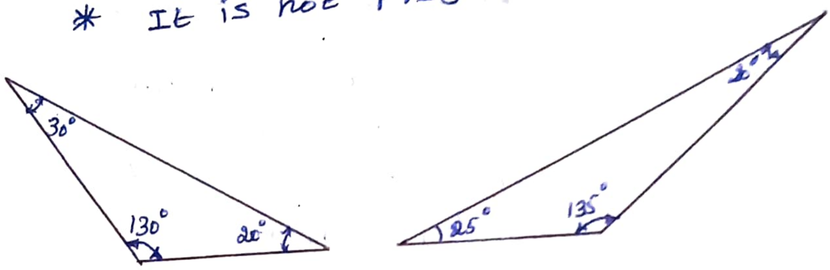
* A number of included angle is not less than 30° or greater than 120°.
* An equilateral triangle is the best conditioned triangle or an ideal triangle.



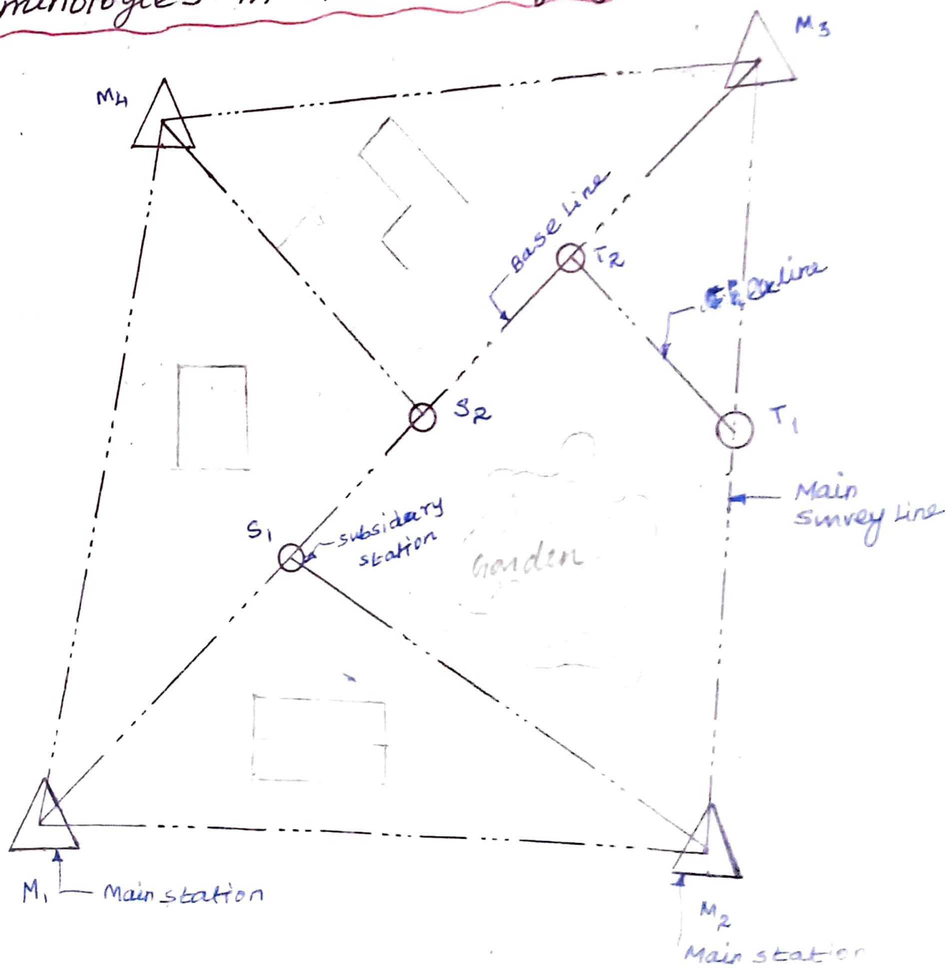
well conditioned Δ^{le}

ILL conditioned triangles:-

* Triangles have ~~..~~ less than 30° and more than 120° are called ill-conditioned triangles.
* It is not preferred in chain surveying



Terminologies in chain surveying:-



Survey stations :-

* survey station is a selected point on the chain line and can be located either at the begining of the chain line or at the end.

Main station :-

* survey stations taken along the boundary of an area as controlling points are referred to as main stations.
* The chain lines connecting the main stations are called main survey line.
* The Main stations are denoted by the symbol " △ "

Subsidary station :-

* station which are taken to run subsidary lines for dividing the area into triangle for checking the accuracy of the triangle and for locating interior details is called Subsidary station.
* It is denoted by ' ⊙ '.

Tie station :-

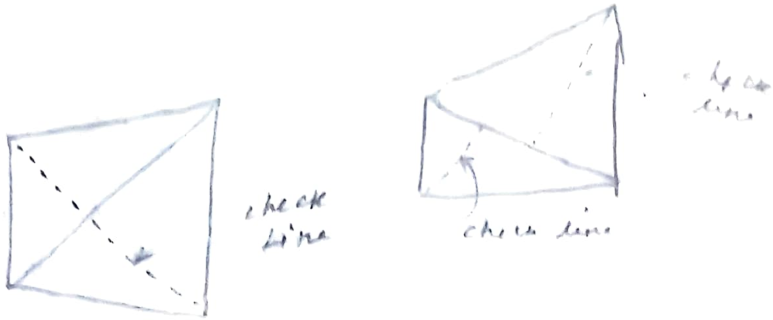
* Tie stations are also subsidary stations taken on the main survey line.
* Lines connecting the tie stations are known as tie lines.

Base Line :-

* It is the line which passes through the centre of the area and the longest one.
* To minimize the accumulation of error.
* It is the longest line

Check Line :-

* A check line measured to check the accuracy of the frame work.
* It is also called a proof line.



selection of survey stations :-

* survey stations must be mutually visible
* survey lines must be as few as possible
* survey lines must be as few as possible so that the frame work can be plotted conveniently.
* The frame work must have one or two base lines.
* The lines must run through level ground as possible.
* The main lines should be form a well-conditioned triangles
* Each triangles should be provided with suffilient check lines.
* As far as possible, the main survey lines should not pass through obstacles
* The main survey lines should fall with in the boundaries of the property to be surveyed.

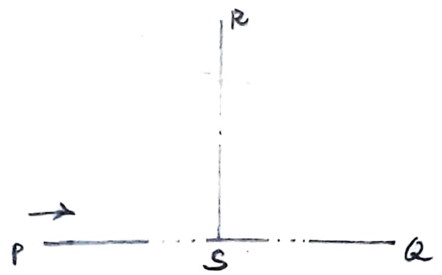
OFFSETS :-

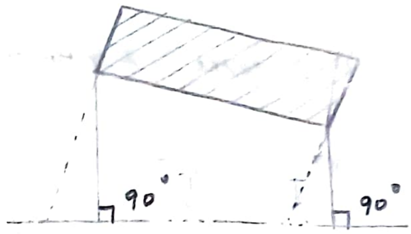
* The lateral measurements taken from an object to the chain line is called offsets.

Types of offset :-

Perpendicular offset :-

Perpendicular offsets or right-angled offsets are one when the lateral measurements are taken perpendicular to the chain line.



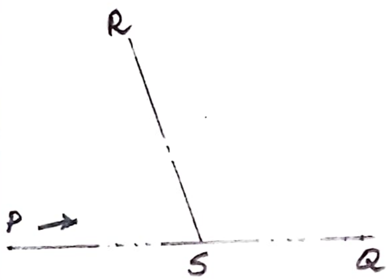


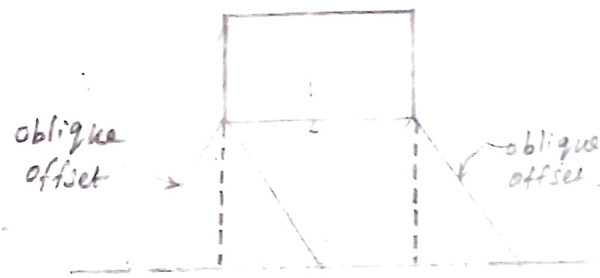
Perpendicular offset are preferred for the following reasons :-

* can be taken quickly.
* Easy to enter in the field book
* progress of survey is not interrupted.
* Easy during plotting.

Oblique offsets :-

* when the angle is other than 90° is called an oblique offset.





CHAIN SURVEY INSTRUMENTS :-

Chain :-

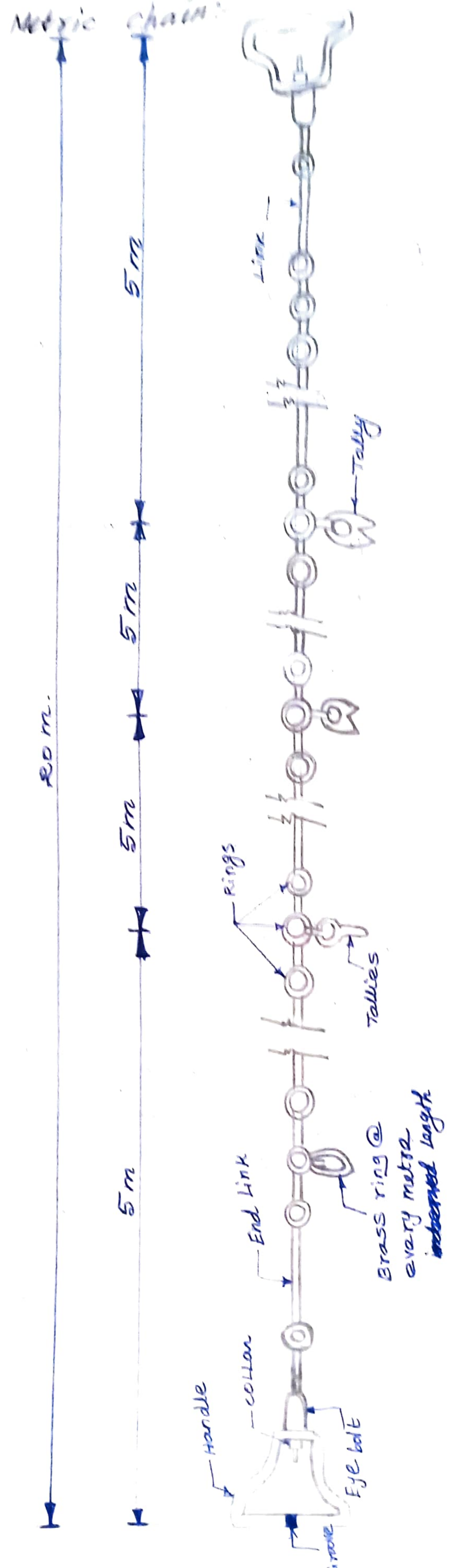
* The most common & accurate method for measuring the linear distance with a help of chain or tape.
* For routine work & accuracy a metallic Chain used.
* More accurate work has to be carried out a Steel tape is used.

chaining :-

The term chaining is used for distance measurement both by chain or tape.

Different types of chain:

1 Metric chain:



* chains are formed of straight links of galvanised mild steel wire bent into rings.
* The links are connected with each other by three small circular or oval ~~o~~ wire rings.
* These rings offer flexibility to the chain.
* Brass handles are provided at each end.
* Brass handles are connected through a swivel joint, so that the chain can be turned round without twisting
* The length of a link is the distance b/w the centres of the two consecutive middle rings.
* Metric chains are available in a length of 20m & 30m.
* The 20m chain is divided into 100 links (each of 0.2m) with tallies connected at every 10 links. (ie, 2m intervals).
* 20 m chain can be used on fairly level ground.
* For 30 m chain is divided into 150 links with each link of 0.2m length.
* Tallies are provided at every 25 links (ie, 5 m interval/length).
* After every meter a brass ring is attached.

Engineer's chain :-

* The chain is 100 feet in Length and divided into 100 links.
* The details of construction are the same as for metric chain.
* Tallies are provided at 10 links (10 feet)
* It is used for all engineering works.

Gunter's chain:-

* This chain is 66 feet of length with 100 links of 0.66 ft long.
* It is measure the distances for miles & furlongs.
* Also it is used for measuring land where the unit of area is an acre.

Revenue chain:-

* It is 33 feet long & dividied into 16 links.
* It is commonly used for cadastral survey.

Testing for the chain:-

* 1m length of a chain should be accurate to 2mm. when measured by a standardised tape or steel band.
* Thus the following limits of accuracy are fixed.

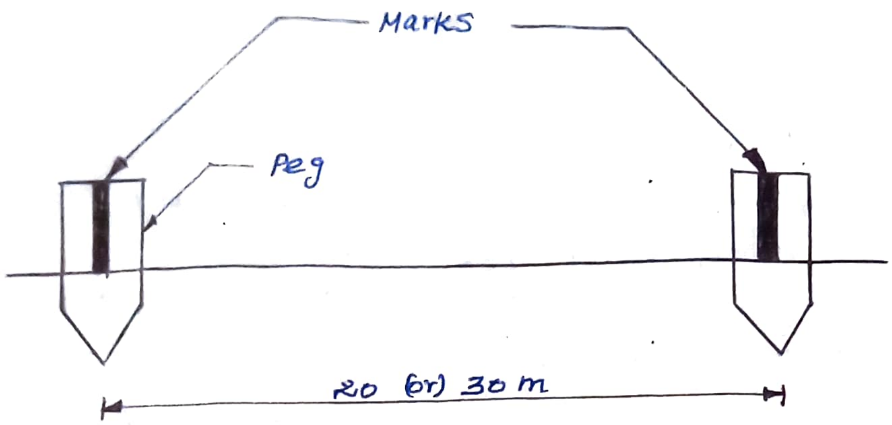
20 m chain ± 5 mm

30 m chain ± 8 mm

Specification:-

* when a tension of 80N is applied at the ends of the chain & compared against a certified steel band (tape),
* standardized at 20°c every meter length should be accurate to with in ± 2mm.
* The accuracy of an overall length of 20m chain should be with in ± 5mm & 30m chain should be with in ± 8mm.

Procedure :-



* Two pegs at a required distance of 20 m or 30 m are inserted on a flat ground.
* The overall length of the chain is compared with the marks and the distance is noted.
* If the chain is found to be too long, it may be adjusted by closing the opened joints of rings;
* Reshaping the elongated links.
* Removing one or more circular rings and replacing the worn out rings.
* If chain is found too short, it may be adjusted by straightening the bent links flattening the circular rings, replacing circular rings by bigger rings & inserting additional rings.

Advantages of chain :-

* Easy & quick to read
* Withstand wear & tear
* Easy to repair and rectify

Disadvantages of chain :-

* consumes more time to open or fold
* heavy to handle
* error due to sagging is more
* shortens or elongates due to frequent use.

Unfolding the chain :-

* The leather strap is removed.
* Both handles of the chain in the left hand, the chain is thrown well forward with the right hand.
* The leader then takes one of the handles of the chain and moves forward until the chain is extended to full length.
* the chain is checked and kinks of bent links are removed.

Folding the chain :-

* During its use, the links of a chain get bent and the length is shortened.
* on the other hand, the length of a chain may be increase by stretching of links & usage, and rough handling through hedges, fences etc.,
* Therefore, it becomes necessary to check the length of the chain before commencing the survey work.
* Before checking, it should be ensured that the links are not bent, rings are circular, openings are not too wide & mud is not clinging to them.

Band chain :-

* It is also called steel band.
* It is a ribbon of steel with brass swivel at each end.
* It is 20 to 30m in length and 16mm wide.
* It is wound on an open steel cross or in a metal real in a closed case.
* The graduations marked on the steel ribbon as follows.

(i) Brass studs divide the band @ 0.2 m & numbered @ every 1m

(ii) 1st & Last links are subdivided into cm & mm

(iii) Brass tallies are provided at every 5 m length.

Tapes :-

Tapes are available in a variety of materials, length & weights.

(i) Cloth or Linen Tape :-

* This is closely woven linen or synthetic material & is varnished to resist the moisture.
* These are available in lengths of 10 m to 30 m and width is 12 mm to 15 mm.

Dis-advantages :-

(i) It is affected by moisture and gets shrunk.

(ii) Its length gets altered by stretching

(iii) It is likely to twist & does not remain straight in strong winds.

(ii) Metallic Tape :-

* Metallic tape is made up of varnished strip of water proof linen inter woven with small brass, copper, or bronze wire & does not stretch as easily as a cloth tape.

* Metallic tapes are light & flexible and are not easily broken.

* It is available in 10, 20, 30 & 50m length.

* It is commonly used for measuring offsets.

(iii) Steel Tape :-

* steel tapes vary in quality and accuracy of graduation.

* A steel tape consist of a light strips of width 6mm to 10mm & is more accurately graduated.

* steel tapes are available in 1m, 2m, 10m, 20m, 30m & 50m

* At the end of the tape a brass ring is attached, the outer end of which is zero point of the tape.

* steel tape cannot be used in ground with vegitation & weeds.

(iv) Invar Tape :-

* Invar tape is made up of an alloy of nickel (36%) and steel having low co-efficient of thermal expansion ($0.122 \times 10^{-6}/^{\circ}C$).

* These are available in length of 30, 50 and 100m and in a width of 6mm.

Advantages:-

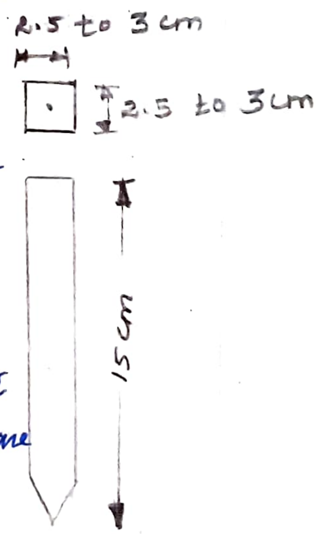
* Highly precise
* It is less affected by temperature changes when compared to the other tapes.

Dis-advantages:-

* It is soft & so deforms easily.
* It requires much attention in handling.

Accessories for chaining:-

* Peg
* Arrows
* Plumb bob
* Ranging rods
* offset rods
* cross staff.



Peg :-

* wooden pegs are used to mark the positions of the stations or terminal points of a survey line
* They are made of stout timber, generally 2.5 cm or 3 cm square and ~~with the help of~~ 15 cm long, tapered at the end.
* They are driven in the ground with the help of a wooden hammer & kept about 4 cm projecting above the surface.

Ranging Rods:-

* Ranging rods have a length of either 2m or 3m, the 2m length being more common.
* They are shod at the bottom with a heavy iron point, and are painted

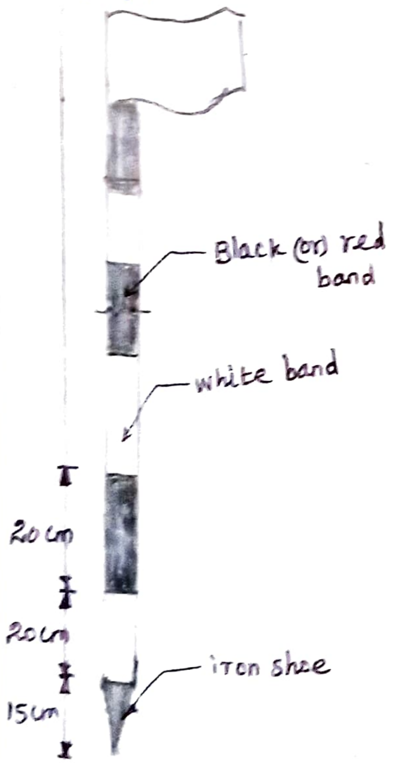
r alternative bands of either black and white or red & white or black & red & white in succession, each band being 20 cm deep.

* Ranging rods are used to range some intermediate points in the survey line.

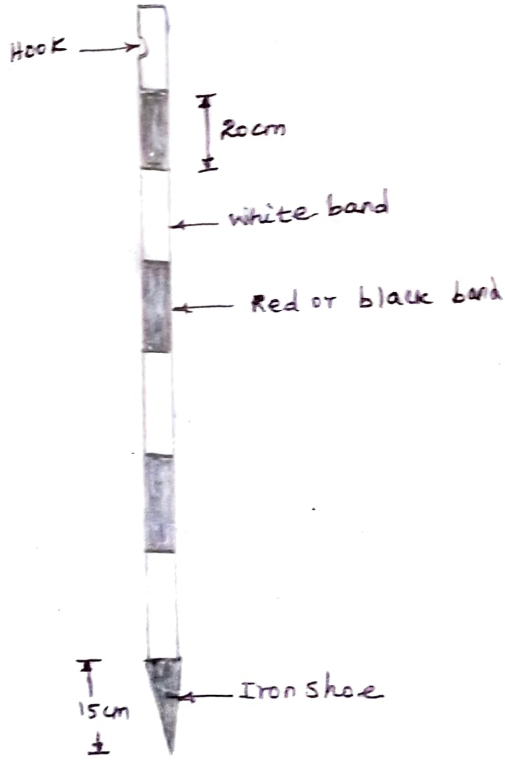
* They are circular or octogonal in cross-section of 3 cm nominal diameter, made of well seasoned, straight grained timber.

* The rods are almost visible at a distance of about 200 metres.

* When used on long lines each rod should have a red, white or yellow flag, about 30 to 50 cm square tied on near its top.



Ranging Rod



Offset Rod

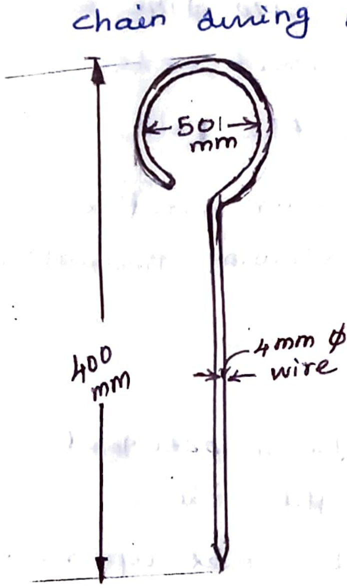
Offset Rod :-

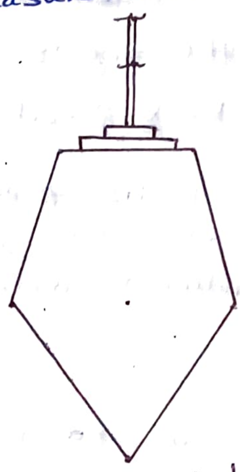
* It is similar to ranging rod.
* Length 3 m

* It is made up of wood and circular c/s with bottom end fixed at sharp iron shoe.
* other end hook provided.

Arrows :-

* Arrows are made up of tempered steel wire of 4 mm dia.
* pointed (sharp) at one end & other end bent ring. of 50 mm dia.
* over all length 400 mm.
* Arrows are used to counting the number of chain during measurement.





plumb-bob

plumb-bob :-

* solid metallic cone placed upside down and suspended from a thick thread.
* Generally used by mason, to check verticality of masonry and other work.
* In survey work it is used for chaining along sloping ground to transfer the points to the ground.
* It is used for centering the theodolite, compass and Plane table.
* verticality of ranging rods are checked.

Cross staff :-

* Instrument used for setting out perpendicular

* These are three types. They are

(i) open cross staff

(ii) French cross staff

(iii) Adjustable cross staff

Open cross staff :-

* It consists of a wooden block round or square in shape.

* 150 mm dia & 38 cm deep

* It is provided with two fine saw cuts at right angles to each other.

* Wooden block fixed on the pole.

* It is made up of four metal arms with vertical slits for viewing through mutually perpendicular directions.

French cross staff :-

* It is a brass tube in octogonal shape with slits on all eight sides.

* It can be used to set up right angles or a 45° line.

Adjustable cross staff :-

* It is a brass cylindrical tube of 25mm dia & 100 mm deep.

* It is divided in the centre.

* upper cylinder is provided ~~with~~ an arrangement to be rotated relative to the lower one.

* Lower part is graduated to degrees and sub divisions, while upper one carries a vernier.

Ranging :-

Ranging is the process of establishing or fixing intermediate points in a straight line between two terminal stations or points.

Types of Ranging :-

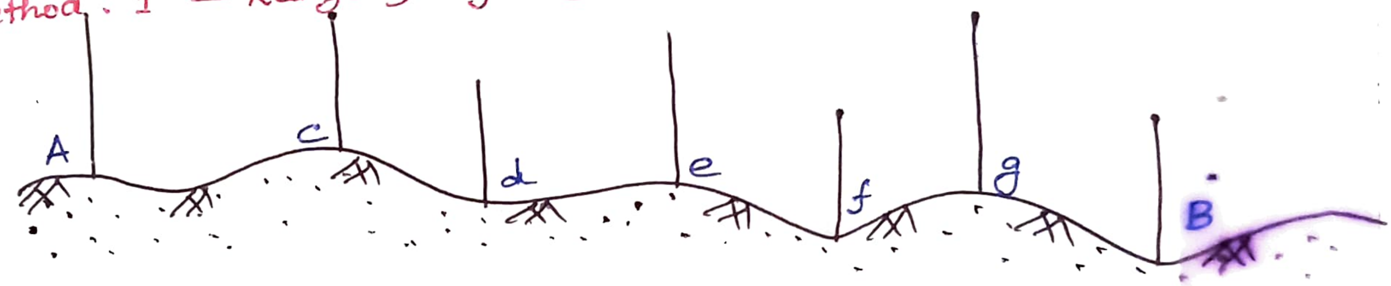
* Direct Ranging
* In-direct (or) Reciprocal ranging.

Direct Ranging :-

1. Explain the method of direct ranging in details.

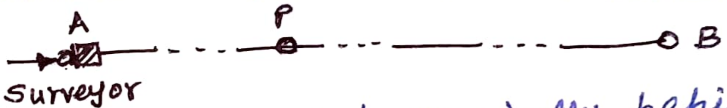
Direct ranging is a process in which ranging rods are placed on a straight line (intermediate points along the chain line) by direct observation from end stations.

Method : 1 — Ranging by eye :-



Let A & B be two end stations & c, d, e, f & g etc. be the intermediate points to be stabilished.

Procedure :-



* Ranging rods are erected vertically behind each end of the line.
* Surveyor stands behind the ranging rods at the end stations A & B of the line.
* one of the surveyors, says the surveyor at A, directs the assistant to hold a ranging rod vertically at arms length from the point where the intermediate point is to be estabilished.
* The assistant is directed to move the rod to the right or left until the three ranging rods appear to be exactly in a Straight line.
* The codes of signals used is stated below.
* The signals given by the surveyor

S.No	Signal by the surveyor	Action by the Assistant
1	Rapid sweep with right hand	Move considerably to the right
2	slow sweep with right hand	Move slowly to the right
3	Right arm extended	continue to move to the right
4	Right arm up & moved to the right	plumb the rod to the right
5	Rapid sweep with left hand	move considerably to the left
6	slow sweep with left hand	move slowly to the left
7	Left arm extended	continue to move to the left
8	Left arm up & moved to the left	plumb the rod to the left
9	Both hands above head & then brought down	correct
10	Both arms extended forward horizontally & hands depressed briskly	Fix the rod.

Method : 2 — Ranging by line ranger :-

* It is a simple instrument used for fixing intermediate points on chain line.
* In this instrument two-right angled isosceles triangular prism are placed one above the other (fig:
* To establish a point between the end stations A & B, the surveyor holds the instrument at the level of the eye and stands approximately in line near P,
* Rays of light from 'A' passes through the upper prism get reflected appears to the eye perpendicular to 'AB'.
* ||| ly another ray from 'B' reaches the eye after reflect
* That the images of ranging rods at station A & B appear in upper & lower prism directly infront of the supervisior.

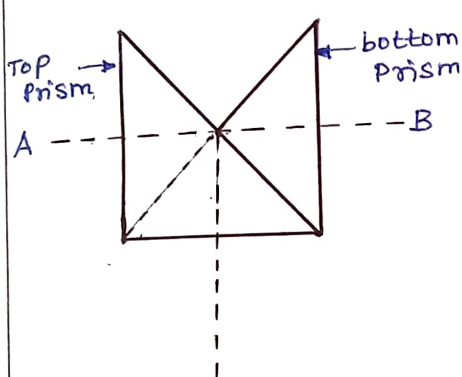
* If the alignment is correct both the images appear one above the other in a vertical line otherwise get seperated. (Fig : b)

* The surveyor has to move perpendicular to chain line till he gets the correct alignment. (Fig : c)

* Then the required point 'p' is vertically below the centre of the instrument.

* The Instrument is very handy & simple to operate.

* It is quite useful to establish intermediate points more rapidly and there is no necessity to go to the end stations.



(Fig : a)

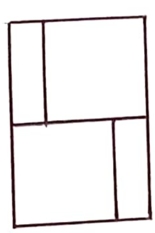


Fig : b

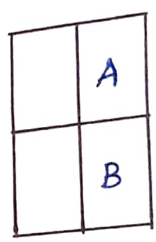
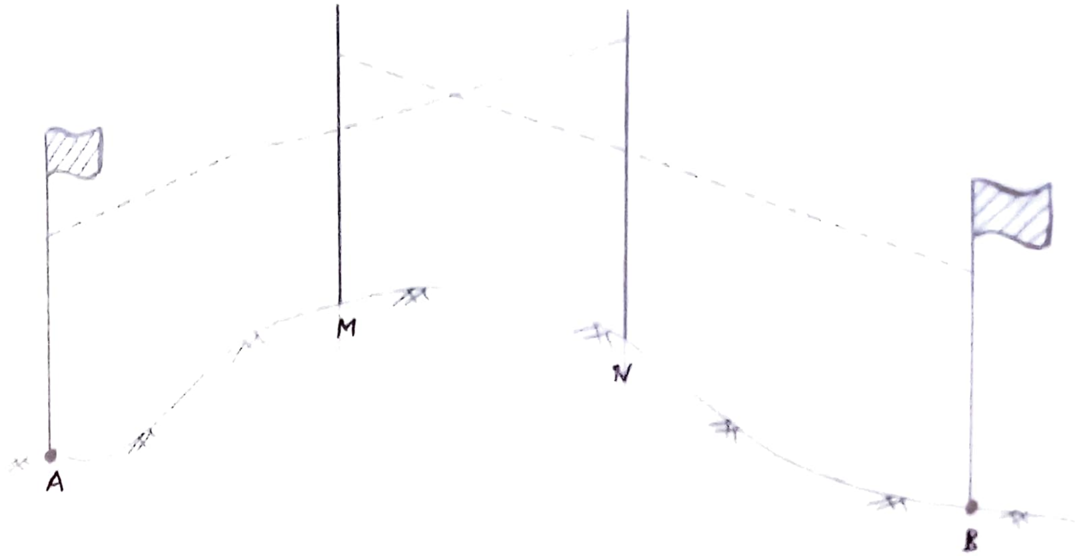


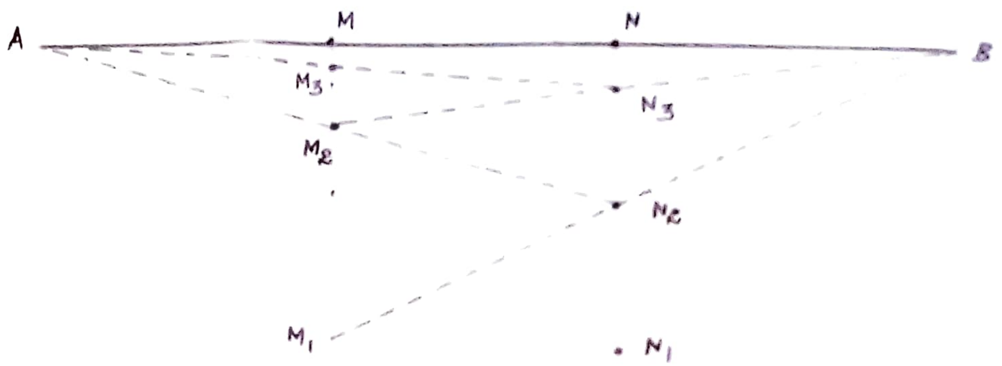
Fig : c

2 Explain the method of reciprocal ranging (In-direct ranging) in detail.

* In-direct or Reciprocal ranging is resorted to when both the ends of the survey line are not intervisible

When end stations are not intervisible due to rising ground between them and due to long distance between the ends. For this type location indirect ranging is used also known as indirect (or) Reciprocal ranging.





* Let A & B be the two end stations of a line with a rising ground between them, and the M & N be the two intermediate points to be established on the chain line.

* Intermediate points M_1 & N_1 very near to the chain line (by judgement) in such a way that from M_1B & N_1B are visible, and N_1A & M_1A are visible.

* Two surveyors station themselves at M_1 & N_1 with ranging rods.

* The person at M_1 then directs the person at N_1 to move a new position N_2 in line with M_1B.

* The person at N_2 then directs the person at M_1 to move a new position M_2. in line with N_2A.

1. A steel tape 20m standardised at 55°F with a pull of 10 kg was used for measuring a base line. Find the correction per tape length, if the temperature at the time of measurement was 80°F and the pull exerted was 16 kg. weight of 1 cubic cm of steel = 7.86 g; Wt of tape = 0.8 kg and $E = 2.109 \times 10^6$ kg/cm². co-efficient of expansion of tape per 1°F $= 6.2 \times 10^{-6}$

Solution:-

Given Data:-

Length of tape $(L) = 20\,m$

Standardised Temp. $(T_0) = 55°F$

Mean Temp. $(T_m) = 80°F$

Co-efficient of expansion $(\alpha) = 6.2 \times 10^{-6}$

Young's Modulus $(E) = 2.109 \times 10^{6}$ kg/cm²

Standard pull $(P_0) = 10$ kg

Actual pull P (or) $(P_m) = 16$ kg

Wt of 1 cm³ $= 7.86$ gm.

Density $= 7.86$ gm/cc

Wt. of tape $= 0.8$ kg.

To find:-

Correction per tape length $= ?$

Solution:-

Temperature Correction (C_T):-

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

$$= 6.2 \times 10^{-6} (80 - 55) \times 20$$

$$\boxed{C_t = 3.1 \times 10^{-3}\ m.}$$

correction for pull :-

$$C_p = \frac{(P_m - P_o) L}{AE}$$

$$C_p = \frac{(16-10) \times 20}{A \times 2.109 \times 10^6}$$

$$C_p = \frac{5.6899 \times 10^{-5}}{A} \quad \text{---------} \; (1)$$

To find Area of tape (A):-

Density of tape $= 7.86 \; g/cc$

Volume of tape = c/s area × Length

$$V = A(20 \times 100) \; cm$$

$$V = 2000\,A$$

Wt of tape = Density × volume

$$0.80 \times 1000 \; gm = 7.86 \; g/cc \times 2000\,A$$

$$\frac{0.80 \times 1000}{7.86} = 2000\,A$$

$$\therefore A = \frac{0.80 \times 1000}{7.86 \times 2000}$$

$$\boxed{A = 0.051 \; cm^2}$$

Substitute the Area value in eqn (1)

$$C_p = \frac{5.6899 \times 10^{-5}}{0.051}$$

$$\boxed{C_p = 1.118 \times 10^{-3} \; m}$$

Sag correction :-

$$C_{sag} = \frac{L(WL)^2}{24 n^2 P_m^2}$$

$$C_{sag} = \frac{20 \times (0.8)^2}{24 \times 1^2 \times 16^2}$$

$$\boxed{C_{sag} = 2.0833 \times 10^{-3}\ m}$$ Negative.

Total correction $= C_t + C_p - C_{sag}$

$= 3.1 \times 10^{-3} + 1.11 \times 10^{-3} - 2.083 \times 10^{-3}$

$= 2.126 \times 10^{-3}\ m.$

Total correction $= 0.00213\ m.$

5, A nominal distance of 30 m was set out with a 30 m steel tape from 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 10 kg and at a mean temperature of 70° F. The top of one peg was 0.25 m below the top of the another. The top of the higher peg was 460 m. above 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 standardised at a temperature of 60°F, in catenary, under a pull of (a) 8 kg (b) 12 kg (c) 10 kg. Take radius of earth = 6370 km

Density of tape $= 7.86\ g/cm^3$

section of tape $= 0.08\ cm^2$

Coefficient of expansion $= 6 \times 10^{-6}$ per 1°F

Young's modulus (E) $= 2 \times 10^6\ kg/cm^2$

Solution :-

Given Data :

$P_m = 10\ kg$

$\alpha = 6 \times 10^{-6}$ per $1^\circ F$

$E = 2 \times 10^6\ kg/cm^2$

$A = 0.08\ cm^2$

$T_m = 70^\circ F$

$T_o = 60^\circ F$

$L = 30\ m$

$h = 0.25\ m$

Density $= 7.86\ g/cm^3$

(Density x l) $W = 7.86\ kg/m^2$

(i) Correction for standardisation (or) absolute length :-

= Nil.

(ii) Correction for slope :-

$$S_c = \frac{h^2}{2L} = \frac{(0.25)^2}{2 \times 30}$$

$$\boxed{S_c = 0.0010 \quad (\text{Negative})}$$

(iii) Temperature correction :-

$$C_t = \alpha (T_m - T_o) L$$

$$= 6 \times 10^{-6} (70 - 60) \times 30$$

$$\boxed{C_t = 0.0018\ m} \text{ additive}$$

(iv) Correction for pull :-

$$C_p = \frac{(P_m - P_o) L}{A E}$$

(a) when $P_o = 8\ kg$

$$\therefore C_p = \frac{(10 - 8) \times 30}{0.08 \times 2 \times 10^6}$$

$$\boxed{C_p = 0.0004\ m} \text{ additive}$$

(b) $P_o = 12$ kg

$$C_p = \frac{(10-12)\times 30}{0.08\times 2\times 10^6}$$

$\boxed{C_p = -0.0004\ m}$ Negative

(c) $P_o = 10$ kg

$$C_p = \frac{(10-10)\times 30}{0.08\times 2\times 10^6}$$

$\boxed{C_p = 0}$

$\boxed{w.L = W}$

(v) Sag correction :-

$$C_{sag} = \frac{L\,(wL)^2}{24\,n^2 P_m^2}$$

A × density

Wt of tape per m run $= 0.08 \times 1 \times 100 \times \frac{7.86}{1000}$

$= 0.06288$ kg/m

∴ Total wt of tape $= 0.06288 \times 30$

$W = 1.886$ kg.

(a) $P_o = 8$ kg

$$C_s = \frac{L\cdot W^2}{24\,P^2} - \frac{LW^2}{24\,P_o^2} \qquad \boxed{n=1}$$

$$= \frac{30\times(1.886)^2}{24\times 8^2} - \frac{30\times(1.886)^2}{24\times(10)^2}$$

$= 0.0695 - 0.0445$

$\boxed{C_s = 0.0250\ m}$

(b) $P_o = 12$ kg.

$$C_s = \frac{30\times(1.886)^2}{24\times 12^2} - \frac{30\times(1.886)^2}{24\times 10^2}$$

$= 0.0309 - 0.0445$

$$\boxed{C_s = -0.0136\,m}$$

(c) $P_o = 10$ kg.

$$C_s = 0$$

Final correction :-

(a) Total correction $= -0.0010 + 0.0018 + 0.0004 + 0.0250$

$= 0.0262$ m.

(b) Total correction $= -0.0010 + 0.0018 - 0.0004 - 0.0136$

$= -0.0132$ m.

(c) Total correction $= -0.0010 + 0.0018 + 0 + 0$

$= 0.0008$ m.

Chain Triangulation :-

* Triangulation survey is the system of survey, in which the area to be surveyed will be divided into number of triangles & the area of the triangle is calculated by measuring the length of the sides, or angles of the triangle.

Survey station :-

Survey station is defined as any points on the chain line. They are upto two types.

(i) Main survey station

(ii) Subsidery (or) tie station.

~~Main survey station:-~~

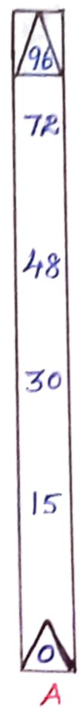
~~It is the point which is either at beginning of the chain line or at end of the chain line.~~

Traversing is the type of survey which is connected to form of survey work of the survey line.

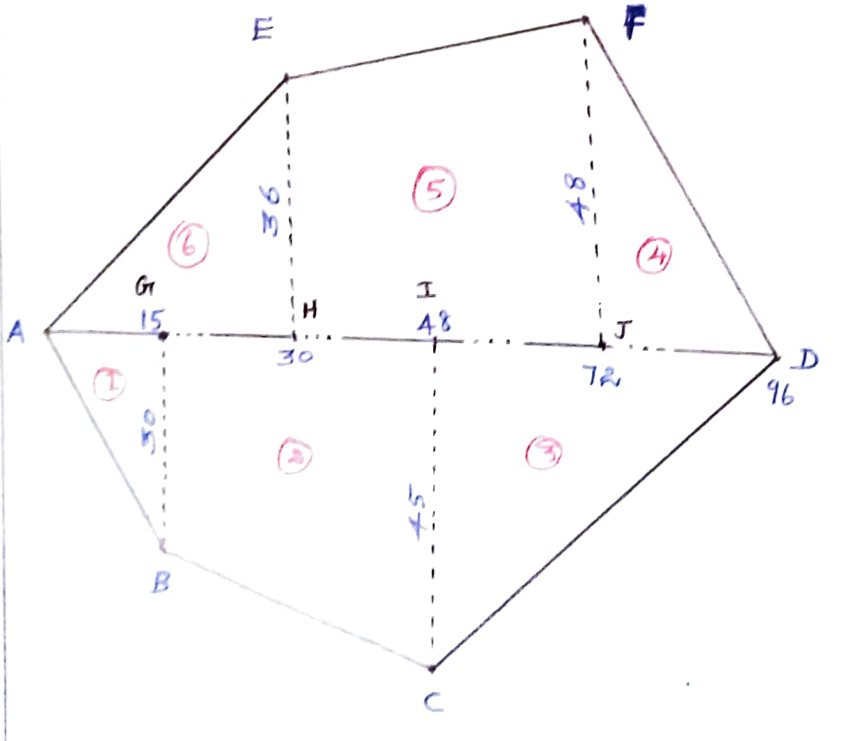
Cross-staff Problem:-

Anna Univ Apr May 2005

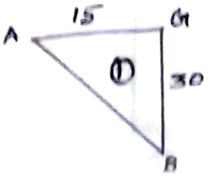
1 Plot the following cross staff survey and calculate the area.



Solution:-



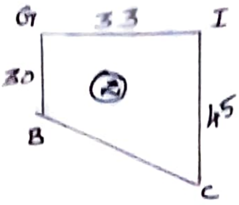
In Δ^{le} ABG



$A_1 = \frac{1}{2} bh = \frac{1}{2} \times 15 \times 30$

$A_1 = 225 \ m^2$

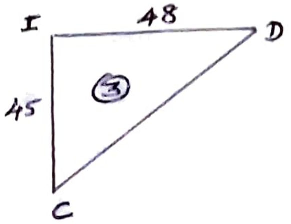
In trapezoidal BCIG



$A_2 = \left(\frac{a+b}{2}\right) h$

$= \left(\frac{45+30}{2}\right) \times 33$

$A_2 = 1237.5 \ m^2$

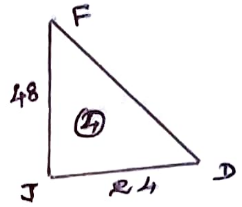


In Δ^{le} ICD

$A_3 = \frac{1}{2} bh$

$= \frac{1}{2} \times 48 \times 45$

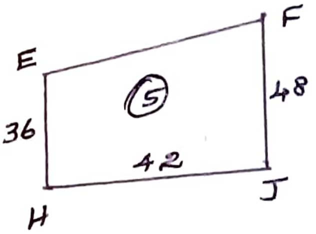
$A_3 = 1080 \ m^2$



In Δ^{le} DJF

$A_4 = \frac{1}{2} bh = \frac{1}{2} \times 24 \times 48$

$A_4 = 576 \ m^2$

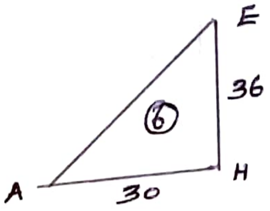


In Trapezoidal JFEH

$A_5 = \left(\frac{a+b}{2}\right) h$

$= \left(\frac{36+48}{2}\right) \times 42$

$A_5 = 1764 \ m^2$



In Δ^{le} AEH

$A_6 = \frac{1}{2} bh = \frac{1}{2} \times 30 \times 36$

$A_6 = 540 \ m^2$

Total Area $= A_1 + A_2 + A_3 + A_4 + A_5 + A_6$

$= 225 + 1237.5 + 1080 + 576 + 1764 + 540$

$A = 5422.50 \ m^2$

$\boxed{40.46 \ m^2 = 1 \text{ cent}}$

Area $= 134.02$ cents

COMPASS SURVEYING

* In traverse work, the survey lines are measured by chain (or) Tape.
* The directions are identified by an angle measuring instruments.
* The instruments used commonly

Angle measuring instruments :-
(i) compass
(ii) Theodolite
(iii) Box sextant

What are the instruments used for the direct measurement of direction?

* surveyor's compass
* Prismatic Compass.

Angle :-

The direction of a survey line with respect to another survey line meeting with in it is known as angle.

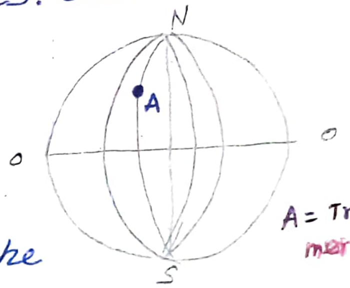
Bearing :-

Bearing of a survey line is the horizontal angle made by the line with reference to a meridian. It is measured in the clockwise direction.

Meridian :-

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

True meridian :-



A = True meridian

The line passing through the geographical north pole & the south pole of any point on the earth surface is known as true meridian.

True bearing:

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

Magnetic meridian :-

Magnetic meridian is the direction sh... by the magnetic needle was freely movin... and balanced, and the magnetic needle is free from any other attractive force.

(or)

It is the direction indicated by a freely suspended and balanced magnetic needle unaffecte... by local attractive forces.

Magnetic bearing:-

The horizontal angle which a line makes with the magnetic meridian is calle... magnetic bearing.

Arbitrary meridian :-

Arbitrary meridians of a point is the direction towards a permanent & Prominent mark or signal, such as church spire or top of chimney.

Arbitrary bearing:-

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

Sexagesimal system :-

1 circumference = 360° degree

1 degree = $60'$ minutes

1 minutes = $60''$ seconds

Types of Compass :-

* Trough compass
* Tubular Compass
* Prismatic Compass
* Surveyor's Compass

Prismatic compass :-

* It consists of a circular box about 100 mm dia
* Magnetic needle balanced on a hard steel pointed pivot
*

- centesimal system - 1 ... - 100 centi grads
- Hour system - 1 centinod

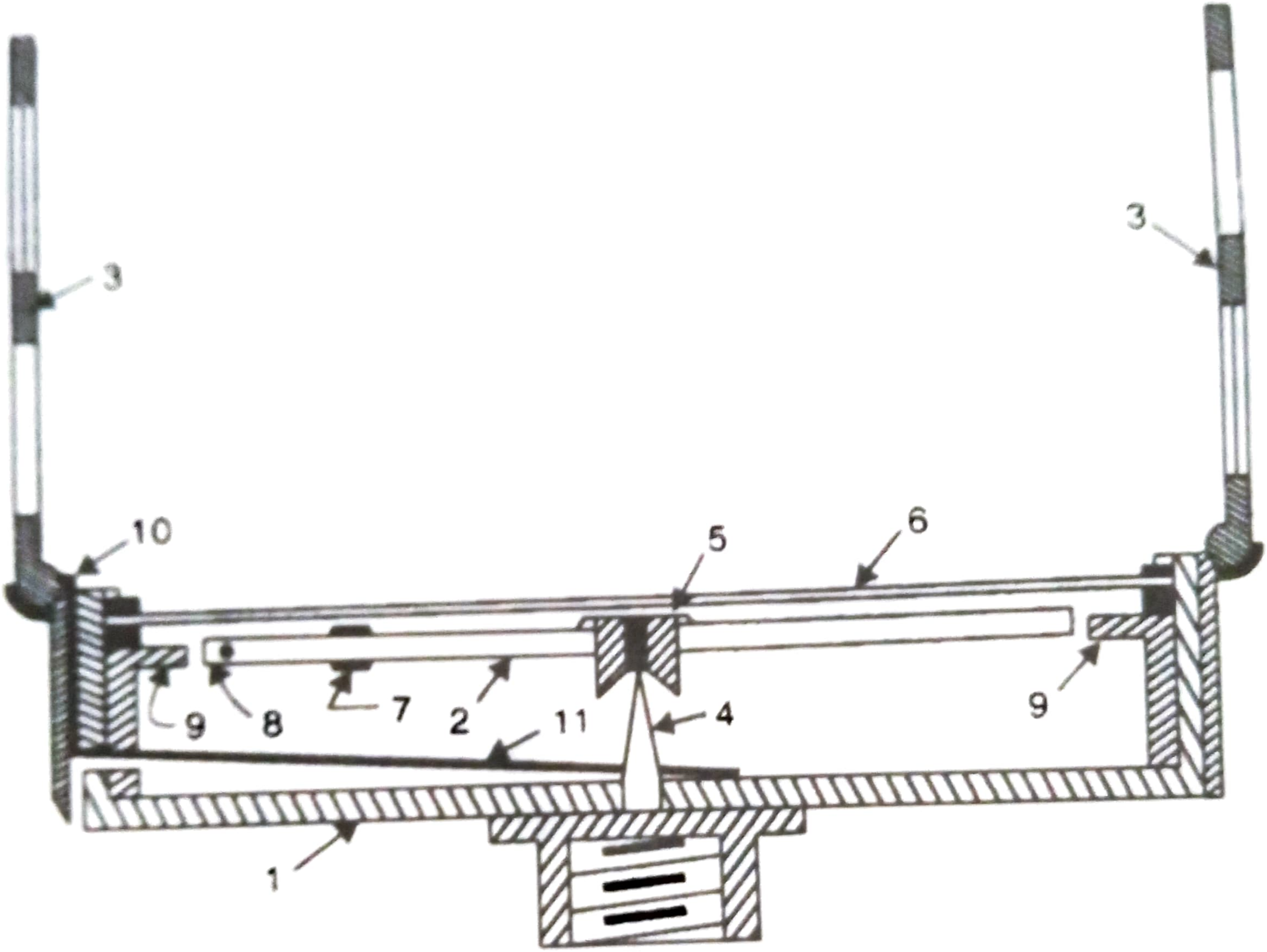
(a) Line of sigh... magnetic meridian

FIG 5.16 SYSTEM OF GRADUATIONS IN THE SURVE...

The difference between surveyor's and prismatic compass is given in Table 5.3

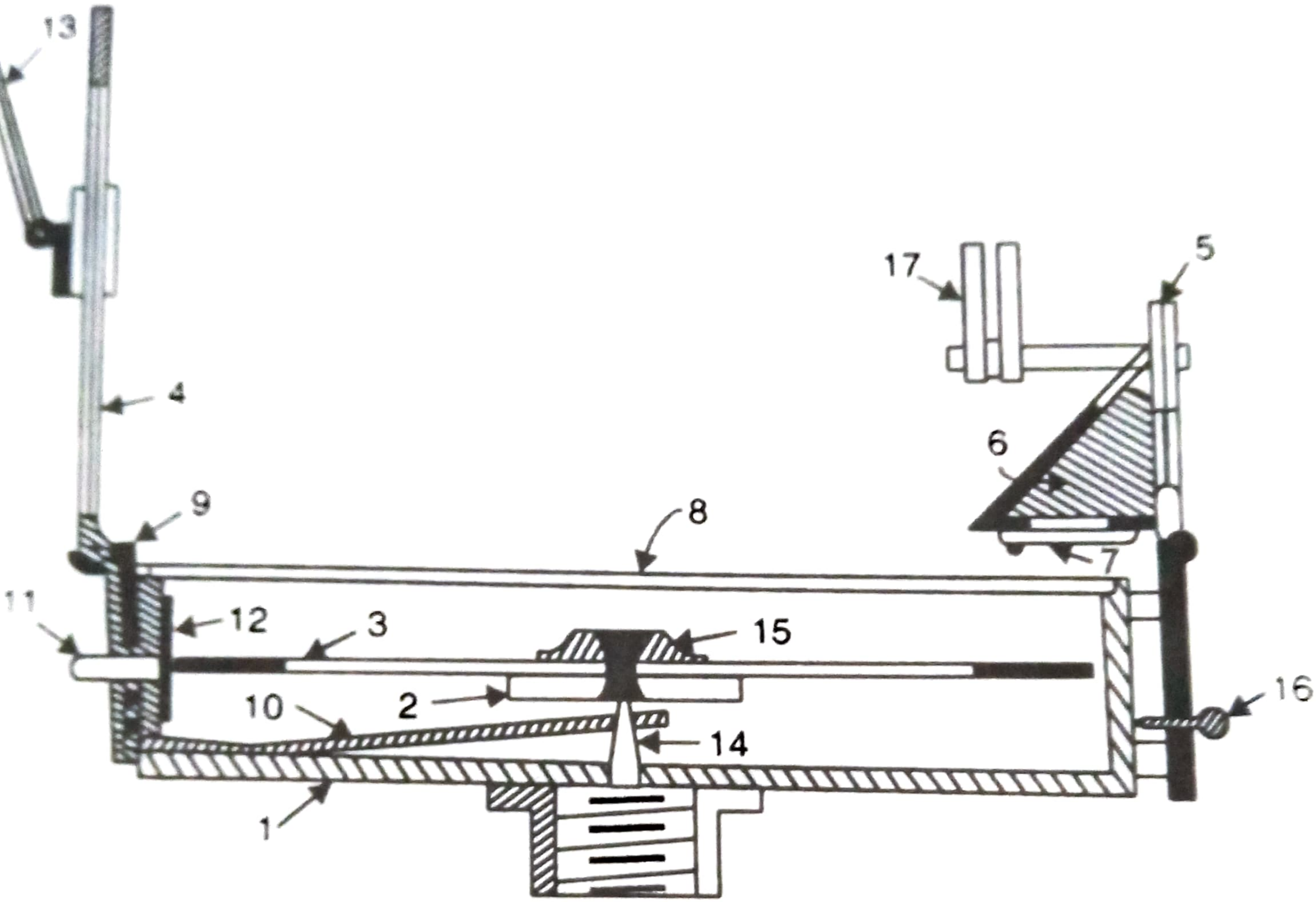
TABLES 5.3. DIFFERENCE BETWEEN SURVEYOR'S AND PRISMATIC COMPASS

Item	Prismatic Compass	Surveyor's Compass
(1) ***Magnetic Needle***	The needle is of 'broad needle' type. The needle does not act as index.	The needle is of 'edge bar' type. The needle acts as the index also.
(2) ***Graduated Card***	(*i*) The graduated card ring is attached with the needle. The ring does not rotate along with the line of sight. (*ii*) The graduations are in W.C.B. system, having 0° at South end, 90° at West, 180° at North and 270° at East. (*iii*) The graduations are engraved inverted.	(*i*) The graduated card is attached to the box and not to the needle. The card rotates along with the line of sight. (*ii*) The graduations are in Q.B. system, having 0° at N and S and 90° at East and West. East and West are interchanged. (*iii*) The graduations are engraved erect.
(3) ***Sighting Vanes***	(*i*) The object vane consists of metal vane with a vertical hair. (*ii*) The eye vane consists of a small metal vane with slit.	(*i*) The object vane consists of a metal vane with a vertical hair. (*ii*) The eye vane consists of a metal vane with a fine slit.
(4) ***Reading***	(*i*) The reading is taken with the help of a prism provided at the eye slit. (*ii*) Sighting and reading taking can be done simultane-ously from one position of the observer.	(*i*) The reading is taken by directly seeing through the top of the glass. (*ii*) Sighting and reading taking cannot be done simultaneously from one position of the observer.
(5) ***Tripod***	Tripod may or may not be provided. The instrument can be used even by holding suitably in hand.	The instrument cannot be used without a tripod.



1. Box
2. Magnetic needle
3. Sight vanes
4. Pivot
5. Jewel bearing
6. Glass top
7. Counter weight
8. Metal pin
9. Circular graduated arc
10. Lifting pin
11. Lifting lever

FIG. 5.14. THE SURVEYOR'S COMPASS.



1. Box
2. Needle
3. Graduated ring
4. Object vane
5. Eye vane
6. Prism
7. Prism cap
8. Glass cover
9. Lifting pin
10. Lifting lever
11. Brake pin
12. Spring brake
13. Mirror
14. Pivot
15. Agate cap
16. Focusing stud
17. Sun glass

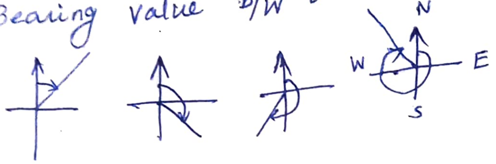
FIG. 5.12. THE PRISMATIC COMPASS.

System of Bearing

* Whole circle bearing system (WCB)
* Quadrantal bearing system (QB) (or) Reduced bearing (RB)

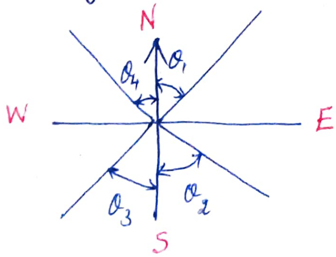
W.C.B

* The bearing of a line measured clockwise from the Magnetic North ~~pole~~ towards any ~~the~~ line is known as WCB.
* WCB of a line is obtained by prismatic compass.
* Used in India & United kingdom.
* Bearing value b/w 0° to 360°



QB (or) RB

* The bearing of a line measured clockwise or anticlockwise from Magnetic North or South ~~pole~~ towards the Eastward or westward is known as the Q.B (or) R.B.
* It consists of four quadrants NE, SE, NW & SW.
* The QB or RB is obtained by surveyor's Compass.
* Bearing value b/w 0° to 90°



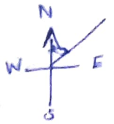
Sl No	WCB	Quadrant	RB
1	Between 0° and 90°	NE	RB = WCB
2	B/W 90° & 180°	SE	RB = 180° – WCB
3	B/W 180° & 270°	SW	RB = WCB – 180°
4	B/W 270° & 360°	NW	RB = 360° – WCB

Problem:

Convert the following WCB into quadrental bearing (RB or QB) (i) 56° (ii) 132° (iii) 253° 30′ (iv) 320° 50′.

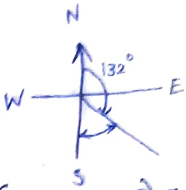
Solution:-

(i) 56°



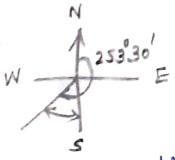
$QB = N\ 56°\ E$

(ii) 132°



$QB = S(180° - 132°)E$
$= S 48° E$

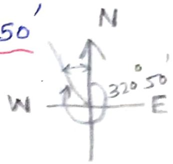
(iii) 253° 30′



$QB = S(270° - 253°30')W$

$QB = S\ 16°30'\ W$

(iv) 320° 50′



$QB = N(360° - 320°50')W$

$QB = N\ 39°10'\ W$

Convert the following WCB into Quadrantal bearing (i) 68°45′ (ii) 132° 15′ (iii) 236°30′ (iv) 335°45′

Solution:-

(i) 68°45′

The bearing is in 1st quadrant

$QB = N\ 68°45'\ E$

(ii) 132°15′

The bearing is in the second quadrant

$QB = 180° - 132°15'$

$QB = S\ 47°45'\ E$

(iii) $236°30'$

The bearing is in 3^{rd} quadrant

$QB = S(236°30' - 180°)W$

$QB = S\,56°30'\,W$

(iv) $335°45'$

The bearing is in 4^{th} quadrant

$QB = N(360° - 335°45')W$

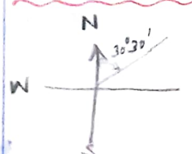
$QB = N\,24°15'\,W$

Convert QB in to WCB

1 Convert the following quadrantal bearing into whole circle bearing (i) $N\,30°30'\,E$ (ii) $S\,45°10'\,E$ (iii) $S\,50°30'\,W$ and (iv) $N\,75°20'\,W$.

Solution

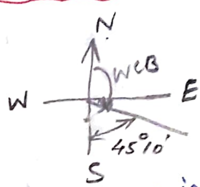
(i) $N\,30°30'\,E$



The bearing is in 1^{st} quadrant

$WCB = 30°30'$

(ii) $S\,45°10'\,E$

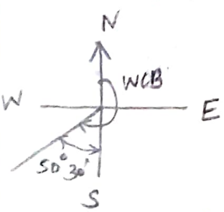


The bearing is in 2^{nd} quadrant

$WCB = 180° - 45°10'$

$WCB = 134°50'$

(iii) $S\,50°30'\,W$

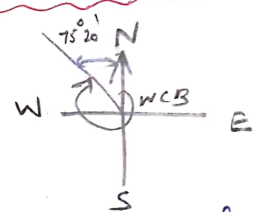


The bearing is in third quadrant

$WCB = 180° + 50°30'$

$= 230°30'$

(iv) $N\,75°20'\,W$.



The bearing is in fourth quadrant

$WCB = 360° - 75°20'$

$= 284°40'$

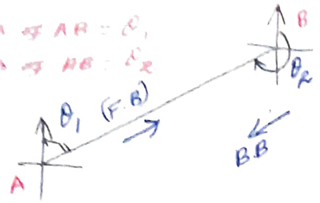
Fore bearing:-

Bearing of a line measured in the direction of progress of a survey is called fore bearing (F.B).

Back Bearing (B.B):-

Bearing of a line measured in the reverse direction of a survey ~~line~~ is called back bearing (BB).

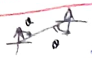
FB of AB = θ_1,
BB of AB = θ_2



Ⓧ Relationship b/w FB + BB

In general BB of a line is equal to FB of line $\pm 180^\circ$.

$\boxed{B.B = FB \pm 180^\circ}$



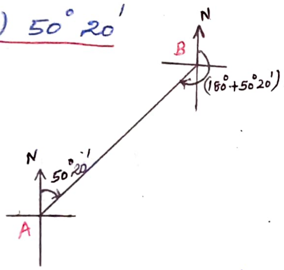
In WCB system

Problem:-

1, Convert the following fore bearings into back bearing: (i) $50^\circ 20'$ (ii) $120^\circ 10'$ (iii) $S\ 50^\circ 20'\ W$ (iv) $N\ 50^\circ 30'\ E$ (v) $220^\circ 40'$ (vi) $S\ 20^\circ 10'\ E$

Solution:-

(i) $50^\circ 20'$

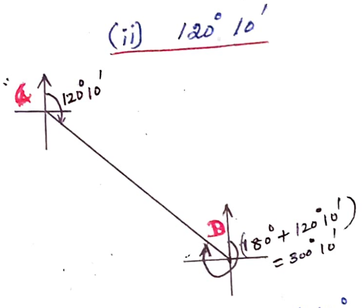


FB of $AB = 50^\circ 20'$

$\boxed{B.B \text{ of } AB = FB \pm 180^\circ}$

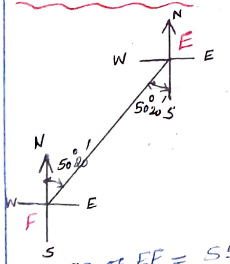
BB of $AB = 50^\circ 20' + 180^\circ$
$= 230^\circ 20'$

(ii) $120^\circ 10'$



BB of AB = FB $\pm 180^\circ$
$= 120^\circ 10' + 180^\circ$
$= 300^\circ 10'$

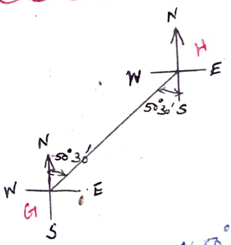
(iii) $S\ 50^\circ 20'\ W$



FB of EF = $S\ 50^\circ 20'\ W$

only change in opp. direction. BB of EF = $N\ 50^\circ 20'\ E$

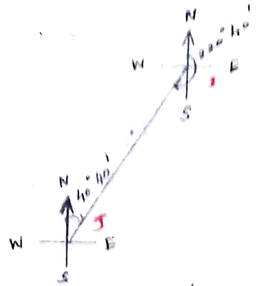
(iv) $N\ 50^\circ 30'\ E$

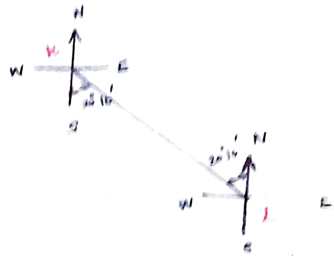


FB of GH = $N\ 50^\circ 30'\ E$

BB of GH = $S\ 50^\circ 30'\ W$

(v) $220° 40'$ (vi) ∴ $S\ 20° 10'\ E$





FB of IJ $= 220° 40'$
BB of IJ $= 220° 40' - 180°$
$= 40° 40'$

FB of KL $= S\ 20° 10'\ E$
BB of KL $= N\ 20° 10'\ W$

2, The following of the observed fore bearing of the lines. (i) AB $12° 25'$ (ii) BC $119° 40'$ (iii) CD $220° 36'$
(iv) DE $350° 15'$

Solution :-

(i) FB of $AB = 12° 25'$
∴ BB of $AB = 12° 25' + 180°$
$= 192° 25'$

(ii) FB of $BC = 119° 40'$
BB of $BC = 119° 40' + 180°$
$= 299° 40'$

(iii) FB of $CD = 220° 36'$
BB of $CD = 220° 36' - 180°$
$= 40° 36'$

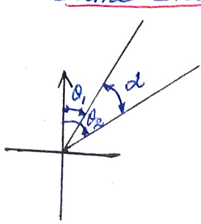
(iv) FB of $DE = 350° 15'$
BB of $DE = 350° 15' - 180°$
$= 170° 15'$

Problems.

Interior Angle calculation

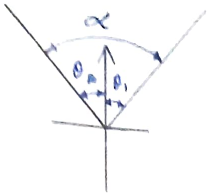
In Reduced Bearing system four cases are available.

① * When the two line formed included angle are in the same side of the same angle.



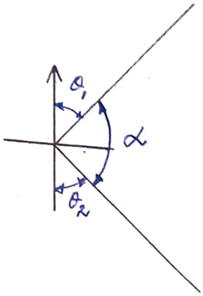
$$\boxed{\alpha = \theta_2 - \theta_1}$$

(1) * When the two line formed included angle are in the opposite side of the same angle.



$$\boxed{\alpha = \theta_1 + \theta_2}$$

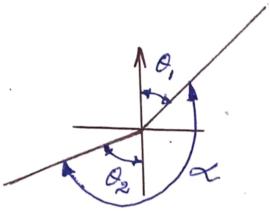
(2) * When the two line formed included angle are same side of the different angle.



$\alpha = 180 - (\theta_1 + \theta_2)$

ie, $\boxed{\alpha = 180 - \theta_1 - \theta_2}$

(3) * When the two line formed included angle are opposite side of the different angle.



$$\boxed{\alpha = 180^\circ - \theta_1 + \theta_2}$$

Clockwise direction :-

$\angle B$ = B.B of AB – FB of BC

[B.B. of previous line – FB of next line]

Anticlockwise direction:-

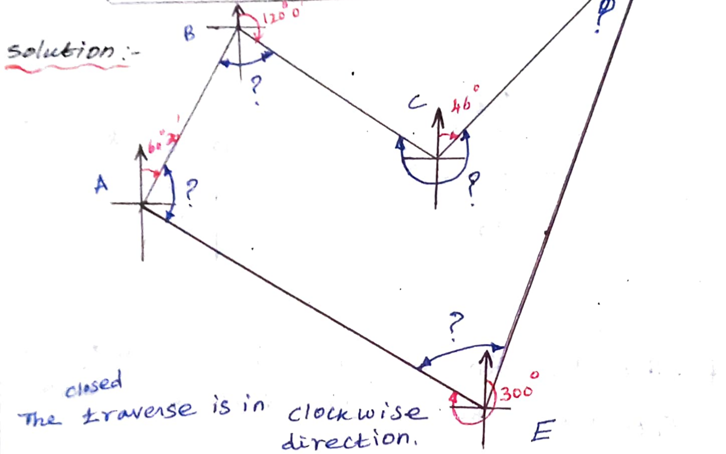
$\angle B$ = FB of AB – BB of BC

[FB of next line – BB of previous line]

The following bearings are taken in a closed ABCDE. Calculate the interior angle.

Line	Bearing
AB	60° 30′
BC	122° 0′
CD	46° 0′
DE	202° 30′
EA	300° 0′

Solution :-



The closed traverse is in clockwise direction.

Line	F.B	B.B.
AB	60° 30′	(60° 30′ + 180°) 240° 30′
BC	122° 0′	(122° + 180°) 302° 0′
CD	46° 0′	(46° + 180°) 226° 0′
DE	202° 30′	(202° 30′ − 180°) 22° 30′
EA	300° 0′	(300 − 180) 120

$\angle B$ = B.B of AB − FB of BC

= 240° 30′ − 122°

= 118° 30′

$\angle C$ = BB of BC − FB of CD

$= 302^\circ - 46^\circ$

$= 256^\circ 0'$ (exterior angle)

[illegible] is 104°.

$\angle D$ = BB of CD − FB of DE

$= 226^\circ 0' - 202^\circ 30'$

$= 23^\circ 30'$

$\angle E$ = BB of DE − FB of EA

$= 22^\circ 30' - 300^\circ$

$= -277^\circ 30'$ [Exterior angle]

ie, $= -277^\circ 30' + 360^\circ$

$= 82^\circ 30'$

$\angle A$ = BB of EA − FB of AB

$= 120^\circ - 60^\circ 30'$

$= 59^\circ 30'$

$\therefore$ Total Included angle $= \angle A + \angle B + \angle C + \angle D + \angle E$

$= 59^\circ 30' + 118^\circ 30' + \overset{104}{\cancel{256}}{}^\circ + 23^\circ 30' + 82^\circ 30'$

$= 540^\circ$

Check:

$(2n-4)\,90$

where n = number of angles $\quad n = 5$

$(2\times5-4)\,90 = 540^\circ$//

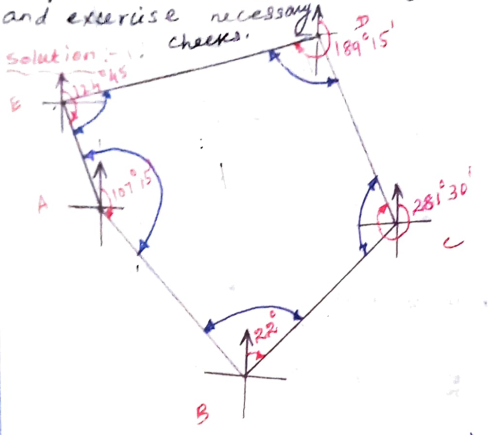
Hence it is Correct.

... bearings of the sides of a traverse ...BCDEA are as follows.

Line	FB	B.B
AB	107° 15'	287° 15'
BC	22° 0'	202° 0'
CD	281° 30'	101° 30'
DE	189° 15'	9° 15'
EA	124° 45'	304° 45'

compute the interior angles of the traverse and exercise necessary checks.

Solution :-



It is the traverse in anticlockwise direction.

$\angle B$ = F.B of BC − B.B of AB

$= 22^\circ - 287^\circ 15'$

$= -265^\circ 15'$ (Exterior angle

$\angle B = -265^\circ 15' + 360^\circ$

$= 94^\circ 45'$

$\angle C$ = FB of CD − BB of BC

$= 281^\circ 30' - 202^\circ 0'$

$= 79^\circ 30'$

$\angle D$ = FB of DE − B.B of CD

$= 189^\circ 15' - 101^\circ 30'$

$= 87^\circ 45'$

$\angle E$ = FB of EA − BB of DE

$= 124^\circ 45' - 9^\circ 15'$

$= 115^\circ 30'$

$\angle A$ = FB of AB − B.B of EA

$= 107^\circ 15' - 304^\circ 45'$

$= -197^\circ 30'$ (Exterior angle)

$\therefore \angle A = -197^\circ 30' + 360^\circ$

$= 162^\circ 30'$

$\therefore$ Total Included angle $= \angle A + \angle B + \angle C + \angle D + \angle E$

$= 162^\circ 30' + 94^\circ 45' + 79^\circ 30' + 87^\circ 45' + 115^\circ 30'$

$= 540^\circ 0'$

Check

$(2n-4)\,90 = (2 \times 5 - 4) \times 90$

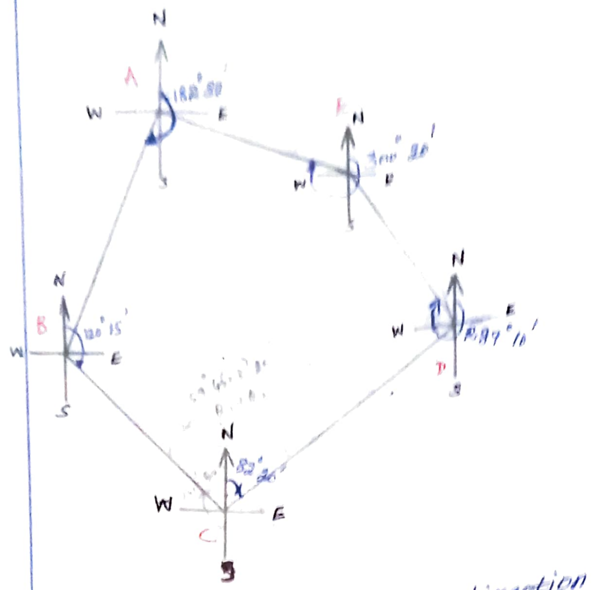
$= 540$

Hence it is correct.

3, The bearing of the sides of a traverse ABCDE are as follows.

Side	F.B	B.B
AB	$182^\circ 30'$	$2^\circ 30'$
BC	$120^\circ 15'$	$300^\circ 15'$
CD	$82^\circ 20'$	$262^\circ 20'$
DE	$289^\circ 10'$	$109^\circ 10'$
EA	$300^\circ 20'$	$120^\circ 20'$

Compute the interior angles of the traverse.



It is in anticlockwise direction.

$\angle A$ = FB of AB − B.B of EA

$= 182°30' - 120°20'$

$= 62°10'$.

$\angle B$ = FB of BC − B.B of AB

$= 120°15' - 2°30'$

$= 117°45'$

$\angle C$ = FB of CD − BB of BC

$= 82°20' - 300°15'$

$= {*}142°5'$

$\angle D$ = FB of DE − BB of CD

$= 289°10' - 262°20'$

$= 26°50'$

$\angle E$ = FB of EA − BB of DE

$= 300°20' - 109°10'$

$= 191°10'$

Total Included angle $= \angle A + \angle B + \angle C + \angle D + \angle E$

$= 540°$

<u>Check</u>

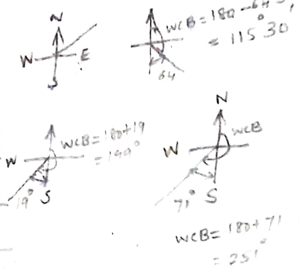
$(2n-4)\,90 = (2\times5-4)\,90$

$= 540$

Hence it is correct.

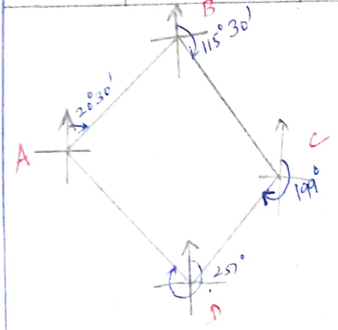
4) Calculate the included angles from the following reduced bearing of a traverse ABCD.

Line	FB	B.B
AB	N 20°30' E	S 20°30' W
BC	S 64°30' E	N 64°30' W
CD	S 19°0' W	N 19°0' E
DA	S 71°0' W	N 71°0' E



Solution:-

Line	R.B FB	R.B B.B	WCB FB	WCB BB
AB	N 20°30' E	S 20°30' W	20°30' +180°	200°30'
BC	S 64°30' E	N 64°30' W	115°30' +180	295°30'
CD	S 19°0' W	N 19°0' E	199°0' −180	19°0'
DA	S 71°0' W	N 71°0' E	251°0'	71°0'



It is in ~~anti~~ clock wise direction

$\angle$ = B.B of previous line − FB of next line

$\angle A$ = BB of DA − FB of AB

$= 71^\circ 0' - 20^\circ 30'$

$= 50^\circ 30'$

$\angle B$ = BB of AB − FB of BC

$= 200^\circ 30' - 115^\circ 30'$

$= 85^\circ 0'$

$\angle C$ = BB of BC − FB of CD

$= 295^\circ 30' - 199^\circ 0'$

$= 96^\circ 30'$

$\angle D$ = BB of CD − FB of DA

$= 19^\circ 0' - 251^\circ$

$= -232^\circ$ (exterior angle)

$\therefore \angle D$ interior angle $= -232^\circ + 360^\circ$

$= 128^\circ 0'$

$\therefore$ Total included angle $= \angle A + \angle B + \angle C + \angle D$

$= 50^\circ 30' + 85^\circ 0' + 96^\circ 30' + 128^\circ 0'$

$= 360^\circ$

Check

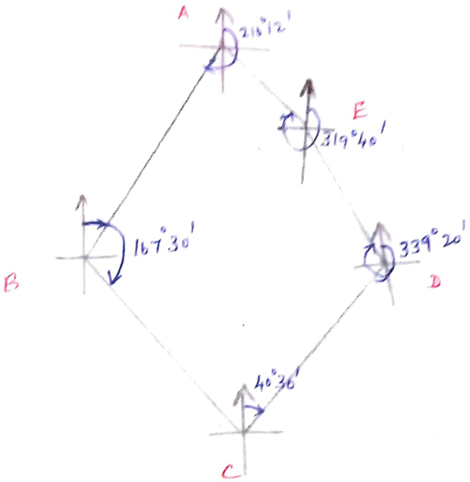
$(2n-4)90 = (2\times4-4)90 = 360^\circ$

Hence it is correct.

Calculate the included angle from the following reduced bearing of a traverse ABCDE.

Line	F.B
AB	S 30°12' W
BC	S 12°30' E
CD	N 40°30' E
DE	N 20°40' W
EA	N 40°20' W

Line	F.B	B.B	W.C.B F.B	W.C.B B.B
AB	S 30°12' W	N 30°12' E	210°12'	30°12'
BC	S 12°30' E	N 12°30' W	167°30'	347°30'
CD	N 40°30' E	S 40°30' W	40°30'	220°30'
DE	N 20°40' W	S 20°40' E	339°20'	159°20'
EA	N 40°20' W	S 40°20' E	319°40'	139°40'



It is in anticlock wise direction.

$\angle A$ = FB of AB − BB of EA
$= 210°12' - 139°40' = \underline{70°32'}$

$\angle B$ = F.B of BC − BB of AB
$= 167°30' - 30°12' = \underline{137°18'}$

$\angle C$ = FB of CD − B.B of BC
$= 40°30' - 347°30' = -307°0' + 360$
Exterior angle
$= \underline{53°0'}$

$\angle D$ = FB of DE − BB of CD

$= 339°20' - 220°30'$

$= 118°50'$

$\angle E$ = FB of EA − BB of DE

$= 319°40' - 159°20'$

$= 160°20'$

$\therefore$ Total included angle $= \angle A + \angle B + \angle C + \angle D + \angle E$

$= 70°32' + 137°18' + 53° + 118°50' + 160°20'$

$= 540°$

Check

$(2n-4)\,90 = (2\times5-4)\times 90 = 540°$

[illegible]

Hence it is correct.

Type-II

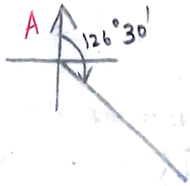
Calculation of bearing from included Angles

1. The following are the interior angles that were observed while surveying among the enclosed traverse ABCDE, $\angle A = 100°30'$; $\angle B = 90°30'$; $\angle C = 66°30'$; $\angle D = 122°45'$; $\angle E = 159°45'$. Find the bearing of the remaining side of the traverse. If the F.B of the line AB is $126°30'$.

Solution:-

The F.B of AB is $126°30'$

$\therefore$



$\therefore$ The F.B of AB is $126°30'$

ie, lies in the 2nd quadrant and

the traverse is in anticlockwise direction

Formula for Anticlockwise direction

$$\boxed{\text{FB of } BC = \text{FB of } AB + \angle B \pm 180^\circ}$$

Formula for clockwise direction

$$\boxed{\text{F.B of } BC = \text{FB of } AB - \angle A \pm 180^\circ}$$

F.B of Line BC = FB of $AB + \angle B \pm 180^\circ$

$= 126^\circ 30' + 90^\circ 30' - 180^\circ$

$\boxed{\text{FB of } BC = 37^\circ 0'}$

FB of CD = FB of $BC + \angle C \pm 180^\circ$

$= 37^\circ + 66^\circ 30' + 180^\circ$

$\boxed{\text{FB of } CD = 283^\circ 30'}$

FB of DE = FB of $CD + \angle D \pm 180^\circ$

$= 283^\circ 30' + 122^\circ 45' - 180^\circ$

$\boxed{\text{FB of } DE = 226^\circ 15'}$

FB of EA = FB of $DE + \angle E \pm 180^\circ$

$= 226^\circ 15' + 159^\circ 45' - 180^\circ$

$\boxed{\text{FB of } EA = 206^\circ 0'}$

FB of AB = F.B of $EA + \angle A \pm 180^\circ$

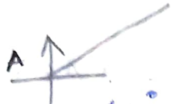
$= 206^\circ + 100^\circ 30' - 180^\circ$

$\boxed{\text{FB of } AB = 126^\circ 30'}$

Hence it is correct.

2. The following interior angle measured with Sextant in a closed traverse. The bearing a line AB as measured 60° with the Prismatic compass. calculate the bearing of all other lines. If $\angle A = 140°10'$; $\angle B = 90°8'$; $\angle C = 60°22'$; $\angle D = 69°20'$

Solution:-



FB of AB = 60° (It is in 1st quadrant)

∴ It is in clockwise direction.

The formula for clockwise direction.

F.B of BC = FB of AB − $\angle A$ ± 180°

$= 60° - 140°10' + 180°$

$\boxed{\text{FB of BC} = 99°50'}$

FB of CD = FB of BC − $\angle B$ ± 180°

$= 99°50' - 90°8' + 180°$

$\boxed{\text{FB of CD} = 189°42'}$

FB of DA = FB of CD − $\angle C$ ± 180°

$= 189°42' - 60°22' + 180°$

$\boxed{\text{FB of DA} = 309°20'}$

F.B of AB = FB of DA − $\angle A$ ± 180°

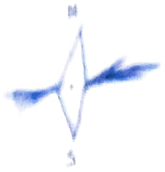
$= 309°20' - 140°10' - 180°$

$\boxed{\text{FB of AB} = 60°}$

Hence it is correct.

Magnetic Dip

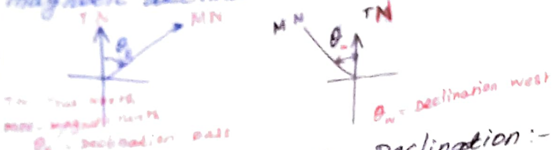
The angle of inclination of the magnetic needle with the horizontal plane is known as the magnetic dip



Magnetic Declination.

The horizontal angle b/w the true meridian & magnetic meridian of a place at the time of observation is known as magnetic declination.

W → (-)
E → (+)



Variation in Magnetic Declination:-

Earth magnetism is not uniform and regular, hence the declination at a place is not constant, and changes from time to time. The changes in magnetic declination are known as magnetic variation.

Types of variation:-

(i) secular variation
(ii) Annual variation
(iii) Diurnal variation → (Day to day)
(iv) Irregular variation

secular variation:-

* The variation of the magnetic meridian over a number of years.
* It is not uniform with reference to place
* changes direction in 150 to 200 years
* It is most impartant variation. & of much concern in a survey work.
*

Annual Variation :- $1'$ or $2'$

* The variation over a period of one year from its mean ~~point~~ position is called annual variation.
* The variation is in the order of one or two minutes.

Diurnal Variation :- $12'$

* The variation of the magnetic needle position over a day.
* Magnitude up to 12 minutes depending on season, time of the day, etc

Irregular Variation :-

* Such variation may be caused due to sudden natural causes such as earthquakes, valcanic erruptions, etc.

Determination of True bearing :-

The direction of magnetic meridian at a place changes with time, important surveys are plotted with reference to true meridian.

$$\boxed{\text{True bearing} = \text{Magnetic bearing} \pm \text{Declination}}$$

Problem :-

1. Calculate the true bearing, if magnetic bearing of a line is $68°30'$ and magnetic declination is $3°30'$ E.

Solution :-

$$\begin{aligned}\text{True bearing} &= \text{Magnetic bearing} \pm \text{Declination} \\ &= 68°30' + 3°30' \\ &= 72°0'\end{aligned}$$

2. If the magnetic bearing of a line is $57^\circ 30'$ and the magnetic declination at the place is $2^\circ 0'$ W. calculate the true bearing.

Solution:

$$T.B = M.B \pm \text{Declination}$$

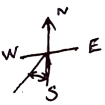
$$= 57^\circ 30' - 2^\circ 0'$$

$$= 55^\circ 30'$$

3. The magnetic of a line AB is S $35^\circ 45'$ W. If the declination is $8^\circ 15'$ E. Find the true bearing.

Solution:

1^{st} the R.B is converted in to WCB



$$R.B \text{ of } AB = S\ 35^\circ 45'\ W$$

$$WCB \text{ of } AB = 180^\circ + 35^\circ 45'$$

$$= 215^\circ 45'$$

True Bearing = Magnetic bearing $\pm$ declination

$$T.B = 215^\circ 45' + 8^\circ 15'$$

$$= 223^\circ 0'$$

$$\text{True bearing in } RB = 223^\circ 0' - 180^\circ$$

$$= S\ 43^\circ 0'\ W$$

4, A survey line AB was found in an old map which was drawn to a magnetic bearing of $220^\circ 45'$ when the magnetic declination was ~~[illegible]~~ $3^\circ 17'$ E. If the present magnetic declination is $2^\circ 45'$ W. calculate the required magnetic bearing to set the line now.

True bearing = Magnetic bearing ± declination

T.B of AB = M.B of AB + declination in ...

$= 220°45' + 3°17'$

$= 224°02'$

The present declination = $2°45'$ W

~~Magnetic bearing of AB = True bearing.~~

T.B = M.B − declination in west

$224°02' = MB - 2°45'$

$\therefore MB = 224°02' + 2°45'$

$= 226°47'$

∴ The line has to be set now to the bearing of 226°47'

Local attraction :

The magnetic needle does not point towards magnetic north due to the substance like magnetic rock, ranging rod, key punch iron button etc. which defect the magnetic needle from its normal position. Such disturbing substance are known as local attraction.

(or)

The disturbance due to the presence of magnetic field is called local attraction.

Sources of local attraction:-

The materials which may influence magnetic action are presence of

(i) Magnetic rocks
(ii) Iron ore deposits
(iii) Steel structure
(iv) Railways
(v) Iron lamp posts.
(vi) Transmission towers.

commonly used iron materials,

bunch of iron keys
knife, iron buttons etc.,
chain, arrows etc.

Problem:

1. The following bearings were observed in a compass traverse.

Line	F.B	B.B.
AB	305° 00'	125° 30'
BC	75° 30'	254° 30'
CD	115° 30'	297° 00'
DE	165° 30'	345° 30'
EA	225° 0'	44° 0'

At which of stations would local attraction be suspected? Find the corrected bearing of the line.

Solution:-

observed Reading.

Line	FB	B.B	F.B ~ B.B
AB	305° 0'	125° 30'	179° 30'
BC	75° 30'	254° 30'	179° 0'
CD	115° 30'	297° 0'	181° 30'
DE	165° 30'	345° 30'	180° 0'
EA	225° 0'	44° 0'	181° 0'

station D is unaffected by local attractions,

∴ The observed FB & B.B of DE are correct.

* observed B.B of CD is correct
* observed FB of EA is correct

Correct FB of EA = $225^\circ 0'$

∴ correct B.B of EA = $225^\circ 0' - 180^\circ$

$= 45^\circ 0'$

observed B.B of EA = $44^\circ 0'$

correction at A = corrected B.B of A − observed BB of A

$= 45^\circ 0' - 44^\circ 0'$

$= 1^\circ 0'$

observed ~~corrected~~ FB of AB = $305^\circ 0'$

∴ Corrected FB of AB = $305^\circ 0' + 1^\circ 0'$

$= 306^\circ 0'$

corrected BB of AB = $306^\circ 0' - 180^\circ$

$= 126^\circ 0'$

observed BB of AB = $125^\circ 30'$

∴ correction at B = corrected BB of B − observed BB of B

$= 126^\circ 0' - 125^\circ 30'$

$= 0^\circ 30'$

observed FB of BC = $75^\circ 30'$

∴ corrected FB of BC = $75^\circ 30' + 0^\circ 30'$

$= 76^\circ 0'$

Correct BB of BC = $76° + 180°$
$= 256° 0'$

observed BB of BC = $254° 30'$

∴ correction at C = $256° 0' - 254° 30'$
$= 1° 30'$

observed FB of CD = $115° 30'$

∴ corrected FB of CD = $115° 30' + 1° 30'$
$= 117° 0'$

∴ Correct BB of CD = $117° 0' + 180°$
$= 297° 0'$

observed BB of CD = $297° 0'$

~~corrected BB of~~

correction at D = $297° 0' - 297° 0'$
$= 0°$

Line	FB	B.B	Correction	Corrected FB	corrected B.B
AB	305° 0'	125° 30'	A = 1° 0'	306° 0'	126° 0'
BC	75° 30'	254° 30'	B = 0° 30'	76° 0'	256° 0'
CD	115° 30'	297° 0'	C = 1° 30'	117° 0'	297° 0'
DE	165° 30'	345° 30'	D = 0°	165° 30'	345° 30'
EA	225° 0'	44° 0'	E = 0° 0'	225° 0'	45° 0'

2. The following bearings were recorded while conducting a closed traverse using a compass. Find out the stations affected by local attraction and determine the corrected bearings.

[illegible] Univ [illegible]/June 2007.

Line	AB	BC	CD	DA
FB	45° 45'	96° 55'	29° 45'	324° 48'
B.B	226° 10'	277° 05'	209° 10'	144° 48'

Line	FB	BB	FB ~ BB
AB	$45°45'$	$226°10'$	$180°25'$
BC	$96°55'$	$277°05'$	$180°10'$
CD	$29°45'$	$209°10'$	$179°25'$
DA	$324°48'$	$144°48'$	$180°0'$

The ~~lines~~ stations ~~DA~~ D & A is ~~correct~~ unaffected by local attraction.

∴ The observed FB & B.B of DA is correct, & also BB of CD, FB of AB is correct

correct FB of AB $= 45°45'$

∴ corrected BB of AB $= 45°45' + 180°$

$= 225°45'$

observed BB of AB $= 226°10'$

∴ error $= 225°45' - 226°10'$

$= -0°25'$

∴ observed FB of BC $= 96°55'$

∴ corrected FB of BC $= 96°55' - 0°25'$

$= 96°30'$

corrected BB of BC $= 96°30' + 180°$

$= 276°30'$

observed BB of BC $= 277°05'$

∴ error $= 276°30' - 277°05'$

$= -0°35'$

observed FB of CD $= 29°45'$

∴ corrected FB of CD $= 29°45' - 0°35'$

$= 29°10'$

corrected BB of CD = 29°10' + 180°
= 209°10'
observed BB of CD = 209°10'
error = 209°10' - 209°10'
= 0

Hence it is

Line	observed FB	observed BB	correction	corrected FB	corrected BB
AB	45°45'	226°10'	A = 0°0'	45°45'	225°45'
BC	96°55'	277°05'	B = -0°25'	96°30'	276°30'
CD	29°45'	209°10'	C = -0°35'	29°10'	209°10'
DA	324°48'	144°48'	D = 0°0'	324°48'	144°48'

3 The following bearings were observed while traversing with a compass.

Line	FB	BB	Line	FB	B.B
AB	75°05'	254°20'	DE	224°50'	44°05'
BC	115°20'	296°35'	EA	304°50'	125°05'
CD	165°35'	345°35'			

Mention which stations were affected by local attraction and determine the corrected bearings.

Solution:-

Line	FB	B.B	FB ~ B.B
AB	75°05'	254°20'	179°15'
BC	115°20'	296°35' ✓	181°15'
CD	165°35' ✓	345°35' ✓	180°0'
DE	✓ 224°50'	44°05'	180°45'
EA	304°50'	125°05'	179°45'

The stations C & D is free from local attraction.

∴ The observed FB & BB of CD is correct.

∴ The observed FB of DE & B.B of BC is also correct.

The correct observed FB of DE = 224°50'

∴ Correct BB of DE = 224°50' − 180°

= 44°50'

Observed BB of DE = 44°05'

error = 44°50' − 44°05'

= 0°45'

∴ Observed FB of EA = 304°50'

∴ Corrected FB of EA = 304°50' + 0°45'

= 305°35'

∴ Correct BB of EA = 305°35' − 180°

= 125°35'

Observed BB of EA = 125°05'

error = 125°35' − 125°05'

= 0°30'

Observed FB of AB = 75°05'

∴ Corrected FB of AB = 75°05' + 0°30'

= 75°35'

∴ Corrected BB of AB = 75°35' + 180°

= 255°35'

Observed BB of AB = 254°20'

error = 255°35' − 254°20'

= 1°15'

Observed FB of BC = 115°20'

∴ Corrected FB of BC = 115°20' + 1°15'

= 116°35'

∴ Corrected BB of BC = 116°35' + 180°

= 296°35'

observed BB of BC = $296^\circ 35'$

$\therefore$ error = $296^\circ 35' - 296^\circ 35'$

= 0.

Line	observed FB	observed BB	Correction	corrected FB	corrected BB
AB	$75^\circ 05'$	$254^\circ 20'$	A = $0^\circ 30'$	$75^\circ 35'$	$255^\circ 35'$
BC	$115^\circ 20'$	$296^\circ 35'$	B = $0^\circ 45'$	$116^\circ 35'$	$296^\circ 35'$
CD	$165^\circ 35'$	$345^\circ 35'$	C = 0	$165^\circ 35'$	$345^\circ 35'$
DE	$224^\circ 50'$	$44^\circ 05'$	D = 0	$224^\circ 50'$	$44^\circ 50'$
EA	$304^\circ 50'$	$125^\circ 05'$	E = $1^\circ 15'$	$305^\circ 35'$	$125^\circ 35'$

4. The following bearing observed in running a prismatic compass traverse in a place where local attraction is subjected.

Line	FB	BB
AB	$92^\circ 30'$	$272^\circ 30'$
BC	$10^\circ 15'$	$190^\circ 0'$
CD	$211^\circ 0'$	$34^\circ 0'$
DE	$112^\circ 30'$	$288^\circ 0'$
EA	$15^\circ 30'$	$197^\circ 15'$

Solution :-

Line	FB	BB	FB ~ BB
AB	$92^\circ 30'$ ✓	$272^\circ 30'$ ✓	$180^\circ 0'$
BC	$10^\circ 15'$ ✓	$190^\circ 0'$	$179^\circ 45'$
CD	$211^\circ 0'$	$34^\circ 0'$	$177^\circ 0'$
DE	$112^\circ 30'$	$288^\circ 0'$	$175^\circ 30'$
EA	$15^\circ 30'$	$197^\circ 15'$ ✓	$181^\circ 45'$

The stations A & B is free from local attraction.

$\therefore$ The observed FB of AB & BB of AB is correct, & also FB of BC & B.B of EA is correct.

Correct observed FB of BC = $10^\circ 15'$

Corrected BB of BC = $10^\circ 15' + 180^\circ$

$= 190^\circ 15'$

Observed BB of BC = 190°

Correction @ station C = $190^\circ 15' - 190^\circ$ $0^\circ \ldots$

Observed FB of CD = $211^\circ 0'$

Corrected FB of CD = $211^\circ 0' + \text{error}$

$= 211^\circ 0' + 0^\circ 15'$

$= 211^\circ 15'$

∴ Corrected BB of CD = $211^\circ 15' - 180^\circ$

$= 31^\circ 15'$

Observed BB of CD = $34^\circ 0'$

Correction at station D = $21^\circ 15' - 34^\circ 0' = -2^\circ 45'$

Observed FB of DE = $112^\circ 30'$

∴ Corrected FB of DE = $112^\circ 30' - 2^\circ 45'$

$= 109^\circ 45'$

Corrected BB of DE = $109^\circ 45' + 180^\circ 0'$

$= 289^\circ 45'$

Observed BB of DE = $288^\circ 0'$

Correction at station E = $289^\circ 45' - 288^\circ$

$= 1^\circ 45'$

Observed FB of EA = $15^\circ 30'$

Corrected FB of EA = $15^\circ 30' + 1^\circ 45'$

$= 17^\circ 15'$

Correct BB of EA = $17^\circ 15' + 180^\circ = 197^\circ 15'$

Observed BB of EA = $197^\circ 15'$

Correction at station A = $197^\circ 15' - 197^\circ 15'$

$= 0$

Line	observed FB	observed BB	Correction	Corrected FB	corrected BB
AB	92° 30'	272° 30'	A = 0	92° 30'	272° 30'
BC	10° 15'	190° 0'	B = 0	10° 15'	190° 15'
CD	211° 0'	34° 0'	C = 0° 15'	211° 15'	31° 15'
DE	112° 30'	288° 0'	D = -2° 45'	109° 45'	289° 45'
EA	15° 30'	197° 15'	E = 1° 45'	17° 15'	197° 15'

Method II

1. Following bearing where measured in a closed traverse running the direction. Calculate the included angles of traverse angles. and also correct the bearing of the lines.

Line	FB	B.B
AB	290° 15'	109° 15'
BC	219° 45'	39° 45'
CD	88° 5'	268° 0'
DE	78° 35'	259° 40'
EA	35° 0'	214° 0'

Solution:-

It is an Anti-clockwise direction.

$\boxed{\angle A = \text{F.B of next line} - \text{B.B of previous line}}$

$\angle B = 219°45' - 109°15' = 110°30'$

$\angle A = 290°15' - 214°0' = 76°15'$

$\angle C = 88°5' - 39°45' = 48°20'$

$\angle D = 78°35' - 268°0' = -189°25'$ (exterior angle)

$= -189°25' + 360°$

$\angle D = 170°35'$

$\angle E = 35°0' - 259°40' = -224°40'$ (exterior angle)

$\therefore \angle E = -224°40' + 360°$

$\angle E = 135°20'$

$\therefore$ Total included angle $= \angle A + \angle B + \angle C + \angle D + \angle E$

$= 76°15' + 110°30' + 48°20' + 170°35' + 135°20'$

Total Included angle = $541^\circ 0$

Check

$(2n-4)90 = (2\times5-4)90 = 540^\circ$

Error $= 541 - 540$

$= 1^\circ$

sharing for 5 station

$e = \frac{1^\circ}{5} = 0^\circ 12'$

$\angle A = 76^\circ 15' - 0^\circ 12' = 76^\circ 03'$

$\angle B = 110^\circ 30' - 0^\circ 12' = 110^\circ 18'$

$\angle C = 48^\circ 20' - 0^\circ 12' = 48^\circ 08'$

$\angle D = 170^\circ 35' - 0^\circ 12' = 170^\circ 23'$

$\angle E = 135^\circ 20' - 0^\circ 12' = 135^\circ 08'$

Line	FB	B.B	FB ~ B.B
AB	$290^\circ 15'$	$109^\circ 15'$	$181^\circ 0'$
BC	$219^\circ 45'$	$39^\circ 45'$	$180^\circ 0'$
CD	$88^\circ 05'$	$268^\circ 0'$	$179^\circ 55'$
DE	$78^\circ 35'$	$259^\circ 40'$	$181^\circ 05'$
EA	$35^\circ 0'$	$214^\circ 0'$	$179^\circ 0'$

~~The station Code is free from local attraction.~~

$\angle C$ = FB of CD − B.B of BC

FB of CD = $\angle C$ + BB of BC

$= 48^\circ 08' + 39^\circ 45' = 87^\circ 53'$

BB of CD $= 87^\circ 53' + 180^\circ 0' = 267^\circ 53'$

$\angle D$ = FB of DE − BB of CD

FB of DE = $\angle D$ + BB of CD

$= 170°23' + 267°53' = 438°16' - 360°$ (Exceed 360°) ∴ −360°

FB of DE $= 78°16'$

B.B of DE $= 78°16' + 180° = 258°16'$

$\angle E$ = FB of EA − B.B of DE

FB of EA = $\angle E$ + BB of DE

$= 135°08' + 258°16' = 393°24' - 360°$ (Exceed 360°) ∴ −360°

FB of EA $= 33°24'$

BB of EA $= 33°24' + 180° = 213°24'$

$\angle A$ = FB of AB − BB of EA

FB of AB = $\angle A$ + BB of EA

$= 76°03' + 213°24' = 289°27'$

B.B of AB $= 289°27' - 180° = 109°27'$

$\angle B$ = FB of BC − BB of AB

FB of BC = $\angle B$ + BB of AB

$= 110°18' + 109°27' = 219°45'$

BB of BC $= 219°45' - 180° = 39°45'$

$\angle C$ = FB of CD − BB of BC

FB of CD = $\angle C$ + BB of BC

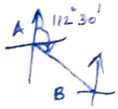
$= 48°08' + 39°45' = \underline{87°53'}$

Line	Corrected FB	Corrected BB
AB	289°27'	109°27'
BC	219°45'	39°45'
CD	87°53'	267°53'
DE	78°16'	258°16'
EA	33°24'	213°24'

In running a compass traverse for the following observation were made with a prismatic compass. Compute the interior angle and correct for observation error. Also calculate the corrected bearing of the line

Line	FB	B.B
AB	112°30'	292°30'
BC	14°15'	200°15'
CD	315°0'	130°0'
DA	215°15'	32°15'

Solution:-



It is in Anti-clockwise direction

$$\boxed{\angle A = \text{FB of next line} - \text{BB of Previous line}}$$

$\therefore \angle A = 112°30' - 32°15' = 80°15'$

$\angle B = 14°15' - 292°30' = -278°15'$ (exterior angle)

$= -278°15' + 360°$

$\angle B = 81°45'$

$\angle C = 315°0' - 200°15' = 114°45'$

$\angle D = 215°15' - 130°0' = 85°15'$

$\therefore$ Total included angle $= \angle A + \angle B + \angle C + \angle D$

$= 80°15' + 81°45' + 114°45' + 85°15'$

$= 362°0'$

check:

$(2n-4)90 = (2\times4-4)90 = 360°$

Error $= 362 - 360° = 2°$

sharing for four stations

$e = \frac{2°}{4} = 0°30'$

$\angle A = 80°15' - 0°30' = 79°45'$

$\angle B = 81°45' - 0°30' = 81°15'$

$\angle C = 114°45' - 0°30' = 114°15'$

$\angle D = 85°15' - 0°30' = 84°45'$

Line	FB	BB	FB ~ BB
AB	112°30'	292°30'	180°0'
BC	14°15'	200°15'	186°0'
CD	315°0'	130°0'	185°0'
DA	215°15'	32°15'	183°0'

$\angle B$ = FB of BC − BB of AB

FB of BC = $\angle B$ + BB of AB

$= 81°15' + 292°30' = 373°45' - 360$

$\boxed{\text{FB of BC} = 13°45'}$

B.B of BC $= 13°45' + 180° = \underline{193°45'}$

$\angle C$ = FB of CD − BB of BC

FB of CD = $\angle C$ + BB of BC

$= 114°15' + 193°45' = \underline{308°0'}$

BB of CD $= 308°0' - 180° = \underline{128°0'}$

$\angle D$ = FB of DA − BB of CD

FB of DA = $\angle D$ + BB of CD $= 84°45' + 128°0'$

$= \underline{212°45'}$

BB of DA $= 212°45' - 180° = \underline{32°45'}$

$\angle A$ = FB of AB − BB of BC

FB of AB = $\angle A$ + BB of DA $= 79°45' + 32°45'$

$= 112°30'$

BB of AB $= 112°30' + 180° = 292°30'$

Line	Corrected FB	Corrected BB
AB	113° 30'	293° 30'
BC	13° 45'	193° 45'
CD	308° 0'	128° 0'
DA	212° 45'	32° 45'

The following bearing of closed traverse

Line	FB	BB
AB	80°10'	259°0'
BC	120°20'	301°50'
CD	170°50'	350°50'
DE	230°10'	49°30'
EA	310°20'	130°15'

Compute the interior angle and correct them for observational error. Determine the correct bearing of the line

Solution:-

Line	FB	B.B
AB	80°10'	259°0'
BC	120°20'	301°50'
CD	170°50'	350°50'
DE	230°10'	49°30'
EA	310°20'	130°15'

It is in a clockwise direction

$\angle A$ = BB of EA − FB of AB
= 130°15' − 80°10' = 50° 5'

$\angle B$ = BB of AB − FB of BC
= 259°0' − 120°20' = 138°40'

$\angle C$ = BB of BC − FB of CD
= 301°50' − 170°50' = 131°0'

$\angle D$ = BB of CD − FB of DE
= 350°50' − 230°10' = 120°40'

... DE − FB of EA

$= 49°30' - 310°20' = -260°50' + 360$

$= 99°10'$

Total Included angle $= 50°5' + 138°40' + 131°0' + 120°40' + 99°10'$

$= 539°35'$

Checked angle $= (2n-4)90 = (2\times5-4)90$

$= 540°$

correction $= 540 - 539°35' = 0°25'$

correction Per angle $= \dfrac{0°25'}{\text{no. of angle}} = \dfrac{0°25'}{5}$

$= 0°5'$

$\angle A = 50°5' + 0°5' = 50°10'$

$\angle B = 138°40' + 0°5' = 138°45'$

$\angle C = 131°0' + 0°5' = 131°05'$

$\angle D = 120°40' + 0°5' = 120°45'$

$\angle E = 99°10' + 0°5' = 99°15'$

Line	FB	BB	B.B ~ F.B
AB	80°10'	259°0'	178°50'
BC	120°20'	131°50'	181°30'
CD	170°50'	350°50'	180°0'
DE	230°10'	49°30'	180°40'
EA	310°20'	130°15'	180°5'

$\angle D$ = BB of CD − FB of DE

FB of DE = BB of CD − $\angle D$

FB of DE $= 350°50' - 120°45' = 230°5$

BB of DE $= 230°5' - 180° = 50°5'$

$\angle E$ = BB of DE − FB of EA

FB of EA = ~~$\angle E$~~ BB of DE − $\angle E$

$= 50°5' - 99°15' = -49°10' + 360$

FB of EA $= 310°50'$

BB of EA $= 310°50' - 180° = 130°50'$

FB of AB = BB of EA − ∠A = 130° 50' − 50° 10'

FB of AB = 80° 40'

BB of AB = 80° 40' + 180° = 260° 40'

∠B = BB of AB − FB of BC

FB of BC = BB of AB − ∠B = 260° 40' − 138° 45'

FB of BC = 121° 55'

BB of BC = 121° 55' + 180° = 301° 55'

∠C = BB of BC − FB of CD

FB of CD = BB of BC − ∠C = 301° 55' − 131° 5'

FB of CD = 170° 50'

BB of CD = 170° 50' + 180° = 350° 50'.

Line	corrected FB	corrected BB
AB	80° 40'	260° 40'
BC	121° 55'	301° 55'
CD	170° 50'	350° 50'
DE	230° 5'	50° 5'
EA	310° 50'	130° 50'

Errors in compass surveying

* What are the errors in compass surveying?

  * Instrumental error
  * Personal (or) observational error
  * Natural error

Instrumental error:-

(i) The needle is not perfectly straight.
(ii) The pivot is bend
(iii) The pivot is not centre of graduated circle
(iv) The plane of sight is not vertical
(v) The vertical hair is too thick or loose.
(vi) Plane of sight not passing through the graduated ring.

(vi) Movement of the needle not free and not horizontal

(vii) Needle lost in its magnetism

Personal Error:

* The compass is not centred properly over the station.
* The compass is not properly levelled
* The stations are not accurately bisected.
* The observations are not properly recorded.
* The graduated ring is read in the wrong direction.

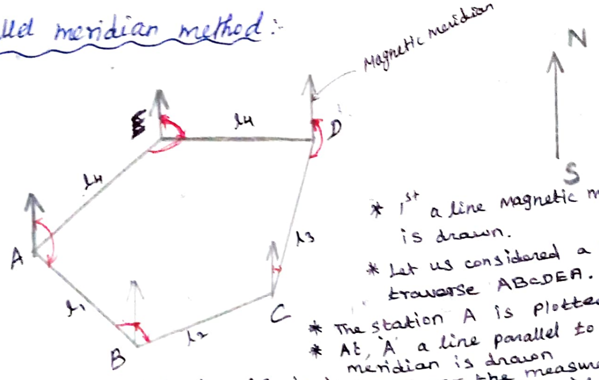
Natural Error:-

* Variations in declination
* Change in atmosphere, (clouds and storms causing magnetic changes).
* Local attraction due to presence of iron or steel materials on the site.
* Irregular Variations due to magnetic storms, earthquake, sun & moon effects.

Plotting of compass Traverse

* Parallel meridian method
* Included angle method
* Rectangular co-ordinates method

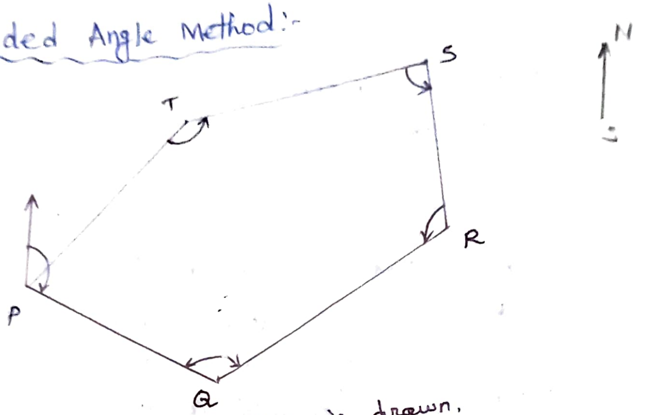
(i) Parallel meridian method:-



* 1st a line magnetic meridian is drawn.
* Let us considered a closed traverse ABCDEA.
* The station A is plotted.
* At A a line parallel to the meridian is drawn
* The line AB is drawn as per the measured bearing of line AB & the length of AB marked

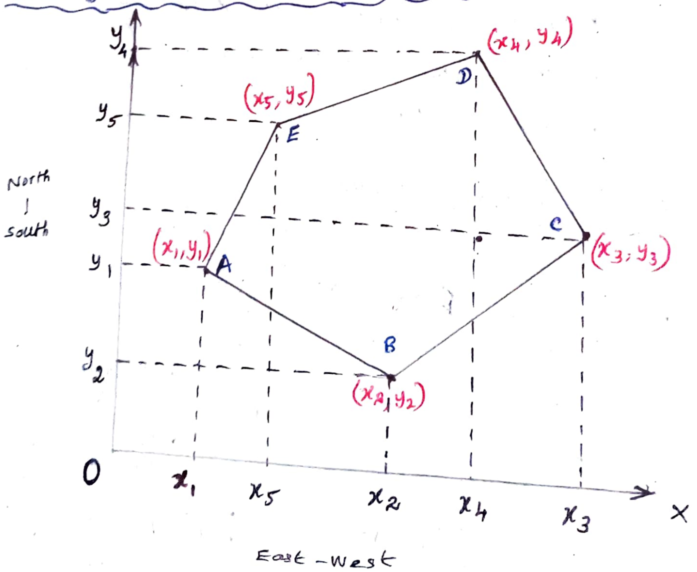
* At B, a line parallel to the meridian is drawn.
* The length & bearing of BC is drawn.
* similarly all the lines are drawn.
* In the case of a closed traverse the last point should coincide with the starting point.
* If it is not coinciding, the closing error is adjusted.

(ii) Included Angle Method:-



* The magnetic meridian line is drawn.
* The included angles are computed.
* The first line is drawn with reference to the magnetic meridian.
* other lines are drawn in sequence by drawing the included angles.

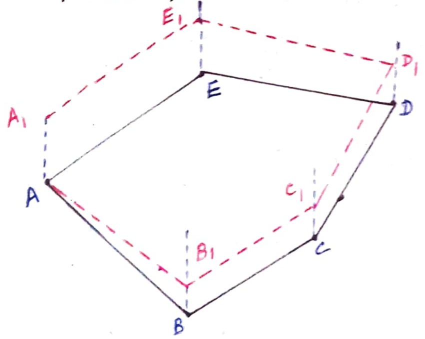
(iii). Rectangular co-ordinates Method:-

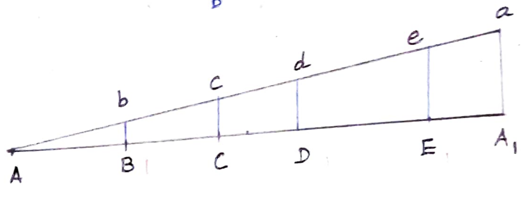


* This is a more accurate method, which is generally followed in theodolite traversing.
* In this method each point is plotted by its co-ordinates with reference to two ⊥r axes.
* These axes are called as axes of co-ordinates
* The point of intersection of these lines is called the origin.
* The axes ox represents the east-west line and oy represents the north-south line.

Adjustment of closing error

* carefully conducted compass survey of a closed traverse will finish at the starting point.
* This is a case of no closing error.
* But many times the traverse fails to close due to some error.
* Such an error may occur due to
    (i) Mistakes made in the measurements of length
    (ii) measurement in bearing
    (iii) Error in plotting.
* If the closing error is with in reasonable limits, the error is adjusted graphically by Bowditch's rule.
* If the error is beyond the limit, the field work should be repeated.
* Let us ~~considered~~ a closed traverse ABCDEA is plotted, to a suitable scale (RF = 1/400)





* A Perimeter
* Actual distance of ABCDE is marked

* In this case the closing error is AA_1

* In order to adjust the error the following steps are followed.

(i) A horizontal distance AA_1 representing the perimeter of the traverse to a suitable scale is drawn.

(ii) On this line distances AB, BC, CD, DE & EA_1 are set off based on the actual measured lengths.

(iii) A_1a a line parallel is drawn at A_1 of the scale equal to the amount of closing error.

(iv) The points A & a are joined.

(v) From points B, C, D & E the lines Bb, Dc, Dd and Ee are drawn parallel to A_1a.

(vi) These intercepts represent the quantity by which the points have to be shifted.

(vii) Now the corrections are transferred to the respective stations of the points ABCD & E are got.

(viii) The points ABCD & E are joined which forms the corrected closed traverse.

<u>Note</u> :-

* The angular error of closure should not exceed $15'\sqrt{N}$ minutes

where,

N $\longrightarrow$ Number of sides of the traverse.

* Relative closing error $= \dfrac{\text{Amount of closing error}}{\text{Perimeter of traverse}}$

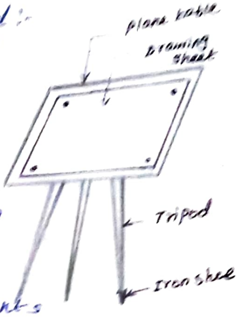
This value should not exceed $1/600$

* Plane table surveying is a graphical method of surveying in which the field work and plotting are done simultaneously.

* Plane table surveying is generally suitable for filling in of the details b/w the stations previously fixed by triangulation or theodolite traversing.

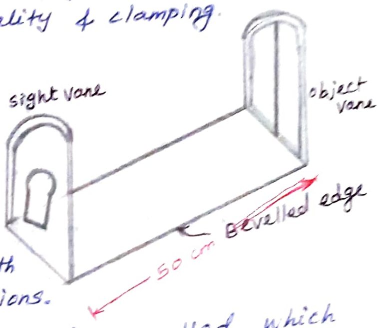
Instruments used in plane table surveying:

* Drawing Board with mounted tripod:-



  * The drawing board is made of well seasoned wood.
  * sizes varies from 40 cm x 30 cm to 75cm x 60 cm
  * Its base is mountable on a tripod with adequate adjustments for levelling, verticality & clamping.

* Alidade :-

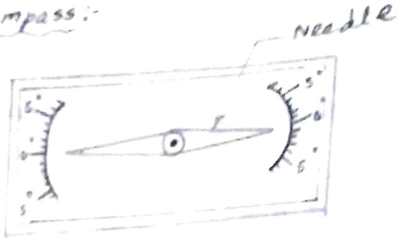


  * The alidade is made of metal (brass or gun metal) or boxwood.
  * straight edge with a length of 50cm & with graduations.
  * The ruling or working edge is bevelled which is called as fiducial edge.
  * The alidade may be a plain one or fixed with a telescope.
  * The plain alidade is fitted with two vanes one at each end which are hinged with the ruler.
  * one vane is an object vane, which is provided with a horse hair and the other one is a sight vane provided with a narrow slit.

Sprit level

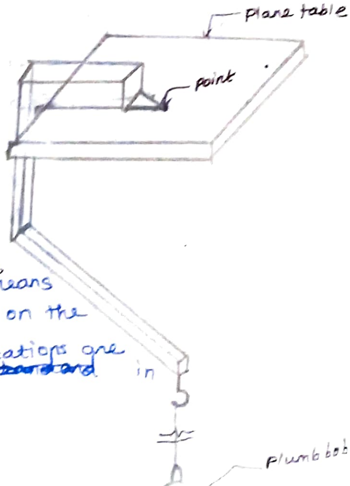
* It is a small metal tube containing a small bubble of sprit.
* The bubble can be seen on the top of a graduated glass tube
* It is used for levelling the plane table

Trough compass :-



* It consist of a long narrow rectangular box about 150 x 30 x 20 mm having a needle of 125 mm in length of its centre.
* The needle being balanced upon a steel pin
* Inside the box and its end is fixed level with the needle a small flat curved scale of only a few degree on each side of the zero.
* The compass is used to ~~rotate~~ plot the north-south direction (magnetic meridian) of the plane table.

Plumbing fork (or) U-fork :-



* It consists of metal frame having two arms of equal length.
* It is used for center over the point) up to plane table also beginning of work, it means (transfering the ground point on the sheet.)
* The plot point of the ground stations ~~standard~~ are in vertical line.

Water proof sorts.

[illegible]

Paper [illegible]

Drawing paper

The paper should be good quality to sustain the effects of atmosphere.

Advantages (or) Merits of plane table surveying:

* It is suitable for small size surveys and to prepare small scale maps.
* The observations and plotting are done simultaneously in field.
* The errors & mistakes in plotting can be checked by drawing check lines.
* Irregular objects can be plotted accurately as the lay of land is in view
* No great skill is required.
* It is most rapid & useful for filling in details
* It is less costly than theodolite survey.
* It is advantagous in magnetic areas, where compass survey is not reliable.

Dis-advantages (or) De-merits of plane table surveying

* It is ~~most inconvenient~~ not suitable for in rainy seasons and wet climate.
* It is not very accurate for large surveys as compass & theodolite surveying.
* It is not suitable for accurate work.
* The number of accessories required in survey work is more & they are likely to be lost.
* The absence of measurements (field notes) are inconvenient, if the survey is to be replotted to some different scale.

Working Operations (ii)

(i) Fixing the table :-

* The table should be set up a convenient height for working. (Generally 1m height)
* The tripod stand is placed over the required station with legs well spread apart and firmly fixed on the ground.
* Then the table is fixed on the tripod head & fixed using the wing nuts provided at the bottom.

(iii) Centering the table :-

* The drawing sheet is fixed on the table by the help of pins or cello tape.
* The plane table is placed, such that the Point on the paper is properly set over the station on the ground.
* The one end of U-fork is placed against the plotted point.
* The table is adjusted such that the plumb-bob at the other end of the Plumbing fork is exactly centered on the station. This operation is called centering.

i) Levelling the table :-

... single ... of the use by exposing it alternately to damp and a dry atmosphere. For work of high precision, fibre glass sheets or paper backed with sheet aluminium are often used.

11.2. WORKING OPERATIONS

Three operations are needed

(a) *Fixing* Fixing the table to the tripod

(b) ***Setting*** (*i*) Levelling the table (*ii*) Centring (*iii*) Orientation

(c) ***Sighting the points.***

Levelling. For small-scale work, levelling is done by estimation. For work of accuracy an ordinary spirit level may be used. The table is levelled by placing the level on the board in two positions at right angles and getting the bubble central in both directions. For more precise work, a Johnson Table or Coast Survey Table may be used.

Centring. The table should be so placed over the station on the ground that the point plotted on the sheet corresponding to the station occupied should be exactly over the station on the ground. The operation is known as *centring* the plane table. As already described this is done by using a plumbing fork.

Orientation. Orientation is the process of putting the plane-table into some fixed direction so that line representing a certain direction on the plan is parallel to that direction on the ground. *This is essential condition to be fulfilled when more than one instrument station is to be used*. If orientaion is not done, the table will not be parallel to itself at different positions resulting in an overall distortion of the map. The processes of centring and orientation are dependent on each other. For orientation, the table will have to be rotated about its vertical axis, thus disturbing the centring. If precise work requires that the plotted point should be exactly over the ground point, repeated orientation and shifting of the whole table are necessary. It has been shown in §11.9 that centring is a needless refinement for small scale work.

There are two main methods of orienting the plane table :

(*i*) Orientation by means of trough compass.

(*ii*) Orientation by means of backsighting.

(*i*) ***Orientation by trough compass***. The compass, though less accurate, often proves a valuable adjunct in enabling the rapid approximate orientation to be made prior to the final adjustment. The plane table can be oriented by compass under the following conditions :

(*a*) When speed is more important that accuracy.

(*b*) When there is no second point available for orientation.

(*c*) When the traverse is so long that accumulated errors in carrying the azimuth forward might be greater than orientation by compass.

(*d*) For approximate orientation prior to final adjustment

(*e*) In certain resection problems.

For orientation, the compass is so placed on the plane table that the needle floats centrally, and a fine pencil line is ruled against the long side of the box. At any other station, where the table is to be oriented, the compass is placed against this line and the table is oriented by turning it until the needle floats centrally. The table is then clamped in position.

(*ii*) ***Orientation by back sighting***. Orientation can be done precisely by sighting the points already plotted on the sheet. Two cases may arise :

(*a*) When it is possible to set the plane table on the point already plotted on the sheet by way of observation from previous station.

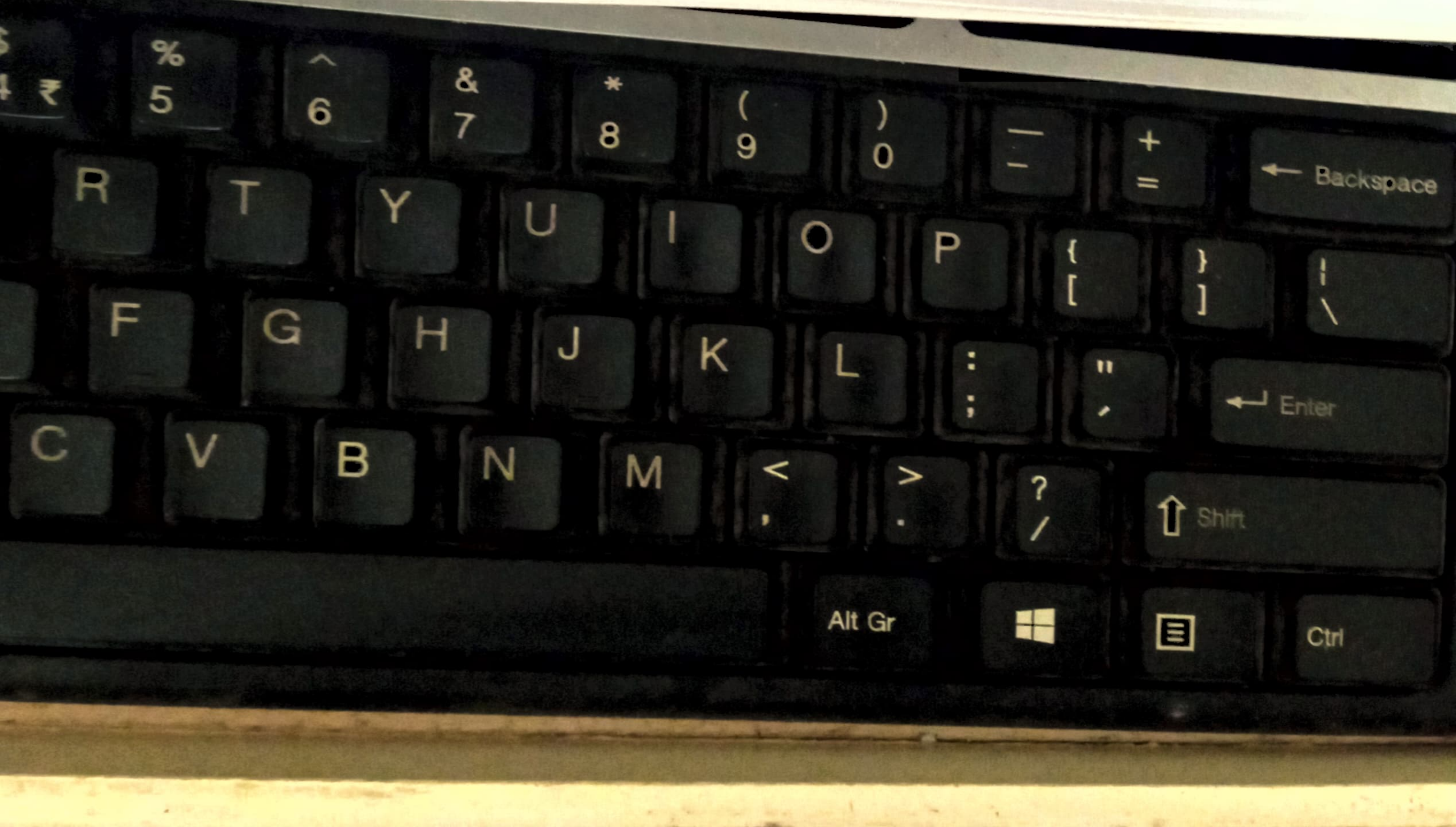
(*b*) When it is not possible to set the plane table on the point.

Case (*b*) presents a problem of *Resection* and has been dealt in § 11.6. When conditions are as indicated in (*a*), the orientation is said to be done by *back sighting*.

To orient the table at the next station, say *B*, represented on the paper by a point *b* plotted by means of line *ab* drawn from a previous station *A*, the alidade is kept on the line *ba* and the table is turned about its vertical axis in such a way that the line of sight passes through the ground station *A*. When this is achieved, the plotted line *ab* will be coinciding with the ground line *AB* (provided the centring is perfect) and the table will be oriented. The table is then clamped in position.

The method is equivalent to that employed in azimuth traversing with the transit. Greater precision is obtainable than with the compass, but an error in direction of a line is transferred to succeeding lines.

Sighting the points. When once the table has been set, *i.e.*, when levelling, centring and orientation has been done, the points to be located are sighted through the alidade. The alidade is kept pivoted about the plotted location of the instrument station and is turned so that the line of sight passes or bisects the signal at the point to be plotted. A ray is then drawn from the instrument station along the edge of the alidade. Similarly, the



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1. Set the table at T, level it and transfer the point on to the sheet by means of plumbing fork, thus getting point t representing T. Clamp the table.

2. Keep the alidade touching t and sight to A. Draw the ray along the fiducial edge of the alidade. Similarly, sight different points B, C, D, E, etc., and draw the corresponding rays. A pin may be inserted at t and the alidade may be kept touching the pin while sighting the points.

3. Measure TA, TB, TC, TD, TE etc., in the field and plot their distances to some scale along the corresponding rays, thus getting a, b, c, d, e etc. Join these if needed.

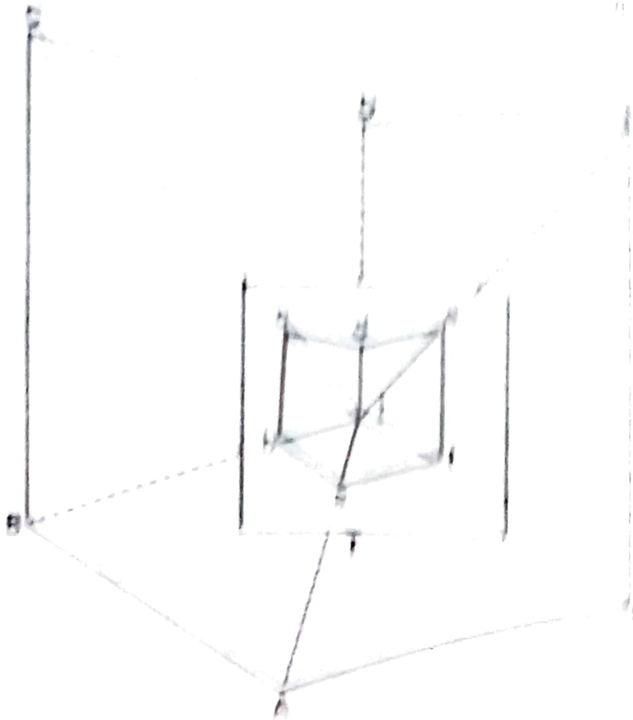


FIG. 11.8. RADIATION.

11.5. INTERSECTION (GRAPHIC TRIANGULATION)

Intersection is resorted to when the distance between the point and the instrument station is either too large or cannot be measured accurately due to some field conditions. The location of an object is determined by sighting at the object from two plane table stations (previously plotted) and drawing the rays. The intersection of these rays will give the position of the object. It is therefore very essential to have at least two instrument stations to locate any point. The distance between the two instrument stations is measured and plotted on the sheet to some scale. The line joining the two instrument stations is known as the *base line*. No linear measurement other than that of the base line is made. The point of intersection of the two rays forms the *vertex* of a triangle having the two rays as two sides and the base line as the third line of the triangle. Due to this reason, intersection is also sometimes known as *graphic triangulation*.

Procedure (Fig. 11.9) : The following is the *procedure* to locate the points by the method of intersection:

(1) Set the table at A, level it and transfer the point A on to the sheet by way of plumbing fork. Clamp the table.

(2) With the help of the trough compass, mark the north direction on the sheet.

(3) Pivoting the alidade about a, sight it to B. Measure AB and plot it

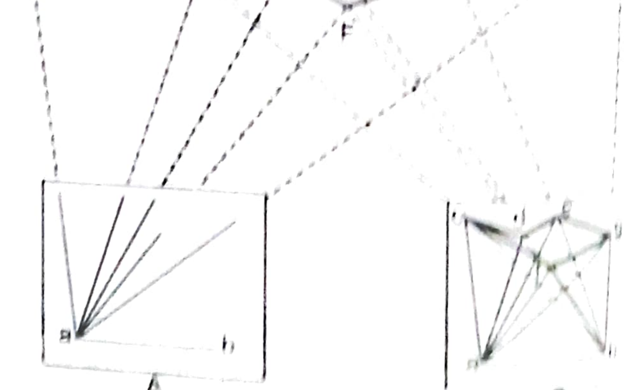


FIG. 11.9. INTERSECTION.

... get b. The base line ab is thus drawn.

... the alidade about a, sight the details C, D, E etc. and draw corresponding ...

... the table at B and set it there. Orient the table roughly by compass and ... backsighting A.

... the alidade about b, sight the details C, D, E etc. and draw the ... along the edge of the alidade to intersect with the previously drawn ... The positions of the points are thus mapped by way of intersection.

... of intersection is mainly used for mapping details. If this is to be used ... which will be used as subsequent plane table station, the point should ... of intersection of at least three or more rays. Triangles should be well ... and the angle of intersection of the rays should not be less than 45° in such ... triangulation can also proceed without preliminary measurement of the base, ... length of the base line influences only the scale of plotting.

11.8 TRAVERSING

Plane table traverse involves the same principles as a transit traverse. At each successive ... the table is set, a foresight is taken to the following station and its location is plotted by measuring the distance between the two stations as in the radiation method described earlier. Hence traversing is not much different from radiation as far as working principles are concerned — the only difference is that in the case of radiation the observations are made to those points which are to be detailed or mapped while in the case of traversing the observations are made to those points which will subsequently be used as instrument stations. The method is widely used to lay down survey lines between the instrument stations of a closed or unclosed traverse.

Procedure (Fig. 11.10)

(1) Set the table at A. Use plumbing fork for transferring A on to the sheet. Draw the direction of magnetic meridian with the help of trough compass.

(2) With the alidade pivoted about a, sight it to B and draw the ray. Measure AB and scale off ab to some scale. Similarly, draw a ray towards E, measure AE and plot e.

(3) Shift the table to B and set it. Orient the table accurately by backsighting A. Clamp the table.

(4) Pivoting the alidade about b, sight to C. Measure BC and plot it on the drawn ray

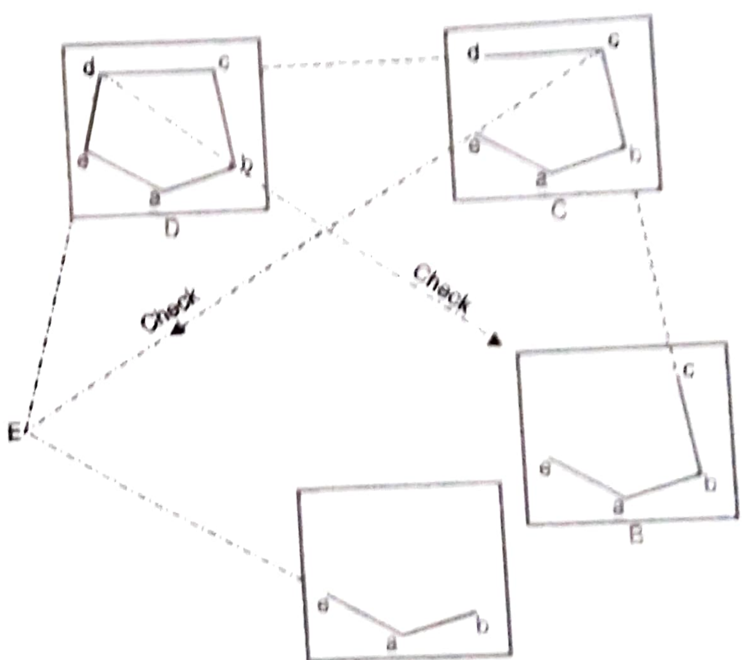


FIG. 11.10 TRAVERSING

to the same scale. Similarly, the table can be set at other stations and the traverse completed.

It is to be noted here that the orientation is to be done by back-sighting. If there are n stations in a closed traverse, the table will have to be set on at least $(n-1)$ stations to know the error of closure though the traverse may be closed even by setting it on $(n-2)$ stations. At any station a portion of the traverse may be checked if two or more of the preceding stations are visible and are not in the same straight line with the station occupied.

11.7. RESECTION

Resection is the process of determining the plotted position of the station occupied by the plane table, by means of sights taken towards known points, locations of which have been plotted.

The method consists in drawing two rays to the two points of known location on the plan after the table has been *oriented*. The rays drawn from the unplotted location of the station to the points of known location are called resectors, the intersection of which gives the required location of the instrument station. If the table is not correctly oriented at the station to be located on the map, the intersection of the two resectors will not give the correct location of the station. The problem, therefore, lies in orienting table at the stations and can be solved by the following four methods of orientation.

(*i*) Resection after orientation by compass.

(*ii*) Resection after orientation by backsighting.

(*iii*) Resection after orientation by three-point problem.

(*iv*) Resection after orientation by two-point problem.

(*i*) **Resection after orientation by compass**

The method is utilised only for small-scale or rough mapping for which the relatively large errors due to orienting with the compass needle would not impair the usefulness of the map.

The method is as follows (Fig. 11.11).

(1) Let C be the instrument station to be located on the plan. Let A and B be two visible stations which have been plotted on the sheet as a and b. Set the table at C and orient it with compass. Clamp the table.

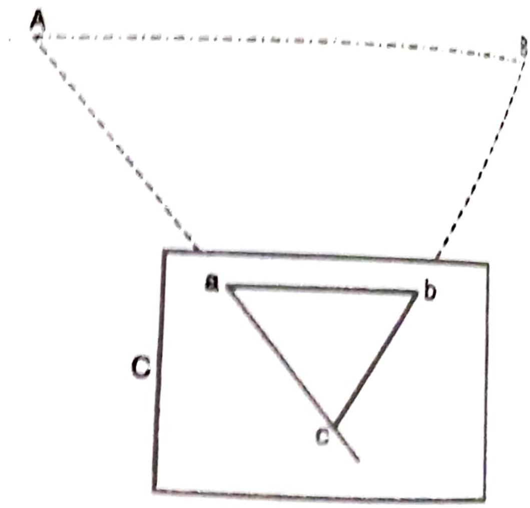


FIG. 11.11. RESECTION AFTER ORIENTATION BY COMPASS.

(2) Pivoting the alidade about a, draw a resector (ray) towards A; similarly, sight B from b and draw a resector. The intersection of the two resectors will give c, the required point.

(*ii*) **Resection after orientation by backsighting**

If the table can be oriented by backsighting along a previously plotted backsight line, the station can be located by the intersection of the backsight line and the resector drawn through another known point. The method is as follows (Fig. 11.12) :

(1) Let C be the station to be located on the plan and A and B be two visible points which have been plotted on the sheet as a and b. Set the table at A and orient it by backsighting B along ab.

(2) Pivoting the alidade at a, sight C and draw a ray. Estimate roughly the position of C on this ray as c_1.

(3) Shift the table to C and centre it approximately with respect to c_1. Keep the alidade on the line $c_1 a$ and orient the table by back-sight to A. Clamp the table which has been oriented.

(4) Pivoting the alidade about b, sight B and draw the resector bB to intersect the ray $c_1 a$ in c. Thus, c is the location of the instrument station.

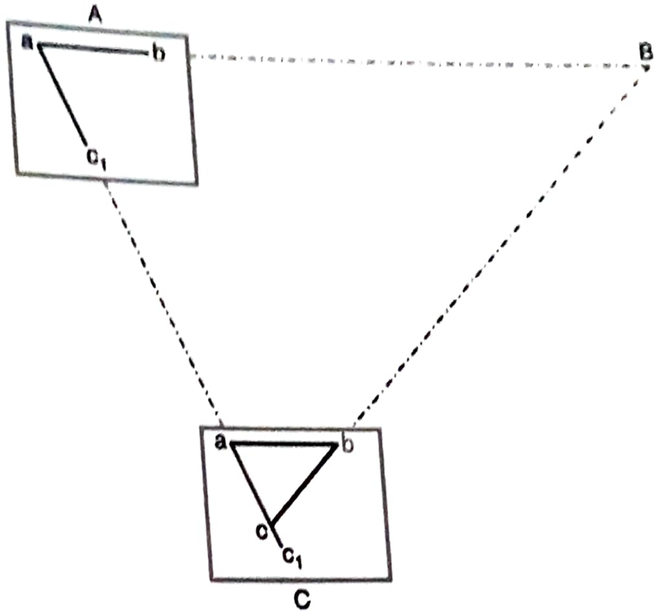


FIG. 11.12. RESECTION AFTER ORIENTATION BY BACKSIGHTING.

Resection by Three-point Problem and Two-point Problem

Of the two methods described above, the first method is rarely used as the errors due to local attraction etc., are inevitable. In the second method, it is necessary to set the table on one of the known points and draw the ray towards the station to be located. In the more usual case in which no such *ray* has been drawn, the data must consist of either :

(*a*) Three visible points and their plotted positions (The three-point problem).

(*b*) Two visible points and their plotted positions (The two-point problem).

11.8. THE THREE-POINT PROBLEM

Statement. ***Location of the position, on the plan, of the station occupied by the plane table by means of observations to three well-defined points whose positions have been previously plotted on the plan."***

In other words, it is required to orient the table at the station with respect to three visible points already located on the plan. Let P (Fig. 11.13) be the instrument station and A, B, C be the points which are located as a, b, c respectively on the plan. The table is said to be correctly oriented at P when the three resectors through a, b and

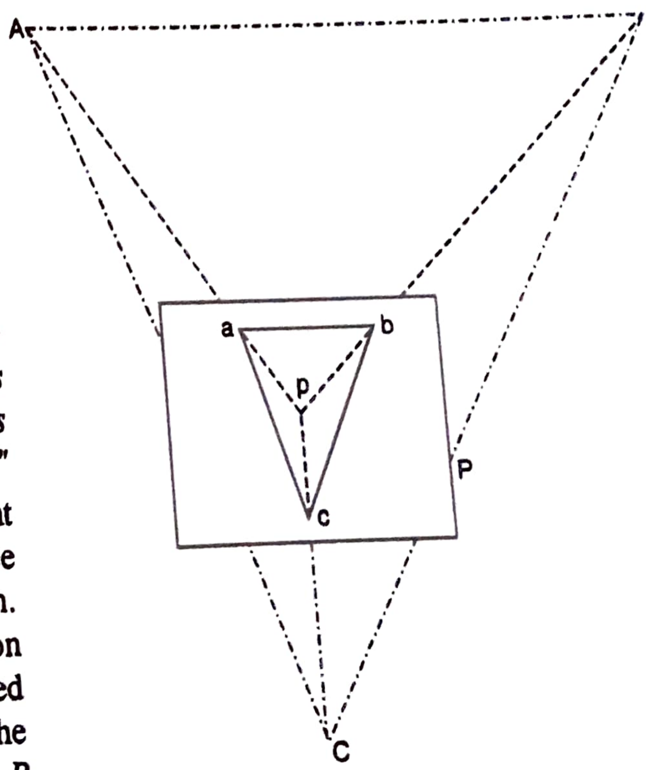


FIG. 11.13.
CONDITION OF CORRECT ORIENTATIO

c meet at a point and not in a triangle. The intersection of the three resectors in a po... gives the location of the instrument station. *Thus, in three-point problem, orientation an... resection are accomplished in the same operation.*

The following are some of the important methods available for the solution of t... problem

(*a*) Mechanical Method (Tracing Paper Method)

(*b*) Graphical Method

(*c*) Lehmann's Method (Trial and Error Method)

1. MECHANICAL METHOD (TRACING PAPER METHOD)

The method involves the use of a tracing paper and is, therefore, also known a... *tracing paper method.*

Procedure (Fig. 11.14)

Let *A*, *B*, *C* be the known points and *a*, *b*, *c* be their plotted positions. Let *P* be the position of the instrument station to be located on the map.

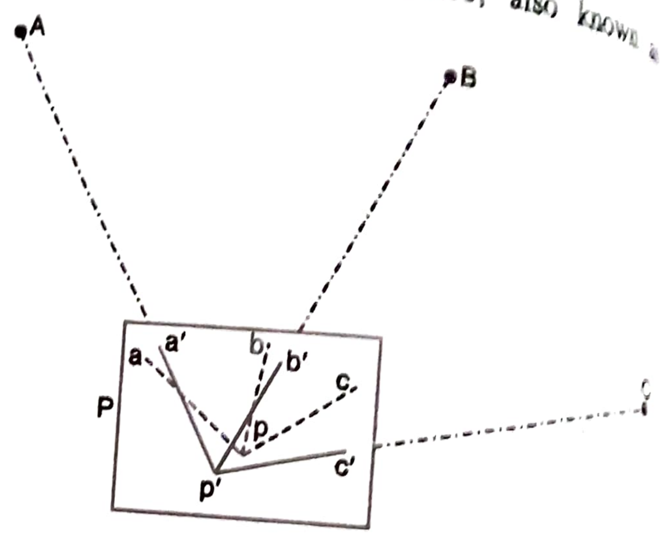


FIG. 11.14.

(1) Set the table on *P*. Orient the table approximately with eye so that *ab* is parallel to *AB*.

(2) Fix a tracing paper on the sheet and mark on it *p'* as the approximate location of *P* with the help of plumbing fork.

(3) Pivoting the alidade at *p'*, sight *A*, *B*, *C* in turn and draw the corresponding lines *p'a'*, *p'b'* and *p'c'* on the tracing paper. These lines will not pass through *a*, *b*, and *c* as the orientation is approximate.

(4) Loose the tracing paper and rotate it on the drawing paper in such a way that the lines *p'a'*, *p'b'* and *p'c'* pass through *a*, *b* and *c* respectively. Transfer *p'* on to the sheet and represent it as *p*. Remove the tracing paper and join *pa*, *pb* and *pc*.

(5) Keep the alidade on *pa*. The line of sight will not pass through *A* as the orientation has not yet been corrected. To correct the orientation, loose the clamp and rotate the plane table so that the line of sight passes through *A*. Clamp the table. The table is thus oriented.

(6) To test the orientation, keep the alidade along *pb*. If the orientation is correct, the line of sight will pass through *B*. Similarly, the line of sight will pass through *C* when the alidade is kept on *pc*.

2. GRAPHICAL METHODS

There are several graphical methods available, but the method given by Bessel is more suitable and is described first.

Bessel's Graphical Solution (Fig. 11.15)

(1) After having set the table at station *P*, keep the alidade on *b a* and rotate the table so that *A* is bisected. Clamp the table.

(2) Pivoting the alidade about b, sight to C and draw the ray xy along the edge of the alidade [Fig. 11.15 (a)].

(3) Keep the alidade along ab and rotate the table till B is bisected. Clamp the table.

(4) Pivoting the alidade about a, sight to C. Draw the ray along the edge of the alidade to intersect the ray xy in c' [Fig. 11.15 (b)]. Join cc'.

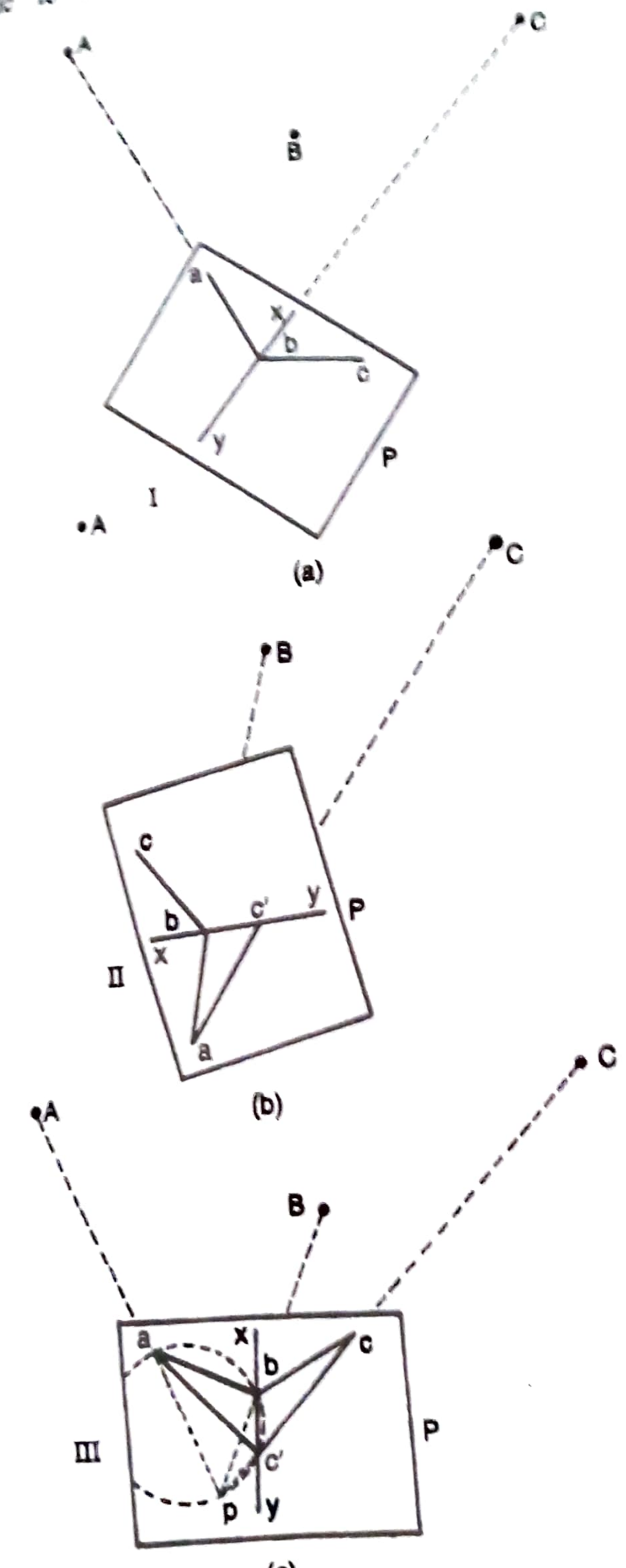


FIG. 11.15. THREE-POINT PROBLEM : BESSEL'S METHOD.

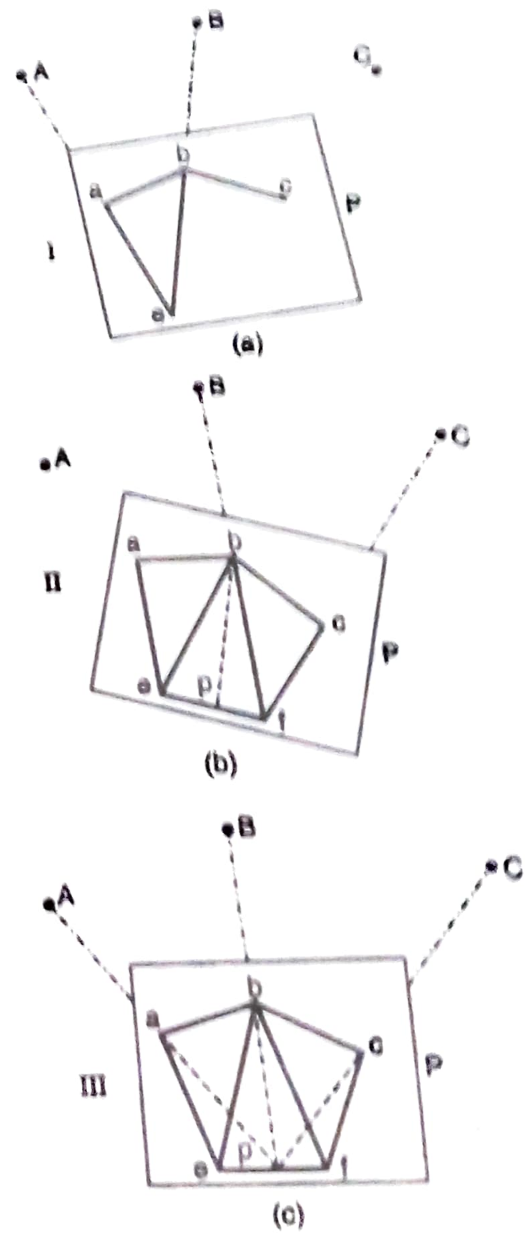


FIG. 11.16

(5) Keep the alidade along $c'c$ and rotate the table till C is bisected. Clamp the table. *The table is correctly oriented* [Fig. 11.15 (c)].

(6) Pivoting the alidade about b, sight to B. Draw the ray to intersect $c'c$ in p. Similarly, if alidade is pivoted about a and A is sighted, the ray will pass through p if the work is accurate.

The points a, b, c' and p form a quadrilateral and all the four points lie along the circumference of a circle. Hence, this method is known as *"Bessel's Method of Inscribed Quadrilateral"*.

In the first four steps, the sighting for orientation was done through a and b, and rays were drawn through c. However, any two points may be used for sighting and the rays drawn towards the third point, which is then sighted in steps 5 and 6.

Alternative Graphical Solution. (Fig. 11.16)

(1) Draw a line ae perpendicular to ab at a. Keep the alidade along ea and rotate the plane table till A is bisected. Clamp the table. With b as centre, direct the alidade to sight B and draw the ray be to cut ae in e [Fig. 11.16 (*a*)].

(2) Similarly, draw cf perpendicular to bc at c. Keep the alidade along fc and rotate the plane table till C is bisected. Clamp the table.

With b as centre, direct the alidade to sight B and draw the ray bf to cut cf in f [Fig. 11.16 (*b*)].

(3) Join e and f. Using a set square, draw bp perpendicular to ef. Then p represents on the plan the position P of the table on the ground.

(4) To orient the table, keep the alidade along pb and rotate the plane table till B is bisected. To check the orientation, draw rays aA, cC, both of which should pass through p, as shown in Fig. 11.16 (*c*).

3. LEHMANN'S METHOD

We have already seen that the three-point problem lies in orienting the table at the point occupied by the table. In this method, the orientation is done by trial and error and is, therefore, also known as *the trial and error method.*

Procedure. (Refer Fig. 11.17)

(1) Set the table at P and orient the table approximately so that ab is parallel to AB. Clamp the table.

(2) Keep the alidade pivoted about a and sight A. Draw the ray. Similarly, draw rays from b and c towards B and C respectively. If the orientation is correct, the three rays will meet at one point. If not, they will meet in three points forming one small *triangle of error.*

(3) The *triangle of error* so formed will give the idea for the further orientation.

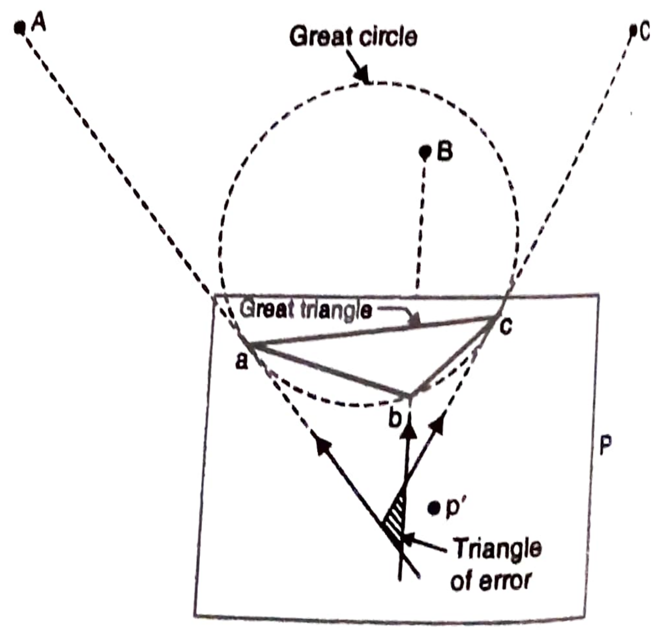


FIG. 11.17. TRIANGLE OF ERROR METHOD.

The orientation will be correct only when the triangle of error is reduced to one point. To do this, choose the point p' as above. This approximate choice of the position may be done with the help of Lehmann's Rules described later.

(4) Keep the alidade along $p'a$ and rotate the table to sight A. Clamp the table. This will give next approximate orientation (but more accurate than the previous one).

(5) Keep the alidade at b to sight B and draw the ray. Similarly, keep the alidade at c and sight C. Draw the ray. These rays will again meet in one triangle, the size of which will be smaller than the previous triangle of error if p' has been chosen judiciously keeping in the view the Lehmann's Rules.

(6) Thus by successive trial and error, the triangle of error can be reduced to a point.

The final and correct position of the table will be such that the rays Aa, Bb and Cc meet in one single point, giving the point p.

The whole problem, thus, involves a fair knowledge of Lehmann's Rules for the approximate fixation of p' so that the triangle of error may be reduced to a minimum.

The lines joining A, B, C (or a, b, c) form a triangle known as the *Great Triangle*. Similarly, the circle passing through A, B, C or (a, b, c) is known as the *Great Circle*.

Lehmann's Rules

(1) If the station P is outside the great triangle ABC, the triangle of error will also fall outside the great triangle and the point p' should be chosen outside the triangle of error. Similarly, if the station P is inside the great triangle, the triangle of error will also be inside the great triangle and the point p' should be chosen inside the triangle of error (Fig. 11.18).

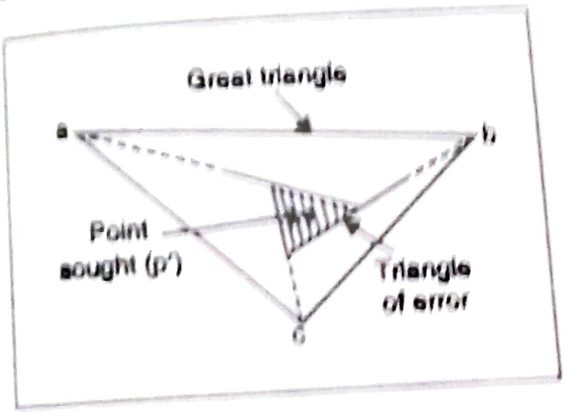


FIG. 11.18

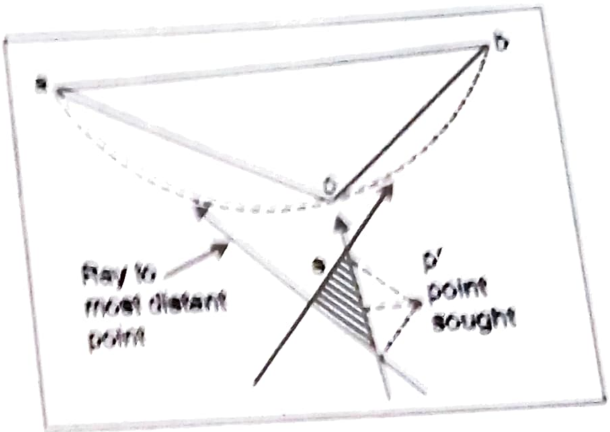


FIG. 11.19

(2) The point p' should be so chosen that its distance from the rays Aa, Bb, and Cc is proportional to the distance of P from A, B and C respectively.

(3) The point p' should be so chosen that it is to the same side of all the three rays Aa, Bb, and Cc. That is, if point p' is chosen to the right of the ray Aa, it should also be to the right of Bb and Cc (Fig 11.19).

Though the above rules are sufficient for the location of p', the following sub-rule may also be useful :

(3 a) If the point P is outside the great circle, the position of p' should be so chosen that the point e (got by the intersection of the two rays drawn to nearer points) is midway between the point p' and the ray to the most distant point (Fig. 11.19).

(3 b) When P is outside the great triangle but inside the great circle (as) in case of the segments of great circle, the point p' must be so chosen that the ray to middle point may lie between p' and the point e which is the intersection of the rays to the other two extreme points (Fig. 11.20).

Special Cases

The following are few rules for special cases:

(4a) If the positions of A, B, C and P are such that P lies on or near the line of AC of the great triangle, the point p' must be so chosen that it is in between the two parallel rays drawn to A and C and to the right (or to the same side of each of the rays) of each of the three rays to satisfy Rule 3 (Fig. 11.21).

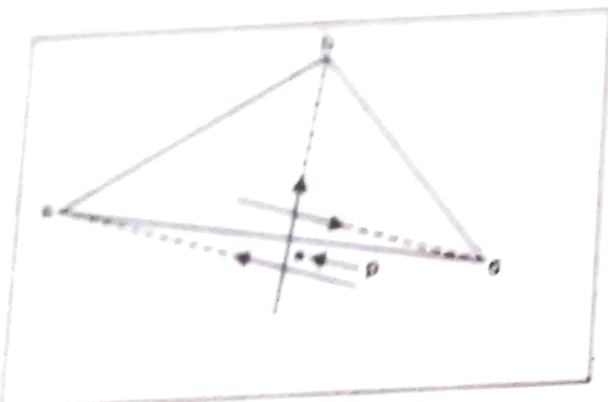


FIG. 11.21

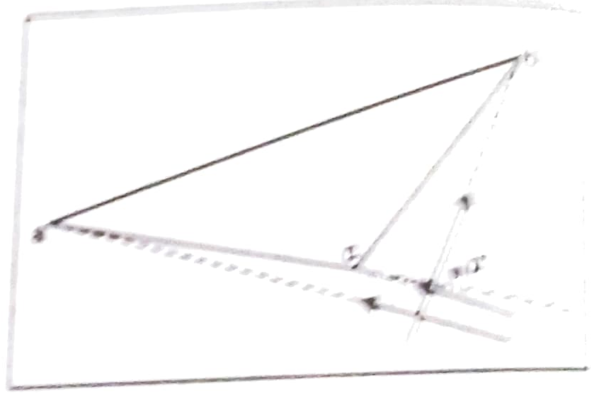


FIG. 11.22

(4b) If the point P (as in 4 a) lies on or near the prolonged line AC, the point p' must be chosen outside the parallel rays and to the right of each of the three rays to satisfy both Rules 2 and 3 (Fig. 11.22).

(4c) If A, B and C happen to be in one straight line the great triangle will be of one straight line only and the great circle will be having abc as its arc the radius of which is infinite. In such cases, the point p' must be so chosen that the rays drawn to the middle point is between the point p' and the point e got by the intersection of the rays to the extreme point (Fig. 11.23).

(4d) If the positions A, B, C and P are such that P lies on the great circle, the point p' cannot be determined by three-point problem because three rays will intersect in one point even when the table is not at all oriented (Fig. 11.24).

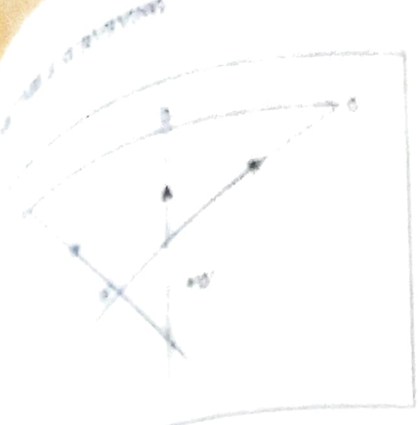


FIG. 11.23

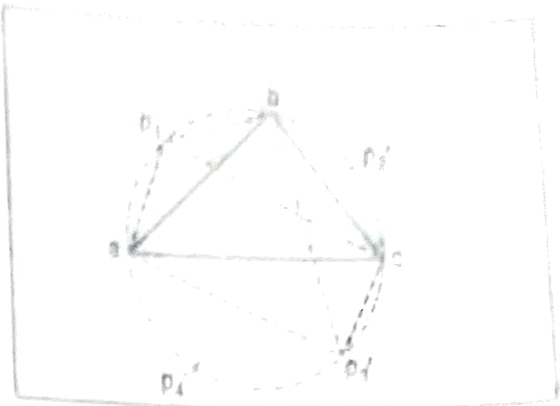


FIG. 11.24

TWO-POINT PROBLEM

Statement. "*Location of the position on the plan, of the station occupied by the table by means of observations to two well defined points whose positions have previously plotted on the plan.*"

Let us take two points *A* and *B*, the plotted positions of which are known. Let *C* be the point to be plotted. The whole problem is to orient the table at *C*.

Procedure. Refer Fig. 11.25

1. Choose an auxiliary point *D* near *C*, to assist the orientation at *C*. Set the table at *D* in such a way that *ab* is approximately parallel to *AB* (either by compass or by alignment). Clamp the table.
2. Keep the alidade at *a* and sight *A*. Draw the resector. Similarly, draw a resector from *b* and *B* to intersect the previous one in *d*. The position of *d* is thus got, the degree of accuracy of which depends upon the approximation that has been made in keeping *ab* parallel to *AB*. Transfer the point *d* to the ground and drive a peg.
3. Keep the alidade at *d* and sight *C*. Draw the ray. Mark a point c_1 on the ray by estimation to represent the distance *DC*.
4. Shift the table to *C*, orient it (tentatively) by taking backsight to *D* and centre it with reference to c_1. The orientation is, thus, the same as it was at *D*.

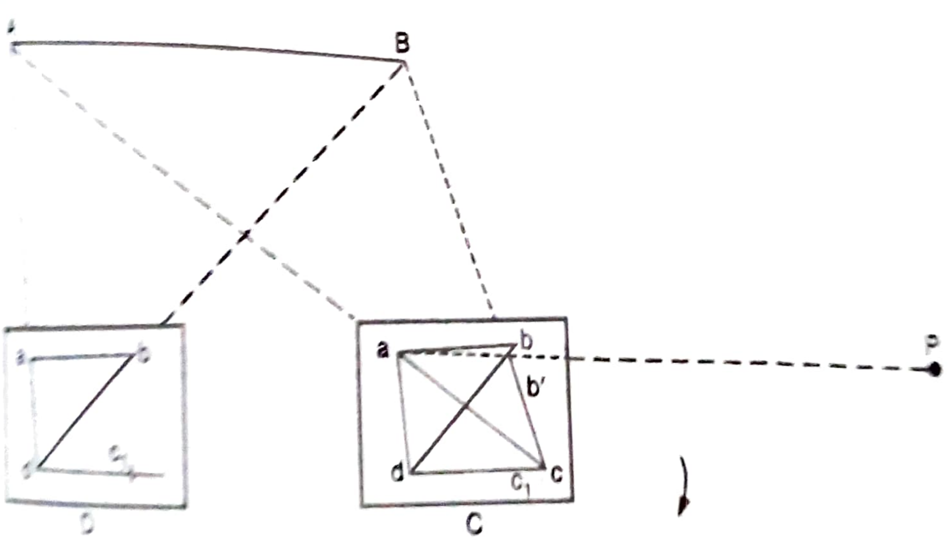


FIG. 11.25. TWO-POINT PROBLEM.

(3) Keep the alidade pivoted at a and sight it to A. Draw the ray to intersect with the previously drawn ray from b to c. Thus c is the point representing the station C with reference to the approximate orientation made at D.

(4) Pivoting the alidade about c, sight B. Draw the ray to intersect with the ray drawn from d to B in b_1. Then b_1 is the approximate representation of B with respect to the orientation made at D.

(5) The angle between ab and ab_1 is the error in orientation and must be corrected for. In order that ab and ab_1 may coincide (or may become parallel) keep a pole P in line with ab_1 and at a great distance. Keeping the alidade along ab, rotate the table till P is bisected. Clamp the table. The table is thus correctly oriented.

(6) After having oriented the table as above, draw a resector from a to A and another from b to B, the intersection of which will give the position C occupied by the table.

It is to be noted here that unless the point P is chosen infinitely distant, ab and ab_1 cannot be made parallel. Since the distance of P from C is limited due to other considerations, two-point problem does not give much accurate results. At the same time, more labour is involved because the table is also to be set on one more station to assist the orientation.

Alternative Solution of Two-point Problem (Fig. 11.26)

(1) Select an auxiliary point D very near to B and orient the table there by estimation (making bd approximately parallel to BA). If D is chosen in the line BA, orientation can be done accurately.

(2) With b as centre, sight B and draw a ray BD. Measure the distance BD and plot the point d to the same scale to which a and b have been previously plotted. Since the distance BD is small, any small error in orientation will not have appreciable effect on the location of d. The dotted lines show the first position of plane table with approximate orientation.

(3) Keep the alidade along da and rotate the table to sight A, for orientation. Clamp the table. The firm lines show the second position with correct orientation.

(4) With d as centre, draw a ray towards C, the point to be actually occupied by the plane table.

(5) Shift the table to C and orient by backsighting to D.

(6) Draw a ray to A through a, intersecting the ray dc in c. Check the orientation

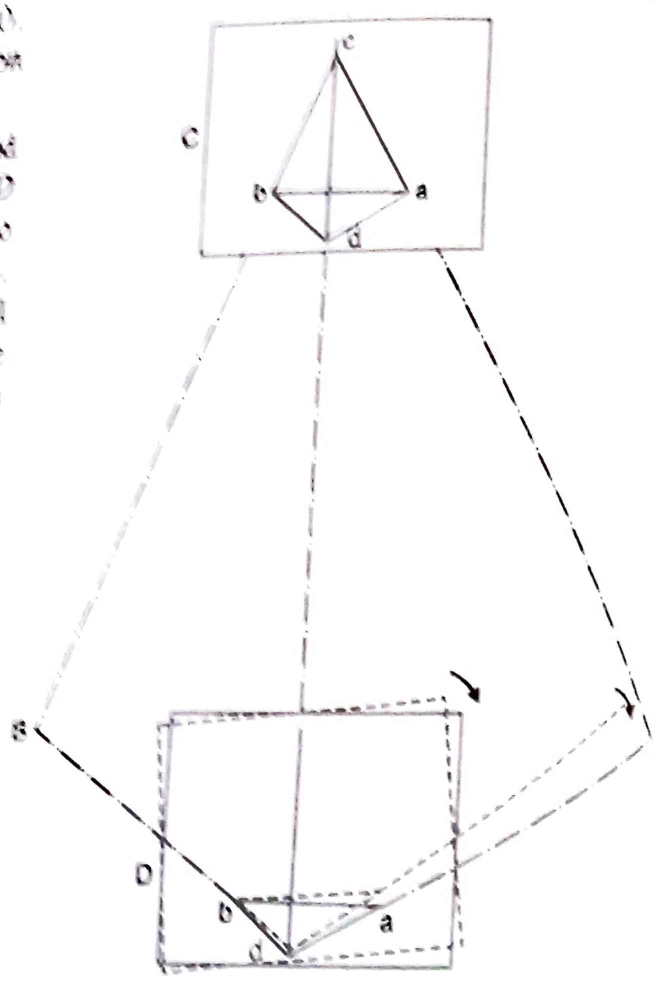


FIG. 11.26. TWO-POINT PROBLEM

[illegible] through p. The ray BB should pass through [illegible]. If the orientation is correct [illegible] be noted that the two point problem and three point problem give both [illegible] as well as fixing.

ERRORS IN PLANE TABLING

The degree of precision to be attained in plane tabling depends upon the character [illegible] the quality of the instrument, the system adopted and upon the degree to [illegible] is deliberately sacrificed for speed. The various sources of errors may be [illegible]

1. **Instrumental Errors**: Errors due to bad quality of the instrument. This includes [illegible] described for theodolite, if telescopic alidade is used.
2. Errors of plotting
3. Error due to manipulation and **sighting**. These include
   (a) Non-horizontality of board
   (b) Defective sighting
   (c) Defective orientation
   (d) Movement of board between sights
   (e) Defective or inaccurate centring

(a) ***Non-horizontality of board***

The effect of non-horizontality of board is more severe when the difference in elevation between the points sighted is more.

(b) ***Defective sighting***

The accuracy of plane table mapping depends largely upon the precision with which points are sighted. The plain alidade with open sight is much inferior to the telescopic alidade in the definition of the line of sight.

(c) ***Defective orientation***

Orientation done with compass is unreliable, as there is every possibility of local attraction. Erroneous orientation contribute towards distortion of the survey. This orientation should be checked at as many stations as possible by sighting distant prominent objects already plotted.

(d) Movement of board between sights

Due to carelessness of the observer, the table may be disturbed between any two sights resulting in the disturbance of orientation. To reduce the possibility of such movement, the clamp should be firmly applied. It is always advisable to check the orientation at the end of the observation from a station.

(e) Inaccurate centring

It is very essential to have a proper conception of the extent of error introduced by inaccurate centring, as it avoids unnecessary waste of time in setting up the table by repeated trials.

Let p be the plotted position of P (Fig. 11.27), while the position of exact centring should have been p', so that linear error in centring is $= e = pp'$ and the angular error in centring is $APB - apb = (\alpha + \beta)$.

$$= 30 \times \frac{}{10000} \text{ cm} = 0.03 \text{ mm (negligible)}$$

Case (*ii*) Scale : 1 cm = 2 m ; $\therefore$ $s = \frac{1}{200}$

$$aa' = e\,s = \frac{30}{200} = 1.5 \text{ mm (large)}.$$

11.11. ADVANTAGES AND DISADVANTAGES OF PLANE TABLING

Advantages

(1) The plan is drawn by the out-door surveyor himself while the country is before his eyes, and therefore, there is no possibility of omitting the necessary measurements.

(2) The surveyor can compare plotted work with the actual features of the area.

(3) Since the area is in view, contour and irregular objects may be represented accurately.

(4) Direct measurements may be almost entirely dispensed with, as the linear and angular dimensions are both to be obtained by graphial means.

(5) Notes of measurements are seldom required and the possibility of mistakes in booking is eliminated.

(6) It is particularly useful in magnetic areas where compass may not be used.

(7) It is simple and hence cheaper than the theodolite or any other type of survey.

(8) It is most suitable for small scale maps.

(9) No great skill is required to produce a satisfactory map and the work may be entrusted to a subordinate.

Disadvantages

(1) Since notes of measurements are not recorded, it is a great inconvenience if the map is required to be reproduced to some different scale.

(2) The plane tabling is not intended for very accurate work.

(3) It is essentially a tropical instrument.

(4) It is most inconvenient in rainy season and in wet climate.

(5) Due to heavyness, it is inconvenient to transport,

(6) Since there are so many accessories, there is every likelihood of these being lost.

PROBLEMS

1. (*a*). Discuss the advantages and disadvantages of plane table surveying over other methods.

(*b*) Explain with sketches, the following methods of locating a point by plane table survey. Also discuss the relative merits and application of the following methods :

(*i*) Radiation (*ii*) Intersection

(*iii*) Resection. (*A.M.I.E*)

2. Describe briefly the use of various accessories of a plane table.

3. Discuss with sketches, the various methods of orienting the plane table.

4. (*a*) A plane table survey is to be carried out at a scale of 1 : 5000. Show that at this scale, accurate centring of the plane table over the survey station is not necessary. What error would be caused in position on a map if the point is 45 cm out of the vertical through the station?

(*b*) Define three-point problem and show how it may be solved by tracing paper method.

5. Describe, with the help of sketches, Lehmann's Rules.

6. What is two-point problem ? How is it solved ?

7. What is three-point problem ? How is it solved by (*i*) Bessel's method (*ii*) Triangle of error method.

8. What are the different sources of errors in plane tabling ? How are they eliminated ?

9. (*a*) Describe the method of orienting plane table by backsighting.

(*b*) Distinguish between 'resection' and 'intersection' methods as applied to plane table surveying.

(*c*) How does plane table survey compare with chain surveying in point of accuracy and expediency? (*A.M.I.E*)

10. (*a*) Compare the advantages and disadvantages of plane table surveying with those of chain surveying.

(*b*) State th

UNIT - III

Levelling and Applications.

Define Levelling :-

Levelling is the branch of surveying used for determination of relative elevations of points above or below the earth surface.

Datum :- The elevation of a point is the verti distance above or below a reference surface i called datum.

Level surface :-

* A surface parallel to the mean spheroida surface of the earth is called level surf
* It is normal to the direction of gravity every point and is a curved surface.
*

4.3. LEVELLING INSTRUMENTS

The instruments commonly used in *direct levelling* are :

(1) A level

(2) A levelling staff.

1. LEVEL

The purpose of a level is to provide a horizontal line of sight. Essentially, a level consists of the following four parts :

(*a*) A *telescope* to provide line of sight

(*b*) A *level tube* to make the line of sight horizontal

(*c*) A *levelling head* (tribrach and trivet stage) to bring the bubble in its centre of run

(*d*) A *tripod* to support the instrument.

There are the following chief types of levels :

(*i*) Dumpy level (*ii*) Wye (or Y) level

(*iii*) Reversible level (*iv*) Tilting level.

(i) DUMPY LEVEL

The dumpy level originally designed by Gravatt, consists of a telescope tube firmly secured in two collars fixed by adjusting screws to the stage carried by the vertical spindle.

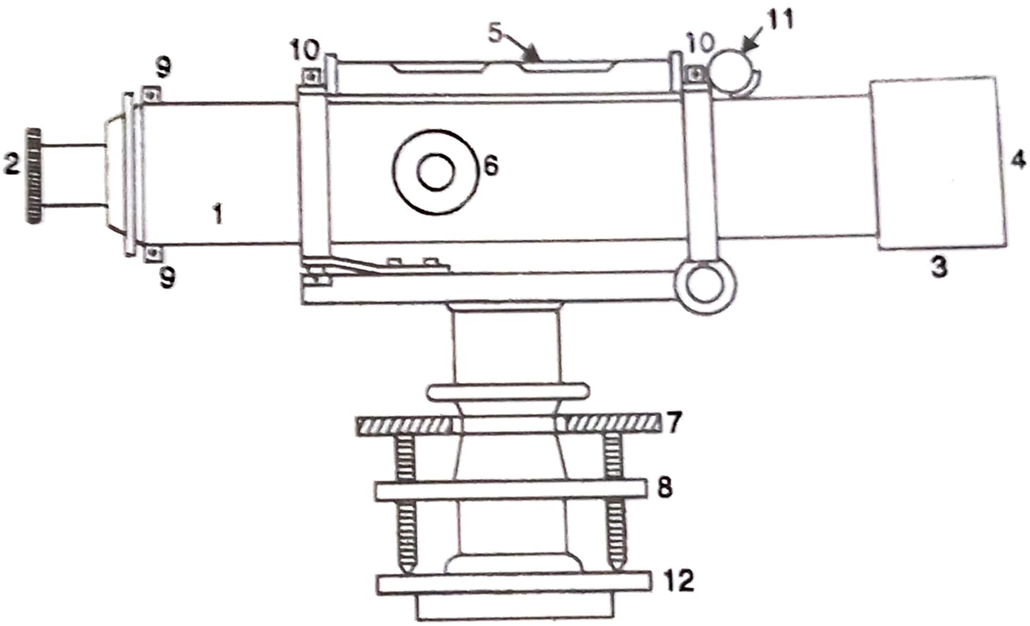


FIG. 9.2. DUMPY LEVEL

1. TELESCOPE
2. EYE-PIECE
3. RAY SHADE
4. OBJECTIVE END
5. LONGITUDINAL BUBBLE
6. FOCUSING SCREWS
7. FOOT SCREWS
8. UPPER PARALLEL PLATE (TRIBRACH)
9. DIAPHRAGM ADJUSTING SCREWS
10. BUBBLE TUBE ADJUSTING SCREWS
11. TRANSVERSE BUBBLE TUBE
12. FOOT PLATE (TRIVET STAGE).

it is ... photograph of a ... Vickers Instruments Ltd

9.4. LEVELLING STAFF

A levelling staff is a straight rectangular rod having graduations, the foot of the staff representing zero reading. The purpose of a level is to establish a horizontal line of sight. The purpose of the levelling staff is to determine the amount by which the station (*i.e.*, foot of the staff) is above or below the line of sight. Levelling staves may be divided into two classes : (*i*) Self-reading staff, and (*ii*) Target staff. A *Self Reading Staff* is the one which can be read directly by the instrument man through the telescope. A *Target Staff*, on the other hand, contains a moving target against which the reading is taken by staff man.

(i) SELF-READING STAFF

There are usually three forms of self-reading staff :

(*a*) Solid staff ; (*b*) Folding staff ; (*c*) Telescopic staff (Sopwith pattern).

Figs. 9.11 (*a*) and (*b*) show the patterns of a solid staff in English units while (*c*) and (*d*) show that in metric unit. In the most common forms, the smallest division

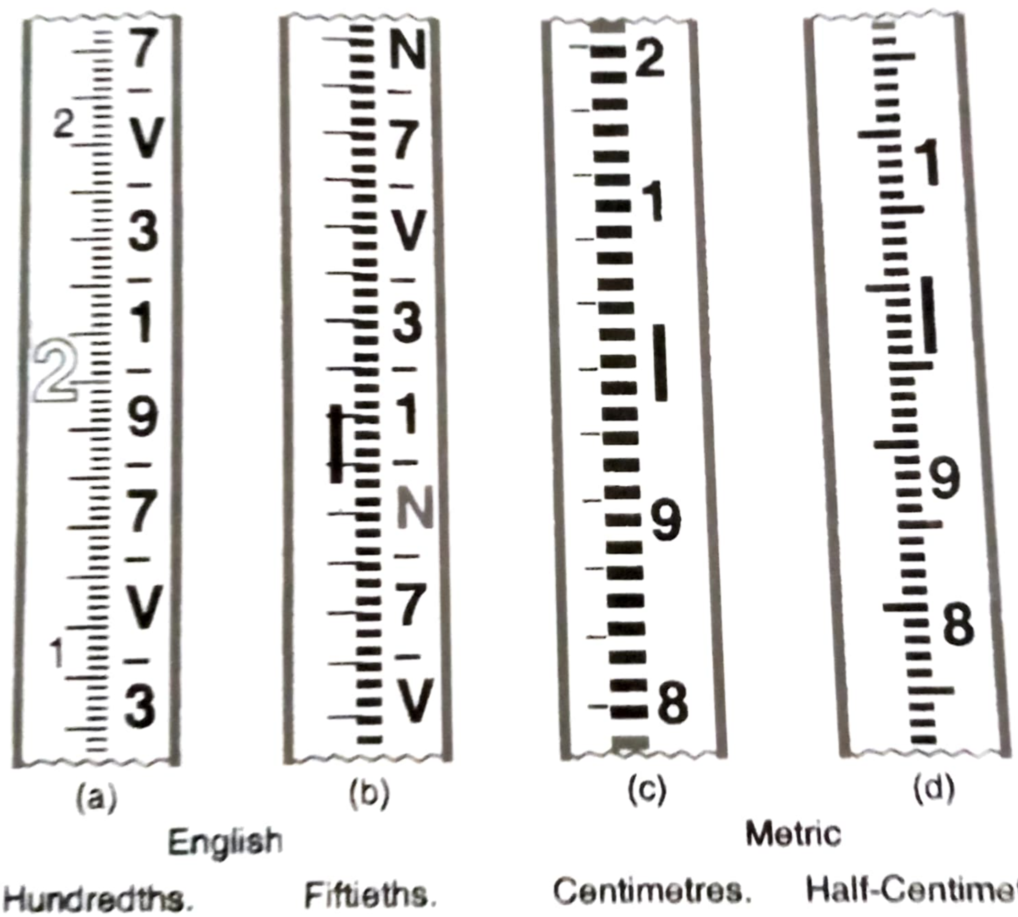


FIG. 9.11. (BY COURTESY OF M/S VICKERS INSTRUMENTS LTD.)

ft. to 5 mm. However, some staves may have fine graduations upto 2 mm
generally made of well seasoned wood having a length of 10 feet or 3 metres
9.12 shows a sopwith pattern staff arranged in three telescopic lengths. When
ded, it is usually of 14 ft (or 5 m) length. The 14 ft. staff has solid top
6" sliding into the central box of 4' 6" length. The central box, in turn, slides
box of 5' length. In the 5 m staff, the three corresponding lengths are usually
m and 2 m.

9.13 shows a folding staff usually 10 ft long having a hinge at the middle
h. When not in use, the rod can be folded about the hinge so that it becomes
to carry it from one place to the other.

a self-reading staff is always seen through the telescope, all readings appear
rted. The readings are, therefore, taken from above downwards.

levelling staves graduated in English units generally have whole number of feet
red to the left side of the staff (shown by hatched lines in Fig. 9.12). The
of the feet are marked in black to the right-hand side. The top of these

ESCOPIC STAFF FIG. 9.13 FOLDING STAFF FIG. 9.14 TARGET STAFF
(BY COURTESY OF M/S VICKERS INSTRUMENTS LTD.)

black graduations indicates the odd tenth while the bottom shows the even tenth. The hundredths of feet are indicated by alternate white and black spaces, the top of a black space indicating odd hundredths and top of a white space indicating even hundredths. Sometimes when the staff is near the instrument, the red mark of whole foot may not appear in the field of view. In that case, the staff is raised slowly until the red figure appears in the field of view, the red figure thus indicating the whole foot.

Folding Levelling Staff in Metric Units

Fig. 9.15 (*a*) shows a 4 m folding type levelling staff (IS : 1779-1961). The staff comprises two 2 m thoroughly seasoned wooden pieces with the joint assembly. Each piece of the staff is made of one longitudinal strip without any joint. The width and thickness of staff is kept 75 mm and 18 mm respectively. The folding joint of the staff is made of the detachable type with a locking device at the back. The staff is jointed together in such a way that

(*a*) the staff may be folded to 2 m length

(*b*) the two pieces may be detached from one another, when required, to facilitate easy handling and manipulation with one piece, and

(*c*) when the two portions are locked together, the two pieces become rigid and straight.

A circular bubble, suitably cased, of 25-minute sensitivity is fitted at the back. The staff has fittings for a plummet to test and correct the back bubble. A brass is screwed on to the bottom brass cap. The staff has two folding handles with spring acting locking device or an ordinary locking device.

Each metre is subdivided into 200 divisions, the thickness of graduations being 5 mm. Fig. 9.15 (*b*) shows the details of graduations. Every decimetre length is figured with the corresponding numerals (the metre numeral is made in red and the decimetre numeral in black). The decimetre numeral is made continuous throughout the staff.

(*ii*) **TARGET STAFF**

Fig. 9.14 shows a target staff having a sliding target equipped with vernier. The rod consists of two sliding lengths, the lower one of approx. 7 ft and the upper one of 6 ft. The rod is graduated in feet, tenths and hundredths, and the vernier of the target enables the readings to be taken upto a thousandth part of a foot. For readings below 7 ft the target is slided to the lower part while for readings above that, the target is fixed to the 7 ft mark of the upper length. For taking the reading, the level man directs the staff man to raise or lower the target till it is bisected by the line of sight. The staff holder then clamps the target and takes the reading. The upper part of the staff is graduated from top downwards. When higher readings have to be taken, the target is set at top (*i.e.* 7 ft mark) of the sliding length and the sliding length carrying the target is raised until the target is bisected by the line of sight. The reading is then on the back of the staff where a second vernier enables readings to be taken to a thousandth of a foot.

Relative Merits of Self-Reading and Target Staffs

(*i*) With the self-reading staff, readings can be taken quicker than with the target staff.

one time through ... field of view is not merely dependent upon the size of the hole in the cross-hair reticule, but it also increases as the magnification of the telescope decreases.

9.6. TEMPORARY ADJUSTMENTS OF A LEVEL

Each surveying instrument needs two types of adjustments : (1) temporary adjustments, and (2) permanent adjustments. *Temporary adjustments or Station adjustments* are those which are made at every instrument setting and preparatory to taking observations with the instrument. *Permanent adjustments* need be made only when the fundamental relations between some parts or lines are disturbed (See Chapter 16).

The temporary adjustments for a level consist of the following :

(1) Setting up the level (2) Levelling up (3) Elimination of parallax.

1. **Setting up the Level.** The operation of setting up includes (*a*) fixing the instrument on the stand, and (*b*) levelling the instrument approximately by leg adjustment. To fix the level to the tripod, the clamp is released, instrument is held in the right-hand and is fixed on the tripod by turning round the lower part with the left hand. The tripod legs are so adjusted that the instrument is at the convenient height and the tribrach is approximately horizontal. Some instruments are also provided with a small circular bubble on the tribrach.

2. **Levelling up.** After having levelled the instrument approximately, accurate levelling is done with the help of foot screws and with reference to the plate levels. The purpose of 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.

(*a*) **Three Screw Head**

1. Loose the clamp. Turn the instrument until the longitudinal axis of the plate level is roughly parallel to a line joining any two (such as *A* and *B*) of the levelling screws [Fig. 9.29 (*a*)].

2. 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* [see Fig. 9.29 (*a*)].

3. Turn the upper plate through 90°, *i.e.*, until the axis on the level passes over the position of the third levelling screw *C* [Fig. 9.29 (*b*)].

4. Turn this levelling screw until the bubble is central.

5. Return the upper part through 90° to its original position [Fig. 9.29 (*a*)] and repeat step (2) till the bubble is central.

(*a*) (*b*)

FIG. 9.29. LEVELLING-UP WITH THREE FOOT SCREWS.

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

7. Repeat steps (2) and (4) till the bubble is central in both the positions.

8. Now rotate the instrument through 180°. The bubble should remain in the centre of its run, provided it is in correct adjustment. The vertical axis will then be truly vertical. If not, it needs *permanent adjustment*.

Note. *It is essential to keep the same quarter circle for the changes in direction and not to swing through the remaining three quarters of a circle to the original position.*

(*b*) **Four Screw Head**

1. 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* [Fig. 9.30 (*a*)].

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

3. Turn the upper part through 90° until the spirit level axis is parallel to the other two diagonally opposite screws such as *A* and *C* [Fig. 9.30 (*b*)].

4. Centre the bubble as before.

5. Repeat the above steps till the bubble is central in both the positions.

6. Turn through 180° to check the permanent adjustment as for three screw instrument.

(*a*) (*b*)

FIG. 9.30. LEVELLING-UP WITH FOUR-FOOT

In modern instruments, three-foot screw levelling head is used in preference to a four foot screw levelling head. The three-screw arrangement is the better one, as three points of support are sufficient for stability and the introduction of an extra point of support leads to uneven wear of the screws. On the other hand, a four-screw levelling head is simpler and lighter as a three-screw head requires special casting called a *tribrach*. A three-screw instrument has also the important advantage of being more rapidly levelled.

3. **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 : (*i*) by focusing the eye-piece for distinct vision of the cross-hairs, and (*ii*) by focusing the objective to bring the image of the object in the plane of cross-hairs.

(*i*) ***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-haris are seen sharp and distinct. In some telescopes, graduations are provided at the eye-piece so that one can always remember the particular graduation position to suit his eyes. This may save much of time.

(*ii*) ***Focusing the objective***

The telescope is now directed towards the staff and the focusing screw in turned till the image appears clear and sharp. The image so formed is in the plane of cross-hairs.

9.7. THEORY OF DIRECT LEVELLING (SPIRIT LEVELING)

A level provides horizontal line of sight, *i.e.*, a line tangential to a level surface at the point where the instrument stands. The difference in elevation between two points is the vertical distance between two level lines. Strictly speaking, therefore, we must have a level line of sight and not a horizontal line of sight ; but the distinction between a level surface and a horizontal plane is not an important one in plane surveying.

Neglecting the curvature of earth and refraction, therefore, the theory of direct levelling is very simple. With a level set up at any place, the difference in elevation between any two points within proper lengths of sight is given by the difference between the rod readings taken on these points. By a succession of instrument stations and related readings, the difference in elevation between widely separated points is thus obtained.

SPECIAL METHODS OF SPIRIT LEVELLING

(*a*) **Differential Levelling.** It is the method of direct levelling the object of which is solely to determine the difference in elevation of two points regardless of the horizontal positions of the points with respect of each other. When the points are apart, it may be necessary to set up the instruments serveral times. This type of levelling is also know as *fly levelling*.

(*b*) **Profile Levelling.** It is the method of direct-levelling the object of which is to determine the elevations of points at measured intervals along a given line in order to obtain a profile of the surface along that line.

(*c*) **Cross-Sectioning.** Cross-sectioning or cross-levelling is the process of taking levels on each side of a main line at right angles to that line, in order to determine a vertical cross-section of the surface of the ground, or of underlying strata, or of both.

(*d*) **Reciprocal Levelling.** It is the method of levelling in which the difference in elevation between two points is accurately determined by two sets of reciprocal observations when it is not possible to set up the level between the two points.

(*e*) **Precise Levelling**. It is the levelling in which the degree of precision required is too great to be attained by ordinary methods, and in which, therefore, special, equipment or special precautions or both are necessary to eliminate, as far as possible, all sources of error.

TERMS AND ABBREVIATIONS

(i) **Station**. In levelling, a station is that point where the level rod is held not where level is set up. It is the point whose elevation is to be ascertained or the point that is to be established at a given elevation.

(ii) **Height of Instrument (H.I.)** For any set up of the level, the height of instrument is the elevation of plane of sight (line of sight) with respect to the assumed datum. It does not mean the height of the telescope above the ground where the level stands.

(iii) **Back Sight (B.S.)**. Back sight is the sight taken on a rod held at a point of known elevation, to ascertain the amount by which the line of sight is *above* that point and thus to obtain the height of the instrument. *Back sighting* is equivalent to measuring up from the point of known elevation to the line of sight. It is also known as a plus sight as the back sight reading is always added to the level of the datum to get the height of the instrument. *The object of back sighting is, therefore, to ascertain the height of the plane of sight.*

(iv) **Fore Sight (F.S.)**. Fore sight is a sight taken on a rod held at a point of unknown elevation, to ascertain the amount by which the point is *below* the line of sight and thus to obtain the elevation of the station. *Fore sighting* is equivalent to measuring *down* from the line of sight. It is also known as a *minus* sight as the fore sight reading is always subtracted (except in speical cases of tunnel survey) from the height of the instrument to get the elevation of the point. *The object of fore sighting is, therfore to ascertain the elevation of the point.*

(v) **Turning Point (T.P.)**. Turning point or *change point* is a point on which both minus sight and plus sight are taken on a line of direct levels. The minus sight (fore sight) is taken on the point in one set of instrument to ascertain the elevation of the point while the plus sight (back sight) is taken on the same point in other set of the instrument to establish the new height of the instrument.

(vi) **Intermediate Station (I.S.)**. Intermediate station is a point, intermediate between two turning points, on which only one sight (minus sight) is taken to determine the elevation of the station.

STEPS IN LEVELLING (Fig. 9.31)

There are two steps in levelling : (*a*) to find by how much amount the line of sight is above the bench mark, and (*b*) to ascertain by how much amount the next point is below or above the line of sight.

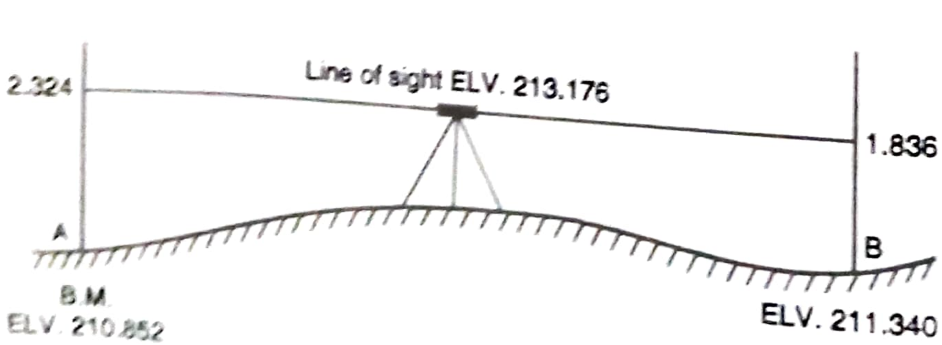


FIG. 9.31

A level is set up approximately midway between the bench mark (or a point of known elevation) and the point, the elevation of which is to be ascertained by direct levelling. A back sight is taken on the rod held at the bench mark. Then

$$H.I. = Elev.\ of\ B.M. + B.S. \quad \ldots(1)$$

Turning the telescope to bring into view the rod held on point B, a foresight (minus sight) is taken. Then

$$Elev. = H.I. - F.S. \quad \ldots(2)$$

For example, if elevation of B.M. = 210.852 m, B.S. = 2.324 m and F.S. = 1.836 m.

$$H.I. = 210.852 + 2.324 = 213.176\ m$$

Then

$$Elev.\ of\ B = 213.176 - 1.836 = 211.340\ m$$

It is to be noted that if a back sight is taken on a bench mark located on the roof of a tunnel or on the ceiling of a room with the instrument at a lower elevation, the back sight must be subtracted from the elevation to get the height of the instrument. Similarly, if a foresight is taken on a point higher than the instrument, the foresight must be added to the height of the instrument, to get the elevation of the point.

9.7 DIFFERENTIAL LEVELLING

The operation of levelling to determine the elevation of points at some distance apart is called **differential levelling**, and is usually accomplished by direct levelling. When two points are at such a distance from each other that they cannot both be within range of the level at the same time, the difference in elevation is not found by single setting but the distance between the points is divided in two stages by turning points on which the staff is held and the difference of elevation of each of succeeding pair of such turning points is found by separate setting up of the level.

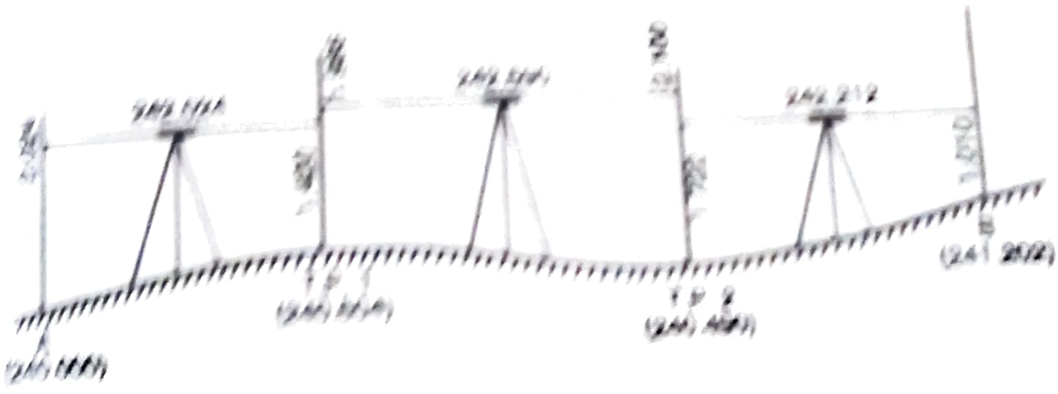


FIG. 9.32

Referring to Fig. 9.32, A and B are the two points. The distance AB has been divided into three parts by choosing two additional points on which staff readings (both plus sight and minus sight) have been taken. Points 1 and 2 thus serve as **turning points**.

The R.L. of point A is 240.000 m. The height of the first setting of the instrument therefore = 240.000 + 2.102 = 242.102. If the following F.S. is 1.420, the R.L. of T.P. 1 = 242.102 − 1.420 = 240.682 m. By a similar process of calculations, R.L. of T.P. 2 = [illegible] m and of B = 241.202 m.

9.9. HAND SIGNALS DURING OBSERVATIONS

When levelling is done at construction site located in busy, noisy areas, it becomes difficult for the instrument man to give instructions to the man holding the staff at the other end. through vocal sounds. In that case, the following hand signals are found to be useful (Table 9.1 and Fig. 9.33)

TABLE 9.1. HAND SIGNALS

Refer Fig. 9.33	*Signal*	*Message*
(a)	Movement of left arm over 90°	Move to my left
(b)	Movement of right arm over 90°	Move to my right
(c)	Movement of left arm over 30°	Move top of staff to my left
(d)	Movement of right arm over 30°	Move top of staff to my right
(e)	Extension of arm horizontally and moving hand upwards	Raise height peg or staff
(f)	Extension of arm horizontally and moving hand downwards	Lower height peg or staff
(g)	Extension of both arms and slightly thrusting downwards	Establish the position .
(h)	Extension of arms and placement of hand on top of head.	Return to me

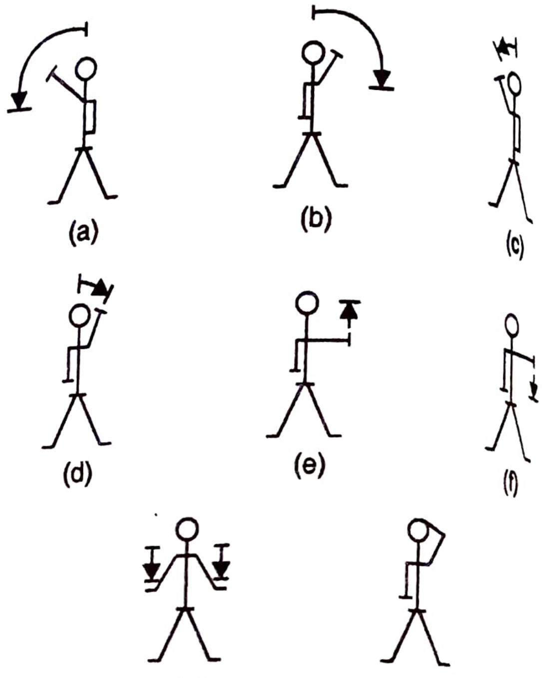


FIG. 9.33. HAND SIGNALS.

9.10. BOOKING AND REDUCING LEVELS

There are two methods of booking and reducing the elevation of points from the observed staff readings : (1) *Collimation or Height of Instrument* method ; (2) *Rise and Fall* method.

(1) HEIGHT OF INSTRUMENT METHOD

In this mehtod, the height of the instrument (*H.I.*) is calculated for each setting of the instrument by adding back sight (plus sight) to the elevation of the *B.M.* (First point). The elevation of reduced level of the turning point is then calculated by subtracting from *H.I.* the fore sight (minus sight). For the next setting of the instrument, the *H.I.* is obtained by adding the *B.S* taken on *T.P.* 1 to its *R.L.* The process continues till the *R.L.* of the last point (a fore sight) is obtained by subtracting the staff reading from height of the last setting of the instrument. If there are some intermediate points, the *R.L.* of those points is calculated by subtracting the intermediate sight (minus sight) from the height of the instrument for that setting.

The following is the specimen page of a level field book illustrating the method of booking staff readings and calculating reduced levels by height of instrument method.

Station	*B.S*	*I.S.*	*F.S.*	*H.I.*	*R.L.*	*Remarks*
A	0.865			561.365	560.500	B.M. on Gate
B	1.025		2.105	560.285	559.260	
C		1.580			558.705	Platform
D	2.230		1.865	560.650	558.420	
E	2.355		2.835	560.270	557.815	
F			1.760		558.410	
Check	6.475		8.565 6.475		558.410 560.500	Checked
			2.090	Fall	2.090	

Arithmetic Check. The difference between the sum of back sights and the sum of fore sights should be equal to the difference between the last and the first *R.L.* Thus

$$\Sigma B.S. - \Sigma F.S. = \textbf{Last } R.L. - \textbf{ First } R.L.$$

The method affords a check for the *H.I.* and *R.L.* of turning points but not for the intermediate points.

(2) RISE AND FALL METHOD

In rise and fall method, the height of instrument is not at all calculated but the difference of level between consecutive points is found by comparing the staff readings on the two points for the same setting of the instrument. The difference between their staff readings indicates a *rise* or *fall* according as the staff reading at the point is *smaller* or *greater* than that at the preceding point. The figures for 'rise' and 'fall' worked out thus for all the points give the vertical distance of each point above or below the preceding one, and if the level of any one point is known the level of the next will be obtained by adding its rise or subtracting its fall, as the case may be.

The following is the specimen page of a level field book illustrating the method of booking staff readings and calculating reduced levels by rise and fall method :

Station	*B.S.*	*I.S.*	*F.S.*	*Rise*	*Fall*	*R.L.*	*Remarks*
A	0.865					560.500	B.M. on Gate
B	1.025		2.105		1.240	559.260	
C		1.580			0.555	558.705	Platform
D	2.230		1.865		0.285	558.420	
E	2.355		2.835		0.605	557.815	
F			1.760	0.595		558.410	
Check	6.475		8.565 6.475	0.595	2.685 0.595	558.410 560.500	Checked
		Fall	2.090	Fall	2.090	2.090	

Arithmetic Check. The difference between the sum of back sights and sum of fore sights should be equal to the difference between the sum of rise and the sum of fall and should also be equal to the difference between the *R.L.* of last and first point. Thus,

$$\Sigma B.S. - \Sigma F.S. = \Sigma \text{ Rise} - \Sigma \text{ Fall} = \text{Last } R.L. - \text{First } R.L.$$

This provides a complete check on the intermediate sights also. The arithmetic check would only fail in the unlikely, but possible, case of two more errors occurring in such a manner as to balance each other.

It is advisable that on each page the rise and fall calculations shall be completed and checked by comparing with the difference of the back and fore sight column summations, before the reduced level calculations are commenced.

Comparison of the Two Methods. The height of the instrument (or collimation level) method is more rapid, less tedious and simple. However, since the check on the calculations for intermediate sights is not available, the mistakes in their levels pass unnoticed. The rise and fall method though more tedious, provides a full check in calculations for all sights. However, the height of instrument method is more suitable in case, where it is required to take a number of readings from the same instrument setting, such as for constructional work, profile levelling etc.

Example 9.1. *The following staff readings were observed successively with a level, the instrument having been moved after third, sixth and eighth readings : 2.228 ; 1.606 ; 0.988 ; 2.090 ; 2.864 ; 1.262 ; 0.602 ; 1.982 ; 1.044 ; 2.684 metres.*

Enter the above readings in a page of a level book and calculate the R.L. of points if the first reading was taken with a staff held on a bench mark of 432.384 m.

Solution.

Since the instrument was shifted after third, sixth and eighth readings, these readings will be entered in the *F.S.* column and therefore, the fourth, seventh and ninth readings will be entered on the *B.S.* column. Also, the first reading will be entered in the *B.S.* column and the last reading in the *F.S.* column. All other readings will be entered in the *I.S.* column.

The reduced levels of the points may be calculated by rise and fall method as tabulated below :

Station	B.S.	I.S.	F.S.	Rise	Fall
1	2.228				
2					

( ) The following consecutive readings were taken with a level & a 4m levelling staff on continuously sloping ground at a common interval of 20 meters 0.385, 1.030, 1.925, 2.825, 3.730, 4.685, 0.625, 2.005, 3.110, 4.485. The reduced level of the first point was 208.125 m. Rule out a page of a level field book and enter the above readings. Calculate the RL of points by rise & fall method & also the gradient of the line joining the first & the last point.

Solution:

Chainage	Station	B.S	I.S	F.S	Rise	Fall	R.L.	Remark
0	1	0.385					208.125	B.M
20	2		1.030			0.645	207.480	
40	3		1.925			0.895	206.585	
60	4		2.825			0.900	205.685	
80	5		3.730			0.905	204.780	
100	6	0.625		4.685		0.955	203.325	
120	7		2.005			1.380	202.445	
140	8		3.110			1.105	201.340	
160	9			4.485		1.375	199.965	
	Σ -	1.010		9.170	0.00	8.160		

Check

$$\Sigma .BS \sim \Sigma F.S = \Sigma Rise \sim \Sigma fall = last\ RL \sim 1^{st}\ RL$$

$$1.010 \sim 9.170 = 0.00 \sim 8.160 = 208.125 \sim 199.965$$

$$8.160 = 8.160 = 8.160$$

Hence ok

Gradient

$$\text{Gradient of line} = \frac{1^{st}\ RL \sim Last\ RL}{\text{Tot. chainage length}} = \frac{8.160}{160}$$

$$= \frac{1}{19.61}$$

1 in 19.61 ...

( ) The following consecutive readings where taken with a level and 5 m levelling staff at a common interval of 20 m,

0.385, 1.030, 1.925, 2.825, 3.730, 4.685, 4.685 2.005, 3.110, 4.485. The instrument change at 6th reading. The RL of the 1st point 209.135 m. The rule out of a level field book and enter the above readings, calculate the RL with the point by Rise & fall method. Also the gradient of the line joining 1st & last point. The readings are taken along slope of the hill.

distance triangle the triangles

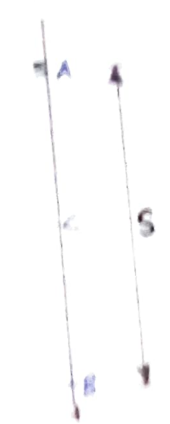
Solution:

Station	B.S	I.S	F.S	Rise	Fall	RL	Remarks
1	0.385					209.135	RL of 1st point
2		1.030			0.645	208.490	
3		1.925			0.895	206.585	
4		2.825			0.900	205.685	
5		3.730			0.905	204.780	
6	0.625		4.685		0.955	203.825	C.P
7		2.005			1.380	202.445	
8		3.110			1.105	201.340	
9			4.485		1.375	199.765	Last RL

elevation

in then

$$\text{slope} = \frac{\text{Last vertical reading} \sim 1^{st} \text{ vertical reading}}{\text{horizontal length}}$$



$$\text{slope} = \frac{1^{st}\ \text{RL} \sim \text{Last RL}}{\text{horizontal length}} = \frac{209.135 - 199.765}{8 \times 20}$$

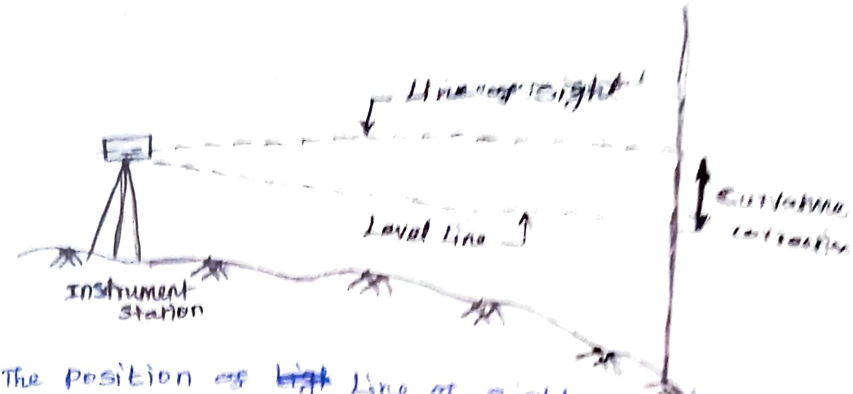
slope = 0.051 m

$\Rightarrow \frac{1}{0.051} = 19.60$

i.e, gradient or slope

= 1 in 20 grade

Curvature correction (C_c)



* The position of ~~high~~ Line of sight is horizontal and the level line is curved downwards and parallel to the mean spheroidal surface of the earth.

* The vertical distance b/w the line of sight and the level line at a particular place is referred to as curvature correction.

$$\boxed{C_c = 0.07857 \, D^2} \text{ in m} \quad (\text{negative})$$

$C_c \rightarrow$ curvature correction

$D \rightarrow$ horizontal distance in km

Correction for Refraction (C_r)

* The density of air varying, the rays of light are refracted, when they pass through layers of air.

* Because of this, the line of sight is refracted towards the surface of the earth in a curved path.

* Under Normal atmospheric conditions radius of this curve is seven times as that of the earth.

$$C_r = \frac{1}{7} \frac{D^2}{2R}$$

ie, $C_r = \frac{1}{7}\, 0.07857\, D^2$

$\boxed{C_r = 0.01121\, D^2}$ in m (positive)

Combined correction:

* Combined correction is negative.

Combined correction for curvature & refraction is

$\boxed{C = -0.06728\, D^2}$ in m

1, Find the correction for refraction for horizontal distance of 600 m and 1.5 km.

Solution:-

Correction for refraction

$$C_r = 0.01121\, D^2$$

Note:-

$D \rightarrow$ substitute in km

$D = 600$ m

$D = 0.6$ km

For 600 m

$$C_r = 0.01121\, D^2$$

$$= 0.01121 \times (0.6)^2$$

$$= 4.0356 \times 10^{-3} \text{ m}$$

$$C_r = 0.004 \text{ m}$$

For 1.5 km

$$C_r = 0.01121\, D^2$$

$$= 0.01121 \times (1.5)^2$$

$$= 0.025 \text{ m}$$

2, Find the correction for curvature and for refraction for a distance of

a) 1200 m (b) 2.48 km.

Solution:

a) 1200 m

correction for curvature

$C_c = 0.07857\ D^2$ in m

$= 0.07857 \times (1200 \times 10^{-3})^2$ in m

$= 0.11314$ m.

correction for refraction

$C_r = 0.01121\ D^2$

$= 0.01121 \times (1200 \times 10^{-3})^2$ in m

$= 0.016$ m.

b) 2.48 km

correction for curvature

$C_c = 0.07857\ D^2$ in m

$= 0.07857 \times (2.48)^2$

$= 0.4832$ m.

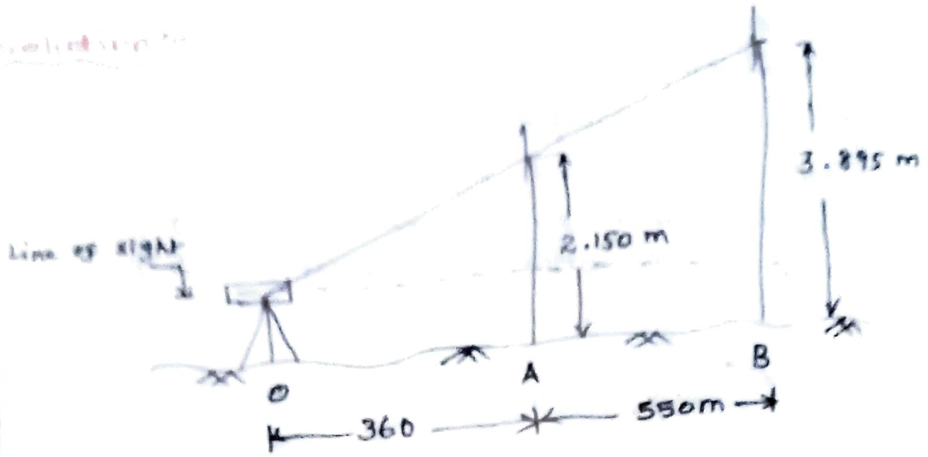
correction for refraction.

$C_r = 0.01121\ D^2$ in m

$= 0.01121 \times (2.48)^2$

$= 0.0689$ m.

3, A level is set up at a statio 'O'. The reading on a staff, when held at 'A' 360 m away from 'O' is 2.150 m and reading on the staff. When held at 'B' 550 m away in 3.895 m. Find the true difference from elevation from A & B.



In 'A' from instrument is small the correction for curvature is negligible.

Combined correction for staff 'A'

$D = 360$ m

$D = 360 \times 10^{-3}$ km

$$C = 0.06728 D^2$$

$$= 0.06728 \times (360 \times 10^{-3})^2$$

$$= 0.0087 \text{ m.}$$

combined correction for staff 'B'.

$D = 550$ m

$D = 550 \times 10^{-3}$ km

$$C = 0.06728 D^2$$

$$= 0.06728 \times (550 \times 10^{-3})^2$$

$$= 0.0203 \text{ m.}$$

$\therefore$ correct staff reading for A

= staff 'A' − combined correction for staff 'A'.

$$= 2.150 - 0.0087$$

$$= 2.1413 \text{ m.}$$

$\therefore$ correct staff reading for B

= staff 'B' − combined correction for staff B

$$= 3.895 - 0.0203$$

$$= 3.875 \text{ m.}$$

A level was set up at a station 'O'. Staff readings were taken on the two points P & Q situated at 380 m and 560 m respectively, and the readings were 2.255 and 3.875 respectively. Find the true difference in elevation b/w 'P & Q'.

(i) D = 380 m

Combined correction for 'P'

$= 0.06728\, D^2$

$= 0.06728 \times (380 \times 10^{-3})^2$

$= 0.0097$ m.

Combined corrector (or) correction for curvature and refraction for 'Q'

$= 0.06728\, D^2$

$= 0.06728 \times (560 \times 10^{-3})^2$

$= 0.0211$ m.

Corrected staff reading at P

$= 2.255 - 0.0097$

$= 2.2453$ m.

Corrected staff reading at Q

$= 3.875 - 0.0211$

$= 3.8539$ m

∴ True difference in elevation b/w P & Q

$= 3.8539 - 2.2453$

$= 1.609$ m.

Computation of volumes of irregular boundaries or solids

The volumes of irregular boundaries or solids like earthwork in embankment or cutting are determined by measuring the areas of cross-section at regular intervals and applying any one of the following rules.

1. End area rule
2. Mid area rule
3. Mean (or) average area rule
4. Trapezoidal rule
5. Prismoidal (or) simpson's rule.

End area rule :-

$$V = \text{common interval} \times \left[\begin{array}{c}\text{Sum of all area of} \\ \text{c/s except last one}\end{array}\right]$$

$$\boxed{V = d\left[A_1 + A_2 + A_3 + \cdots\cdots A_{n-1}\right]}$$

Mid area rule :-

$$V = \text{common interval} \times \left[\begin{array}{c}\text{sum of all mid section} \\ \text{Area}\end{array}\right]$$

$$\boxed{V = d\left[A_{m_1} + A_{m_2} + A_{m_3} + \cdots + A_{m(n-1)}\right]}$$

Mean (or) Average area rule :-

$$V = \text{Length} \times \text{Average of all c/s area.}$$

$$\boxed{V = L\left[\frac{A_1 + A_2 + A_3 + \cdots + A_n}{n}\right]}$$

Trapezoidal rule :-

$$V = \frac{\text{common interval}}{2}\left[\begin{array}{l}\Sigma \text{ of area} \\ \text{of } 1^{st} \text{ \& last} \\ \text{section}\end{array} + 2(\Sigma ...\right.$$

$$V = \frac{d}{2}\left[(A_1 + A_n) + 2(A_2 + A_3 + A_4 + \dots A_{n-1})\right]$$

Prismoidal (or) simpson's rule :-

$$V = \frac{\text{common interval}}{3}\left[\left(1^{st} + \text{Last ordinate area}\right) + 2\left(\Sigma \text{ of odd section areas}\right) + 4\left(\Sigma \text{ of area of even section}\right)\right]$$

$$V = \frac{d}{3}\left[(A_1 + A_n) + 2(A_3 + A_5 + A_7 + \dots) + 4(A_2 + A_4 + \dots)\right]$$

The height of an embankment of formation width (b) 10m with side slope 1.5 : 1 are found to be 2 m, 3m & 4 m at 0m, 30m, 60 m chainage respectively. Determine the volume of the bank in the 60m length by all methods assuming the ground as level in the transverse direction.

Solution :-

Given Data :-

Formation width (b) = 10 m

Common interval (d) = 30 m

Side slope (S) = 1.5

Height of bank

$h_1 = 2\,m$

$h_2 = 3\,m$

$h_3 = 4\,m$

$L = 60\,m$

c/s at 0 m level

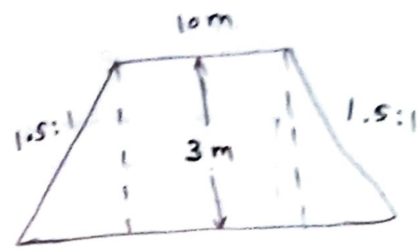
(b + nd) d

$A_1 = (b + sh)\,h$

$A_1 = [10 + (1.5 \times 2)] \times 2$

$\boxed{A_1 = 26\ m^2}$

c/s at 30 m level

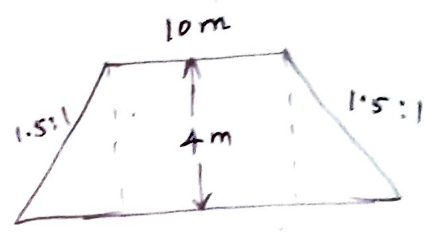


$A_2 = (b + sh)\,h$

$= [10 + (1.5 \times 3)]\,3$

$\boxed{A_2 = 43.50\ m^2}$

c/s at 60 m level



$A_3 = (b + sh)\,h$

$= [10 + (1.5 \times 4)] \times 4$

$\boxed{A_3 = 64\ m^2}$

$A_1 = 26\ m^2$; $A_2 = 43.50\ m^2$; $A_3 = 64\ m^2$

$d = 30\ m$

(i) End area rule :-

$V = d\,[A_1 + A_2 + \dots + A_{n-1}]$

$V = 30\,[26 + 43.50]$

$\boxed{V = 2085\ m^3}$

Mid area rule:-

$$V = d\left[A_1 m_1 + A_2 m_2 + \cdots A_3 m_{n-1}\right]$$

$h_1 = m_1$

$h_2 = m_2$

$$m_1 = \frac{2+3}{2} = 2.5\,m$$

$$m_2 = \frac{3+4}{2} = 3.5\,m$$

$A = (b+sh)h$

$$A_1 = [10 + (1.5 \times 2.5)]\,2.5 = 34.375\,m^2$$

$$A_2 = [10 + (1.5 \times 3.5)]\,3.5 = 53.375\,m^2$$

$$V = d\left[34.375 + 53.375\right]$$

$$\boxed{V = 2632.50\,m^3}$$

(iii) Average area rule :-

$$V = \left[\frac{A_1 + A_2 + A_3 + \cdots + A_n}{n}\right] \times L$$

$$V = \left(\frac{26 + 43.5 + 64}{3}\right) \times 60$$

$$\boxed{V = 2670\,m^3}$$

(iv) Trapezoidal rule:-

$$V = \frac{d}{2}\left[(A_1 + A_n) + 2(A_2 + A_3 + \cdots A_{n-1})\right]$$

$$V = \frac{30}{2}\left[(26+64) + 2(43.5)\right]$$

$$\boxed{V = 2655\,m^3}$$

(v) Prismoidal (or) simpson's rule:-

$$V = \frac{d}{3}\left[(A_1 + A_n) + 2(A_3 + A_5 + \cdots) + 4(A_2 + A_4 + \cdots)\right]$$

$$= \frac{30}{3}\left[(2.6+64) + 2(0) + 4(43.5)\right]$$

$$\boxed{V = 2640\ m^3}$$

2\. A railway embankment is 10m wide with side slopes 1.5 : 1. Assuming the ground to be level in a direction traverse to the centre line, calculate the volume contained in a length of 120m, the centre height at 20m intervals being in meter 2.2, 3.7, 3.8, 4, 3.8, 2.8, 2.5 using all methods.

Given Data:

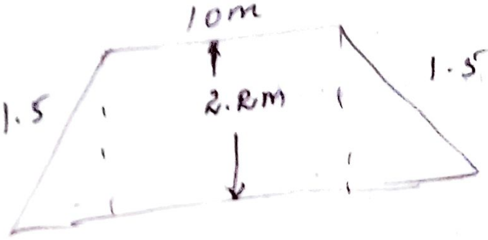
$b = 10m$ $\quad$ $h_1 = 2.2m$; $h_2 = 3.7m$

$S = 1.5$ $\quad$ $h_3 = 3.8m$; $h_4 = 4m$

$L = 120m$ $\quad$ $h_5 = 3.8m$; $h_6 = 2.8m$

$h_7 = 2.5m$

Cross section at 0 m level:-

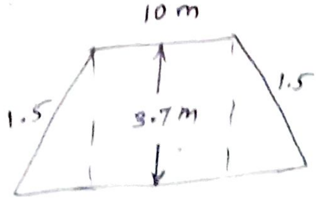


$$A_1 = (b + sh)\,h$$

$$= [10 + (1.5 \times 2.2)] \times 2.2$$

$$\boxed{A_1 = 29.26\ m^2}$$

Cross section at 20m level:-

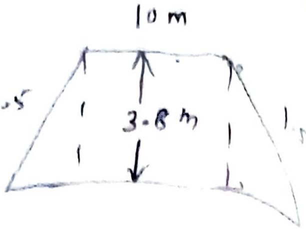


$$A_2 = (b + sh)\,h$$

$$= [10 + (1.5 \times 3.7)] \times 3.7$$

$$\boxed{A_2 = 57.535\ m^2}$$

c/s at 40 m level :

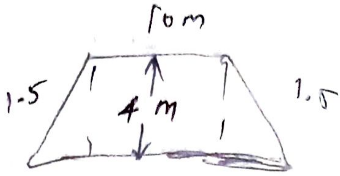


$A_3 = (b+sh)h$

$= [10 + (1.5 \times 3.8)] \times 3.8$

$\boxed{A_3 = 59.66 \, m^2}$

c/s at 60 m level :-

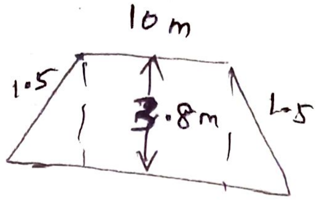


$A_4 = (b+sh)h$

$= (10 + 1.5 \times 4) \times 4$

$\boxed{A_4 = 64 \, m^2}$

c/s at 80 m level :-

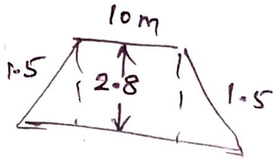


$A_5 = (b+sh)h$

$= (10 + 1.5 \times 3.8) \times 3.8$

$\boxed{A_5 = 59.66 \, m^2}$

c/s at 100 m level :-

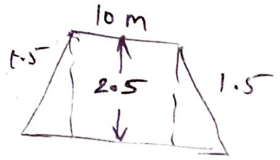


$A_6 = (b+sh)h$

$= (10 + 1.5 \times 2.8) \times 2.8$

$\boxed{A_6 = 39.76 \, m^2}$

c/s at 120 m level :-



$A_7 = (b+sh)h$

$= (10 + 1.5 \times 2.5) \times 2.5$

$\boxed{A_7 = 34.375 \, m^2}$

(i) End Area Rule :-

(except last one)

$V = d[A_1 + A_2 + A_3 + A_4 + A_5 + A_6]$

$$= 20\left[29.26 + 57.539 + 59.66 + 64 + 59.66 + 29.76\right] ~~+34.875~~$$

$$= 20 \times 309.875$$

$$\boxed{V = 6197.5\ m^3}$$

(ii) Mid Area rule :-

$$V = \left[A_{m_1} + A_{m_2} + A_{m_3} + A_{m_4} + A_{m_5} + A_{m_6}\right] d$$

$$A_{m_1} = \frac{A_1 + A_2}{2} = \frac{29.26 + 57.535}{2} = 43.40\ m^2$$

$$A_{m_2} = \frac{A_2 + A_3}{2} = \frac{57.535 + 59.66}{2} = 58.60\ m^2$$

$$A_{m_3} = \frac{A_3 + A_4}{2} = \frac{59.66 + 64}{2} = 61.83\ m^2$$

$$A_{m_4} = \frac{A_4 + A_5}{2} = \frac{64 + 59.66}{2} = 61.83\ m^2$$

$$A_{m_5} = \frac{A_5 + A_6}{2} = \frac{59.66 + 39.76}{2} = 49.71\ m^2$$

$$A_{m_6} = \frac{A_6 + A_7}{2} = \frac{39.76 + 34.375}{2} = 37.07\ m^2$$

$$V = 20\left[43.40 + 58.60 + 61.83 + 61.83 + 49.71 + 37.07\right]$$

$$= 20 \times 312.44$$

$$\boxed{V = 6248.8\ m^3}$$

(iii) Mean Area Rule :-

$$V = L\left[\frac{A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7}{7}\right]$$

$$= 120\left[\frac{29.26 + 57.535 + 59.66 + 64 + 59.66 + 39.76 + 34.375}{7}\right]$$

$$V = 120 \times \frac{344.25}{7}$$

$$\boxed{V = 5901.43\ m^3}$$

(iv) Trapezoidal Rule :

$$V = \frac{d}{2}\left[(A_1 + A_7) + 2(A_2 + A_3 + A_4 + A_5 + A_6)\right]$$

$$= \frac{20}{2}\left[(29.26 + 34.375) + 2(57.535 + 59.66 + 64 + 59.66 + 39.75)\right]$$

$$= \frac{20}{2} \times \left[63.635 + (2 \times 280.615)\right]$$

$$\boxed{V = 6248.65\ m^3}$$

(v) Prismoidal (or) simpson's rule :-

$$V = \frac{d}{3}\left[\underbrace{(A_1 + A_7)}_{(1^{st}+last)} + 2\underbrace{(A_3 + A_5)}_{odd} + 4\underbrace{(A_2 + A_4 + A_6)}_{even}\right]$$

$$= \frac{20}{3}\left[(29.26 + 34.375) + 2(59.66 + 59.66) + 4(57.535 + 64 + 39.76)\right]$$

$$= \frac{20}{3}\left[63.635 + (2 \times 119.32) + (4 \times 161.295)\right]$$

$$\boxed{V = 6316.37\ m^3}$$

3, The cross section areas of an embankment are as given below. calculate the cubic contents of embankment by trapezoidal & prismoidal method.

Distance (m)	0	50	100	150	200	250	300
Area (m^2)	200	540	810	1420	1520	2320	1920

Solution :-

Trapezoidal Method :-

$$V = \frac{d}{2}\left[(1^{st} + \text{Last area}) + 2(\text{other areas})\right]$$

$$= \frac{50}{2}\left[(200 + 1920) + 2(540 + 810 + 1420 + 1520 + 2320)\right]$$

$$V = \frac{50}{2} \times \left[2120 + (2 \times 6610)\right]$$

$$\boxed{V = 3,83,500\ m^3}$$

Prismoidal method :

$$V = \frac{d}{3}\left[(A_1+A_n) + 2(\text{odd area}) + 4(\text{even area})\right]$$

$$= \frac{50}{3}\left[(A_1+A_7) + 2(A_3+A_5) + 4(A_2+A_4+A_6)\right]$$

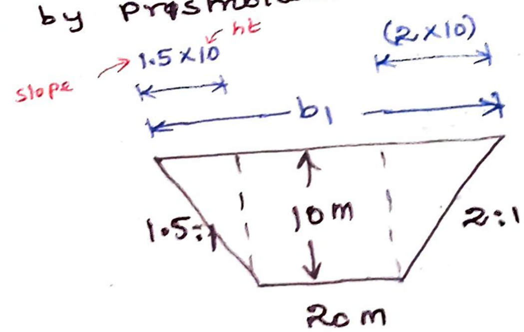
$$= \frac{50}{3}\left[(200+1920) + 2(810+1520) + 4(540+1420+2320)\right]$$

$$= \frac{50}{3}\left[2120 + (2\times 2330) + (4\times 4280)\right]$$

$$\boxed{V = 3,98,333.33\ m^3}$$

4, A cutting of 1000 m length is made in a flat ground with a base width of 20m through out. The depth of cutting is 10m at one end and 15m at another end. The side slopes are 1.5:1 on one side and 2:1 on another side. calculate the volume of earthwork by prismoidal rule.

Solution :-



$L = 1000\ m$

$b = 20\ m$

$S_1 = 1.5$

$S_2 = 2$

c/s at 0m level :-

$$b_1 = b + (S_1 h) + (S_2 h)$$

$$= 20 + (1.5\times 10) + (2\times 10)$$

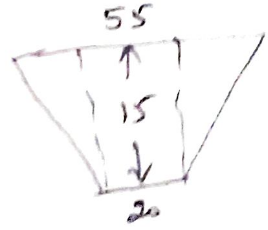
$$b_1 = 55\ m$$

$$A_1 = \left(\frac{b_1+b}{2}\right)h$$

$$A_1 = \left(\frac{20+55}{2}\right)\times 10$$

$$\boxed{A_1 = 375\ m^2}$$

c/s at 1000 m level :-



$$A_2 = \left(\frac{20+55}{2}\right)\times 15$$

$$\boxed{A_2 = 562.50\ m^2}$$

(even Areas)]

UNIT III

Theodolite surveying

Theodolite — vernier & microptic — Description and uses — Temporary & permanent adjustments of vernier transit — Horizontal angles — vertical angles — Heights and distances — Traversing — closing error and distribution — Gale's tables — omitted measurements.

Introduction:

* Theodolite is a precise instrument used for accurate measurement of horizontal & vertical angles.
* It is widely used for various purposes, so it is also called as an universal instrument.

Purpose of theodolite:-

* It is measured horizontal, vertical and deflection angle.
* It is measured in magnetic bearing
* It is measured the horizontal and vertical distance between any two points.
* To find the difference in elevation b/w various points
* It will produce the line ranging.

Types of theodolite:-

* Transit vernier theodolite
* Non-Transit theodolite.

Transit Theodolite:-

* In this type of theodolite is mostly used.
* In this type of theodolite the verniers are provided for reading taken in horizontal and vertical graduated ring.
* The telescope can be rotated as a complete rotation, about its horizontal axis in a vertical plane.

Non-Transit Theodolite (or) optical theodolite:-

* It is one in which the telescope can be rotated only by limited in the vertical plane.

Components of a Transit theodolite.

* Levelling head
* Lower plate
* Upper plate
* Standards
* T-frame
* Plate levels (or) bubble
* Telescope
* Altitude bubble
* plumb bob
* Tubular compass
* Tribrach
* Foot screw
* Tangent screw
* clamp screw
* vertical circle
* Adjusting mirror
* Focusing screw

Base plate (or) Trivet

* It is a circular plate having a central threaded hole for fixing the theodolite on the tripod stand by a wing nut.

Foot screw:-

* The lower part of the foot screw in the trivet by means of the ball and the

~~element~~ and the upper threaded
Part passes through the threaded hole in the tribrach plate.

Tribrach :-

* It is a triangular plate carring ~~carrying~~ three foot screws at its ends.

Levelling head :-

The trivet, foot screws and the tribrach constitute a body which is known as levelling head.

* They are screwed with threads on the levelling head and the upper & lower plates.
* This provision is made for levelling the instrument.

Lower plate :-

* It is attached to the outer axis and is also known as the scale plate.
* The scale is graduated from 0° to 360° in a clockwise direction.

Upper plate :-

* The vernier scales a & b it is attached to the inner axis, ~~its~~
* Its motion is control by the upper clamp screw and the upper tangent screw.
* When the clamp screw is tightened the vernier scales are fixed with the inner axis and for the fine adjustment of the scale, The tangent screw is used.

Plate bubble :-

* Two plate bubbles are mounted at right angles to each other on the upper surface of the vernier plate.

Standard A-frame :-

* Two frames are provided on the upper plate to support the telescope, vertical graduated ring & clamp screw, tangent screw, adjustable mirror and ~~attached~~ the altitude bubble.

Telescope

* It is pivoted b/w the standards at right angles to the horizontal axis.
* It can be rotated about its horizontal axis in a vertical plane.
* A telescope is provided with a focusing screw, clamping screw & tangent screw.

Altitude bubble

* A long sensitive bubble tube is provided on the top of the index bar.
* The bubble it contains is known as the altitude bubble.

Permanent adjustment of theodolite

* Adjustment of the horizontal plate level
* Adjustment of the horizontal axis
* Adjustment of the telescope
* Adjustment of the telescope level
* Adjustment of the vertical circle index

Vertical axis :-

* vertical axis is the axis above which the instrument is rotated in the horizontal plane.

Horizontal axis :-

* The axis about, which the telescope along with the vertical circle of a theodolite may be rotated in a vertical plane is called horizontal axis.
* It is also called tranian axis or transverse axis.

Line of collimation :-

* It is an imaginary line passing through the intersection of the cross hairs at the eye piece and the optical centre of

the objective and its continuation is called line of collimation.

* **Error of collimation**:

  * The angle b/w the line of collimation and the line $\perp^r$ to the horizontal axis is called error of collimation.

Centering:-

The process of setting of a theodolite exactly over the ground station mark by means of a plumbbob is known as centering.

Transiting:-

The process of turning the telescope in vertical plane through 180° above its horizontal axis is known as transiting

Swing:-

The continuous motion of the telescope about the vertical axis in horizontal plane is called swing.

Right swing:-

When the telescope is rotated in the clockwise direction is called right swing.

Face left observation:-

When the vertical circle is on the left of the telescope at the time of observation

The observations of the angles are known as face left observation.

Face right observation:-

When the vertical circle is on the right of the telescope at the time of observation.

The observations of the angles are known as face right observation.

* Setting up the instrument
* Levelling the instrument
* Elimination of the parallax
* Focussing the eye piece
* Focussing the object glass

(i) Setting the theodolite over the station:

* The tripod stand is placed over the required station.
* The theodolite is then lifted from the box and fixed on the top of the stand by means of arrangements provided along with the instruments.

(ii) Levelling by tripod stand:-

* The legs of the tripod stand are placed and fixed on the ground approximately levelling is then using this stand.
* So that the bubble in the stand approximately at the centre of its ~~[illegible]~~ run.

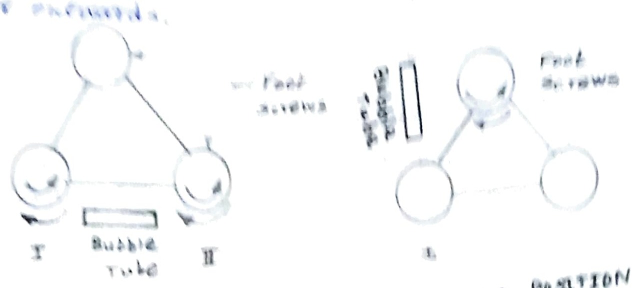
(iii) Centering:

* It is the process of setting the instrument exactly over the station.
* At the time of approximately levelling by means of the tripod stand it should be ensured that the plumb bob suspended from hook under the vertical axis line exactly over the station peg.

(iv) Levelling:

* Before starting the levelling operation, all the foot screws are brought to the centre of the run.

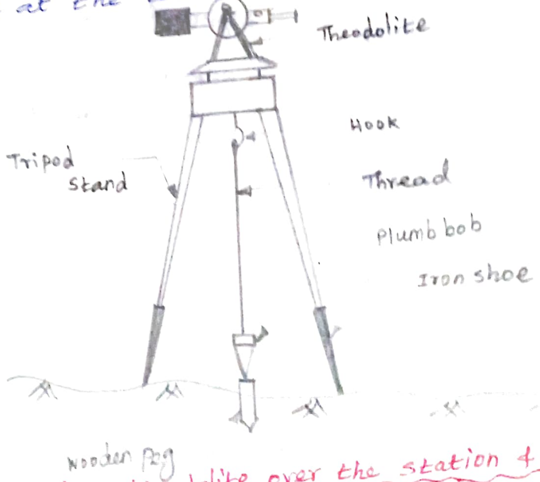
* The plate bubble is placed parallel to any pair of foot screws (1 & 2).
* The bubble is brought to the centre of its run by turning two foot screws equally inwards or outwards.



FIRST POSITION

SECOND POSITION

* Now in the second position the plate bubble is turned through 90° such that its ⊥ to the line joining two foot screws (1 and 2).
* The bubble is brought to centre of its run by turning the third foot screw either clockwise or anticlockwise.
* The procedure is repeated such that the bubble remains in the centre of its run in both the positions.
* Then the plate bubble is rotated 360° about the vertical axis, then also the bubble should be at the centre of its run.



Setting theodolite over the station & centering.

(v) Focussing the eye piece :-

* Focussing is done by adjusting the eye piece for to get a clear view of the cross-hairs
* This is done by directing the telescope to the sky or a piece of white paper is held infront of the object glass and the eye piece is moved in or out such that the cross-hairs appear clear and sharp.

(vi) Focussing the object glass :-

* The telescope is directly towards the object and the focussing screw is turned clockwise or anticlockwise.
* Untill the image appears clearly. and sharpingly.
* This is done to bringing a sharp image of the object (or) target in the plane of cross hair and to elliminate Parallex

(vii) Setting the vernier :-

* Before starting of the work, the vernier 'A' should be set up 0° and B at 180°.
* This is done by fixing the lower clamp and loosening the upper clamp.
* The upper plate is moved untill the vernier 'A' approximately coincides with 0° (ie, 360°) mark and the vernier 'B' at 180°, upper clamp is tightened.
* By turning the upper tangent screw the arrows are exactly ~~Place~~ brought 0° to 180° points.

Permanent Adjustments of Theodolite:-

* Axis of bubble tube should be $\perp^r$ to the vertical axis.
* Line of collimation should be $\perp^r$ to the horizontal axis or tunnion axis.
* Horizontal axis should be $\perp^r$ to the vertical axis.
* Axis of telescope level should be parallel to the line of collimation.
* The vernier should read zero, when the instrument is levelled.

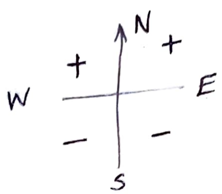
Latitude :

Latitude of the survey line may be defined as its co-ordinate length measured parallel to the meridian are known as latitude.

$$\boxed{\text{Latitude } (L) = l\cos\theta}$$

Note :-

Latitude is always positive in north and negative in south direction.



N → +

S → –

Departure :-

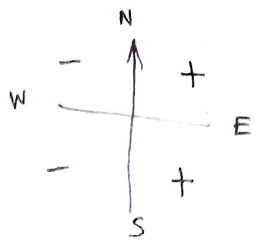
Departure of the survey line may be defined as its co-ordinate length measured $\perp^r$ to the meridian is known as departure.

$$\boxed{\text{Departure } (D) = l\sin\theta}$$

Note :-

It is always + in East direction +

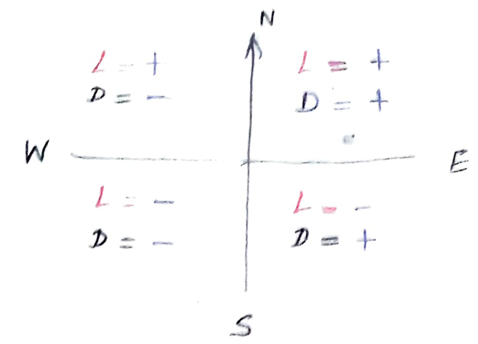
– in West direction



Latitude & departure

$L = l\cos\theta$

$D = l\sin\theta$



1, The following are the length and bearings of various line observer from travels A, B, C, D. Calculate the latitude & departure.

Solution:

Line	length	WCB
AB	262.2	25°20'
BC	312.3	138°40'40"
CD	173.5	209°42'40"
DA	295.6	307°45'30"

Solution :-

Line	Length	WCB	Reduced bearing	Latitude $l\cos\theta$	Departure $l\sin\theta$
AB	262.2	25°20'	N 25°20' E	+236.985	+112.19
BC	312.3	138°40'40"	S 41°19'20" E	−234.53	+206.21
CD	173.5	209°42'40"	S 29°42'40" E	−150.691	−85.99
DA	295.6	307°45'30"	N 52° 14'30" W	+181.01	−233.70

Omitted measurements

The following records are obtained in traverse survey, where the length & bearing of the last line where not recorded. Compute the length ~~along~~ and bearing of line DA

Line	length	Bearing
AB	75.5	30° 24'
BC	180.5	110° 36'
CD	60.25	210° 30'
DA	?	?

Solution :-

Line	length	Bearing	Reduced bearing	Latitude $l\cos\theta$	Departure $l\sin\theta$
AB	75.50	30° 24'	N 30° 24' E	65.12	38.20
BC	180.5	110° 36'	S 69° 24' E	-63.50	168.95
CD	60.25	210° 30'	S 30° 30' W	-51.91	-30.57
DA	~~?~~ l	θ	θ	$l\cos\theta$	$l\sin\theta$

Σ of latitude

$$= 65.12 - 63.51 - 51.91 + l\cos\theta = 0$$

$$l\cos\theta = 50.3 \quad \text{——} \quad (1)$$

Sum of departure

$$38.21 + 168.95 - 30.61 + l\sin\theta = 0$$

$$l\sin\theta = -176.56 \quad \text{——} \quad (2)$$

Simplify these two equations

② $\Rightarrow l \sin\theta = -176.56$

① $\Rightarrow l \cos\theta = 50.3$

$$\frac{②}{①} \Rightarrow \frac{l \sin\theta}{l \cos\theta} = \frac{-176.56}{50.3}$$

$$\tan\theta = -3.5101$$

(the [illegible] west [illegible])

$$\therefore \theta = 74^\circ 5'$$

Sub in θ in ②

$$l \sin 74^\circ 5' = -176.56$$

$$\therefore l = -\frac{176.56}{0.9617}$$

$$l = 183.58$$

			direction
$\therefore$ The latitude get	+ ive		N
Departure get	(−) ive		W

So the angle get North west direction.

2) The following observations were taken from station P & Q.

Line	Length	Bearing	~~Latitude~~	~~Departure~~
PA	120	S $60^\circ 30'$ W		
PQ	200	N $30^\circ 30'$ W		
QB	150.5	N $50^\circ 15'$ W		
AB	l	θ		

Solution:

Line	length	Bearing	Latitude $l\cos\theta$	Departure $l\sin\theta$
PA	120	S 60°30' W	− 59.09	− 104.44
PQ	200	N 30°30' E	+ 172.33	+ 101.51
QB	150.5	N 50°15' W	+ 96.24	− 115.71
AB	l	θ	$l\cos\theta$	$l\sin\theta$

Sum of all the latitude

$$-59.09 + 172.33 + 96.24 + l\cos\theta = 0$$

$$l\cos\theta = -209.48 \quad \text{—— (1)}$$

Latitude is negative ∴ south

Sum of all departure

$$-104.44 + 101.51 - 115.71 + l\sin\theta = 0$$

$$l\sin\theta = 118.64 \quad \text{—— (2)}$$

Departure is +ive ∴ East

$$\frac{(2)}{(1)} = \frac{\cancel{l}\sin\theta}{\cancel{l}\cos\theta} = \frac{118.64}{-209.48}$$

$$\tan\theta = -0.566$$

$$\theta = \tan^{-1}(-0.566)$$

$$\theta = 29^\circ 31' 31'' \quad \text{(Negative)}$$

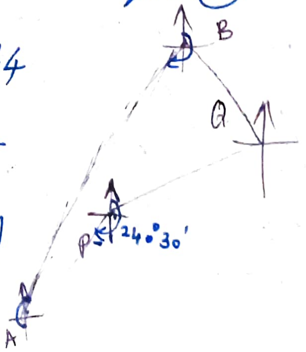
So the bearing of AB = S 29°31'31" E

Substitute the values in eqn. (2)

$$l\sin\theta = 118.64$$

$$l = \frac{118.64}{\sin(29^\circ 31' 31'')}$$

$$\boxed{l = 240.74\ m}$$



$\angle PAB$ = RB of AP − RB of AB

$= 60°30' - 29°31'31''$

$= 30°58'29''$

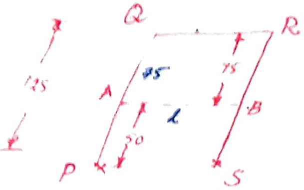
$\angle QBA$ = RB of BQ + RB of AB

$= 50°15' + 29°31'31''$

$= 79°46'31''$

3. The following particulars are given for a traverse survey where the length of the line 'AB' is required to be measured point 'A' is 50 m from P and point 'B' is 75 m from R.

Line	Length	Bearing
PQ	125.50	N 30°15' E
QR	80.25	S 40°30' E
RS	150.75	S 60°30' W



Solution:-

Line	Length	R.B	Latitude $l\cos\theta$	Departure $l\sin\theta$
AQ	75.0	N 30°15' E	+ 64.79	+ 37.78
QR	80.25	S 40°30' E	− 61.02	+ 52.12
RB	75.0	S 60°30' W	− 36.93	− 65.28
BA	l	θ	$l\cos\theta$	$l\sin\theta$

Sum of latitude

$$64.79 - 61.02 - 36.93 + l\cos\theta = 0$$

$$l\cos\theta = 33.16 \quad \text{—— (1)}$$

Sum of departure

$$37.78 + 52.12 - 65.28 + l\sin\theta = 0$$

$$l\sin\theta = -24.62 \quad \text{—— (2)}$$

$$\frac{(2)}{(1)} \Rightarrow \frac{l\sin\theta}{l\cos\theta} = \frac{-24.62}{33.16} = -0.7425$$

$$\theta = 36°35'$$

Substitute in eqn.

$l \sin\theta = -24.62$

$l = \dfrac{-24.62}{\sin 36°35'} = 41.30$ m

∴ The $l\cos\theta$ is positive ie, North

$l\sin\theta$ is negative ie, West

So, it is North west direction.

7. A recorded value of the close traverse is given below, with two distance missing. Calculate the length of BC & EA

Line	length	bearing
AB	100.5	N 30° 30′ E
BC	?	S 45° 0′ E
CD	75.0	S 40° 30′ W
DE	50.5	S 60° 0′ W
EA	?	N 40° 15′ W

Solution:-

Line	length	Bearing	R.B	Latitude $l\cos\theta$	Departure $l\sin\theta$
AB	100.5	N 30°30′E	N30°30′E	+ 86.59	+ 51.01
BC	l_1	S45°0′E	S45°0′E	$-0.707 l_1$	$+0.707 l_1$
CD	75.0	S40°30′W	S40°30′W	− 57.03	− 48.71
DE	50.5	S 60°0′W	S 60°0′W	− 25.25	− 43.73
EA	l_2	N40°15′W	N 40°15′W	$+0.763 l_2$	$-0.646 l_2$

Σ of latitude = 0

$$86.59 - 0.707 l_1 - 57.03 - 25.25 + 0.763 l_2 = 0$$

$$-0.707 l_1 + 0.763 l_2 = -4.31 \quad \text{(1)}$$

~~iiiy~~

Σ of departure = 0

$$51.01 + 0.707 l_1 - 48.71 - 43.73 - 0.646 l_2 = 0$$

$0.707\,l_1 - 0.646\,l_2 = 41.43$ ---------- (2)

(1) $\Rightarrow -0.707\,l_1 + 0.763\,l_2 = -4.31$

(2) $\Rightarrow 0.707\,l_1 - 0.646\,l_2 = 41.43$

(1)+(2) $\Rightarrow \quad +0.117\,l_2 = 37.12$

$$\therefore l_2 = \frac{37.12}{0.117}$$

$$\boxed{l_2 = 317.26\ m}$$

substitute l_2 values in eqn (2)

$$0.707\,l_1 - 0.646 \times 317.26 = 41.43$$

$$0.707\,l_1 = 41.43 + 204.95$$

$$l_1 = \frac{246.38}{0.707}$$

$$\boxed{l_1 = 348.49\ m}$$

BC

$l_1 \cos\theta = 348.49 \times \cancel{\cos}\ 0.707$

$= -246.38\ m$

$l_1 \sin\theta = +246.38\ m$

EA

$l_2 \cos\theta = +317.26 \times (0.763) = +242.07\ m$

$l_2 \sin\theta = -317.26 \times 0.646 = -204.95\ m.$

5, An incomplete traverse table is obtained as follows.

Line	Length	Bearing
AB	100	?
BC	80.5	140°30'
CD	60.0	220° 30'
DA	?	310°15'

II

1. What is centering of theodolite?

* It is the process of setting the instrument exactly over the station
* At the time of approximately levelling by means of the tripod stand it should be ensured that the plumb-bob suspended from hook under the vertical axis line exactly over the station peg.

2. What is Face right observation?

When the vertical circle of a theodolite is on the right of the observer at the time of observation is called Face right observation.

Nov/Dec 2009, Reg. 2004

3. What is meant by Transit?

* Theodolite is precise instrument used for accurate measurement of horizontal and vertical angles.
* The telescope is revolved through a complete revolution about its horizontal axis in a vertical plane.
* Transisting is the process of turning of the telescope about the turion axis in vertical plane through 180°.

4. Mention the methods to be used in measuring horizontal angles using theodolite.

* Horizontal angles using theodolite are measured by
  (i) Repetition method
  (ii) Reiteration Method.

May/June 2009, Reg. 2004

5. What is the use of a tangential screw provided for adjustments in a transit theodolite?

The lower plate is provided with a clamp screw and a tangential screw which control its movements.

When the clamp screw is tightened, the plate is fixed with the outer axis.

For fine adjustment of the lower plate, the tangent screw is rotated to the extent required.

Define closing error. How does it occur?

A carefully conducted compass survey of a closed traverse will finish at the starting point. This is a case of no closing error. But

Anna University 16 marks ...

Unit - II

Chennai Nov/Dec - 2009 Reg: 2008

1. Describe the essential parts of a transit theodolite. (16 marks)

2. Discuss the temporary adjustments of a transit theodolite (16 marks)

Chennai Nov/Dec-2009 Reg: 2004

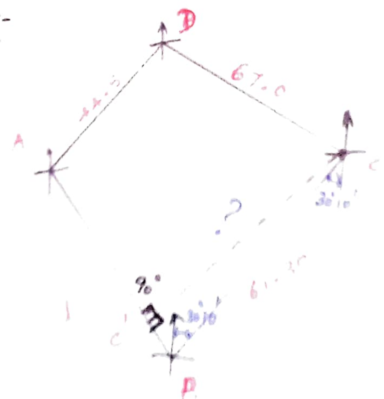
3. A traverse ABCD was to be run but due to an obstruction b/w the stations A & B, it was not possible to measure the length and direction of the line AB. The following data could only be obtained.

16 marks

Line	length (m)	R.B
AD	44.5	N 50°20′ E
DC	67.0	S 69°45′ E
CB	61.3	S 30°10′ W

Determine the length and the direction of BA. Also determine the $\perp^r$ distance of C from AB.

Solution :-



Line	length (m)	RB	Latitude $L\cos\theta$	Departure $L\sin\theta$
AD	44.5	N 50°20′ E	+ 28.4[illegible]	+ 34.[illegible]
DC	67.0	S 69°45′ E	− 23.19	+ [illegible]
CB	61.3	S 30°10′ W	− 53.[illegible]	− [illegible]
AB	l	θ		

[illegible]

[illegible]

[illegible]

$\left[l \cos\theta = 47.79 \text{ m} \right]$ — (1)

∴ [illegible] departure [illegible]

[illegible] $l \sin\theta = 0$

$60.31 + l \sin\theta = 0$

$\left[l \sin\theta = -60.31 \right]$ — (2)

since the latitude of AB as positive and the departure of AB as negative. Hence it is N-W ~~direction~~ quadrant.

$\frac{(2)}{(1)} \Rightarrow \frac{\cancel{l}\sin\theta}{\cancel{l}\cos\theta} = \frac{-60.31}{47.79}$

$\tan\theta = 1.262$

$\theta = \tan^{-1}(1.262)$

$\theta = 51^\circ 36'$

Reduced bearing $\boxed{\theta = N\,51^\circ 36'\,W}$

Bearing of AB $= 360^\circ - 51^\circ 36' = 308^\circ 24'$

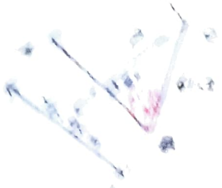
Length of AB $= \sqrt{\Sigma L^2 + \Sigma D^2}$

$= \sqrt{(47.79)^2 + (-60.31)^2}$

$\boxed{\text{Length of AB} = 76.95 \text{ m}}$

In Δ^{le} CBC',

$\angle C'BC = 30^\circ 10' + 51^\circ 36' = 81^\circ 46'$



using sine rule

$$\frac{CC'}{\sin 81°48'} = \frac{61.80}{\sin 90}$$

$$CC' = \frac{61.80 \times \sin 81°48'}{\sin 90}$$

$$CC' = 60.67\,m$$

~~$AC'^2 = AB^2 + BC^2$~~

4) Explain the Method of repetition and reiteration methods. (8 marks)

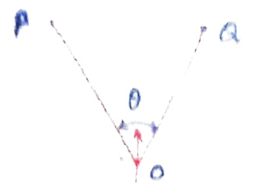
Repetition Method :

* It is very accurate method and also it is measure horizontal angle.
* The Angle is added continuously & the the total angle is divided into the number of repetition
* It is consists of measuring the horizontal angle clockwise by any number of times.
* The angle should be measured clock wise and face right and face left ~~[illegible]~~ position, With three repetition at each face (totally six repetitations)

Procedure :-

* The instrument is set over the station 'O'.
* The temporary adjustments are made.
* The telescope should be in the normal position and the vertical circle at the right hand side.
* The vernier A is set to zero (or) 360°. With the help of upper clamp and tangent screw.
* choosen the lower clamp and direct the

telescope towards the object P. (right ...)



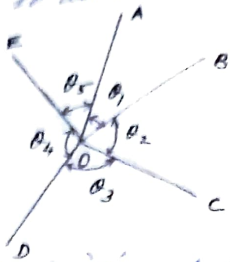
* Tight the lower clamp & bisect P accurately by lower tangent screw, Read both verniers.
* unclamp the upper plate and swing the telescope clockwise. Bisect station 'Q' by upper clamp and tangent screw.
* Read both verniers. Take the average to get angle POQ
* Unclamp the lower plate & swing the telescope clockwise and bisect station 'P' accurately by using the lower clamp & lower tangent screw
* Read both the verniers. check the vernier reading. It should be the same as that obtained in step (7).
* Release the upper plate by using upper clamp and tangent screw and bisect station 'Q' accurately (telescope is turned clockwise). The vernier will read twice the angle 'POQ'
* Repeat the process for required number of times, say three times, and find out the value of angle 'POQ'.
* Repeat the above procedure with the face changed and calculate the angle 'POQ'.
* The average of the two values of angle 'POQ' thus obtained with face left and face right gives an accurate value of horizontal angle.

Reiteration Method:-

* This method is suitable, when several angles are measured from a single station.
* In this method all the horizontal angles are measured succesively and finally the horizon is closed. ie, the angle b/w the last and first station is measured.

* The final reading of the leading vernier should be the same as its initial reading.
* If it does not coincide, the difference is equally distributed to all measured angle.

Procedure:



* To measure angle AOB, BOC, COD, DOE and EOA.
* Set up the instrument at 'O' and level it.
* Set the vernier 'A' to read zero using the upper clamp & tangent screw.
* ~~Direct~~ Then the telescope was turned to 'B' by loosening the lower clamp.
* Similarly, biset the stations C, D, E & finally 'A' and read both verniers in all cases, in a clockwise direction.
* Then the same procedure was repeated with the other face in anti-clockwise direction.

Instrument station

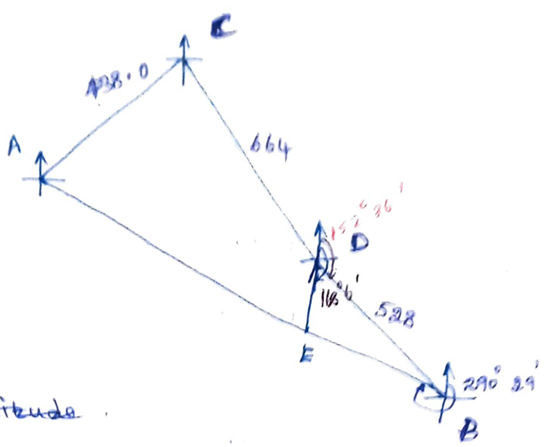
5) List out the permanent adjustments of a theodolite. And explain any two of them.

6, Two stations A & B are fixed on either side a wood. The following traverse is run from A to B along the side of the wood.

Anni ... June 2009 / Reg: 2006

Line	Length (m)	Bearing	Latitude (m)	Departure (m)
A[illegible]	438	48°24'	+ 290.8	+ 327.5
[illegible]D	664	110°12'	229.3	+ 623.2
DB	528	152°36'	516.6	+ 267.8

[illegible]etermine the length of AB from the station D a line DE is carried into the woods on a bearing of 168°6' in order to fix an intermediate point E on AB. Find the length of DE



~~Sum of latitude.~~

$\cancel{= 290.8 - 229.3 - 516.6}$

$\cancel{= -455.10\ m}$

~~Sum of departure~~

$\cancel{= 327.5 + 623.2 + 267.8}$

$\cancel{= 1218.5\ m}$.

Line	Length	Bearing	Latitude	Departure
AC	438	48°24'	+290.8	+327.5
CD	664	110°12'	−229.3	+623.2
DB	528	152°36'	−516.6	+267.8
AB	l	θ	$l\cos\theta$	$l\sin\theta$

Sum of latitude $= 0$

$$290.8 - 229.3 - 516.6 + l\cos\theta = 0$$

$$l\cos\theta = 455.10 \quad \text{——— } (1)$$

Sum of departures = 0

$327.8 + 233.2 + 217.8 + l\sin\theta = 0$ —— (2)

$l\sin\theta = 1218.5$

$\frac{(2)}{(1)} \Rightarrow \frac{l\sin\theta}{l\cos\theta} = \frac{-1218.5}{455.1}$

$\tan\theta = 2.6774$

$\theta = \tan^{-1}(2.6774)$

$\theta = 69°31'$

The Latitude $l\cos\theta = 455.1$ $(+^{ve})$ ∴ the direction is North

Departure $l\sin\theta = -1218.5$ $(-^{ve})$ ie, West

∴ $\boxed{\text{The bearing of AB} = N\,69°31'\,W}$

Length of AB

$l\cos\theta = 455.1$ —— (1)

$l = \frac{455.10}{\cos 69°31'}$

$= 1300.53$ m

Length of AB

$= \sqrt{\Sigma L^2 + \Sigma D^2}$

$= \sqrt{455.1^2 + (1218.5)^2}$

$= 1300.71$ m

$\theta = N\,69°31'\,W$

$\angle EDB = 168°6' - 152°16'$

$= 15°50'$

$DB = 528$ m

$\angle DBE = (155°16' + 180°) - 270°29'$

$= 41°47'$

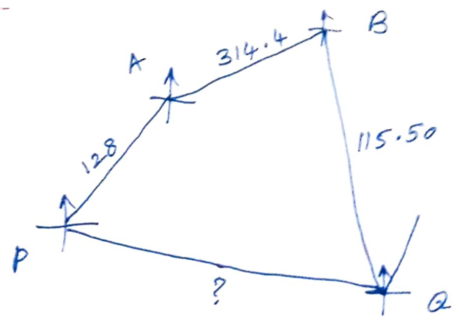
DE = 419.33 m

The bearing of a line PQ was impossible to be measured directly. Hence the following observation were made from two stations A & B

Line	Length	Bearing
AP	128.0 m	S 65°36' W
AB	314.40	N 24°12' E
BQ	115.50	N 70°48' W

compute the length and bearing of PQ & the angles APQ and BQP.

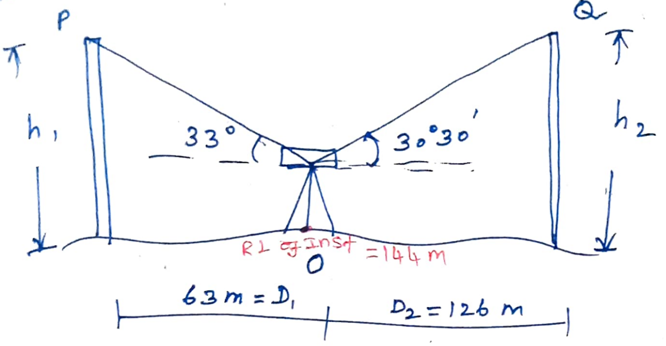
Solution:-



Line	Length	R.B	Latitude $l\cos\theta$	Departure $l\sin\theta$
AP	128.0	S65°36'W	− 52.877	− 116.678
AB	314.4	N24°12'E	+ 286.771	+ 128.88
BQ	115.50	N70°48'W	+ 37.984	− 109.075
PQ	?	θ	$l\cos\theta$	$l\sin\theta$

A theodolite is set up b/w two towers P & Q. The theodolite station was 63 m from P & 126 m from Q. observations were taken to P & Q, and the angle of elevations were 33°0' & 30°30' respectively. The Reduced level of the instrument axis was 144 m. calculate the RL of P & Q.

Solution.



$h_1 = D_1 \tan \alpha_1$

$= 63 \times \tan 33°$

$\underline{h_1 = 40.91 \text{ m}}$

$h_2 = D_2 \tan \alpha_2$

$= 126 \times \tan 30°30'$

$\underline{h_2 = 74.22 \text{ m}}$

RL of P = RL of Instrument + h_1

$= 144 + 40.91$

$= \underline{184.91 \text{ m}}.$

RL of Q = RL of Instrument + h_2

$= 144 + 74.22$

$= \underline{218.22 \text{ m}}$

The following are the latitudes and departures of the lines of a closed traverse ABCD.

Line	Latitude	Departure
AB	−116.10	−44.4
BC	+6.8	+58.2
CD	+80.5	+17.2
DA	+28.8	−31.0

Compute the area of the traverse by the departures and total latitudes method.

Solution

The latitudes of the stations are calculated with reference to station A.

Total Latitude of B = −116.10 m

Total latitude of C = −116.10 + 6.8
= −109.3 m

Total Latitude of D = −116.1 + 6.8 + 80.5
= −28.80 m

Total latitude of A = −116.1 + 6.8 + 80.5 + 28.8
= 0

Algebric sum of Departure

~~total departure~~ at, B = AB + BC
= −44.4 + 58.2
= 13.8 m

Algebric sum of departure at C,
= BC + CD
= +58.2 + 17.2
= 75.4 m

Algebric sum of departure at D,
= CD + DA = 17.2 − 31.0
= −13.8 m

Algebraic sum of departures at A

= DA + AB

= -31.0 + (-44.4)

= -75.4 m

The results are tabulated

Side	Latitude	Departure	Station	Total Latitude	Algebraic sum of adjoining departure	Double Area (col 5 × col 6) + (7)	− (8)
1	2	3	4	5	6	7	8
AB	−116.1	−44.4	B	−116.1	+13.8		1602.18
BC	+6.8	+58.2	C	−109.3	+75.4		8241.22
CD	+80.5	+17.2	D	−28.8	−13.8	397.44	
DA	+28.8	−31.0	A	0	−75.4		0
					Total	397.44	9843.40

Twice ~~the~~ area = Algebraic sum of column (7) & (8)

$= 9843.40 - 397.44$

$= 9445.96\ m^2$

Required area $= \dfrac{9445.96}{2}$

$= \underline{4722.98}\ m^2$

The following table gives the corrected latitudes & departure of the sides of a closed traverse ABCD. calculate the area enclosed by latitude & departure method.

	Latitude		Departure		
Side	N	S	E	W	
AB	108		4		
BC	15		249		
CD		−123	4		
DA	0	~~[illegible]~~		−257	

Solution

The Latitudes of the stations are calculated with reference to 'A'

Total latitude of $B = +108$ m

Total Latitude of $C = (+108 + 15) = 123$ m

Total Latitude of $D = 108 + 15 - 123$

$= 0$

Total Latitude of $A = 108 + 15 - 123 + 0 = 0$

Algebraic Sum of departure at

$B = AB + BC = +4 + 249 = 253$ m

$C = BC + CD = +249 + 4 = 253$ m

$D = CD + DA = +4 - 257 = -253$ m

$A = DA + AB = -257 + 4 = -253$ m

The results are tabulated.

Side	Latitude	Departure	Station	Total Latitude	Algebraic Sum of adjoining departure	Double Area (5×6) +ive	Double Area (5×6) −ive
1	2	3	4	5	6	7	8
AB	108	4	B	+108	+253	27324	
BC	15	249	C	+123	+253	31119	
CD	−123	4	D	0	−253		0
DA	0	−257	A	0	−253		0
				Total		58443	0

~~Algebraic Sum = +58443~~

Twice area = Algebric Sum $= 58443$ m^2

$\therefore$ Required Area $= \dfrac{+58443}{2}$

$= 29221.5$ m^2

In order to set out a line PT the following traverse is run. PT is of 1890 m length and at right angles to a given line PQ

Line	Length (m)	Bearing	Latitude (m)	Departure (m)
PQ		360°	—	—
PR	840.0	119°30′	−413.60	+731.1
RS	1020.0	85°20′	+81.81	+1016.0

Calculate the required length and bearing of ST.

As PT is to be set out at right angles to PQ and

Bearing of QP = 360°

Bearing of PT = 90°

TP = 270°

∴ Latitude of TP = $l \cos\theta = 0$

Departure of TP = −1890 m.

PRST is a closed traverse,

$\Sigma L = 0$

$-413.60 + 81.81 + L + 0 = 0$

Latitude of ST = 331.79 m.

$\Sigma D = 0$

$731.10 + 1016.0 + D - 1890.0 = 0$

Departure of ST = +142.9 m.

Bearing of ST $\tan\theta = \dfrac{142.9}{331.79}$

$\theta = N\,23°19' E$

( ) Discuss the various errors in theodolite surveying & precautions in use of theodolite
(or)
What are the possible sources of error while using theodolite

Ans: The sources of errors in the theodolite observations may be classified as

(i) Instrumental error.
(ii) Personal (or) observational error
(iii) Natural error

Instrumental error:-

* Error due to non-adjustment of plate levels.
* Errors of graduation
* Error due to eccentricity of verniers.
* Error due to eccentricity of centres (the inner and outer axis)
* Error due to non-parallelism of the axis of the telescope level and the line of collimation.
* Error due to horizontal axis not being $\perp^r$ to vertical axis.
* Error due to the line of collimation not $\perp^r$ to the horizontal axis.

Personal Error:-

* The personal errors include
  * errors of manipulation
  * errors in sighting and reading.

Natural error:-

These are caused by

* Wind producing vibration of the instrument
* high temperature

* sunshining on the instrument
* unequal expansion
* Unequal settlement of the tripod

personal Error :-

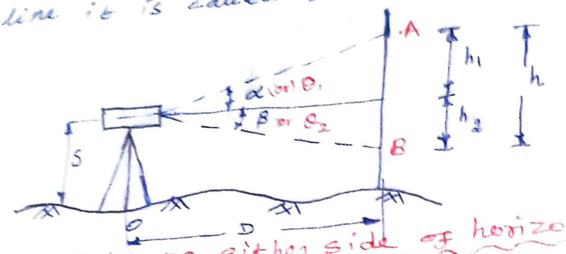
* centering not done properly
* Levelling may not be performed correctly
* clamp screws are not properly tightened
* Improper use of tangent screw
* Focussing may be not done properly
* object not bisected correctly
* Verniers are set correctly.
* Improper reading of verniers.

Measurement of vertical angles

* An angle b/w the line of sight and the horizontal line is called vertical angle.

* When it is above the horizontal line it is known as the angle of elevation.

* When the angle is below the horizontal line it is called the angle of depression.



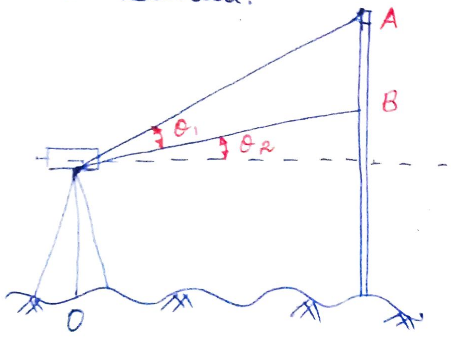
Points on either side of horizontal

* The theodolite is set up at 'O' it is centred and levelled properly.

* The Verniers are set as 0° to 0°

* After loosening the clamp, the telescope is used to measure the angle of elevation by raising slowly.

* Then the reading on both the verniers are noted and the angle of elevation is recorded.

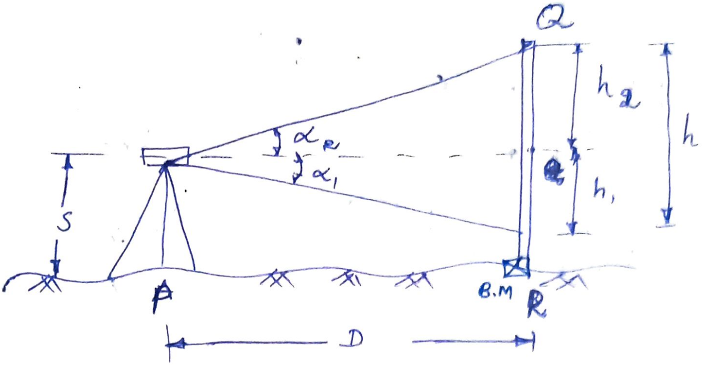


Points above Horizontal

Vertical angle measurement from station O

Methods of Trigonometrical Levelling
(or)
Measurement of Height & distances

case:1 : Base of the object is Accessible



$$\tan\alpha_2 = \frac{h_2}{D}$$

$$\therefore \boxed{h_2 = D\tan\alpha_2}$$

III^{ly} $$\boxed{h_1 = D\tan\alpha_1}$$

$$h = h_1 + h_2$$

$$\boxed{\text{R.L of } Q = \text{RL of BM} + S + h_2}$$

where

P = instrument station

h = vertical distance

h_1 = vertical distance b/w

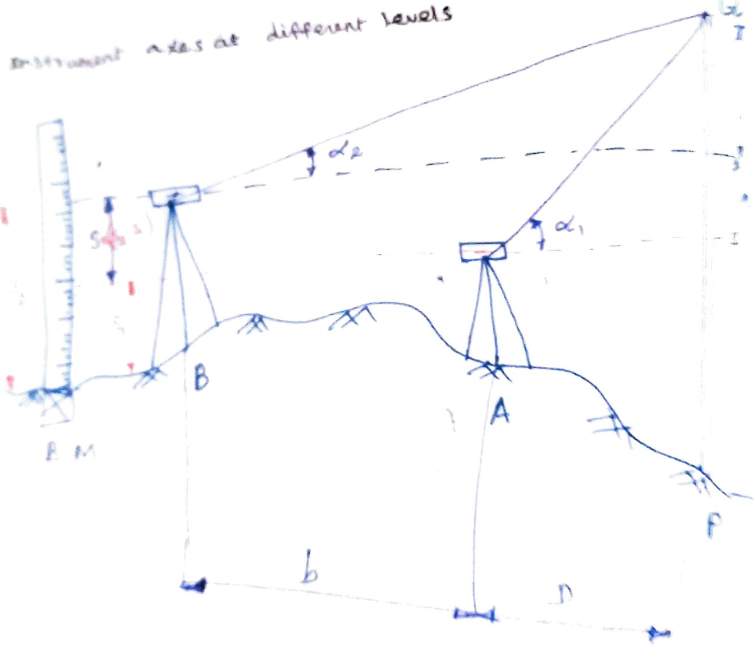
S = staff reading

α = angle of elevation

D = horizontal distance b/w instrument station & object

Case 2 Base of the object inaccessible and the two instrument stations are not in same vertical plane

Instrument axes at different levels



$S = s_2 - S_1$

$h = h_1 - h_2$

$$h_1 = D \tan\alpha_1$$

$$h_2 = (b+D) \tan\alpha_2$$

$$h = S$$

$$S = h_1 - h_2$$

$$S = D\tan\alpha_1 - (b+D)\tan\alpha_2$$

$$= D\tan\alpha_1 - b\tan\alpha_2 - D\tan\alpha_2$$

$$= D\tan\alpha_1 - D\tan\alpha_2 - b\tan\alpha_2$$

$$S = D(\tan\alpha_1 - \tan\alpha_2) - b\tan\alpha_2$$

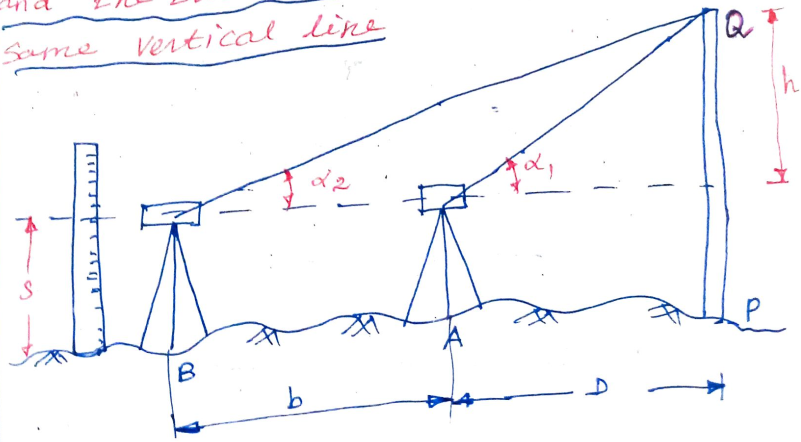
$$S + b\tan\alpha_2 = D(\tan\alpha_1 - \tan\alpha_2)$$

$$\therefore \boxed{D = \frac{S + b\tan\alpha_2}{(\tan\alpha_1 - \tan\alpha_2)}}$$

$$\boxed{\text{RL of } Q = B.M + S_1 + h_1}$$

Instrument axes at the same level

Case : III - Base of the object inaccessible and the two instrument station in the same vertical line



$\tan\alpha_1 = \frac{h}{D}$

$\boxed{h = D\tan\alpha_1}$ —— ①

$\tan\alpha_2 = \frac{h}{b+D}$

$\therefore \boxed{h = (b+D)\tan\alpha_2}$ —— ②

Equating ~~solving~~ the equation ① & ②

$D\tan\alpha_1 = (b+D)\tan\alpha_2$

$D\tan\alpha_1 = b\tan\alpha_2 + D\tan\alpha_2$

$D\tan\alpha_1 - D\tan\alpha_2 = b\tan\alpha_2$

$D(\tan\alpha_1 - \tan\alpha_2) = b\tan\alpha_2$

$$\boxed{\therefore D = \frac{b\tan\alpha_2}{\tan\alpha_1 - \tan\alpha_2}}$$

RL of Q = RL of BM + S + h

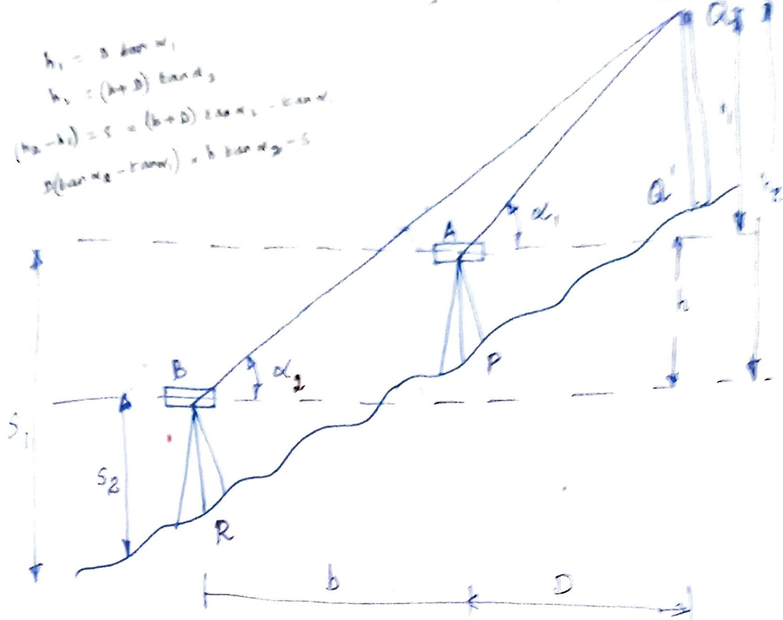
① $\Rightarrow h = D\tan\alpha_1$

$$\boxed{h = \frac{b\tan\alpha_2 \cdot \tan\alpha_1}{\tan\alpha_1 - \tan\alpha_2}}$$

Instrument axes at very different levels

Case : IV - To determine the elevation of point in a steep slope whose base is inaccessible.

The two instrument stations are in the same line. But the difference in elevation b/w the two instrument station cannot be determine by taking staff reading



$h_1 = D \tan\alpha_1$

$h_2 = (b+D)\tan\alpha_2$

$(h_2 - h_1) = S = (b+D)\tan\alpha_2 - \tan\alpha_1$

$D(\tan\alpha_2 - \tan\alpha_1) = b\tan\alpha_2 - S$

RL of Q = RL of R + S_2 + h_1

RL of Q = RL of P + S_1 + h_1

$$\therefore D = \frac{b\tan\alpha_2 - S}{(\tan\alpha_1 - \tan\alpha_2)}.$$

Problem:-

1) A transit theodolite was set up at a distance of 168m from a temple. The angle of elevation in its top was $10^\circ 2'$ and the angle of depression to a foot of the wall was $3^\circ 12'$. The R.L of instrument axis is 158.535 m. Find the height of the temple and RL of the top.

Solution:-

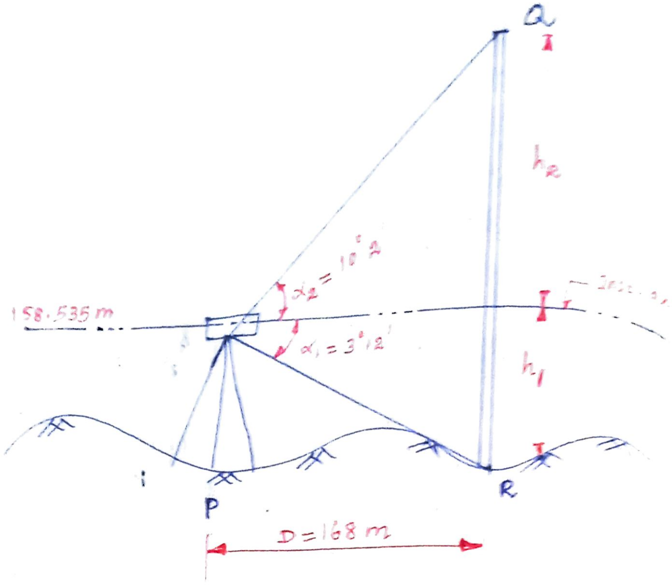
Given Data:-

$D = 168m$

$\alpha_2 = 10^\circ 2'$

$\alpha_1 = 3^\circ 12'$

Ht. of. Inst. axis = 158.535



$h_1 = D \tan \alpha_1$

$= 168 \times \tan 3°12'$

$\boxed{h_1 = 9.39 \text{ m}}$

$h_2 = D \tan \alpha_2$

$= 168 \times \tan 10°2'$

$\boxed{h_2 = 29.72 \text{ m}}$

$h = h_1 + h_2$

$h = 9.39 + 29.72$

$\boxed{h = 39.11 \text{ m}}$

R.L of the top (Q) = Ht of Inst. axis + h_2

= 158.535 + 29.72

= 188.26 m.

2, In order to find the elevation of a tower 'P' a theodolite was set up at 'A' and the angle of elevation of 'P' was found to be 28°45

With the telescope in horizontal, the staff reading over a bench mark of R.L is 75.255 m was 1.285 m. The instrument was set up at another station 'B' 75 m from 'A' away from the tower 'P', such that A, B, P are in same vertical plane. The angle of elevation of 'P' is 21° 30' with the telescope in horizontal, the staff reading over the same bench mark was found to be 1.285 m. Find the R.L of 'P'.

Given Data:-

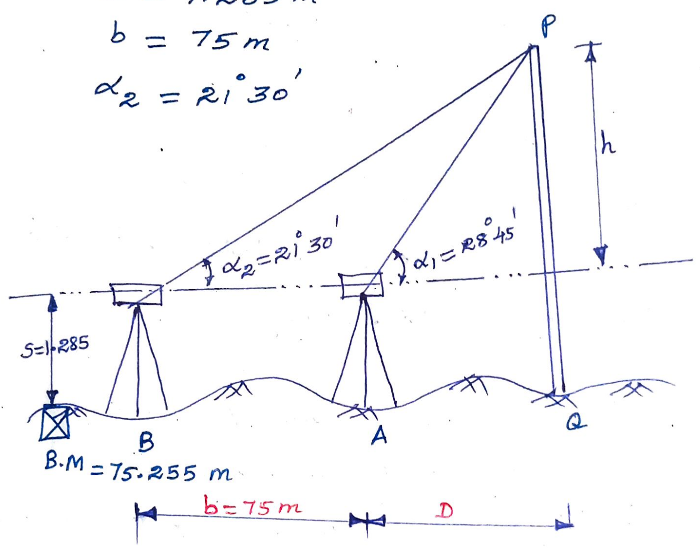
$\alpha_1 = 28° 45'$

$BM = 75.255\,m$

$S = 1.285\,m$

$b = 75\,m$

$\alpha_2 = 21° 30'$



$h = D \tan \alpha_1$

$h = D \tan 28° 45'$ ——— (1)

$h = (D + b) \tan \alpha_2$

$h = (D + 75) \tan 21° 30'$ ——— (2)

Equating the equation ① & ②

$$D \tan 28^\circ 45' = (D + 75) \tan 21^\circ 30'$$

$$D \tan 28^\circ 45' = D \tan 21^\circ 30' + 75 \times \tan 21^\circ 30'$$

$$D \tan 28^\circ 45' - D \tan 21^\circ 30' = 75 \times \tan 21^\circ 30'$$

$$D (\tan 28^\circ 45' - \tan 21^\circ 30') = 75 \times \tan 21^\circ 30'$$

$$\therefore D = \frac{75 \times \tan 21^\circ 30'}{(\tan 28^\circ 45' - \tan 21^\circ 30')}$$

$$\boxed{D = 190.96 \text{ m}}$$

$$h = D \tan \alpha_1$$
$$= 190.96 \times \tan 28^\circ 45'$$

$$\boxed{h = 104.76 \text{ m.}}$$

R.L. of P = RL of B.M + S + h

$$= 75.255 + 1.285 + 104.76$$

$$\boxed{\text{RL of P} = 181.30 \text{ m}}$$

3. Inorder to thus certain the top 'Q' of signa in the observations are made from two instr station 'p' & 'R' at a horizontal distance of 100 m apart. The angle of elevation of 'Q' at 'p' and 'R' are $28^\circ 42'$ and $18^\circ 6'$ respectively. The staff reading upon the B.M are elevation 287.280m, where 2.870 m and 3.750m. When the instrument was on 'p' & 'R' respectively. When the telescope is horizontal.

Determine the elevation at the foot of the signal. If the height of the signal above its base is 3m

Given Data:

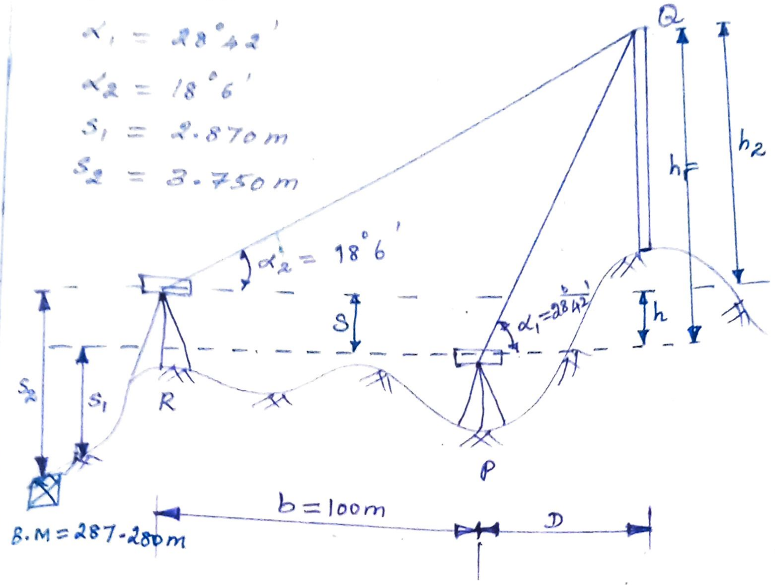
$b = 100\,m$

$\alpha_1 = 28^\circ 42'$

$\alpha_2 = 18^\circ 6'$

$s_1 = 2.870\,m$

$s_2 = 3.750\,m$



$$S = S_2 - S_1 = 3.750 - 2.870 = 0.880\,m$$

$$h = h_1 - h_2$$

$$S = h$$

$$h_1 = D \tan\alpha_1 = D \tan 28^\circ 42' \quad \text{——— (1)}$$

$$h_2 = (b + D)\tan\alpha_2 = (100 + D)\tan 18^\circ 6' \quad \text{——— (2)}$$

$$S = h$$

ie, $S = h_1 - h_2$

$$S = D\tan 28^\circ 42' - (100 + D)\tan 18^\circ 6'$$

$$S = D\tan 28^\circ 42' - D\tan 18^\circ 6' - 100\tan 18^\circ 6'$$

$$S = D(\tan 28^\circ 42' - \tan 18^\circ 6') - 100\tan 18^\circ 6'$$

$$\frac{S + 100\tan 18^\circ 6'}{(\tan 28^\circ 42' - \tan 18^\circ 6')} = D$$

$$D = \frac{0.88 + (100 \times \tan 18°6')}{\tan 28°42' - \tan 18°6'}$$

$$\boxed{D = 152.130\,m}$$

$h_1 = D \tan \alpha_1$

$= 152.13 \times \tan 28°42'$

$$\boxed{h_1 = 83.288\,m.}$$

$h_2 = (b + D) \tan \alpha_2$

$= (100 + 152.13) \times \tan 18°6'$

$$\boxed{h_2 = 82.408\,m}$$

RL of Q = B.M + S_1 + h_1

$= 287.280 + 2.870 + 83.288$

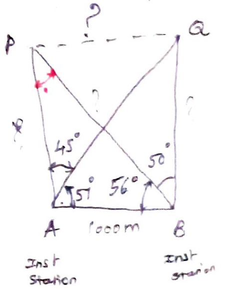
$= 373.438$ m.

RL of Q' = RL of Q − ht of signal above its base

$= 373.438 - 3.00$

$= 370.438$ m.

To find out the distance b/w two inaccessible points P & Q, the theodolite is set up at two stations A & B, 1000 m apart and the following angles were observed PAQ = 45°, QAB = 57°, PBA = 56° & PBQ = 50°, calculate the distance PQ.



Solution :-

In Δ^{le} APB, =

[illegible]

Using sine Rule

$\frac{PR}{\sin [illegible]} = \frac{AR}{\sin [illegible]}$

$PR = \frac{1000 \times \sin [illegible]}{\sin [illegible]}$

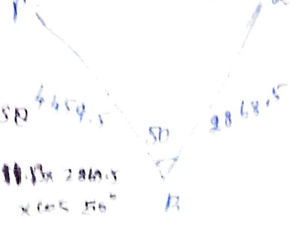
$\boxed{PR = [illegible]\ m}$

$\frac{QR}{\sin 59} = \frac{AR}{\sin 11}$

$RQ = \frac{1000 \times \sin 51}{\sin 11} = 2868.5\ m$

$\boxed{RQ = 2868.5\ m}$

PQ



$PQ^2 = PR^2 + QR^2 - 2\ PR\ QR \cos\theta$

$= [illegible]^2 + 2868.5^2 - 2 \times [illegible] \times 2868.5 \times \cos 50^\circ$

$\boxed{PQ = [illegible]\ m}$

Tacheometric systems
Tangential, stadia and subtense methods
Stadia systems
Horizontal and inclined sights
Vertical and normal staffing
Fixed and movable hairs
Stadia constants
Anallactic lens
Subtense bar

Introduction:

* Tacheometry is a brach of surveying in which both horizontal and vertical distances are measured without the use of a chain or tape.
* Tacheometry is also known as tachymetry or telemetry

Uses of Tacheometry:

* It is mostly used for contouring, in which horizontal distances and elevations are to be determined to give a complete relief map of the ground.
* It is also used for checking measurements taken by chain or tape.

Suitable

* Tacheometry is suitable in rough terrains where chaining is difficult or impossible

* Interms of precision it is not very suitable.

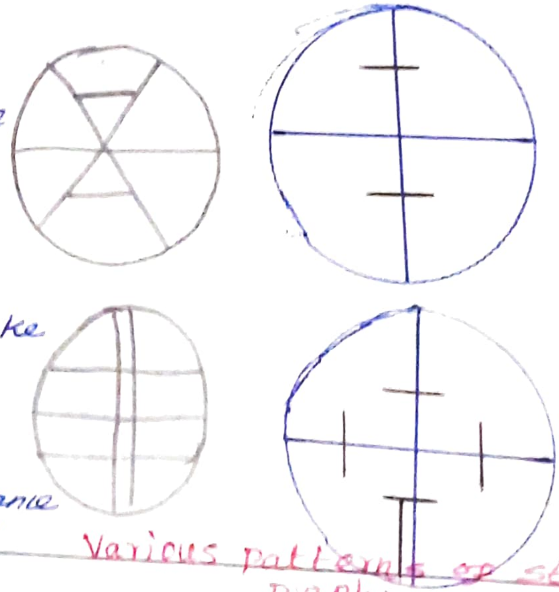
* An accuracy of 1 in 1000 can be achieved with careful handling and reading of instruments, the normal range being 1 in 600 to 1 in 850.

Essential characteristics :-

* The value of the multiplying constant should be 100.
* The value of the additive constant should be zero.
* The telescope should be fitted with an anallactic lens.
* The magnification of the telescope should be 20 to 30 diameters.
* Magnifying power of the eyepiece is kept high.
* For small distance (upto 100 meter) ordinary levelling staff may be used.
* For greater distance a stadia rod may be used.
* stadia rod is usually one piece having 3 to 5m length.
* For smaller distances, a stadia rod graduated in 5mm (ie, 0.005m) may be used.
* For longer distances the rod may be graduated in 10 mm (ie, 0.01m)

Stadia Diaphragms:-

* stadia diaphragm of a theodolite it has three horizontal cross hairs. (top, middle & bottom)
* For findout the vertical distance to take middle hair readings
* Top & bottom hairs are used to find the horizontal distance
* For tangential method, is used only middle hair reading



Various patterns of stadia Diaphragms

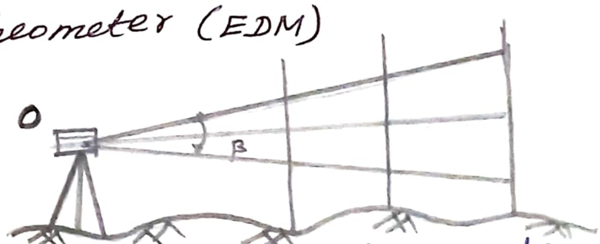
Tacheometric systems (or) Methods :-

1 ... Hair method (or) stadia method

2 Movable Hair method (or)

There are two basic methods tacheometry.

1. stadia method
    a) Fixed hair stadia method
    b) Movable hair stadia method. (or) subtense Method.
2. Tangential method
3. Measurements by means of special Instruments
    a) Beaman stadia Arc
    b) Jeffcot direct reading tacheometer
    c) szepessy direct reading tacheometer
    d) Auto reduction (or) Hammer Fennel tacheometer
    e) Electronic Tacheometer (EDM)



Stadia Method :-

Fixed hair stadia method :-

* In this method the distance among the stadia hairs is kept constant. The horizontal & vertical distance of a point may be determined by fixed hair (fixed stadia interval)
* The vertical distance b/w the stadia wires is termed as stadia interval.
* The readings on the staff corresponding to all the three wires are taken

Movable Hair (or) subtense method :-

* This method is similar to fixed hair stadia method except that the stadia interval is varying.
* staff intercept is constant even though the dist... is varies.
* staff intercept is generally fixed b/w 3 4 6 m.

Tangential Method (or) system :-

* In this method, the stadia hairs are not used.
* The readings are taken in ~~the~~ horizontal cross hair.
* In the tangential method, vertical angles are measured from the central cross hair and the distances are calculated using trigonometric formulae.

Instruments used Tacheometry :-

* Theodolite fitted with a stadia diaphragm (or) a tacheometer. (Tacheometer is similar to theodolite but has some speal features)
* Levelling staff (or) stadia rod.

Special features for tacheometer ie, theodolite used for tacheometry

* Multiplying constant should be 100 ie, $k = 100$
* Additive constant should be zero ie, $c = 0$
* Telescope should be fitted with an anallactic lens.
* Magnification of the telescope should be 20 to 30 diameter.
* Magnifying power of eye piece is kept high.

Staff & stadia rod :-

* For rough work and small work, ~~then~~ a ~~the~~ levelling staff can be used for measuring the intercept.
* For Accurate work, a stadia rod is used.
* stadia rod is similar to levelling staff but may be longer and more accurately and finely divided.
* stadia rods should have bright, bold and clear markings for ease of reading.

Holding the staff

* In the case of a horizontal line of sight, the staff is held vertical.
* In the case of an inclined line of sight, the staff may be held vertical or normal to the line of sight

Holding the staff vertical :-

* The staff must be held truely vertical in accurate work.
* For this purpose, the verticality can be checked by a suspended plumb bob.
* Many times, for accurate work, the stadia rod may be provided with a circular level to check the verticality of the staff.
* Any deviation in verticality can result in serious error in the calculation of distances and elevations.

Holding the staff normal :-

* The staff must be held ~~pe~~ held of sight is more difficult perpendicular to the line of sight.
* ~~Generally~~, The perpendicularity of the staff may be checked by sighting the instrument with the help of a pair of open sights, or a small telescope fixed at right angles to the side of the staff.
* The staff is inclined until the telescope of the tacheometer is bisected by the cross wires of the telescope fitted to the staff.

Merits and demerits of vertical and normal holding :-

* It is a bit easy to ensure that the staff is perfectly vertical.
* A slight error in not keeping the staff vertical causes a series error in computation of ~~error~~ distances.

* In the case of an inclined sight, It is difficult to keep the staff perpendicular to the line of sight during high winds and in rough country.

* Normal holding, the accuracy of the direction of the staff can be judged by the transit man even during high winds.

Methods of reading the staff.

There are three methods of observing the staff for distance and altitude.

* conventional three hair method
* Height of instrument method
* Even-angle method

The observations consist of
(i) staff intercept (S)
(ii) Middle hair reading (r)
(iii) vertical angle (θ)

Conventional three Hair method :-

Advantages
* The staff is easier to be read (only 2 readings are uneven val
* The substructions for finding Staff intercept (S) and checking its accuracy are easier.

Height of Instrument Method :-

* Main Purpose of this method is to facilitate in calculating the elevation of the staff since

$$r = h$$

[r = Middle hair reading
h = Height of instrument]

Disadvantages:
* All the three readings are uneven
* In some cases r cannot be equal to 'h'.
* Difficulty of the field work

Even angle method :-

Advantages
* Even angles are multiples of 20'.
* computation is simple
* The trouble of measuring addi[...]

Errors in Tacheometric surveying:-

Instrumental errors:-

* Permanent adjustment of tacheometer may not be Perfect. (Adjustment of altitude level, accuracy of reading to the vertical circle).
* Graduation of the staff or stadia rod may not be uniform.
* Multiplying constant value may not be correct.

Errors due to manipulation & sighting (or) observation:-

* Inaccurate centering, ~~& levelling~~ & bisection.
* Inaccurate levelling of the instrument
* Incorrect position of the staff (Verticality of the staff has been not correctly)
* Improper focussing of the telescope
* Inaccurate reading to the horizontal and vertical circles.

Errors due to natural causes:-

* During high wind both the staff and the instrument may be subjected to vibration.
* During hot weather condition parts of tacheometer may be subjected to expansion.
* In hot weather there may be proper visibility of staff.
* Unequal refraction

Precautions of errors in tacheometric Surveying:-

Instrumental Errors:-

Tacheometer not be perfect → before starting the survey all the adjustments are properly checked and rectified.

... eliminate such errors the staff and rod should be checked & corrected or should be replaced.

* Multiplying constant value not correct →

before starting of work necessary field tests should be done to avoid this type of error.

observational error :-

* Incorrect centering & levelling :- →

In every setting of the tacheometer, proper centering & levelling of the Plate bubble & altitude bubble should be ~~substated~~. attended.

* Verticality of the staff →

To avoid this error is to be properly checked the verticality of the staff using plumb bob

* Improper focusing :- →

This error can be elliminated by proper focussing before starting of the work & all steps should be taken to prevent parallex.

* Bad visibility :- →

This error can be avoided, if the graduations on the staff are clearly and distinctly seen

Natural error :

High wind :- → In such a situation the work should be suspended or some temporary barear may be used.

* Hot weather condition - expansion: →

This can be avoided by providing some shade.

* Poor visibility during hot weather: →

This is avoided by placing instrument such that there is no direct sunlight on the object glass.

A staff held vertically at a distance of 50 m & 100 m from a transit fitted with stadia hairs, the staff intervals with the telescope normal were 0.494 and 0.994 m respectively. The instrument was then set up near a B.M of RL = 1500 m, and the readings on the staff held on the B.M was 1.495 m. The staff readings at the station 'A' with staff held vertically and the line of sight horizontal were 1.00, 1.85 & 2.70. What is the horizontal distance b/w the B.M & A, and RL of A.

<u>solution</u>

$D_1 = 50 m$; $D_2 = 100 m$

$S_1 = 0.494 m$; $S_2 = 0.994 m$

$D_1 = KS_1 + C$ —— ① ; $D_2 = KS_2 + C$ —— ②

$$K = \frac{D_2 - D_1}{S_2 - S_1} = \frac{100 - 50}{0.994 - 0.494} = 100$$

$$C = \frac{D_1 S_2 - D_2 S_1}{S_2 - S_1} = \frac{(50 \times 0.994) - (100 \times 0.494)}{0.994 - 0.494} = 0.60$$

Line of sight ~~staff~~ is horizontal

$D = KS + C$

RL of BM = 1500.0 m

h = 1.495 m

S = 2.7 − 1.0 = 1.70 m

$D = (100 \times 1.7) + 0.6$

$D = 170.6$ m.

RL of A = RL of Bm + h = 1500.0 + 1.495

= 1501.495 m.

Introduction:-

Tacheometry is a branch of surveying in which both horizontal & vertical distances are measured without the use of chain or tape.

It is also known as tachymetry or telemetry.

Certainity (or) suitable of tacheometric surveying:-

* Hilly areas
* Undulations areas
* rough terrains
* Don't used for chainage areas (river, etc)

Uses of tacheometry:-

* It is used for contouring, in which the horizontal distances and elevations are to be determined. also to prepare map
* Railway, Highway and Irrigation projects (Dam, etc).
* It is also used for checking measurements taken by cha[in] or [tape]

Instruments used in Tacheometry:-

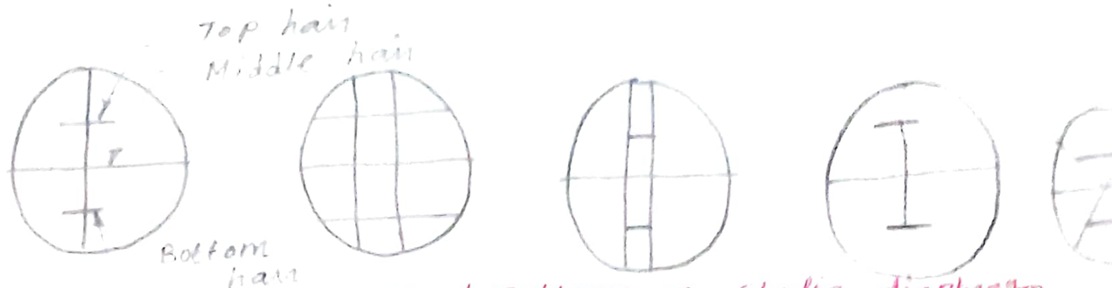
* Tacheometer
* Levelling staff (or) stadia rod.

Tacheometer:-

a) Diaphragm

* In ordinary theodolite, to fixed with stadia diaphragm, then it is called tacheometer.
* stadia diaphragm, means the theodolite has three horizontal cross hairs ~~(ie~~, or stadia hairs (ie, top, bottom & middle hairs)

*



Different Patterns of stadia diaphragm

* For find out the vertical distance to take the middle hair reading.

* For find out the horizontal distance to take the top and bottom hair reading is considered.

* For tangential method is only used for middle hair reading.

Telescope :-

* In ordinary theodolite, the telescope is ~~in~~ small in ~~to~~ length wise to compared with tacheometer.

* Magnifying power of eye piece is high

* Magnification of telescope should be 20 to 30 diameters.

* The telescope ~~with~~ should be fitted with anallactic lens.

* Tacheometer → The value of multiplying constant should be 100. ($K = 100$).

* Additive constant 'C' should be zero.

Types of Telescope in tacheometric surveying :-

(i) external focussing telescope → Theodolite
(ii) external focussing anallatic telescope → tacheometer
(iii) Internal focussing telescope

$$f_1 = f + \left(\frac{f}{i}\right) s \quad \text{——— (4)}$$

Substitute the f_1 values in equation (1)

$$(1) \Rightarrow \quad D = f_1 + d$$

$$= f + \left(\frac{f}{i}\right) s + d$$

$$= \left(\frac{f}{i}\right) s + (f+d)$$

$$\boxed{D = Ks + C}$$

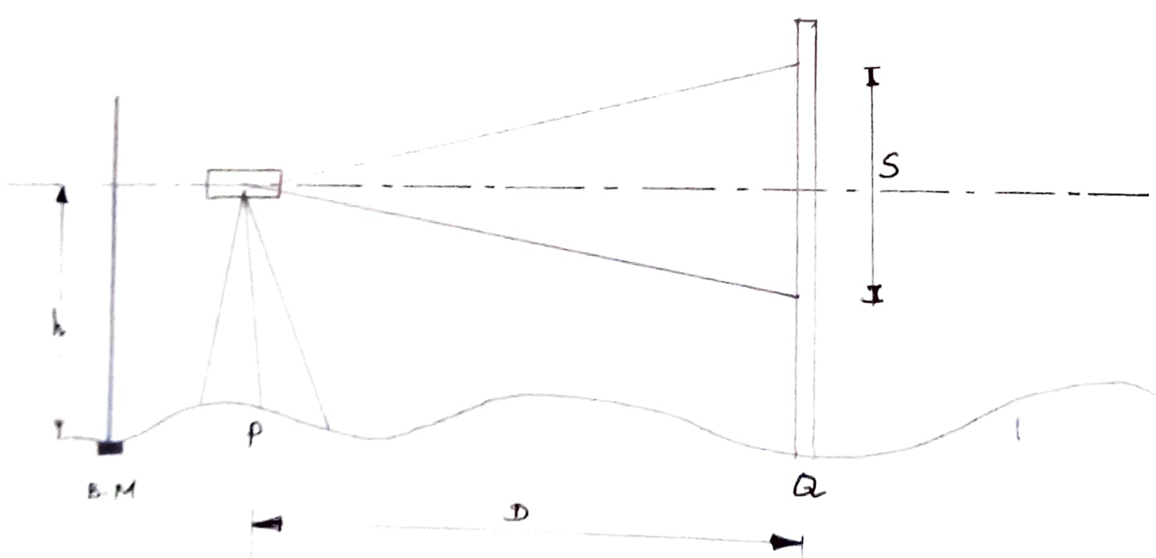
where, $K = \frac{f}{i}$ = multiplying constant

s = staff intercept

$C = (f+d) \rightarrow$ additive Constant

Fixed Hair Method :-

Line of sight horizontal and staff held vertical



where,

$P \rightarrow$ Tacheometer (or) Instrument station

$Q \rightarrow$ Staff station

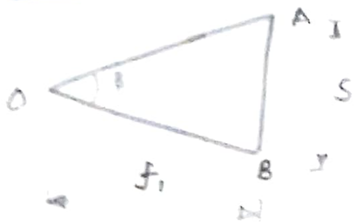
$S \rightarrow$ Staff intercept

From the figure

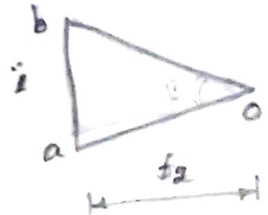
$$D = f_1 + d \quad \text{———————} \quad (1)$$

In Δ^{le} AOB & aob

AOB



aob



$$\sin\beta = \frac{s}{f_1} \qquad\qquad \sin\beta = \frac{i}{f_2}$$

Equating AOB & aob

$$\cancel{\sin\beta} = \frac{s}{f_1} = \frac{i}{f_2} = \cancel{\sin\beta}$$

$$\frac{s}{f_1} = \frac{i}{f_2}$$

ie,
$$\frac{s}{i} = \frac{f_1}{f_2} \quad \text{———————} \quad (2)$$

(or) $\boxed{\frac{1}{f_2} = \frac{s}{i f_1}}$

By the lense formula

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2} \quad \text{———————} \quad (3)$$

Substitute the values $\frac{1}{f_2}$ in eqn. (3)

$$\frac{1}{f} = \frac{1}{f_1} + \frac{s}{i \cdot f_1}$$

$$\frac{1}{f} = \frac{1}{f_1}\left(1 + \frac{s}{i}\right) \quad \text{(or)} \quad f_1 = f\left(1 + \frac{s}{i}\right)$$

$D \rightarrow$ Horizontal distance b/w the instrument station 'P' & staff station 'Q'.

$h_1 \rightarrow$ staff reading at B.M

* To set the tacheometer at instrument station 'P'.
* To set verniers C & D is to be zero. to 180°
* To take the staff reading at B.M

$$D = Ks + C$$

(or)

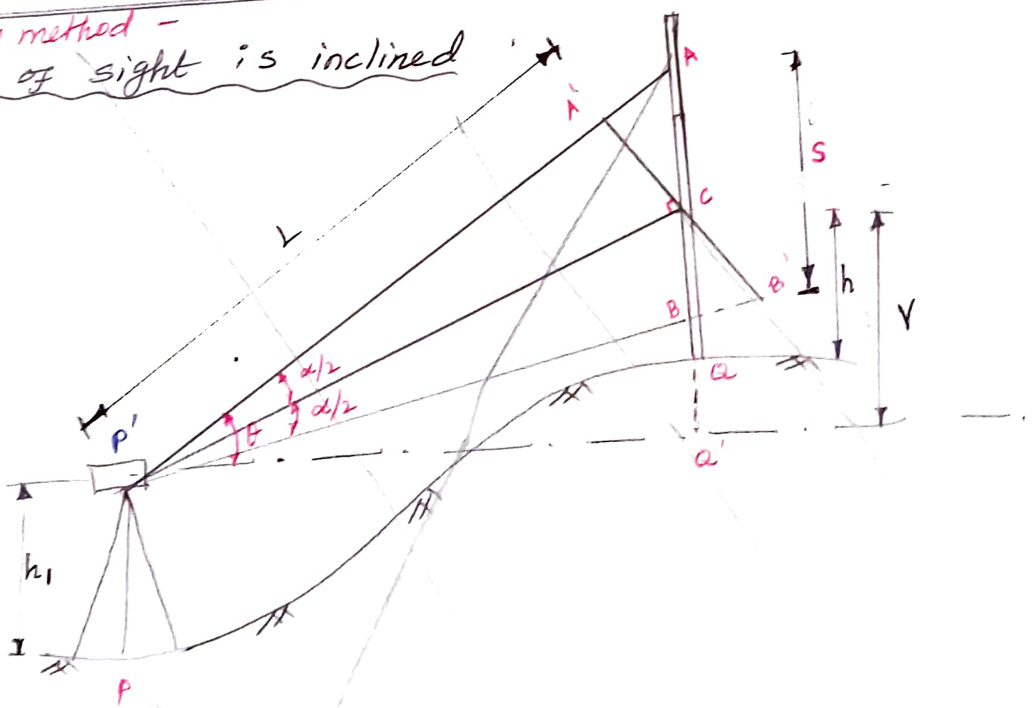
$$D = \frac{f}{i} s + (f+d)$$

RL of Horizontal line of sight = RL of P + h (or) RL of BM + h

RL of Q = RL of Horizontal line of sight − middle hair reading.

Fixed hair method –

Line of sight is inclined



Where,

$P \rightarrow$ Instrument (or) Tacheometer station

$Q \rightarrow$ staff station

$i \rightarrow$ stadia interval

$s \rightarrow$ staff intercept

θ → Inclination at line of sight

L → Length of line of sight ($P'C$)

D → Horizontal distance b/w instrument station & staff station

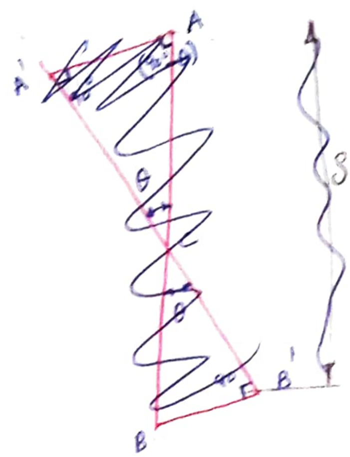
V → vertical height b/w tacheometer line of sight to ~~top~~ middle hair reading.

h_1 → staff reading at B.M

h → middle hair reading

β → Inclination (or) angle b/w the top & bottom hair ~~reading~~

To draw a line $A'CB$ normal to the line of sight OC.



From right angle Δ^{le} $P'Q'C$

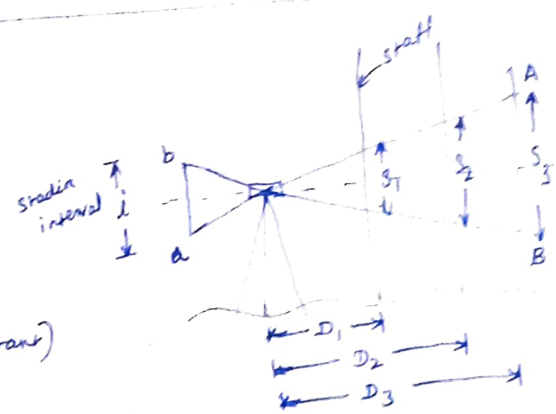
$\lfloor Q'P'C = \theta$

$\lfloor P'Q'C = 90$

$\therefore \lfloor P'CQ' = 90 - \theta$

and also

Principle of Tacheometry:



D_1, D_2, D_3 → Staff distance

S_1, S_2, S_3 → stadia intercept

$$\frac{D_1}{S_1} = \frac{D_2}{S_2} = \frac{D_3}{S_3} = \frac{f}{i} \text{ (constant)}$$

~~f/i = multiplying~~ $\frac{f}{i}$ = multiplying constant

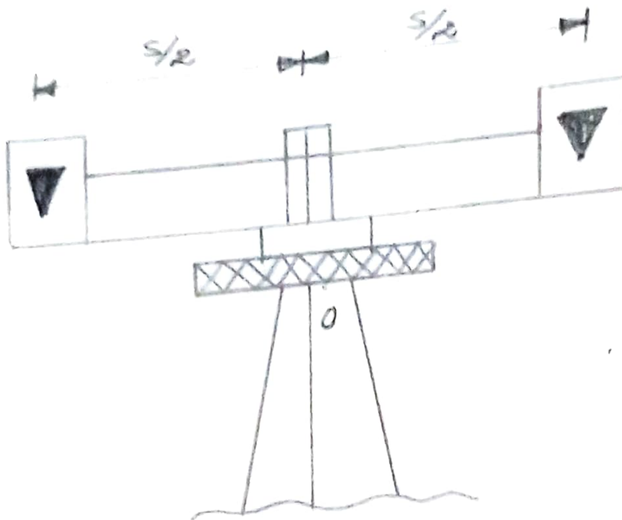
$(f+d)$ = Additive constant

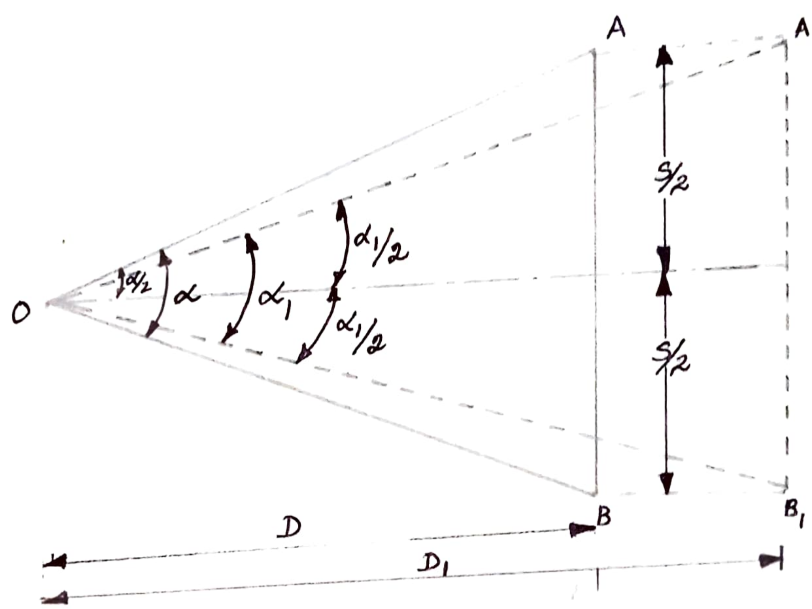
f → focal length

d = horizontal distance b/w vertical axis of tacheometer & objective lense.

Movable Hair Method (or) Subtense Method

Principles :-





* ~~The tacheometric angle is always constant for a given telescope.~~

* ~~The staff intercept (s) and tacheometric angle (α) changes with the same station.~~

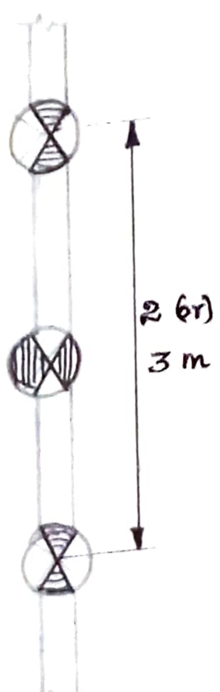
* In the movable hair method the stadia interval (i) is variable, where as the staff intercept (s) is kept constant.

* The staff intercept (s) is generally fixed b/w 3 and 6m

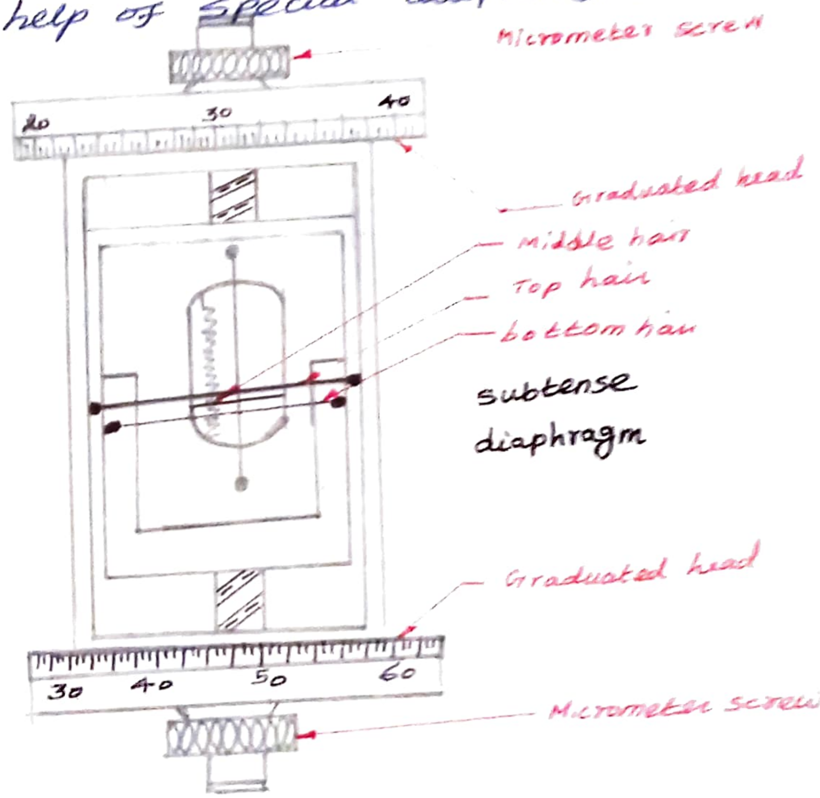
* If the staff intercept (s) is more than staff length, only half the staff intercept is needed. The staff intercept is also called base.

* When the base is horizontal, the method is called horizontal base subtense method and the angle is measured with the horizontal circle of the theodolite.

* If the base is vertical, the method is called vertical base subtense method and the angle is measured with help of special diaphragms.



Rod with target

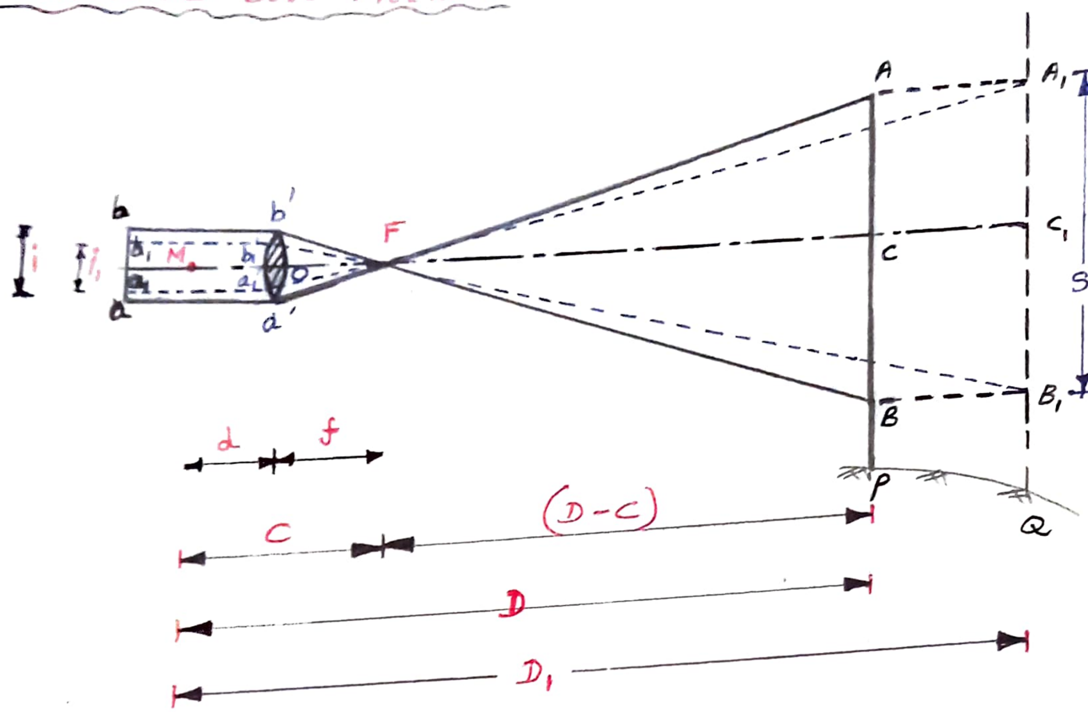


subtense diaphragm

* A drum is provided with a vernier readings to be obtained up to a 1000th of the pitch of the screw moving the legs.

* The movable hair method is rarely used for nowadays.

Vertical subtense bar Method:



The optical diagram with subtense theodolite for a staff at 'P' and dotted lines show it for the staff at 'Q'.

Distance and elevation formula for horizontal sights.

Let,

$S = AB = AB_1$ = staff intercept

$i = ab$ = stadia interval

F = Exterior principal focus of the objective

M = centre of the instrument

From similar Δ^{le} ABF & a'b'F

$$\frac{FC}{S} = \frac{FO}{a'b'} = \frac{f}{i}$$

(or)

$$FC = f\frac{S}{i}$$

$$D = MF + FC$$

$$= (f+d) + \frac{fS}{i}$$

$$\boxed{D = \frac{f}{i}S + (f+d)} \quad (1) \quad \text{(or)} \quad \boxed{D = KS + C}$$

staff intercept (s) is fixed & stadia interval (i) is variable.

$\frac{f}{i}$ varies with staff ~~elevation~~ position

i is measured with the help of micrometer screw.

Let,

m = total number of revolution of micrometer screw

p = Pitch of micrometer screw.

e = Index error

$$\boxed{i = mp}$$

substituting the values 'i' in equation ①

$$D = \frac{f}{i} s + (f+d)$$

$$= \frac{f}{mp} s + (f+d)$$

$$\boxed{D = \frac{ks}{m} + c} \text{ ———— } ②$$

Where,

$k = \frac{f}{p}$ = constant for an instrument

c = additive constant $c = (f+d)$

e = index error

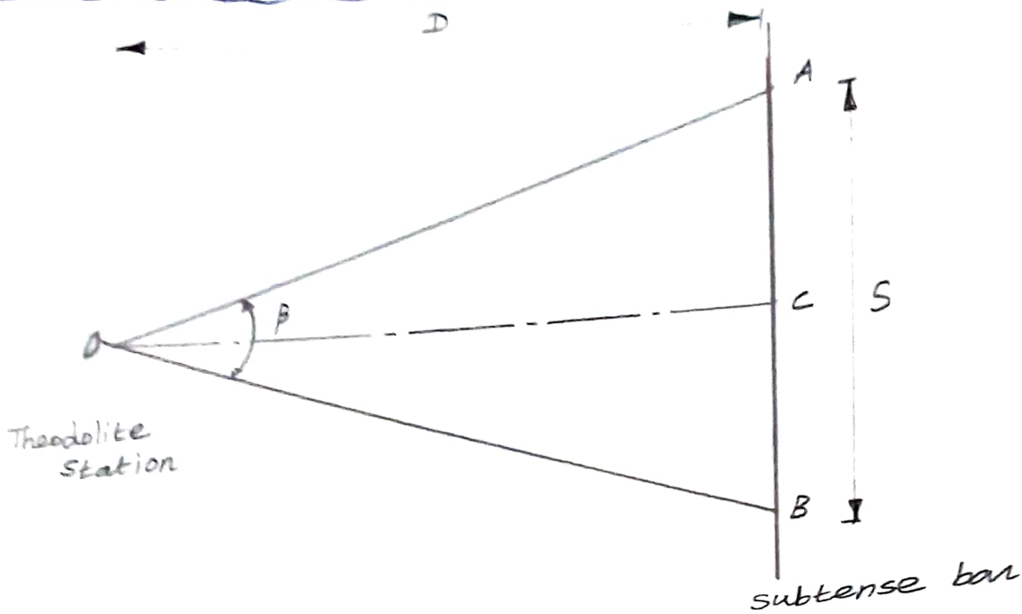
$$\boxed{D = \frac{ks}{m-e} + c} \quad ; \quad \boxed{V = \frac{ks \sin 2\theta}{2m} + c \sin\theta}$$

Distance and elevation formula for inclined sights

If the line of sight is inclined at an angle θ and staff is vertical

$$\boxed{D = \frac{KS}{m-e} \cos^2\theta + c\cos\theta}$$

Horizontal subtense bar Method



In this method, the horizontal distance b/w the instrument station 'O' and the subtense bar station 'C' is calculated by a subtense bar.

A line AB is $\perp^r$ to the line OC

∴ Distance b/w O & C

$$D = \frac{1}{2} S \cdot \cot \beta/2$$

ie,

$$D = \frac{S}{2 \tan \beta/2}$$

If the β is very small, then

$\tan \beta/2 = \beta/2$ where β is in radians (1 rad = 206265 seconds)

$$= \frac{1}{2} \cdot \frac{\beta}{206265''}$$

$$\therefore \boxed{D = \frac{S \times 206265''}{\beta}}$$

where β in seconds

$$\boxed{V = \frac{K \cdot S}{m-e} \cdot \frac{\sin 2\theta}{2} + c \sin\theta}$$

Usually the constant $K = 1000$, then

* Fix to targets on a staff at some distance say S.
* Range a line on fairly level ground and measure distance D_1 & D_2
* Note the micrometer readings m_1 & m_2 to move the stadia hairs.

$$D_1 = \frac{f}{m_1 p} S + (f+d)$$

$$D_1 = \frac{KS}{m_1} + C \quad \text{———— (1)}$$

$$D_2 = \frac{KS}{m_2} + C \quad \text{———— (2)}$$

solving equations (1) & (2)

$$\boxed{K = \frac{(D_1 - D_2)\, m_1 \cdot m_2}{S(m_1 - m_2)}}$$

$$\boxed{C = \frac{D_1 m_1 - D_2 m_2}{m_1 - m_2}}$$

Merits & Demerits of movable Hair method

Merits	Demerits
* More accurate method	* computation is slow
* stadia interval (i) is accurately measured.	

Problem

observations were made from a station P on a subtense bar held at station Q. The vertical angle was 8°15'. The number of revolutions of the micrometer screw was 21.35. The instrument constants were 1000 & 0.4. The intercept was kept at 3 m. Find the horizontal distance b/w P & Q.

Solution:

The horizontal distance in movable hair instrument or subtense method is

$m = 21.35$
$s = 3\ m$
$k = 1000$
$\theta = 8°15'$

$$\boxed{D = \frac{ks\cos^2\theta}{m} + c\cos\theta}$$

$$= \frac{1000 \times 3 \times \cos^2 8°15'}{21.35} + 0.4 \times \cos 8°15'$$

$$= 138.01\ m.$$

The stadia intercept read by means of a fixed hair instrument on a vertically held staff is 1.05 m, the angle of elevation being 5°36'. The instrument constants are 100 and 0.30. What would be the total number of turns registered on a movable hair instrument at the same station for a 1.75 m intercept on a staff held on the same point, the vertical angle in this case being 5°24' and the constants 1000 & 0.5?

Solution:-

observations by fixed hair instrument

$k = 100$
$c = 0.30$
$\theta = 5°36'$
$S = 1.05\ m$

$$\boxed{D = ks\cos^2\theta + c\cos\theta}$$

$$= 100 \times 1.05 \times \cos^2 5°36' + 0.3 \times \cos 5°36'$$

$$\boxed{D = 104.29\ m}$$

observations by movable hair instrument :-

$k = 1000$; $s = 1.75$

$c = 0.50$; $\theta = 5^\circ 24'$

$$\boxed{D = \frac{k s \cos^2\theta}{m} + c\cos\theta}$$

$$104.29 = \frac{1000 \times 1.75 \times \cos^2 5^\circ 24'}{m} + 0.5 \times \cos 5^\circ 24'$$

$$104.29 = \frac{1734.50}{m} + 0.498$$

$$m\,(104.29 - 0.498) = 1734.50$$

$$\therefore m = \frac{1734.50}{103.79}$$

$$\therefore \boxed{m = 16.71}$$

The constant (k) for an instrument is 850, the value of $c = 0.50$, and staff intercept, $s = 3m$. calculate the distance from the instrument to the staff when the micrometer reading are 4.628 and 4.626 and the line of sight is inclined at $+10^\circ 36'$. The staff was held vertical.

Solution :-

sum of micrometer readings $m = 4.628 + 4.626$

$m = 9.254$

$$D = \frac{K S \cos^2\theta}{m} + c\cos\theta$$

$$= \frac{850 \times 3 \times \cos^2 10^\circ 36'}{9.254} + 0.5 \times \cos 10^\circ 26'$$

$$\boxed{D = 226.70\ m}$$

The distance b/w two stations A & B was 258m. A movable hair instrument was used to measure this distance again. The vertical angle was $6^\circ 30'$. The distance b/w the vanes on the subtense bar was 5m. The constants of the instrument were 1000 and 0.50. Find the number of turns of the micrometer screw registered during this measurement.

Solution:-

The horizontal distance is given by

$$\boxed{D = \frac{K s \cos^2\theta}{m} + c\cos\theta}$$

$K = 1000$; $s = 5m$; $c = 0.50$; $\theta = 6^\circ 30'$

$D = 258\,m$

$$D = \frac{1000 \times 5 \times \cos^2 6^\circ 30'}{m} + 0.5 \times \cos 6^\circ 30'$$

$$258 = \frac{4935.925}{m} + 0.494$$

$$\therefore \boxed{m = 19.13}$$

A distance PQ was measured with a tacheometer (constants 100 & 0.5) at P. The vertical angle was $5^\circ 30'$. The cross hair readings were 1.335, 2.335 and 3.335. Find the distance PQ and the RL of Q if the readings at the staff at BM of RL 1030.50 was 2.335. A movable hair instrument was then set up over P and observations were made over the same distance. The vertical angle was the same. The intercept was 3m and the number of turns of the micrometer screw was noted as 14.93. If $c = 0.5$, find the constant K of the instrument.

$$\boxed{D = ks\cos^2\theta + c\cos\theta}$$

$s = 3.335 - 1.335 = 2\text{ m}$

$\theta = 5°30'$; $k = 100$; $c = 0.5$

$D = 100 \times 2 \times \cos^2 5°30' + 0.5 \times \cos 5°30'$

$$\boxed{D = 198.66\text{ m}}$$

$$\boxed{V = \frac{ks\sin 2\theta}{2} + c\sin\theta}$$

$$V = \frac{100 \times 2 \times \sin(2 \times 5°30')}{2} + 0.5 \times \sin 5°30'$$

$$\boxed{V = 19.128\text{ m}}$$

RL of Q = RL of BM + h + V − r

$= 1030.50 + 2.355 + 19.128 - 2.335$

$$\boxed{\text{RL of Q} = 1049.648\text{ m}}$$

with the movable hair instrument :

$$\boxed{D = \frac{ks\cos^2\theta}{m} + c\cos\theta}$$

$D = 198.66\text{m}$

$s = 3\text{ m},$

$\theta = 5°30'$

$c = 0.5$; $m = 14.93$

$$198.66 = \frac{k \times 3 \times \cos^2 5°30'}{14.93} + 0.5 \times \cos 5°30'$$

$$\frac{(198.66 - 0.498) \times 14.93}{2.972} = k$$

$$\boxed{k = 995.48}$$

The number of turns of the micrometer screw recorded was 22.5 for a distance of 60 m & 11.28 for a distance of 120m. Find the constants K & C of the instrument.

Solution -

Two equations can be set up for the two measurement

$$D = \frac{KS}{m} + C$$

$$60 = \frac{K \times 1.5}{22.5} + C$$

$$60 = 0.067\,K + C \quad \text{——— (1)}$$

$$120 = \frac{K \times 1.5}{11.28} + C$$

$$120 = 1.133\,K + C \quad \text{——— (2)}$$

Solving the equations (1) & (2) we get

$$\boxed{K = 904.8}$$

$$\boxed{C = 0.32}$$

The constants can be determined from the formula

$$K = \frac{(D_1 - D_2)\, m_1 m_2}{s\,(m_1 - m_2)} = \frac{(120 - 60)\; 11.28 \times 22.5}{1.5\,(22.5 - 11.28)}$$

$$\boxed{K = 904.81}$$

$$C = \frac{D_1 m_1 - D_2 m_2}{m_1 - m_2} = \frac{120 \times 11.28 - 60 \times 22.5}{(22.5 - 11.28)}$$

$$\boxed{C = 0.32}$$

Effect of angular error on horizontal distance

$$\delta D = \frac{D \cdot \delta \beta}{\beta}$$

$$\boxed{\delta_D = \frac{D \cdot \delta_\beta}{\beta}}$$

where

Angular error $= \delta_\beta \; (-)^{ive}$

Linear error $= \delta_D \; (+)^{ive}$

Distance $= D$

The horizontal angle subtended at a theodolite by a subtense bar with vanes 3 m apart is $12' 33''$. calculate the horizontal distance b/w the instrument and the bar. Also find (i) The error of horizontal distance if the bar was 3° from being normal to the line joining the instrument and bar stations.

(ii) the error of the horizontal distance if there is an error of $1''$ in the measurement of the horizontal angle at the instrument station.

Solution :-

$$\beta = 12' 33'' = (12 \times 60 + 33) = 753''$$

$$D = \frac{206265}{\beta} s = \frac{206265 \times 3}{753}$$

$$\boxed{D = 821.77\,m}$$

(i) The horizontal distance $(D = 821.77\,m)$, if the bar 3° from the normal to line joining the instrument & bar station

$$D' = D \cos \beta = 821.77 \times \cos 3^\circ$$

$$\boxed{D' = 820.64\,m}$$

$$\text{Error} = D' - D = 821.77 - 820.64 = 1.13\,m$$

[illegible] = [illegible] = $\frac{1.13}{[illegible]}$ = [illegible]

[illegible] = [illegible]

[illegible] the displacement [illegible] the angle [illegible]

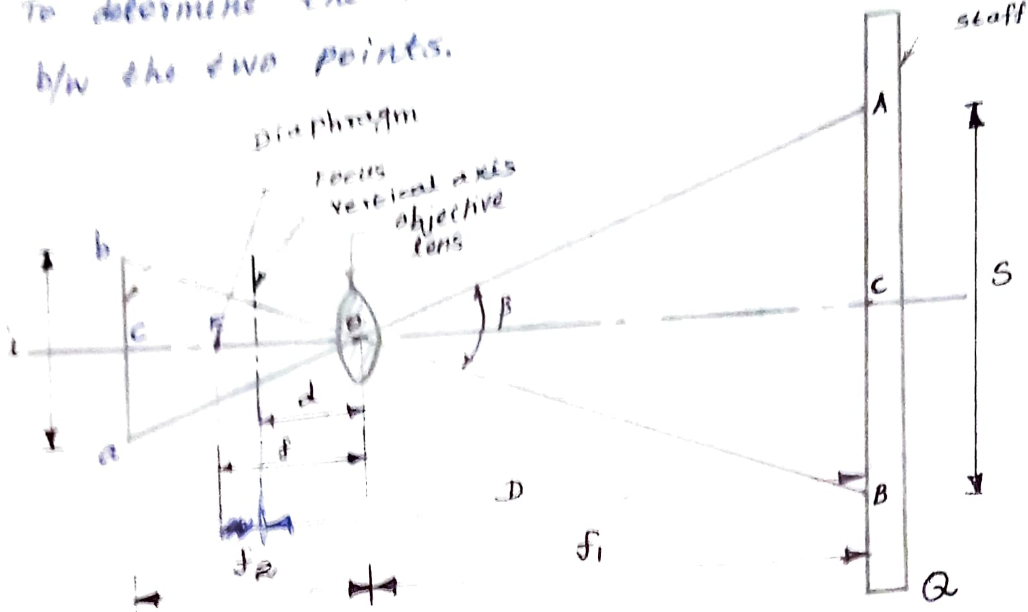
$$\delta_s = \frac{D\ \delta_\theta}{s} = \frac{[illegible]1.77 \times 1}{75.9}$$

$$\delta_s = 1.09\ m$$

stadia system

Principles of stadia tachometry

* To determine the horizontal and vertical distance b/w the two points.



where

O → optical centre of objective

F → Principal focus of objective

a, b, c → bottom, top & middle hairs of the diaphragm

A, B, C → bottom, top & middle hairs of the diaphragm in a staff readings.

ab = i → stadia interval

AB = S → staff intercept

OF = f → focul length

f_1 → Horizontal distance b/w optical centre ie, lens & the staff

f_2 → Horizontal distance b/w cross wires & the optical centre or lens.

D → Horizontal distance b/w vertical axis of the tacheometer and the staff.

d → Horizontal distance b/w vertical axis of the tacheometer and the objective lens.

Stadia rod (or) Levelling staff:-

* For smaller distance (up to 100 meter) ordinary levelling staff may be used.
* For more than 100 meter, then the stadia rod may be used.
* Stadia rod is normally 50 mm to 150 mm width, and 3 to 5 m length.
* Stadia rod is made up of aluminium or wood.
* It has clearly marking the measurements or readings in meters, decimeter and centimeters.
* For smaller distance, → the stadia rod graduation is 5mm (0.005 m) may be used.
* For longer distance the stadia rod graduation is 10mm (0.010m)

Methods (or)

System of tacheometry:- Fixed hair stadia method / movable hair method (or) Substense method

* Stadia methods (or) stadia system
* Tangential methods (or) tangential system.
* Measurements by means of special Instruments
  a) Beaman stadia Arc
  b) Jeffcot direct reading tacheometer
  c) Auto reduction (or) Hammer Fennel tacheometer
  d) Electronic tacheometer (EDM).

Anallatic lense

In a tacheometer an additional convex lense is fitted b/w the eye piece & the object glass at a fixed distance from the object glass. The convex lense is called as an anallatic lense.

Two marks Questions & Answers.

UNIT - I

Tacheometric surveying

1. What are the different systems of tacheometric surveying?

A.U. May/June 2009.

a) stadia systems — Fixed hair method / movable hair method

b) Tangential systems.

a) stadia systems:-

* The diaphragm is provided with two stadia hairs (upper & lower hair)
* There are two kinds of stadia systems ie,
  Fixed hair method
  movable hair method.

(b) Tangential systems:-

* The diaphragm of the tacheometer is not provided with stadia hairs.
* Only the single horizontal hair is used to take the reading.

2. What are the three types of telescopes used in stadia surveying?

A.U. Nov/Dec 2009

(i) external Focussing telescope (ie, stadia theodolite)
(ii) external - focussing anallatic telescope (ie, tacheometer)
(iii) Internal - focussing telescope.

3. Define / what is an anallatic lens?

* Anallatic lens is an additional lens placed b/w diaphragm and the objective at a fixed distance from the objective.
* This lense will be fitted in ordinary transit theodolite
* The anallatic lens is fitted with the telescope then it is called as external focussing alalytic telescope.

Purpose

* Fitting the ~~at~~ anallatic lense is to redule the additive constant to zero.

List the characteristics should a tacheometer have.

* The telescope should be with a magnification of 20 to 30 diameters.
* For a bright image, the aperture of the objective should be of 35 to 45 mm diameter.
* The anallatic lens is fitted with the telescope then the multiplying constant $\frac{f}{i} = k = 100$ and the additive constant $(f+d) = c = 0$
* To obtain a clear staff reading from a long distance, the eye-piece should be greater magnifying power.

Define Fixed hair stadia method

* The distance b/w the stadia hairs is fixed and thus the method is known as fixed hair stadia method.
* The upper and lower hair readings are taken in the staff intercept.
* Staff intercept is varies with the distance b/w the instrument and staff position.

*
Differentiate the principles of stadia and subtense methods.

Stadia method	Subtense method
* The distance b/w the staff and the tacheometer	* The principle of subtense method is just reverse of the stadia principle.
* Tacheometer angle is always constant for a given telescope	
* The staff intercept is varies with the distance b/w the staff and the instrument.	* The staff intercept forms the fixed base and the tacheometric angle changes with the staff position.

List the merits and demerits of movable - hair method in tacheometric survey?

Merits:-

* Movable hair method is more accurate
* Long distances can be taken with greater accuracy than in stadia method.

Demerits:-

* Careful observation is essential
* Lacks speed in the field ie, computations are not quicker
* Variables 'm' and 'i' should be measured accurately
*

8. Explain the use of subtense bar in surveying?

* The subtense bar is an instrument used for measuring the horizontal distance b/w the instrument station and a point on the ground.
* Apart from the subtense bar, in this method, no staff or target rod is needed.
* Further the theodolite needed is also the ordinary transit type.

9. List the instrument error in tacheometry survey. Explain any one with the necessary Precautions.

* Instrumental errors
* Errors of observation (or) personal errors
* Errors due to nautural causes.

Instrumental Errors:- and Precautions

(i) Permanent adjustments of the tacheometer may not be perfect

Precautions:- Before starting the survey all the adjustments should be checked and rectified.

(ii) Graduation of the staff (or) stadia rod may not be uniform.

Precautions:- The staff and rod should be checked and corrected or should be replaced.

(iii) Multiplying constant value may not be correct.

Precautions:- Before starting the work necessary field test should be done to avoid this type of error.

10. Define tacheometry:

Tacheometry is a branch of surveying in which both horizontal and vertical distances are measured without the use of chain or tape.

It is also known as tachymetry or telemetry.

11. Define tacheometer.

It is an ordinary ~~theo~~ transit theodolite fitted with an extra lense called anallatic lense. (or) stadia diaphragm is called a tacheometer.

stadia diaphragm means the theodolite has three horizontal cross hairs or stadia hairs, (top, middle & bottom hairs).

k = 100
c = 0

12. Define subtense bar

* The length of the subtense bar is 2 m (6 ft) for measurement of comparitively short distance in a traverse.
* The length of the bar is made equal to the distance b/w the two targets.

13 Define staff intercept.

The difference of the staff readings corresponding to the top & bottom stadia wires.

14. Define stadia intercept

The difference of the distance b/w the top and bottom cross hairs.

15. What is subtense method.

* stadia interval is variable
* staff intercept is kept fixed while the stadia interval is variable.

16. Explain the tangential method

* The stadia hairs are not for taking readings.
* The readings being taken against the horizontal cross hair.

17 What is the principles of stadia methods?

* It is based on the Principle, that the ratio of the Perpendicular to the base is constant to similar isosceles triangle.

The readings on a staff held vertically 60 m from a tacheometer were 1.460 and 2.055. The line of sight was horizontal. The focal length of the objective lens was 24 cm and the distance from the objective lens to the vertical axis was 15 cm. Calculate the stadia interval.

Solution:

$D = KS + C$ $\qquad$ $k = \frac{f}{i}$

$D = \left(\frac{f}{i}\right)S + (f+d)$ $\qquad$ $C = (f+d)$

$S = 2.055 - 1.460 = 0.595$

$60 \times 100\ \text{cm} = \frac{24}{i} \times 0.595\ \text{m} \times 100\ \text{cm} + (24+15)$

$6000 - 39 = \frac{1428}{i}$

$\therefore i = 2.55\ \text{cm}.$ 0.239 cm

$\boxed{\text{stadia interval } i = 2.55\ \text{cm}}$ 0.239 cm

What is the difference b/w staff intercept & stadia intercept?

staff intercept	stadia intercept
* The distance b/w the targets is kept fixed in a staff intercept	* The distance b/w the stadia hairs is variable.

What are the disadvantages of an anallactic lens?

* The anallactic lens reduces the brilliance of the image.
* It absorbs much of incident light
* It cannot be easily cleaned.
* If the anallactic lens is adjustable, it is a potential source of error.

List some disadvantages of tangential method of tacheometry.

* In general tacheometry compares unfavourable with that of chaining.
* In tangential method the horizontal and vertical distance from the instrument to the staff stations are computed from the observed vertical angles to the vanes fixed at a constant distance. Thus the distances calculated depend very much on the accuracy of the two angles measured

22. consider the horizontal distance equation $D = KS + C$. What are represented by K, S & C?

Apr/May 2011

Equation pertains to tacheometric surveying

$$D = KS + C$$

Where,

D = horizontal distance from instrument and staff station.

$K = \frac{f}{i}$ = multiplying constant

$C = (f+d)$ = additive constant

f = focul length of object glass

i = stadia interval (or) length of image

d = distance b/w optical centre & vertical axis of the instrument.

K & C are called as tacheometric constants.

23 What is parallax? How it can be eliminated?

Apr/May 2011

Parallax is a condition arising when the image formed by the objective is not in the plane of the cross hairs. Accurate sight is possible only when parallax is eliminated. It is eliminated by focussing the eye piece and the objective.

24. What are the multiplying constant and additive constant of a tacheometer?

Apr/May 2010

The K & C are the tacheometric constants.

Where, $K = \frac{f}{i}$ = multiplying constant

$C = (f+d)$ = additive constant

f = focul length of object glass

i = stadia interval

d = distance b/w the vertical axis of the instrument & optical centre.

Holding the staff

* The line of sight is horizontal → staff is held vertical

* Line of sight is inclined → staff is vertical or normal.

Vertical holding (or) Holding the staff is vertical :-

* The staff must be held truely vertical

* For ordinary work → the verticality of the staff can be judged by the eye

* For accurate work → verticality can be checked by suspened plumb bob.

otherwise

* For accurate work → stadia rod may be provided with a circular bubble attached.

Normal holding (or) Holding the staff normal :-

* The staff must be held perpendicular to the line of sight

* The perpendicularity of the staff may be checked by sighting the instrument with the help of a pair of open sights. or a small telescope fixed at right angles to the side of the staff.

(1) An instrument was set up at P and the angle of elevation to a vane 4 m above the foot of the staff held at Q was 9°30'. The horizontal distance b/w P & Q was known to be 2000 m. Determine the RL of the staff station Q, given that the RL of the instrument axis was 2650.38 m.

Solution:

$D = 2000 m$ $\qquad D = \frac{2000 m}{1000} = 2 km$

$\alpha = 9°30'$

$r = 4 m$

Ht of vane above the instrument axis

$h = D \tan\alpha = 2000 \times \tan 9°30'$

$= 334.68 m$

correction for curvature & refraction

$C = 0.06735 D^2$ $\qquad D \rightarrow$ is in km

$= 0.06735 \times 2^2$

$= 0.27 \ (+)^{ive}$

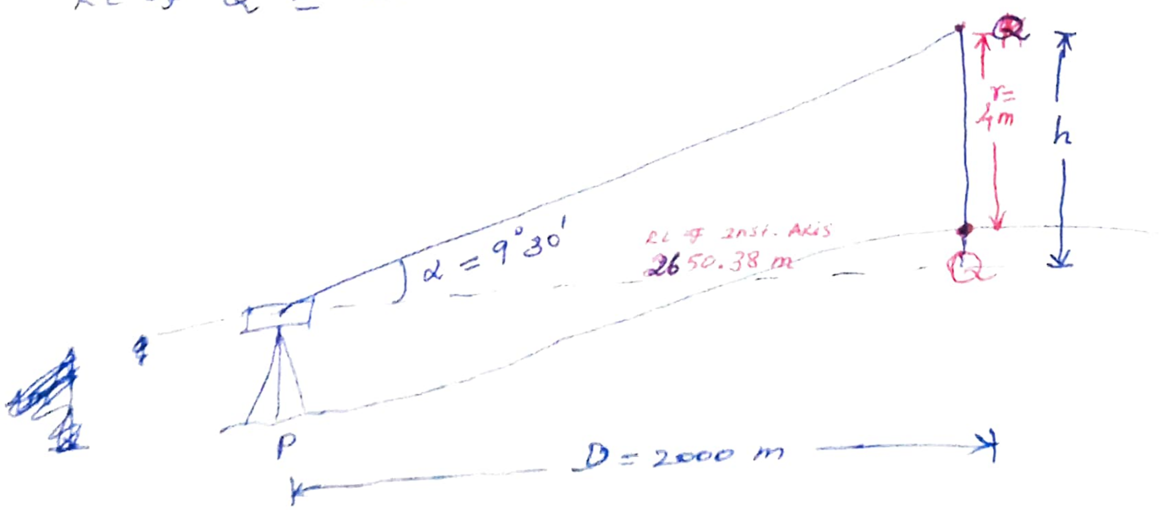
Ht. of vane above the inst. axis $= 334.68 + 0.27$

$= 334.95 m.$

RL of vane $= 334.95 + 2650.38 m.$

$= 2985.33 m.$

RL of Q $= 2985.33 - 4 = 2981.33 m.$



(4)

Trigonometrical levelling

Trigonometrical levelling is the process of determining the elevations of stations from observed vertical angles and horizontal distances.

* vertical angles are measured with a theodolite
* The distances are measured accurately with a tape

Types of Trigonometrical levelling :-

Trigonometrical levelling are conducted considering the concepts of plane surveying or Geodetic surveying.

(i) Observations to find small elevations & short distances
(or)
Plane trigonometrical levelling

(ii) Observations to find higher elevations & large distances
(or)
Geodetic trigonometrical levelling

(i) Observations to find small elevations & short distances :-

* The principles of plane surveying is adopted
* Distance measurements b/w the stations are ~~at~~ not large, then the effect of curvature or refraction is neglected. or proper correction may be applied linearly to the calculated difference in elevation

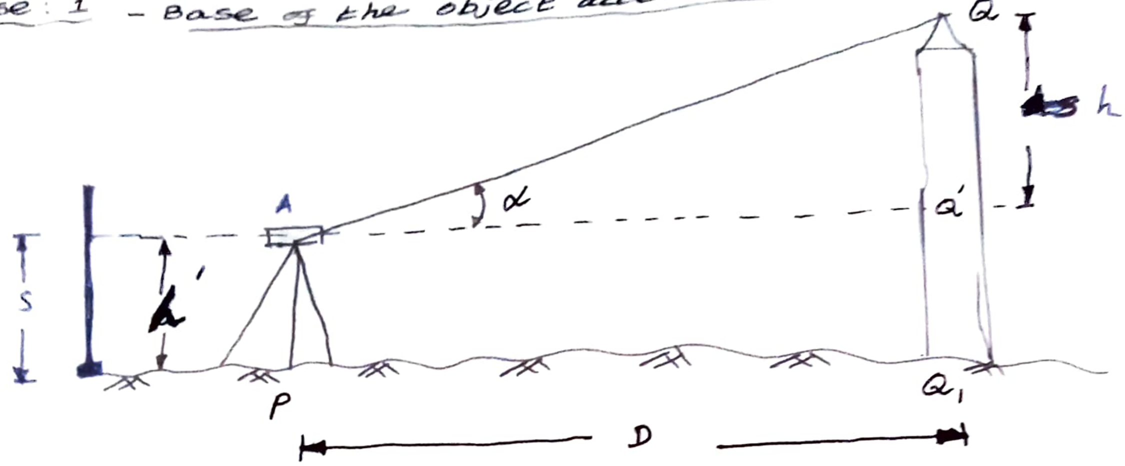
(ii) Observations to find large (higher) elevations & larger distances :-

* It is adopted for geodetic surveying principles
* The effects of curvature & refraction are fully applied
* The corrections of curvature & refraction are applied to all angular measurements.

Surveying

Heights and Distances

case : 1 - Base of the object accessible



Let the horizontal distance b/w the instrument & the object can be measured accurately.

When
- P → Instrument station
- A → centre of Instrument
- Q → points to be observed
- ~~Q1 → reading taken on staff~~

Δ^{le} AQQ'

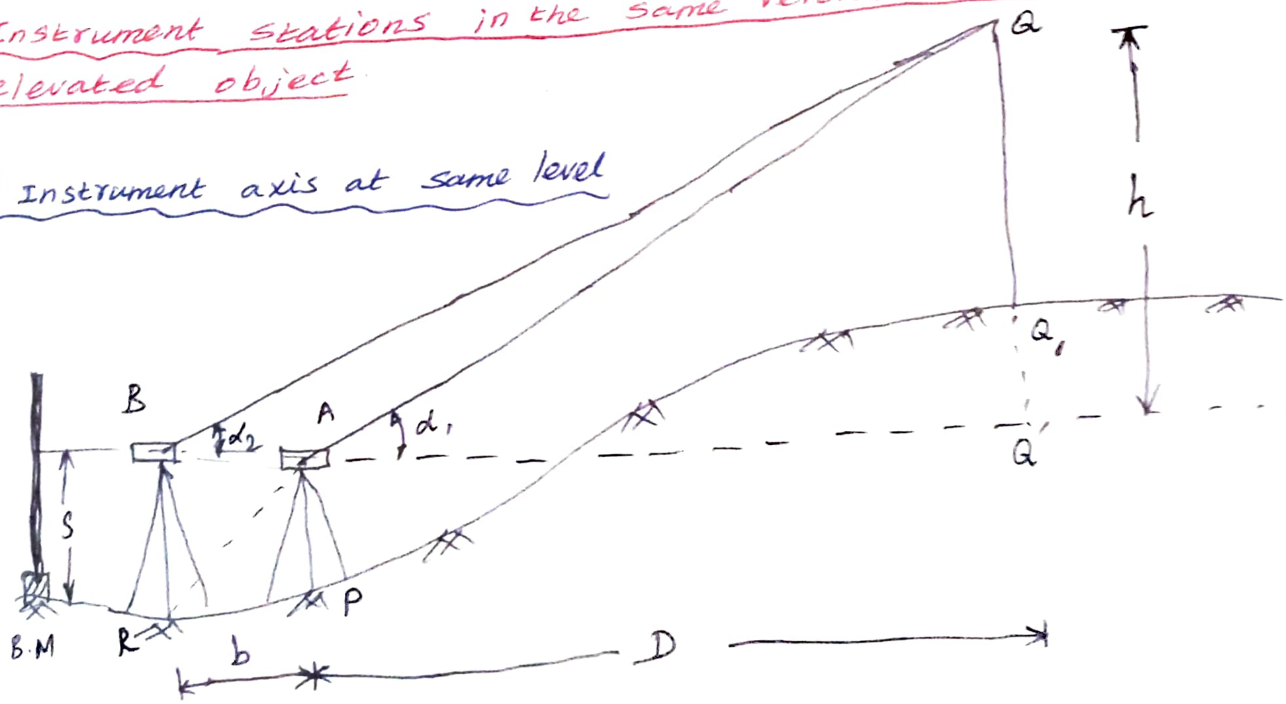
$$\boxed{h = D \tan \alpha}$$

RL of Q = RL of B.M ~~at~~ P + S + h

case : 2 - Base of the objective In-accessible :-

Instrument stations in the same vertical plane with the elevated object

a) Instrument axis at same level



Δ^{le} AQQ'

$$h = D \tan \alpha_1$$

Δ^{le} BQQ'

$$h = (b + D) \tan \alpha_2$$

Equating these two equations

$$D \tan \alpha_1 = (b + D) \tan \alpha_2$$

$$D \tan \alpha_1 - D \tan \alpha_2 = b \tan \alpha_2$$

$$\therefore D(\tan \alpha_1 - \tan \alpha_2) = b \tan \alpha_2$$

$$\therefore \boxed{D = \frac{b \tan \alpha_2}{(\tan \alpha_1 - \tan \alpha_2)}}$$

RL of Q = RL of BM + S + h

Instrument axis at different levels:-

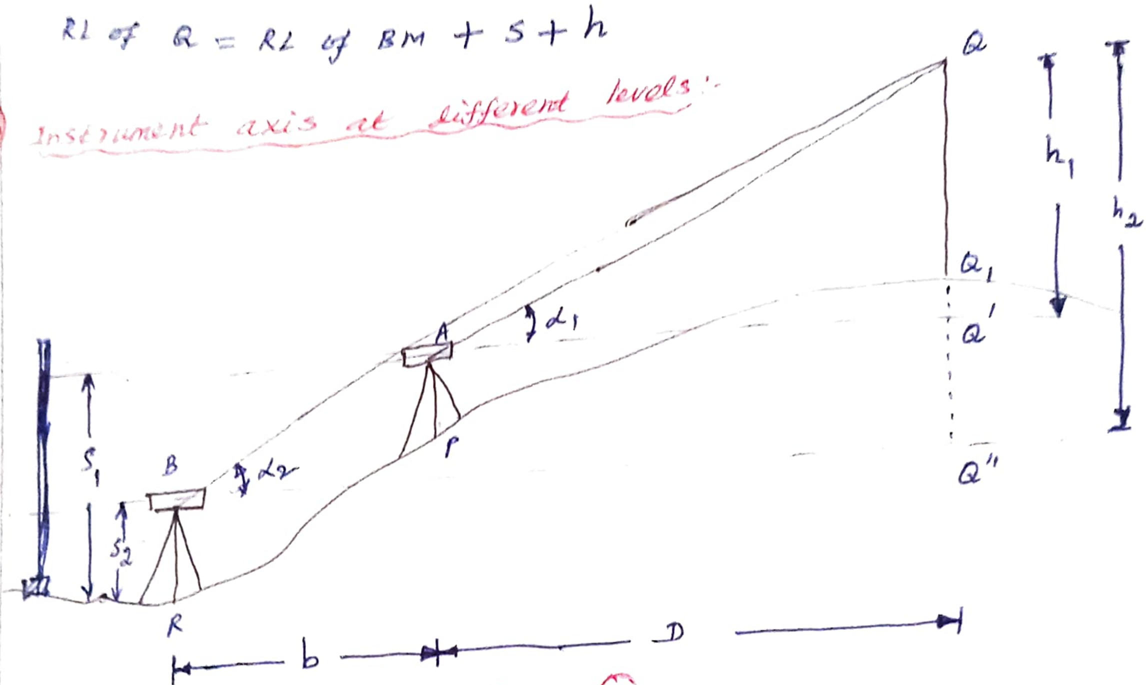


Fig ①

(or)

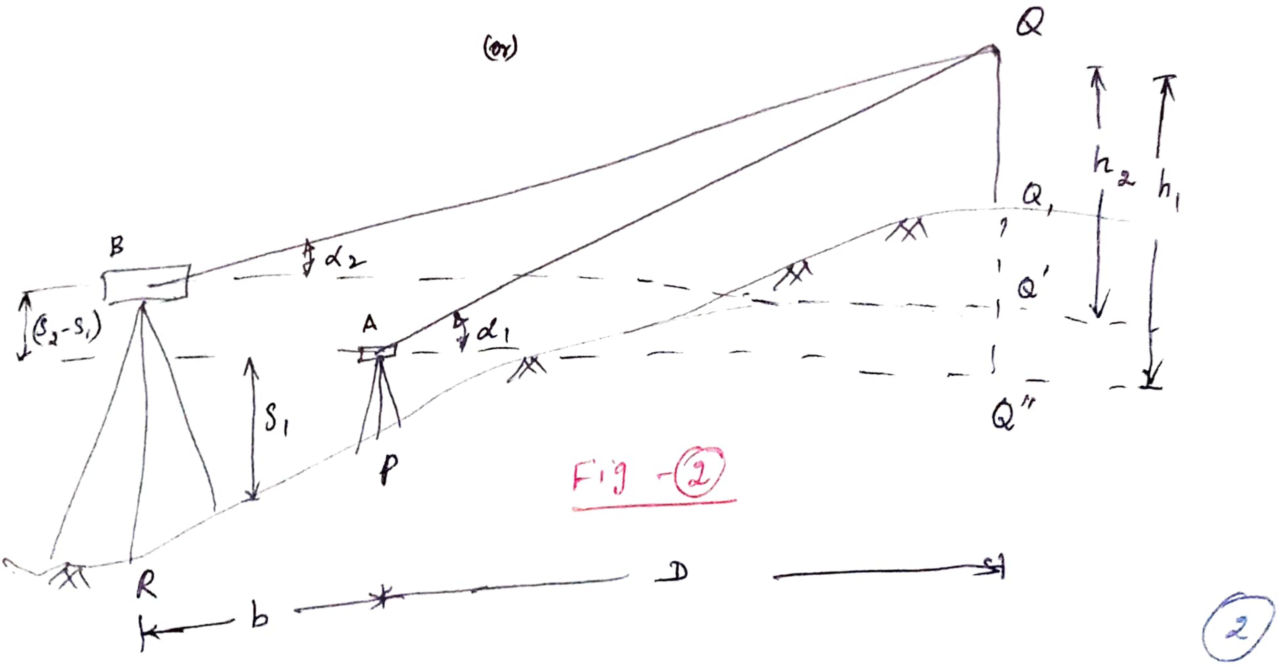


Fig -②

$h_1 = D \tan\alpha_1$

$h_2 = (b+D)\tan\alpha_2$

From fig. (1)

$h_2 - h_1 = (b+D)\tan\alpha_2 - D\tan\alpha_1$

ie, $S = b\tan\alpha_2 + D\tan\alpha_2 - D\tan\alpha_1$

$S = b\tan\alpha_2 + D(\tan\alpha_2 - \tan\alpha_1)$

$S - b\tan\alpha_2 = D(\tan\alpha_2 - \tan\alpha_1)$

$$\therefore D = \frac{S - b\tan\alpha_2}{(\tan\alpha_2 - \tan\alpha_1)}$$

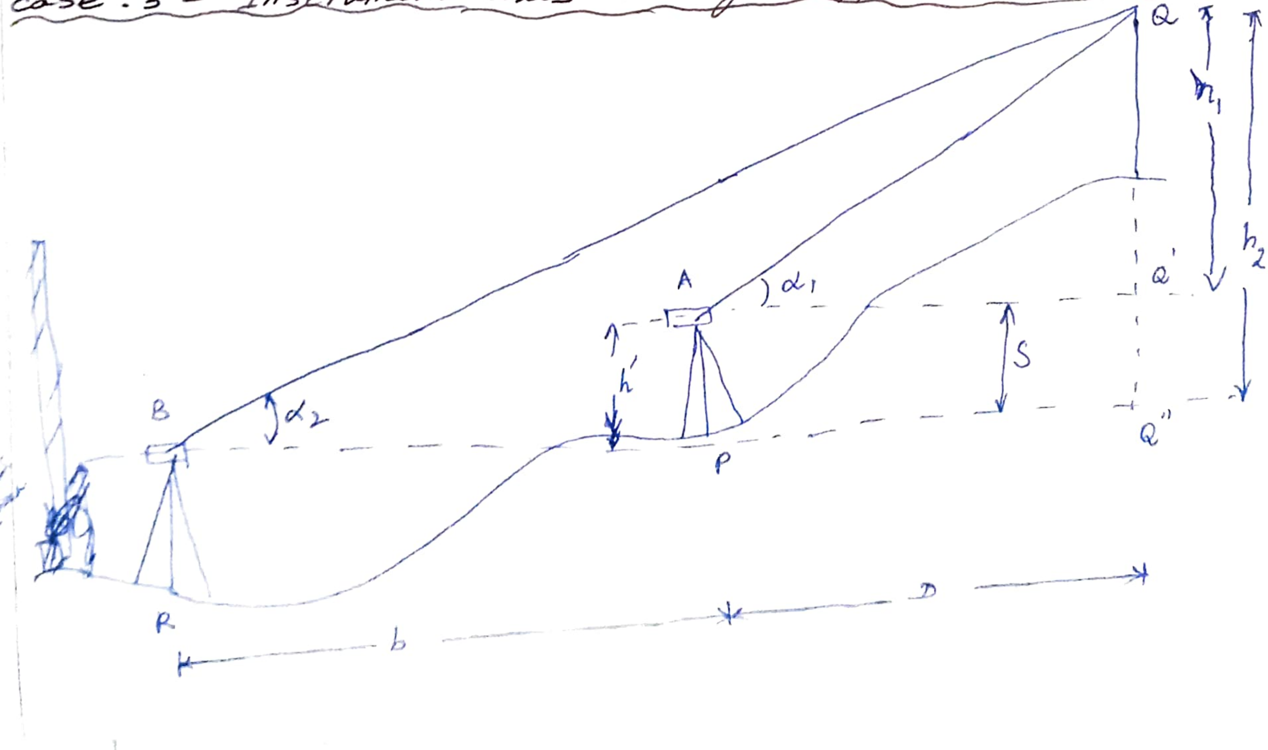
ie, $$\boxed{D = \frac{S + b\tan\alpha_2}{\tan\alpha_1 - \tan\alpha_2}}$$

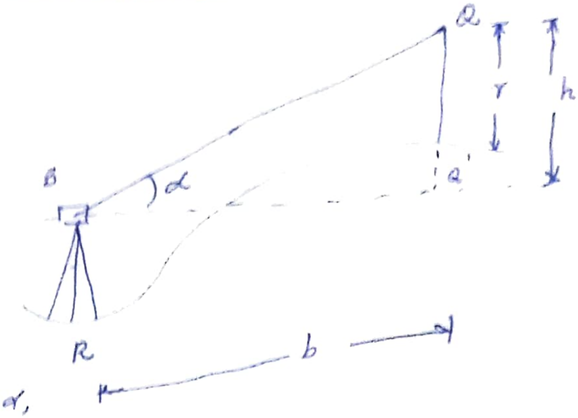
$h_1 = D.\tan\alpha_1$

ie, $$\boxed{h_1 = \frac{(S + b\tan\alpha_2)\tan\alpha_1}{(\tan\alpha_1 - \tan\alpha_2)}}$$

RL of Q = RL of B.M + $S_1 + h_1$

case : 3 – Instrument axis at very different ~~interval~~ levels





$h_1 = D \tan\alpha_1$

$h_2 = (b + D) \tan\alpha_2$

$(h_2 - h_1) = (b + D)\tan\alpha_2 - D\tan\alpha_1$

$S = b\tan\alpha_2 + D\tan\alpha_2 - D\tan\alpha_1$

$S = D(\tan\alpha_2 - \tan\alpha_1) + b\tan\alpha_2$

$$\frac{S - b\tan\alpha_2}{\tan\alpha_2 - \tan\alpha_1} = D$$

ie, $$\boxed{D = \frac{b\tan\alpha_2 - S}{(\tan\alpha_1 - \tan\alpha_2)}}$$

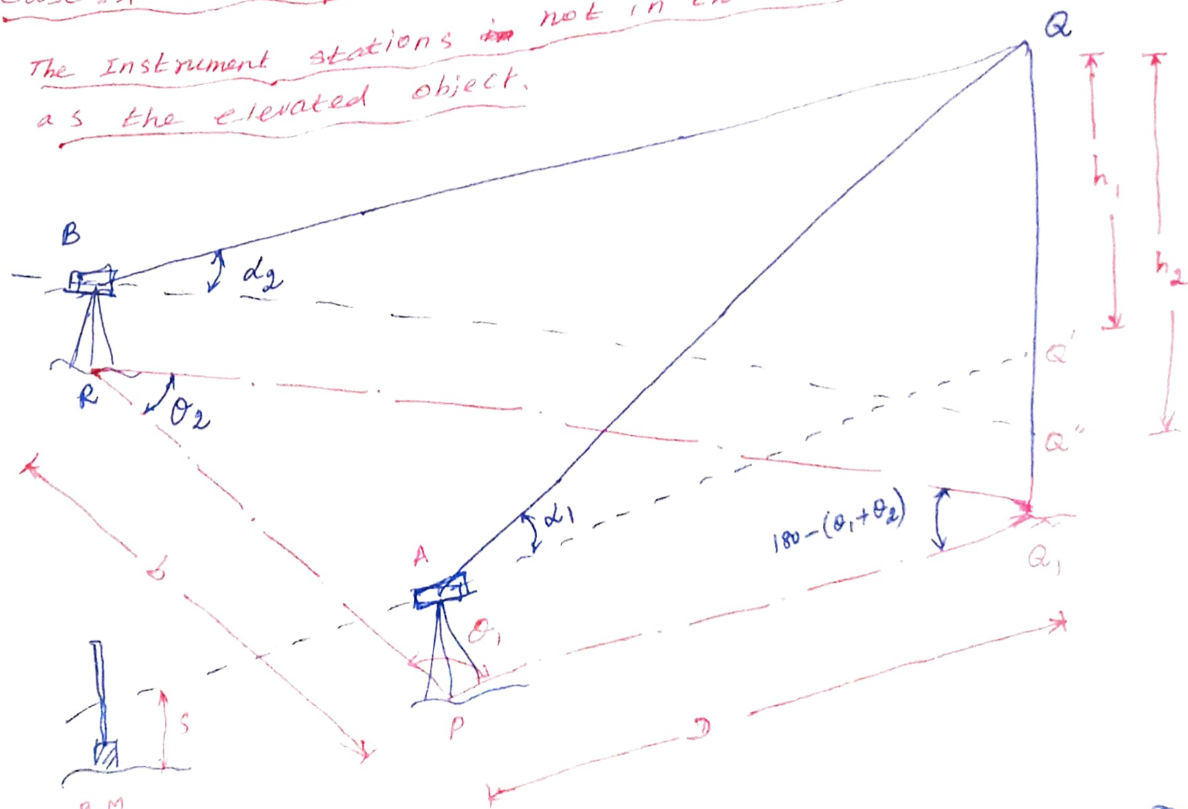
RL of Q = RL of A + h_1 = RL of B + $S + h_1$

RL of Q = RL of BM + B.S. reading at B (at R) + $S + h_1$

Case : A - Base of the object inaccessible

The Instrument stations not in the same vertical plane as the elevated object.

Double plane method



③

From Δ^{le} AQQ',

$$QQ' = h_1 = D \tan\alpha_1 \qquad \text{—— (1)}$$

From Δ^{le} $PQ_1R = 180 - (\theta_1 + \theta_2)$ $\quad \theta_3$

From sine rule

$$\frac{PQ_1}{\sin\theta_2} = \frac{RQ_1}{\sin\theta_1} = \frac{\overset{b}{PR}}{\sin(180-(\theta_1+\theta_2))}$$

i.e.,
$$\frac{D}{\sin\theta_2} = \frac{b}{\sin\theta_3}$$

$$D = \frac{b \cdot \sin\theta_2}{\sin\theta_3}$$

$$RQ_1 = \frac{b \cdot \sin\theta_1}{\sin\theta_3}$$

Substituting the value of D in (1) we get

$$h_1 = D \tan\alpha_1$$
$$= \frac{(b \cdot \sin\theta_2) \cdot \tan\alpha_1}{\sin\theta_3}$$

RL of Q = RL of BM + S + h_1

Check

$$h_1 = RQ_1 \tan\alpha_2 = \frac{b \cdot \sin\theta_1 \tan\alpha_2}{\sin\theta_3}$$

TANGENTIAL SYSTEM

* In tangential method, the horizontal & vertical distances are computed by measuring angles.

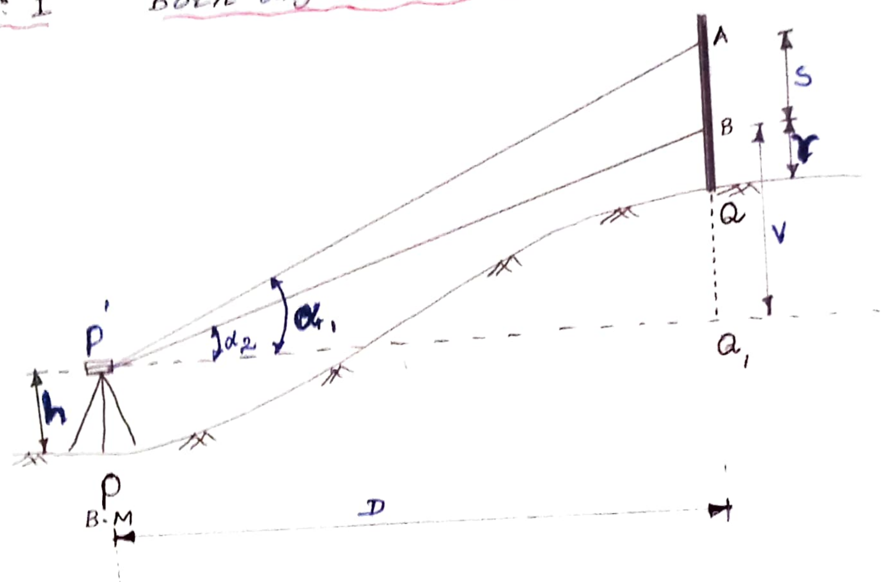
* Two vanes are fixed on the stadia rod or an another target a fixed distance apart.

* These vanes are bisected by the central cross hair. and the vertical angles corresponding to each vane are measured.

* The tangential method is suitable, if the theodolite does not have a stadia diaphragm.

* Two vertical angles are measured - one corresponding to each vane.

Case : 1   Both angles are Angle of Elevation.



Where,

P → Instrument station

Q → Staff station

P' → Position of Instrument axis

V → Vertical distance b/w lower vane & horizontal line of sight

D → Horizontal distance b/w P & Q

S → Staff intercept

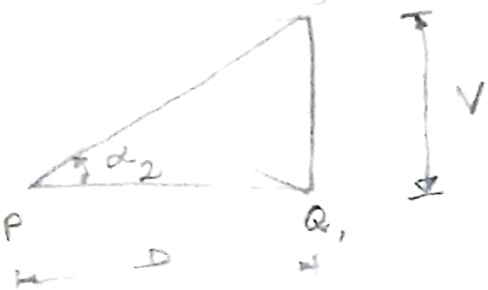
α_1, α_2 → Angle of elevation, corresponding to A & B

$h \rightarrow$ height of instrument

$r \rightarrow$ staff reading at lower vane

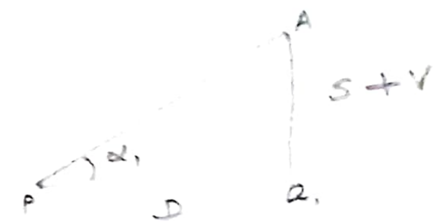
In Δ^{le} $P'BQ_1$

$V = D \tan \alpha_2$ —— ①



In Δ^{le} $P'AQ_1$

$S + V = D \tan \alpha_1$ —— ②



substracting ② − ①

$$S + \cancel{V} - \cancel{V} = D \tan \alpha_1 - D \tan \alpha_2$$

$$S = D \tan \alpha_1 - D \tan \alpha_2$$

$$S = D(\tan \alpha_1 - \tan \alpha_2)$$

$$\boxed{\therefore D = \frac{S}{(\tan \alpha_1 - \tan \alpha_2)}}$$

substituting 'D' values in equation ①

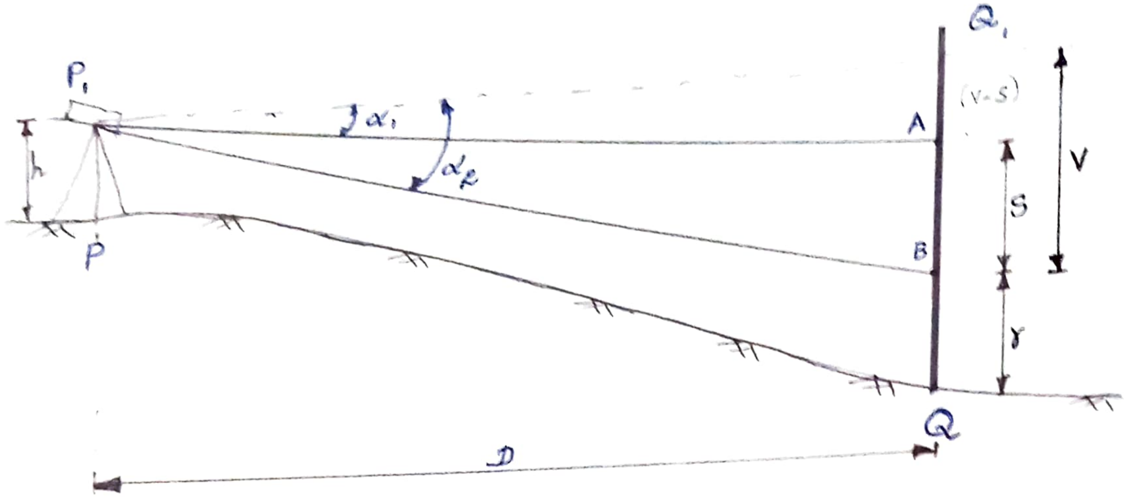
$\therefore$ ① $\Rightarrow$ $V = D \tan \alpha_2$

$$\boxed{V = \frac{S \tan \alpha_2}{(\tan \alpha_1 - \tan \alpha_2)}}$$

R.L of Q

case

Case : 2 Both angles are Angle of depression



In Δ^{le} $P_1 Q_1 A$

$$V - S = D \tan \alpha_1 \quad \text{——— ①}$$

S- V-(V-S)

In Δ^{le} $P_1 Q_1 B$

$$V = D \tan \alpha_2 \quad \text{——— ②}$$

substracting ① - ② we get,

$$\cancel{V} - S - \cancel{V} = D \tan \alpha_1 - D \tan \alpha_2$$

$$-S = D(\tan \alpha_1 - \tan \alpha_2)$$

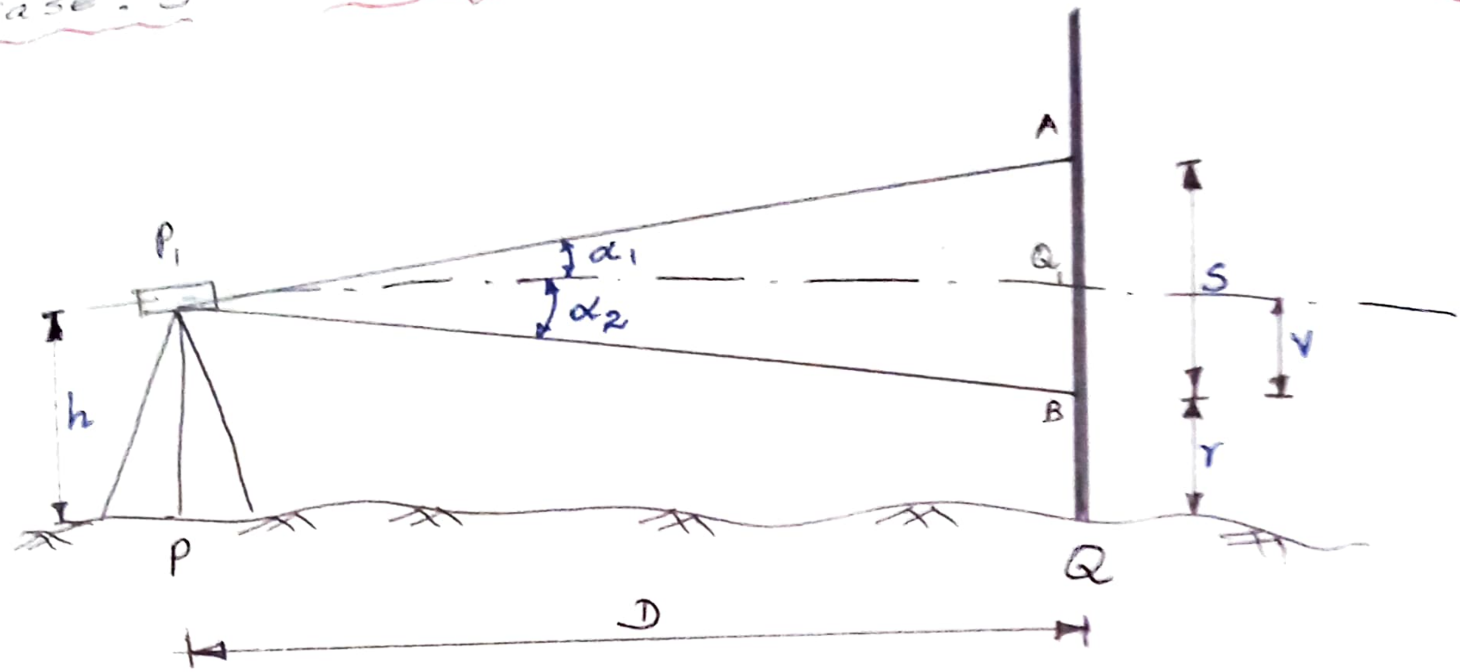
$$\therefore \boxed{D = \frac{S}{\tan \alpha_2 - \tan \alpha_1}}$$

substitute the 'D' values in equation ②

$$\boxed{V = \frac{S \tan \alpha_2}{(\tan \alpha_2 - \tan \alpha_1)}}$$

$$\boxed{RL \text{ of } Q = RL \text{ of BM } P + h - V - \gamma}$$

Case : 3 One angle ...



In Δ^{le} $P_1 Q_1 A$

$$S - V = D \tan \alpha_1 \quad \text{---} \quad (1)$$

In Δ^{le} $P_1 B Q_1$

$$V = D \tan \alpha_2 \quad \text{---} \quad (2)$$

Adding eqn (1) + (2)

$$S - \cancel{V} + \cancel{V} = D \tan \alpha_1 + D \tan \alpha_2$$

$$S = D(\tan \alpha_1 + \tan \alpha_2)$$

$$\boxed{D = \frac{S}{\tan \alpha_1 + \tan \alpha_2}}$$

substituting 'D' values in eqn (2) we get

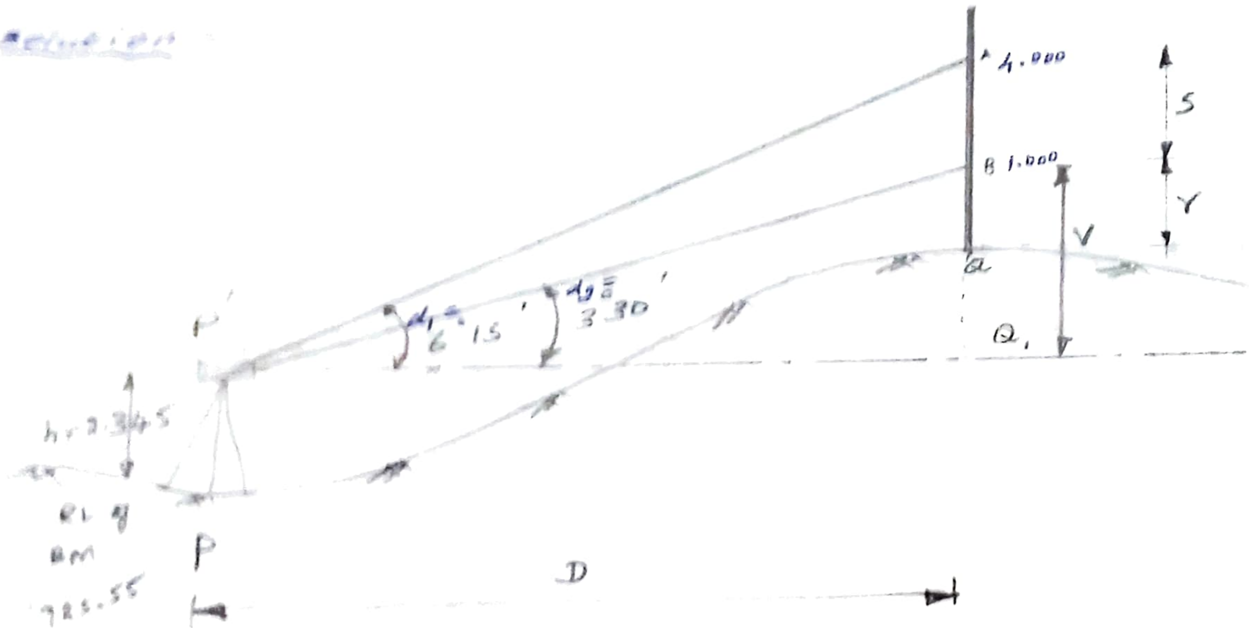
$$\boxed{V = \frac{S \tan \alpha_2}{(\tan \alpha_1 + \tan \alpha_2)}}$$

$$\boxed{RL \text{ of } Q = RL \text{ of } BM \text{ at } P + h - V - r}$$

Problems:

Vertical angles were measured to vanes fixed at the 1m and 4m marks on a staff held at a station Q from the instrument kept at a station P. The vertical angles were 3°30' and 6°15'. The reading at a B.M. of R.L. 785.550 m from P was 2.345m. Find the horizontal distance PQ and the RL of Q.

Solution



$S = 4.000 - 1.000 = 3.00$ m.

$h = 2.345$ m

$\alpha_1 = 6°15'$

$\alpha_2 = 3°30'$

$$\boxed{D = \frac{S}{(\tan\alpha_1 - \tan\alpha_2)}} = \frac{3.000}{(\tan 6°15' - \tan 3°30')}$$

$$\boxed{D = 62.040 \text{ m}}$$

$$V = \frac{S \tan\alpha_2}{(\tan\alpha_1 - \tan\alpha_2)} = D \tan\alpha_2 = 62.04 \times \tan 3°30'$$

$$\boxed{V = 3.795 \text{ m}}$$

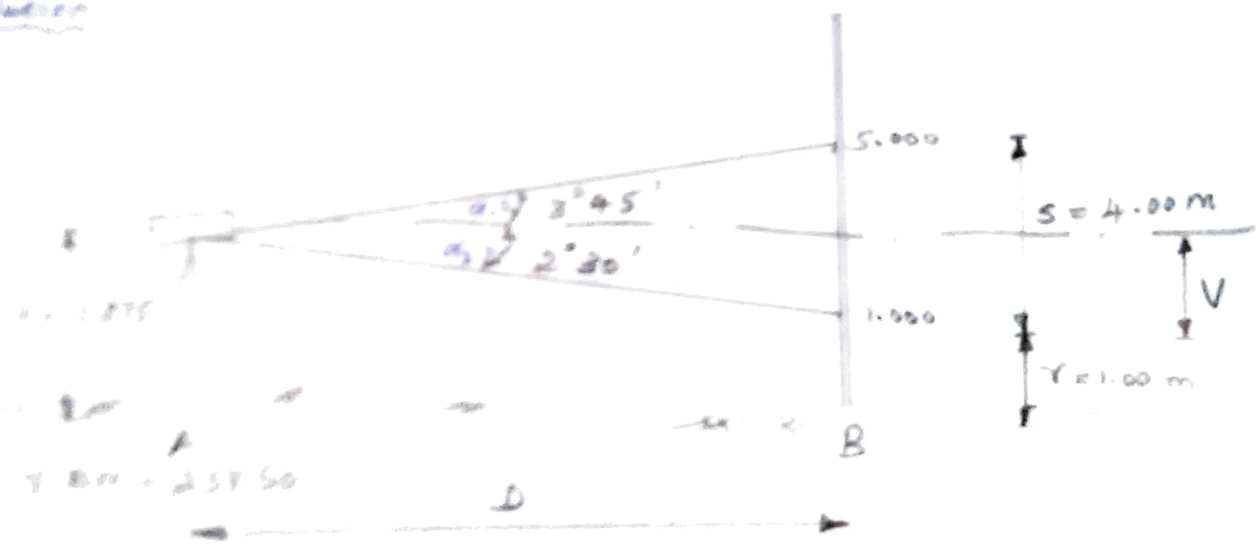
R.L of B = R.L of BM + h + V − r

= [illegible] + [illegible].345 + 3.795 − 1.000

∴ R.L of B = [illegible] m

A theodolite was set up at a station A and vertical angles were measured to vanes kept at a [illegible] B. The angles measured to the 1m & 5m marks were −2°30' and +3°45' respectively. A reading of 1.875 m was also taken on a staff held at a BM of RL 258.50 m. Find the horizontal distance AB & the RL of B.

Solution



$s = 5 - 1 = 4.00\ m$

$r = 1.000\ m$

$\alpha_1 = 3°45'$

$\alpha_2 = 2°30'$

Note: −2°30' (−) sign in angle of depression

$$D = \frac{s}{\tan\alpha_1 + \tan\alpha_2} = \frac{4.00}{(\tan 3°45' + \tan 2°30')}$$

$$\boxed{D = 36.629\ m}$$

$$\boxed{V = \frac{S \tan\alpha_2}{(\tan\alpha_1 + \tan\alpha_2)}} = D \tan\alpha_2$$

$$= 36.629 \times \tan 2^\circ 30'$$

$$\boxed{V = 1.599\ m}$$

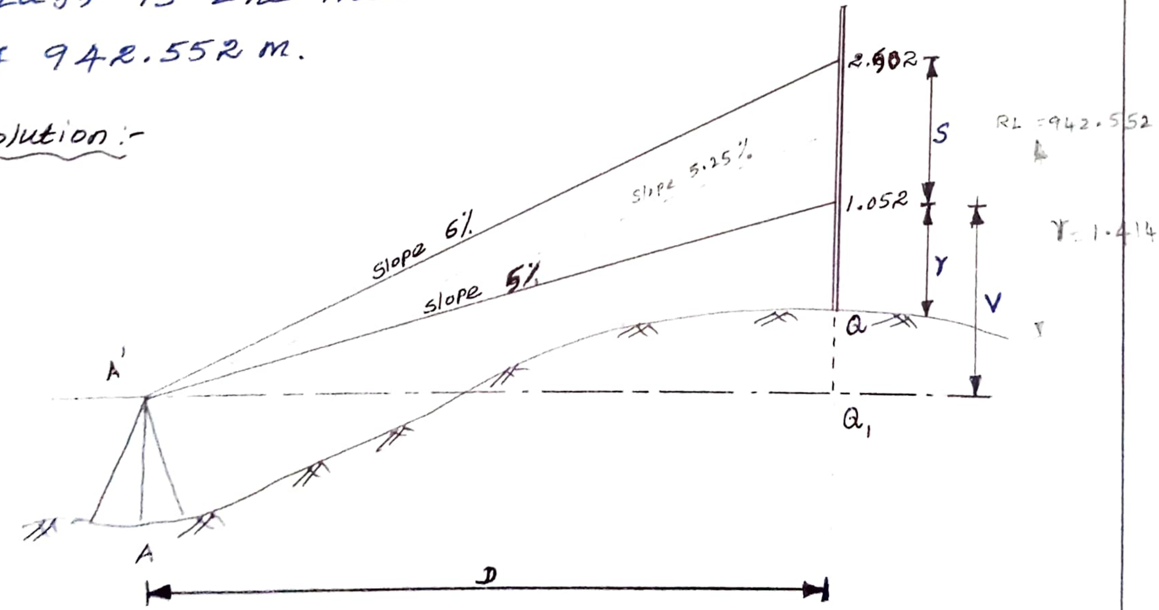
RL of B = RL of BM at A + h − V − r

$$= 258.50 + 1.875 - 1.599 - 1.00$$

$$\boxed{RL \text{ of } B = 257.776\ m}$$

3) An observation with percentage theodolite gave staff readings are 1.052 and 2.502 for angle of elevation of 5% and 6% respectively. on sighting the graduation corresponding to the instrument axis above the ground in the vertical angle was 5.25%. compute the horizontal distance and elevation of the staff is the instrument station has an elevation of 942.552 m.

Solution :-



$S = 2.502 - 1.052 = 1.450$ m

$\tan \alpha_1 = 6\% = \frac{6}{100} = 0.06$

$\tan \alpha_2 = 5\% = \frac{5}{100} = 0.05$

$$D = \frac{S}{\tan \alpha_1 - \tan \alpha_2} = \frac{1.450}{0.06 - 0.05}$$

$\boxed{D = 145 \text{ m}}$

$$V = \frac{S \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = D \tan \alpha_2 = 145 \times 0.05$$

$\boxed{V = 7.25 \text{ m}}$

Let the angles to the graduation corresponding to the height of instrument be $\alpha_3 = 5.25\%$ so that the ~~reading~~ staff intercept,

$\tan \alpha_3 = \frac{5.25}{100} = 0.0525$

$S' =$ staff intercept

$\tan \alpha_1 = \frac{6}{100} = 0.06$

$D = 145$ m

$$D = \frac{S'}{\tan \alpha_1 - \tan \alpha_3} = \frac{S'}{0.06 - 0.0525} = 145$$

$\therefore S' = 145 \times (0.06 - 0.0525)$

$\boxed{S' = 1.088 \text{ m.}}$

Let, r' be the staff reading to the height of instrument

$r = 2.502 - 1.088$

$\boxed{r = 1.414 \text{ m}}$

since the staff readings sighting the graduation corresponding to the line of sight through the instrument axis is 1.414 m.

$\therefore$ RL of Q = RL of Inst. station − r
at 5.25 X

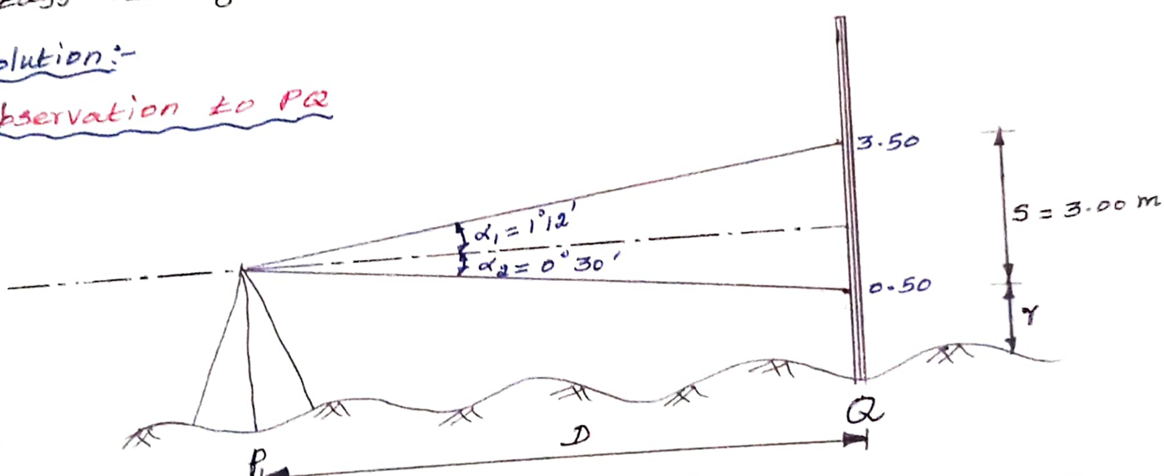
= 942.552 − 1.414

$\boxed{\text{RL of Q} = 941.138 \text{ m}}$

The vertical angles to vane fixed at a staff station Q observed from the instrument station 'P' are 0.50 m and 3.50 m above the foot of the staff held vertically were −0°30' and +1°12' respectively. Then sighted to the another instrument station on to the vanes fixed at the staff station Q are 1 m and 3.50 m above the foot of the staff held vertically. The vertical angles were 2°30' and 5°40' respectively. Find the horizontal distance PQ and QR. Also determine the RL of Q if the level of instrument axis is 125.380 m above the datum when the staff is sighted from instrument at station P.

Solution:-

Observation to PQ



$\alpha_1 = 1°12'$; $\alpha_2 = 0°30'$; $S = 3.5 - 0.5$

$S = 3.00$ m

$$D = \frac{S}{\tan\alpha_1 + \tan\alpha_2} = \frac{3.00}{\tan 1°12' + \tan 0°30'}$$

$\boxed{D = 101.990 \text{ m}}$

$$\boxed{V = \frac{S \tan\alpha_2}{\tan\alpha_1 + \tan\alpha_2}} = D \cdot \tan\alpha_2$$

$V = 101.990 \times \tan 0^\circ 30'$

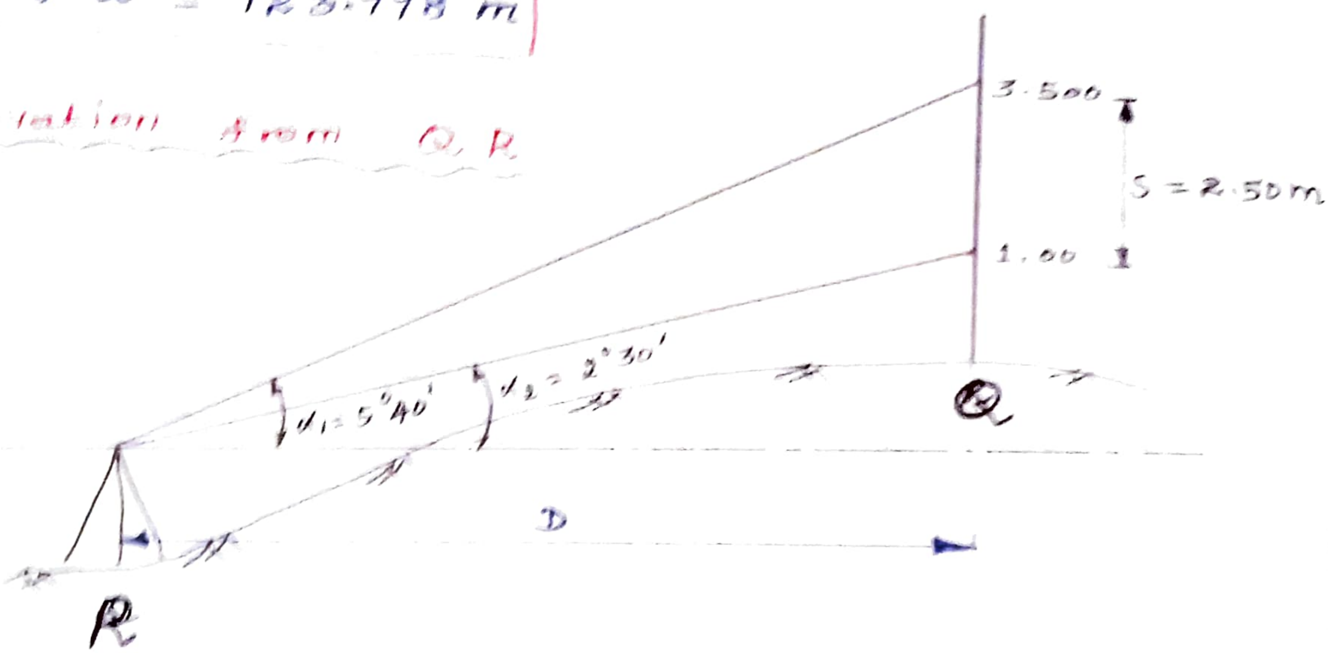
$\boxed{V = 0.882 \text{ m}}$

RL of Q = RL of Inst. axis − V − r

= 125.380 − 0.882 − 0.50

$\boxed{\text{RL of Q} = 123.998 \text{ m}}$

observation from Q, R



$$\boxed{D = \frac{S}{\tan\alpha_1 - \tan\alpha_2}} = \frac{2.50}{\tan 5^\circ 40' - \tan 2^\circ 30'}$$

$\boxed{D = 44.993 \text{ m}}$

$$\boxed{V = \frac{S \tan\alpha_2}{\tan\alpha_1 - \tan\alpha_2}} = D \tan\alpha_2 = 44.993 \times \tan 2^\circ 30'$$

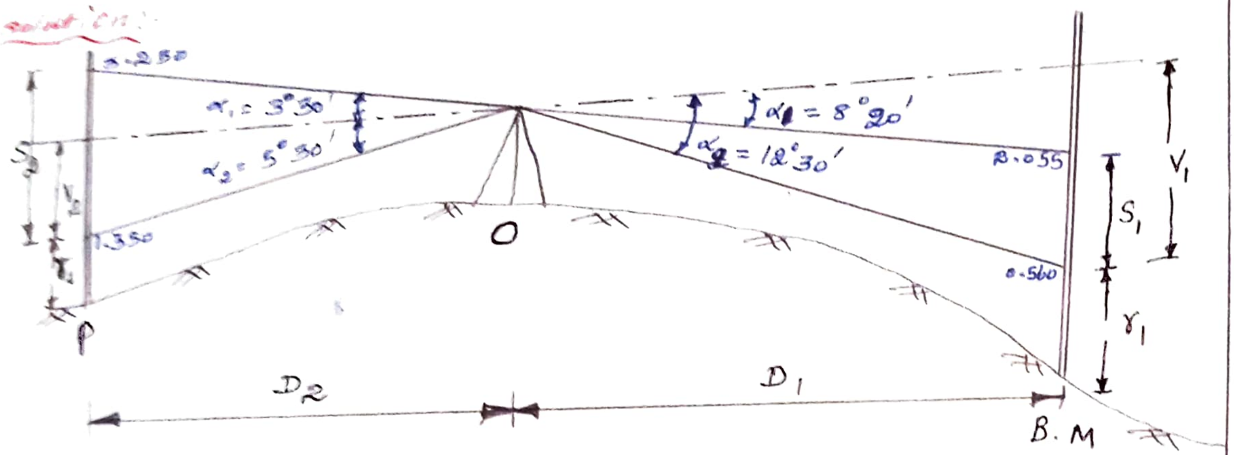
$\boxed{V = 1.964 \text{ m}}$

Two observations are taken by a transit theodolite from station O one to B M with RL of 515.600 m and the other to station 'P' the observations are recorded as under.

Instrument station	Staff at station	Target	Vertical angle	Staff reading	Remarks
O	B.M	lower	$-12°30'$	0.560	RL of B.M = 515.600 m
		upper	$-8°20'$	2.055	
O	P	lower	$-5°30'$	1.350	
		upper	$+3°30'$	3.250	

Find the RL of P and the distance BM and station P.

Solution:



Observation to B.M

$s_1 = 2.055 - 0.560 = 1.495$ m.

$r_1 = 0.560$ m

$$D_1 = \frac{S_1}{\tan\alpha_2 - \tan\alpha_1} = \frac{1.495}{\tan 12°30' - \tan 8°20'}$$

$\boxed{D_1 = 19.876 \text{ m}}$

$V_1 = D_1 \tan\alpha_2 = 19.876 \times \tan 12°30'$

$\boxed{V_1 = 4.406 \text{ m}}$

RL of Inst. axis = RL of BM + r_1 + V_1

= 516.600 + 0.560 + 3.406

$\boxed{\text{RL of Inst. axis} = 520.566\ m}$

Observation to P

$S_2 = 3.250 - 1.350 = 1.900\ m$

$r_2 = 1.350\ m$

$$D_2 = \frac{S_2}{\tan\alpha_2 + \tan\alpha_1} = \frac{1.900}{\tan 3^\circ 30' + \tan 5^\circ 30'}$$

$$\boxed{D_2 = 12.067\ m}$$

$$V_2 = \frac{S_2 \cdot \tan\alpha_2}{\tan\alpha_1 + \tan\alpha_2} = D_2 \tan\alpha_2 = 12.067 \times \tan 5^\circ 30'$$

$$\boxed{V_2 = 1.162\ m}$$

RL of P = RL of Ht of Instrument axis $-V_2 - r_2$

= 520.566 − 1.162 − 1.350

$$\boxed{\text{RL of P} = 518.054\ m}$$

Distance b/w BM and station 'P'

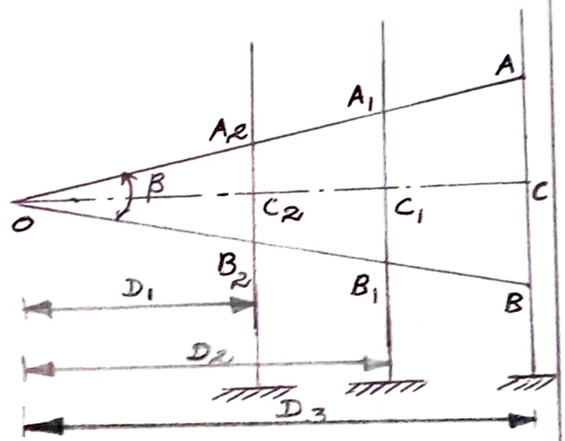
= 19.876 + 12.067

= 31.943 m.

Stadia Method :-

Principle of stadia Method :-

* To determine the horizontal and vertical distance between the two points.
* The stadia method is based on the principle that the ratio of the perpendicular to the base is contant in similar isosceles triangles.



Let two rays OA & OB are equally inclined to OC

Let A_2B_2, A_1B_1 and AB are the staff intercept.

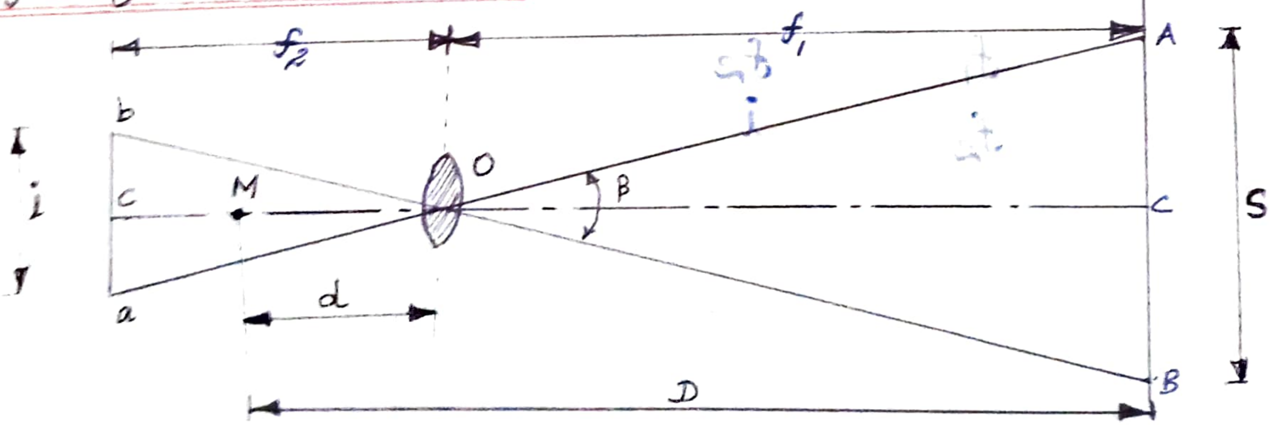
ie, $\dfrac{OC_2}{A_2B_2} = \dfrac{OC_1}{A_1B_1} = \dfrac{OC}{AB} = \text{constant} = k$

$k = \frac{1}{2} \cot \beta/2$ $\beta = 34°22.6$

$\boxed{D = ks + C}$ k → multiplying constant
C → Additive constant

* When the two points are nearly at same elevation then the line of sight will be horizontal.
* When the two points are at different elevation then the line of sight will be inclined.

Line of sight is horizontal :- → Fixed Hair Method :-



* The horizontal distance and elevation can be determined as follows

* Let D be the distance between the two points of staff to the instrument point 'O'

* Considered the point O is an optical centre of the objective of an external focussing telescope.

* Let b, c & a is the corresponding top, axial and bottom hairs of the diaphragm.

* Let A, B & C are the points cut by the three line of sight corresponding to the three wires.

* ab = i = stadia interval (or) interval b/w stadia hair

* AB = S = staff intercept

f = focal length of the objective

f_1 = Horizontal distance of the staff from the optical centre of the objective.

f_2 = Horizontal distance of the ~~staff from~~ cross wires from 'O'.

d = Distance b/w the optical centre 'O' to the vertical axis

* The rays Bob and AOa pass through the optical centre. they are straight.

From the similar Δ^{le} AOB & Δ^{le} aob

$$\frac{OC}{AB} = \frac{Oc}{ab}$$

$$\frac{f_1}{S} = \frac{f_2}{i} \quad \text{(or)} \quad \frac{f_1}{f_2} = \frac{S}{i}$$

By the lens formula

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2}$$

For multiplying ff_1 on both side

$$\frac{ff_1}{f} = \frac{ff_1}{f_1} + \frac{ff_1}{f_2}$$

$$f_1 = f + \frac{f f_1}{f_2} \quad \text{———} (3)$$

substituting these values $\left(\frac{f_1}{f_2} = \frac{s}{i}\right)$ in equation (3)

$$f_1 = f + \frac{f f_1}{f_2} \quad \text{———} (3)$$

$$f_1 = f + f \cdot \frac{s}{i}$$

From the figure, The horizontal distance b/w the axis the staff is $D = f_1 + d$

ie, $\boxed{D - d = f_1}$

$$f_1 = f + f \cdot \frac{s}{i}$$

$$D - d = f + f \cdot \frac{s}{i}$$

$$D = f + f \cdot \frac{s}{i} + d$$

ie, $D = (f + d) + \left(f \cdot \frac{s}{i}\right)$

sine D is not contant

The constant $K = \frac{f}{i}$ is known as multiplying constant

$C = (f + d)$ is known as additive constant.

$$\therefore \boxed{D = KS + C}$$

Where, K & C are the tacheometric constants

Where
D = horizontal Distance
K = multiplying constant
S = staff intercept
C = Additive constant

Problem:1

The two distances of 20m and 100m were accurately measure and intercept on the staff between the outer stadia works ~~where~~ were 0.196 m at the fore distance and 0.996 m at the later distance. calculate the tacheometric constants.

Given Data:-

$D_1 = 20\ m$

$D_2 = 100m$

$S_1 = 0.196\ m$

$S_2 = 0.996\ m$

To find:

Tacheometric constant

$K = ?$

$C = ?$

Solution:-

$$D_1 = KS_1 + C \quad \text{———} \; (1)$$

$$D_2 = KS_2 + C \quad \text{———} \; (2)$$

$$(2) - (1) \Rightarrow \quad D_2 - D_1 = KS_2 - KS_1 + \cancel{C} - \cancel{C}$$

$$D_2 - D_1 = KS_2 - KS_1$$

$$\therefore D_2 - D_1 = K(S_2 - S_1)$$

$$\therefore \boxed{K = \frac{D_2 - D_1}{S_2 - S_1}}$$

Substitute the 'K' values in equation (1)

$$D_1 = KS_1 + C \quad \text{———} \; (1)$$

$$D_1 = \left(\frac{D_2 - D_1}{S_2 - S_1}\right) S_1 + C$$

$$C = D_1 - \left(\frac{D_2 - D_1}{S_2 - S_1}\right) S_1$$

$$C = \frac{D_1(S_2 - S_1) - (D_2 - D_1) S_1}{(S_2 - S_1)}$$

$$C = \frac{D_1 S_2 - D_1 S_1 - D_2 S_1 + D_1 S_1}{S_2 - S_1}$$

$$\boxed{C = \frac{D_1 S_2 - D_2 S_1}{S_2 - S_1}}$$

$D_1 = 20\,m$
$D_2 = 100\,m$
$S_1 = 0.196\,m$
$S_2 = 0.996\,m$

$$C = \frac{(20 \times 0.996) - (100 \times 0.196)}{(0.996 - 0.196)}$$

$$\boxed{C = 0.40}$$

To find k:-

$$k = \frac{D_2 - D_1}{S_2 - S_1} = \frac{100 - 20}{0.996 - 0.196} = \frac{80}{0.8}$$

$$\boxed{k = 100}$$

Result:-

Tacheometric constant

Multiple constant $k = 100$

Additive constant $c = 0.40$

Problem: 2

A tacheometer was set up at station P and observations were taken on a staff held at Q, the vertical circle reading being zero. The readings were 1.980 m, 1.660 m and 1.340 m. The reading from P to a staff held at a B.M of elevation 1020.50 m was 2.85 m. Find the distance PQ and the elevation of point Q. The instrument constants were 100 and 0.50.

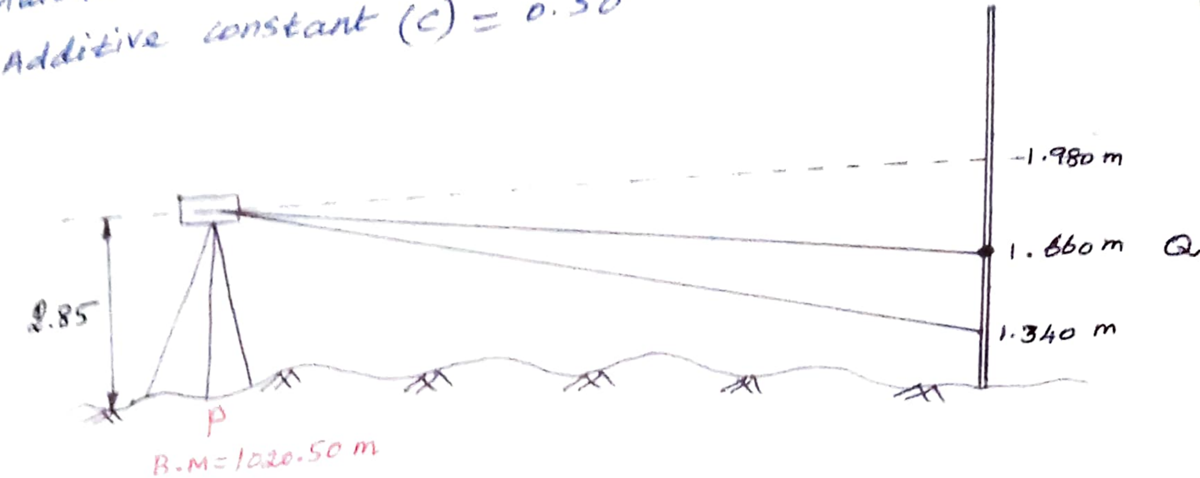
Given Data:-

$S_1 = 1.980\,m$; $S_2 = 1.660\,m$; $S_3 = 1.340\,m$

$S = S_1 - S_3 = 1.980 - 1.340 = 0.640\,m$

Multiple constant $(K) = 100$

Additive constant $(C) = 0.50$



The vertical circle reading being zero

RL of B.M at Inst. station $P = 1020.50 m$

Ht of Instrument $h = 2.85 m$.

Elevation of Line of sight $= \text{RL of BM at } P + h$

$= 1020.50 + 2.85$

$= 1023.350 m$.

$\therefore$ Elevation of point Q $= \text{Line of sight} - S_2$

$= 1023.350 - 1.660$

$= 1021.690 m$.

Problem : 3

Find the stadia constants K and C from the following Data

Instrument at	observation to	Distance	Staff readings
P	Q	50 m	1.354, 1.603, 1.852
P	R	100 m	1.152, 1.650, 2.149

The line of sight was horizontal in both cases.

Solution :-

$$D = ks + c$$

For the observation from P to Q

$$D = ks + c \quad \text{---} \quad (1)$$

$D = 50m$
$s_1 = 1.354$; $s_2 = 1.603$
$s_3 = 1.852$

$$s = s_3 - s_1$$
$$= 1.852 - 1.354$$
$$s = 0.498 \, m.$$

(1) $\Rightarrow$ $50 = k \times 0.498 + c$

$50 = 0.498k + c$ ———————— (2)

For the observation from P to R

$D = 100m$; $s_1 = 1.152m$; $s_2 = 1.650$
$s_3 = 2.149 m$

Staff intercept $(s) = s_3 - s_1 = 2.149 - 1.152$

$$s = 0.997 \, m$$

(1) $\Rightarrow$ $100 = k \times 0.997 + c$

$100 = 0.997k + c$ ———————— (3)

Solving these equations (2) & (3)

(2) $\Rightarrow$ $50 = 0.498k + \cancel{c}$
(3) $\Rightarrow$ $\overset{(-)}{100} = \overset{(-)}{0.997}k \overset{(-)}{+} \cancel{c}$

(2) $-$ (3) $\cdots -50 = -0.499k + 0$

$$\boxed{k = 1002}$$

Substitute the k values in eqn. ②

$$50 = 0.498 \times 100 + C$$

$$C = 50 - 49.8$$

$$\boxed{C = 0.2}$$

Ans

$k = 100$

$C = 0.30$

Problem : 4 :

The readings on a staff held vertically 60 m from a tacheometer were 1.460 and 2.055. The line of sight was horizontal. The focal length of the objective lens was 24 cm and the distance from the objective lens to the vertical axis was 15 cm. calculate the stadia interval.

Given Data :-

focal length $(f) = 24 \text{ cm} = 0.24 \text{ m}$

distance from the objective lens $(d) = 15 \text{ cm} = 0.15 \text{ m}$

Staff intercept $(S) = 2.055 - 1.460 = 0.595 \text{ m}$.

$D = 60 \text{ m}$.

To find :-

Stadia interval $(i) = ?$

Solution :-

$$D = KS + C \quad \text{(or)} \quad D = \left(\frac{f}{i} S\right) + (f + d)$$

$$K = \frac{f}{i} \quad ; \quad C = (f + d)$$

$c = f + d = 0.24 + 0.15 = 0.39\,m$

$D = k s + c$

$40 = k \times 0.3954 + 0.39$

$k = \dfrac{40 - 0.39}{0.3954} = 100.18$

$k = 100.18$

$c = 0.39$

$k = \dfrac{f}{i}$

$100.18 = \dfrac{0.24}{i}$

$i = \dfrac{0.24}{100.18} = 2.40 \times 10^{-3}\,m$

$i = 2.4\,mm$

Fixed hair method → Line of sight is inclined or vertical.

Case i) → staff held vertical

In this case, the vertical angle can be an angle of elevation or depression.

Angle of elevation:

where, P → instrument station

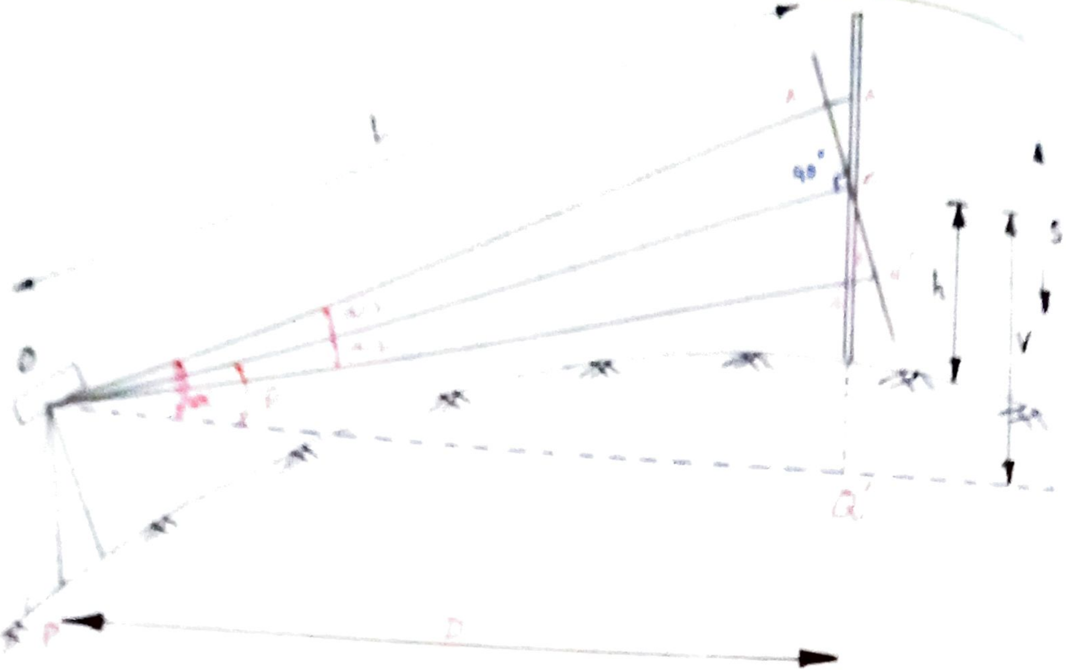
Q → staff station

S → staff intercept

θ → Inclination of the line of sight from the horizontal

L → length measured along line of sight

V → vertical intercept



r → central hair reading
h = height of Instrument
α → Angle b/w the two extreme rays to stadia hair

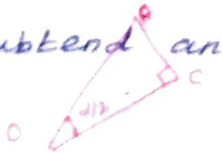
Draw a line $A'CB'$ normal to the line of sight OC

From right angle triangle $OQ'C$ 

$$\angle OCQ' = 90 - \theta$$

$$\angle BCB' = \theta \quad (\text{as } CB' \text{ is } \perp^r \text{ to } OC)$$ 

$$\angle ACA' = \angle BCB' = \theta$$ 

Let the stadia hairs subtend an angle α, 
then

$$\angle COA' = \alpha/2$$

$$\angle CAO = 90 - \frac{\alpha}{2}$$

$$\angle CA'A = 180 - \left(90 - \frac{\alpha}{2}\right)$$

$$= 90 + \frac{\alpha}{2}$$

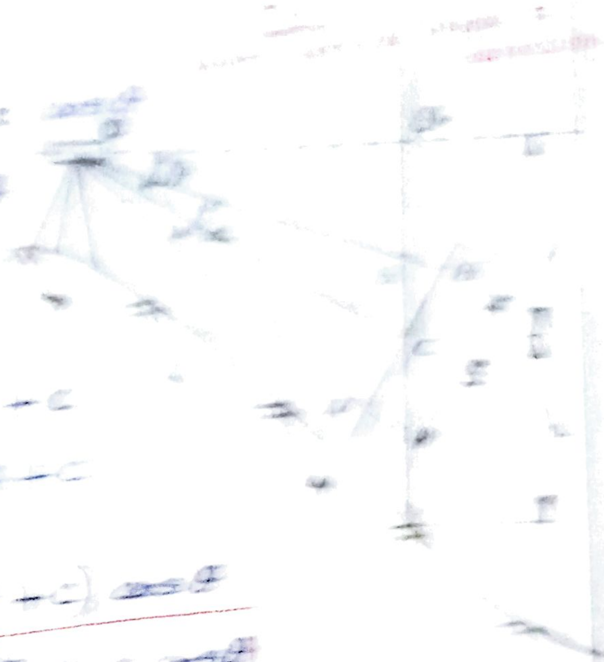
The value of $\frac{\alpha}{2}$ is very small.
Hence the triangles $AA'C$ & $BA'C$ may be assumed right angles

$AB = AC - BC$

$= AC \cos\theta + BC \cos\theta$

$= (AC + BC)\cos\theta$

$= S\cos\theta$



Constant distance FC

$L = k \cdot AB + C$

$L = kS\cos\theta + C$

But $D = L\cos\theta$

$= (kS\cos\theta + C)\cos\theta$

$D = kS\cos^2\theta + C\cos\theta$

$V = GC = L\sin\theta$

$= (kS\cos\theta + C)\sin\theta$

$= kS\cos\theta\sin\theta + C\sin\theta$

$= \frac{1}{2}kS\sin 2\theta + C\sin\theta$

$V = \frac{kS\sin 2\theta}{2} + C\sin\theta$

Elevation of staff station for angle of elevation

$= H.I + V - h$

Elevation of staff for angle of depression $= H.I - V - h$

[illegible]

A tacheometer was set up at station [illegible] and observations were made to a staff held normal to the line of sight over point [illegible]. The vertical angle measured was [illegible]. The three hair readings were [illegible]. The reading from [illegible] ... [illegible] ... horizontal ... [illegible] ... the instrument constants are [illegible] ... [illegible] staff held normal

Given Data:

$s_1 = 1.905$ m $s_2 = 2.480$; $s_3 = 3.055$ m

$\therefore$ Staff intercept $(s) = 3.055 - 1.905$

$s = 1.150$ m

Multiplying constant $k = 100$

Additive constant $c = 0.50$

Vertical angle $\theta = 6^\circ 36'$

RL of B.M = 852.55 m ; $h = 1.855$ m

To find:-

RL of Q = ?

Solution:

Condition:- staff held Normal to the line of sight

Horizontal distance $(D) = L\cos\theta$

Where, $L \rightarrow$ inclined length

$L = ks + c$

$$\therefore \boxed{D = (ks + c)\cos\theta}$$

$$D = (100 \times 1.150 + 0.50) \times \cos 6^\circ 36'$$

$$\boxed{D = 114.735 \text{ m}}$$

$$\boxed{V = (ks + c)\sin\theta}$$

Where,

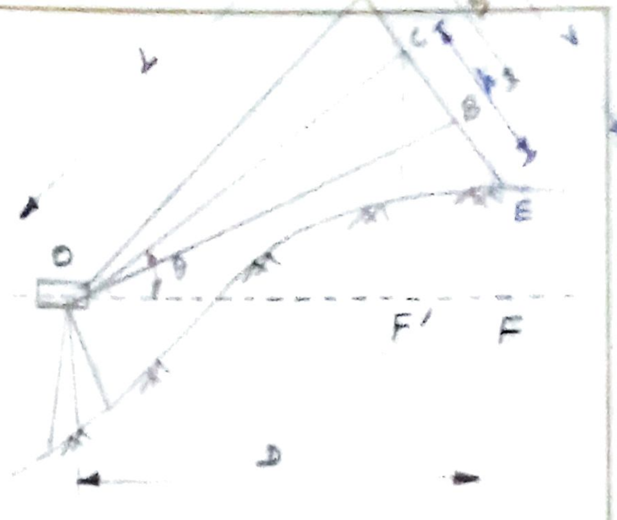
$V \rightarrow$ Vertical height from inst. height to the middle hair reading

$V = (100 \times 1.15 + 0.50) \times \sin 6^\circ 36' = 13.275$ m

RL of Line of sight = RL of BM + h = 852.55 + 1.855 = 854.4

RL of Q = 854.405 + 13.275 − 1.855 = 865.825 m

Staff held Normal :



Staff E is held normal to the line of sight AC.

∴ The staff intercept AB is normal to the line of sight OC

Line of sight at an angle of Elevation

Fig: Angle of elevation

Let,

$AB = S$ = staff intercept

$CE = h$ = central hair reading

θ = angle of elevation

$OC = L$ = inclined distance

Drop perpendicular CF' to horizontal OF

$$L = ks + c$$

$$OF' = (ks+c)\cos\theta$$

But $D = OF' + F'F$

$$\boxed{D = (ks+c)\cos\theta + h\sin\theta}$$

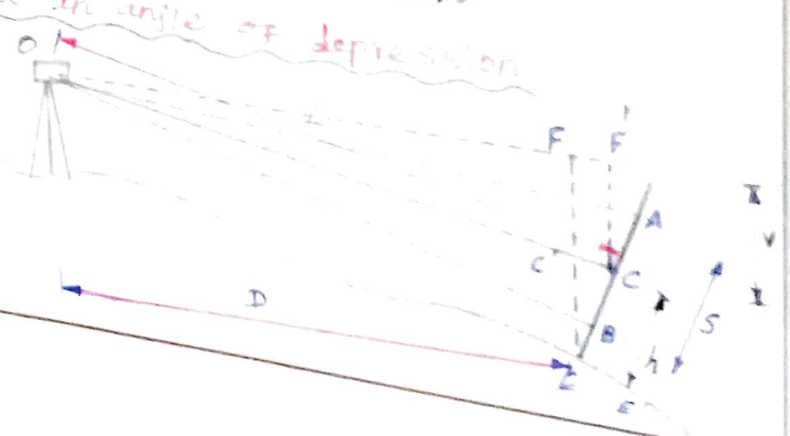
Elevation of the staff station $V = OC \sin\theta$

$= L \sin\theta$

$$\boxed{V = (ks+c)\sin\theta}$$

Elevation of staff station (R.L of E) $= H.I + V - h\cos\theta$ (R.L of line of sight)

Line of sight at an angle of depression



$L = ks + c$

$OF' = L\cos\theta = (ks+c)\cos\theta$

Now, $D = OF' - FF'$

$= OF' - EE'$

$$\boxed{D = (ks+c)\cos\theta - h\sin\theta}$$

Elevation of staff station,

$V = OC\sin\theta$

$= L\sin\theta$

$$\boxed{V = (ks+c)\sin\theta}$$

Elevation of staff station (RL of E), $= H.I - V - h\cos\theta$

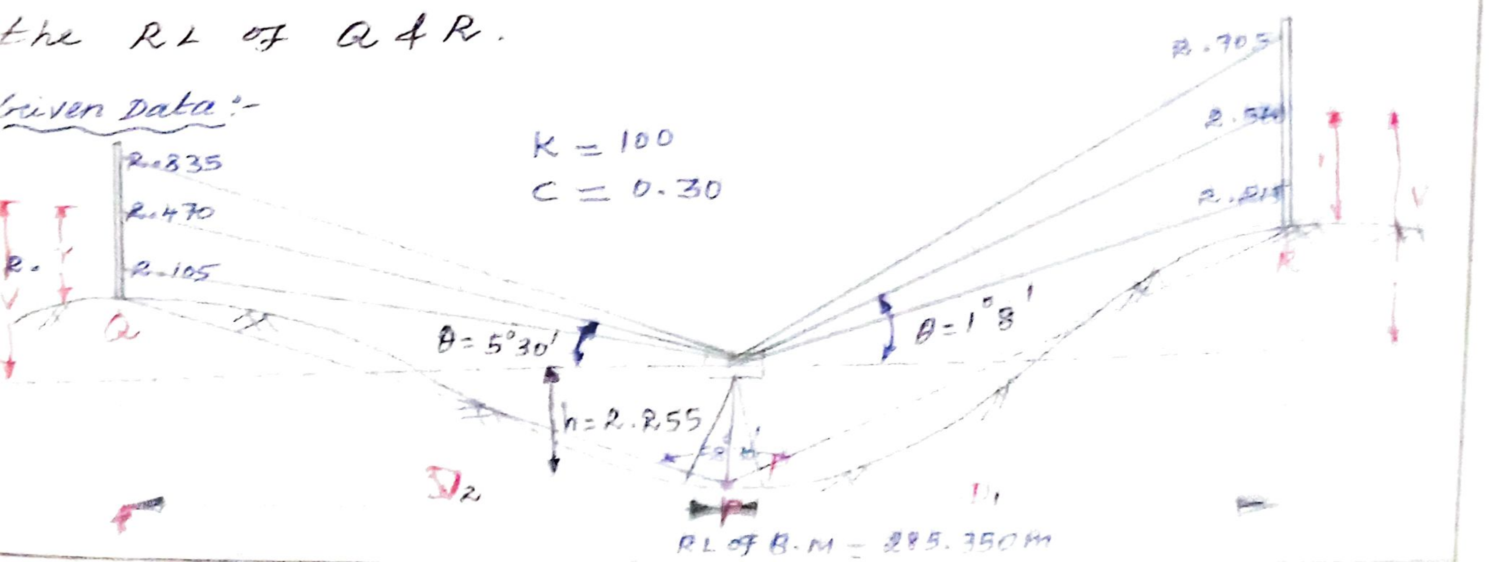
Problem:-

A tacheometer was set up at station 'P' and observations were made to two stations 'Q' & 'R'. The vertical angles to 'Q' & 'R' were $5^\circ30'$ & $1^\circ08'$ respectively. The cross hair readings at 'Q' were 2.105, 2.470 & 2.835 and those at 'R' were 2.215, 2.560 and 2.905. The staff was held vertical in both cases. The instrument constants were 100 & 0.30. The reading from P to a B.M of RL 285.35 m was 2.255. The horizontal angle QPR measured was $58^\circ30'$. Find the distance QR, the gradient from Q to R and the RL of Q & R.

Given Data:-

$K = 100$

$C = 0.30$



observations ~~a~~ from Q to P

$\theta = 5°30'$; staff readings = 2.105, 2.470 & 2.835

$k = 100$; $s = 2.835 - 2.105$

$C = 0.30$; $S = 0.730\ m$

$r = 2.470\ m$

Horizontal distance,

$\boxed{D = ks\cos^2\theta + c\cos\theta}$ → For staff held vertical

$= 100 \times 0.730 \times \cos^2 5°30' + 0.30 \times \cos 5°30'$

$\boxed{D_2 = 72.628\ m}$

Vertical distance

$\boxed{V = \frac{ks\sin 2\theta}{2} + c\sin\theta}$

$= \frac{100 \times 0.730 \times \sin 2\times 5°30'}{2} + 0.3 \times \sin 5°30'$

$\boxed{V = 7.025\ m}$

R.L. of Line of sight = RL of B.M at P + h

= 285.350 + 2.255

= 287.605 m

RL of Q = RL of line of sight + V − r

= 287.605 + 7.025 − 2.470

RL of Q = 292.160 m

~~the~~

observations to R :

$\theta = 1°8'$; staff readings = 2.215, 2.560 & 2.905 m

$k = 100$; $S = 2.905 - 2.215$

$c = 0.30$; $S = 0.690\ m$

$r = 2.560\ m$

Horizontal distance $D_1 =$

$$\boxed{D = Ks\cos^2\theta + C\cos\theta}$$

$= 100 \times 0.690 \times \cos^2 1°08' + 0.3 \times \cos 1°08'$

$D_1 = 69.273\ m$

Vertical distance

$$\boxed{V = \frac{Ks\sin 2\theta}{2} + C\sin\theta}$$

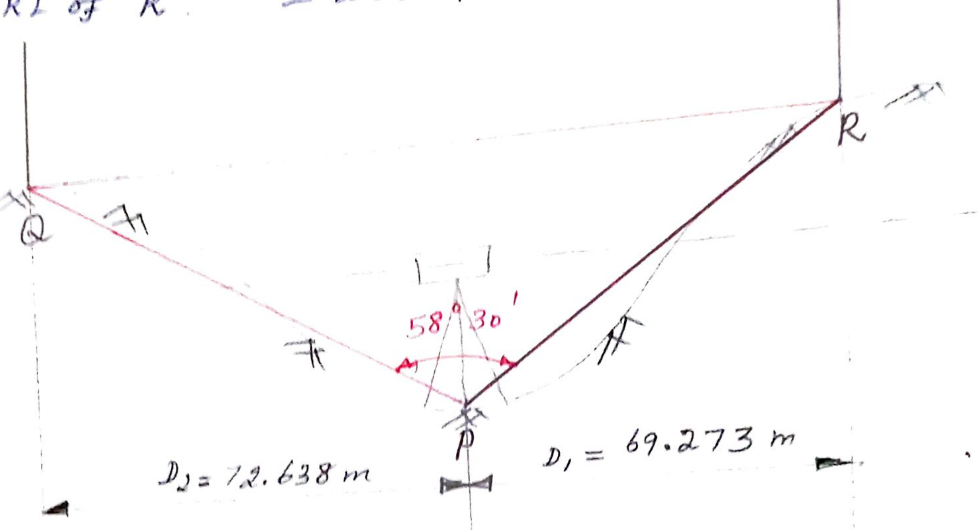
$= \frac{100 \times 0.690 \times \sin 2 \times 1°8'}{2} + 0.3 \times \sin 1°8'$

$V = 1.371\ m.$

RL of B.M at R = RL of line of sight + V − r

$= 287.605 + 1.371 - 2.560$

RL of R $= 286.416\ m$



$\therefore QR^2 = PQ^2 + PR^2 - 2\,PQ.PR\cos\theta$

$= \left[(72.638)^2 + (69.273)^2\right] - 2 \times 72.638 \times 69.273 \times \cos 58°30'$

$QR^2 = 4777.605$

$\therefore \boxed{QR = 69.12\ m}$

Gradient from Q to R

Difference in elevation of Q & R =

RL of Q ~ RL of R

= 292.160 ~ 286.416

= 5.744 m

$$\text{Gradient from Q to R} = \frac{\text{Difference in elevation of Q \& R}}{\text{Horizontal Length of QR}}$$

$$= \frac{5.744}{69.12} = 0.083$$

$$\boxed{\text{Gradient} = 1 \text{ in } 12}$$

Problem:-

To determine the elevation of a point P, a tacheometer was set up at a station 'A' & observations were made to staff held vertically at 'P'. check. the instrument was set up another point 'B' and observations were taken to a staff held at 'P'. The RL of the B.M was 235.455 m. The instrument constants were 100 and 0.30. Determine the RL of 'P' from the following data recorded.

Instrument at	Staff at	vertical angle	Hair reading	Reading at BM
A	P	3° 45'	2.235, 2.795 3.355	1.75
B	P	2° 30'	0.945, 1.490, 2.035	2.25

D = ... V = 7.3...

RL of line of sight = ...

Solution:-

Instrument at A & Staff at P

$\theta = 3° 45'$

Staff readings = 2.235, 2.795 & 3.355 m.

$\therefore$ S = 3.355 − 2.235 = 1.120 m

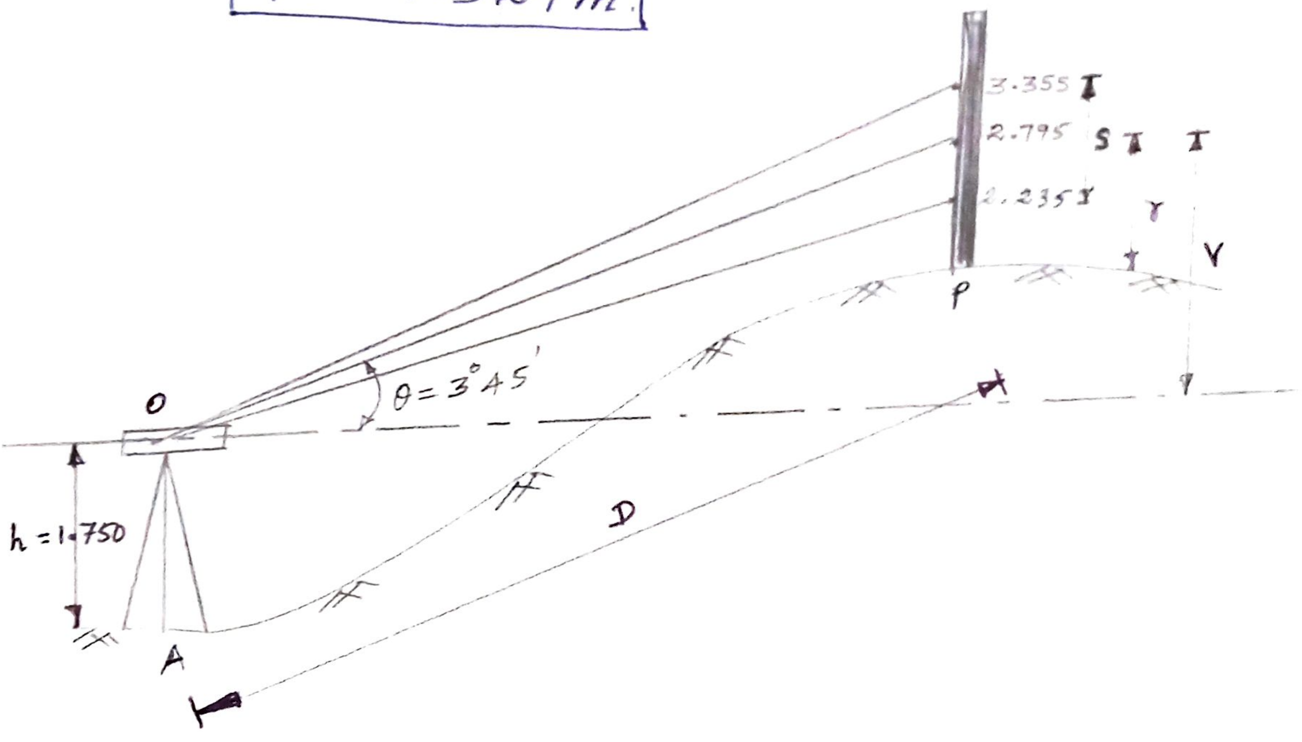
$k = 100$
$c = 0.30$

Horizontal distance

$$D = ks\cos^2\theta + c\cos\theta$$
$$= 100 \times 1.12 \times \cos^2 3^\circ 45' + 0.3 \times \cos 3^\circ 45'$$
$$\boxed{D = 111.82\,m}$$

Vertical distance

$$V = \frac{ks\sin 2\theta}{2} + c\sin\theta$$
$$= \frac{100 \times 1.12 \times \sin 2 \times 3^\circ 45'}{2} + 0.3 \times \sin 3^\circ 45'$$
$$\boxed{V = 7.329\,m}$$



RL of B.M = 235.455 m

RL of Line of sight = RL of BM + h = 235.455 + 1.750
= 237.205 m

RL of P = RL of Line of sight + V − r
= 237.205 + 7.329 − 2.795

$\boxed{\text{RL of P} = 241.718\ m}$

observations from [illegible]

$\theta = 2^\circ 30'$

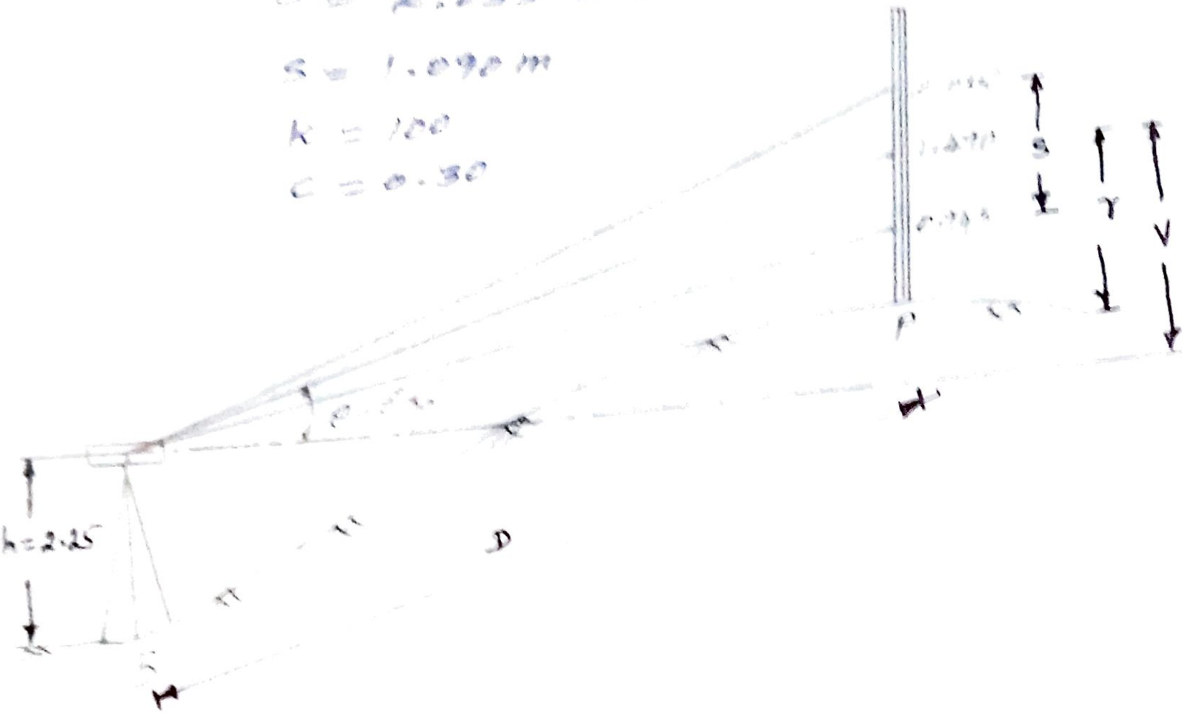
staff readings = 0.945 , 1.490 & 2.035

$s = 2.035 - 0.945$

$s = 1.090\ m$

$k = 100$

$c = 0.30$



$$D = ks\cos^2\theta + c\cos\theta$$

$$= 100 \times 1.09 \times \cos^2 2^\circ 30' + 0.3 \times \cos 2^\circ 30'$$

$$\boxed{D = 109.092\ m}$$

$$V = \frac{ks \sin 2\theta}{2} + c\sin\theta$$

$$= \frac{100 \times 1.09 \times \sin 2 \times 2^\circ 30'}{2} + 0.3 \times \sin 2^\circ 30'$$

$$\boxed{V = 4.763\ m}$$

RL of BM = 235.455 m

RL of Line of sight = RL of BM + h = 235.455 + 2.250

= 237.705 m

RL of P = RL of Line of sight + V − r

= 237.705 + 4.763 − 1.490

RL of P = 240.978 m

Problem :-

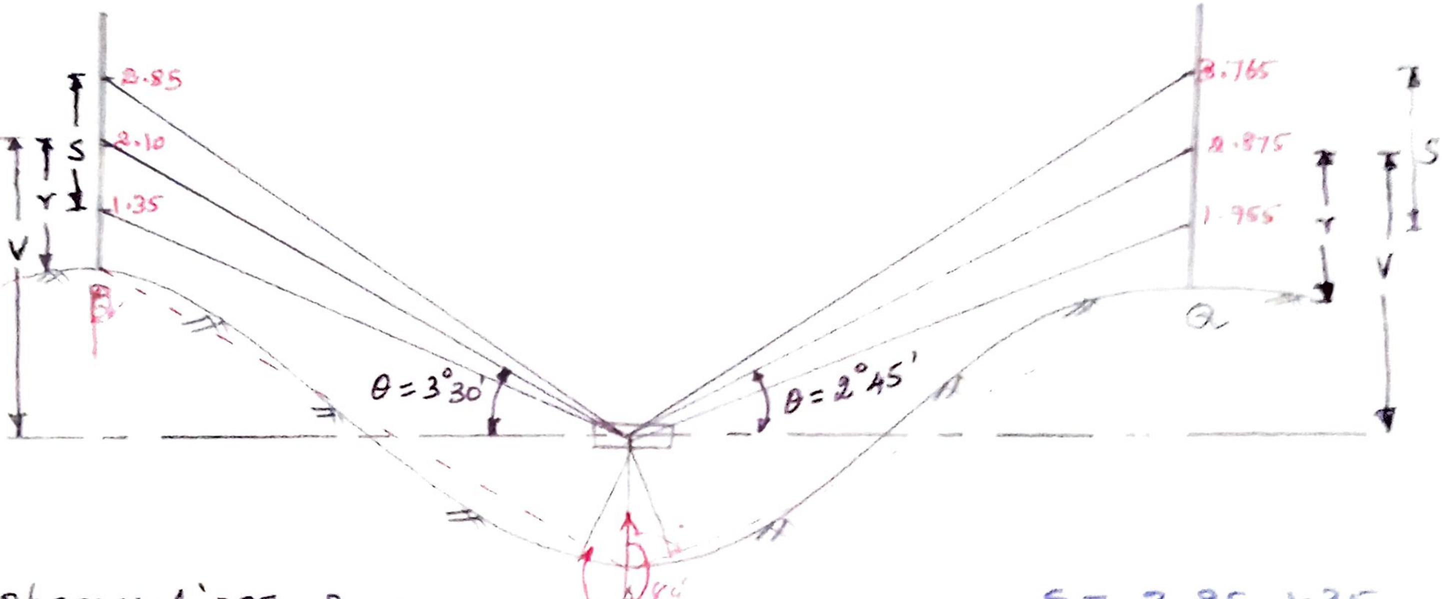
Find the gradient from P to Q using the data is given in table

Instrument at	Staff at	Line	Bearing	Vertical angle	Cross hair readings
A	P	AP	$84°26'$	$3°30'$	1.35, 2.10, 2.85
A	Q	AQ	$142°24'$	$2°45'$	1.955, 2.875, 3.765

The staff was held normal to the line of sight in both cases. Assume $K=100$, $C=0.30$

Solution :-

Condition :- The staff held Normal



Observations from A to P

$S = 2.85 - 1.35$

$S = 1.50\,m$

$D = (KS + C)\cos\theta$

$= (100 \times 1.50 + 0.3) \times \cos 3°30'$

$\boxed{D = 150.020\,m}$

$V = (KS + C)\sin\theta \quad = (100 \times 1.50 + 0.3)\sin 3°30'$

$\boxed{V = 9.176\,m}$

Assuming the horizontal line of sight as datum

Elevation of Point $P = V - r = 9.176 - 2.10$

$\boxed{\text{Elevation of point } P = 7.076\,m}$

observations from A to Q.

$\theta = 2°45'$

$s = 3.765 - 1.955$

$= 1.810 m$

$D = (ks + c)\cos\theta$

$= [(100 \times 1.81) + 0.3] \times \cos 2°45'$

$\boxed{D = 181.091 m}$

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

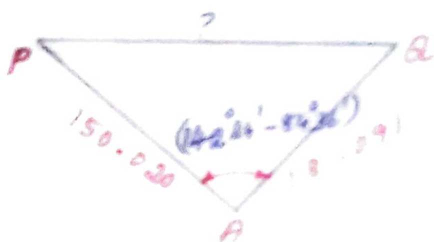
$= (100 \times 1.81 + 0.3)\sin 2°45'$

$\boxed{V = 8.689 m}$

Assuming horizontal line of sight as datum

Elevation of $Q = V - r = 8.689 - 2.875$

$\boxed{\text{Elevation of } Q = 5.814 m.}$



Gradient from P to Q:

$\theta = 142°24' - 84°36'$

$= 57°48'$

using cosine formula

$PQ^2 = AP^2 + AQ^2 - 2 AP . AQ \cos\theta$

$= 150^2 + 181.091^2 - 2 \times 150 \times 181.091 \times \cos 57°48'$

$PQ = 162.309 m.$

Gradient from P to Q $= \dfrac{\text{Elevation of P} \sim \text{Elevation of Q}}{\text{Length of PQ}}$

$= \dfrac{7.076 \sim 5.814}{162.309} = 0.00778$

$= 1 \text{ in } 128$

Problem

To determine the tacheometric constants K & C, the instrument was set up at O. Distances of 30 m, 60 m & 90 m were carefully measured and stations P, Q & R were carefully marked. A stadia rod was kept at the three stations and the following readings were obtained

Instrument at	staff at	Distance from O	cross hair readings
O	P	30	1.135, 1.284, 1.433
O	Q	60	1.025, 1.325, 1.624

Determine the instrument constants.

Solution

Two distance equations can be formed and solved for the constants.

$$30 = KS + C$$

$$30 = K(1.433 - 1.135) + C$$

$$30 = 0.298\,K + C \quad \text{———} \quad (1)$$

$$60 = KS + C$$

$$60 = K(1.624 - 1.025) + C$$

$$60 = 0.599\,K + C \quad \text{———} \quad (2)$$

(2) − (1) $\Rightarrow$

$$\begin{array}{rcl} 60 & = & 0.599\,K + \cancel{C} \\ (-)\,30 & = & (-)\,0.298\,K\ (-)\!+\cancel{C} \\ \hline 30 & = & 0.301\,K \end{array}$$

$\boxed{K = 99.67}$ $\boxed{C = 0.299}$

Find the tacheometric constants from the observations

Instrument at	Staff at	Distance	Vertical angle	Staff readings
P	A	80	2° 24'	1.285 2.185
P	B	140	1° 36'	0.905 2.332

Solution

The horizontal distance is given by

$D = K s \cos^2\theta + C \cos\theta$

$80 = K(2.185 - 1.285) \cdot \cos^2 2°24' + C \cos 2°24'$

$80 = 0.899 K + 0.999 C$ ---- (1)

$140 = K(2.332 - 0.905) \cdot \cos^2 1°36' + C \cos 1°36'$

$140 = 1.426 K + 0.9996 C$ (2)

Solving equations (1) & (2)

$K = 99.88$

$C = 0.63$

Two sets of tacheometric readings were taken from an instrument station A (R.L = 100.000 m) to a staff station B as shown below

Instruments	P	Q
Multiplying constant	100	95
Additive constant	0.30	0.45
Height of instrument	1.45 m	1.45 m
Staff held	vertical	normal

Instrument	Instrument station	Staff station	Vertical angle	Staff readings
P	A	B	5°44'	1.090, 1.440, 1.795
Q	A	B	5°44'	?

Determine (i) The distance b/w instrument station and staff station

(ii) The R.L. of staff station B

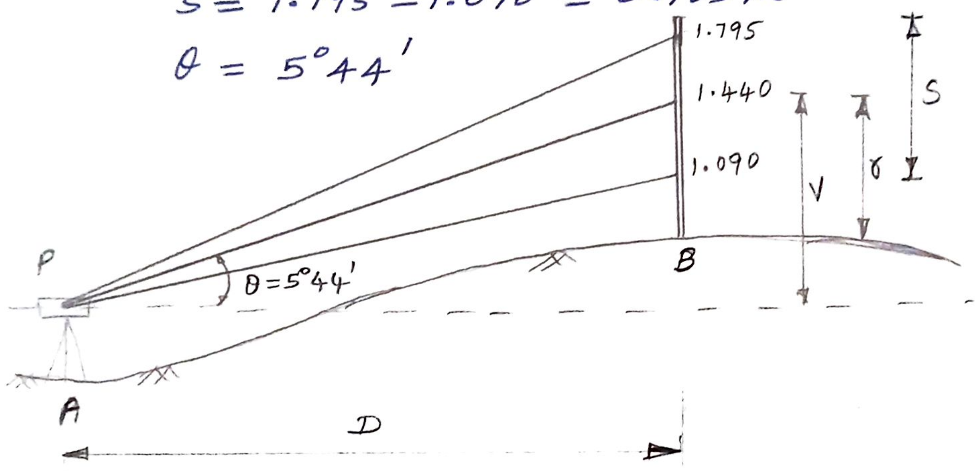
(iii) Stadia readings with instrument Q.

Solution:-

Instrument P at station A and staff held vertical at B

$S = 1.795 - 1.090 = 0.705\,m$

$\theta = 5°44'$



$$AB = D = ks\cos^2\theta + c\cos\theta$$

$$= 100 \times 0.705 \times \cos^2 5°44' + 0.3 \times \cos 5°44'$$

$$\boxed{D = 70.095\,m}$$

$$V = \frac{ks\sin 2\theta}{2} + c\sin\theta$$

$$= \frac{100 \times 0.705 \times \sin 2\times 5°44'}{2} + 0.3 \times \sin 5°44'$$

$$\boxed{V = 7.038\,m}$$

RL of B = RL of A + H.I + V − r

= 100 + 1.40 + 7.038 − 1.440

$\boxed{\text{RL of B} = 106.998\,m}$

$AB = (ks + c)\cos\theta + \gamma \sin\theta$

$= (95\, s + 0.45)\cos 5^\circ 44' + \gamma \sin 5^\circ 44'$

$70.095 = 94.525\, s + 0.448 + 0.0999\,\gamma$

$70.095 - 0.448 = 94.525\, s + 0.0999\,\gamma$

$69.647 = 94.525\, s + 0.0999\,\gamma$

$\div$ by 0.0999

$697.167 = 946.196\, s + \gamma$

$\gamma = 697.167 - 946.196\, s$ ——— (1)

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

$= (95\, s + 0.45)\sin 5^\circ 44'$

$V = 9.49\, s + 0.045$

RL of B = RL of $A + H.I + V - \gamma\cos\theta$

$= 100 + 1.450 + 9.49\, s + 0.45 - \gamma\cos 5^\circ 44'$

$106.998 = 101.9 + 9.49\, s - 0.995\,\gamma$

$5.098 = 9.49\, s - 0.995\,\gamma$

$\div$ by 0.995

$5.124 = 9.538\,s - r$

$r = 9.538\,s - 5.124$ ——— (2)

From equation (1) + (2)

(1) = (2)

$697.167 - 946.196\,s = 9.538\,s - 5.124$

$697.167 + 5.124 = 9.538\,s + 946.196\,s$

$702.291 = 955.734\,s$

$\therefore \boxed{s = 0.735\ m}$

(2) $\Rightarrow$

$r = 9.538 \times 0.735 - 5.124$

$\boxed{r = 1.885\ m.}$

stadia lower reading $= r - \frac{s}{2}$

$= 1.885 - \frac{0.735}{2}$

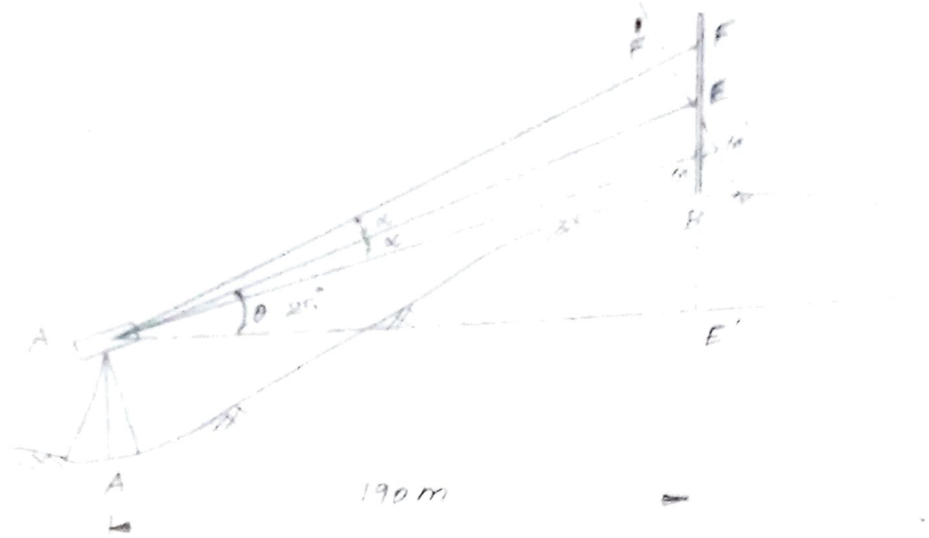
$= 1.517\ m.$

stadia upper reading $= r + s/2 = 1.885 + \frac{0.735}{2}$

$= 2.252\ m$

A tacheometer is fitted with an anallactic lens and the constants are 100 & 0. The reading corresponding to the cross wire on a staff held vertical on a point B was 2.295 m when sighted from A. If the vertical angle was +25° and the horizontal distance AB was 190 m. calculate the stadia wire readings and thus show that the two intercept intervals are equal. Using these values calculate the level of B if that of A was 50.000 m & the ht. of instrument is 1.35 m.

Solution



$$D = ks\cos^2\theta + c\cos\theta$$

$$190 = 100 \times s \times \cos^2 25 + 0 \times \cos 25'$$

$$190 = 82.139\,s$$

$$\therefore \boxed{s = 2.313\ m}$$

From figure, $F'G' = s\cos\theta = 2.313 \times \cos 25^\circ$

$$= 2.096\ m.$$

$$A'E = \frac{A'E'}{\cos\theta} = \frac{190}{\cos 25^\circ} = 209.642\ m.$$

$$\therefore\ 2\alpha = \frac{2.096}{209.642} = 36''$$

$$\therefore\ \alpha = 18''$$

Now, $E'G = D\tan(25^\circ - \alpha)$

$$= 190 \times \tan(25^\circ - 18'')$$

$$= 88.578\ m.$$

$$E'E = D\tan\theta = 190 \times \tan 25^\circ$$

$$= 88.598\ m.$$

$$E'F = D \tan(25^\circ + \alpha)$$

$$= 190 \times \tan(25^\circ + 18'')$$

$$= 88.618 \text{ m.}$$

Stadia intercept interval, $GE = 88.598 - 88.578$

$= 0.020$ m

Stadia intercept interval, $FE = 88.618 - 88.598$

$= 0.020$ m

$\therefore$ The two intercepts are equal

$S = GE + FE = 0.020 + 0.020$

$S = 0.040$ m

Middle cross wire reading $= 2.295$ m

upper stadia wire reading $= 2.295 + 0.020$

$= 2.315$ m

Lower stadia wire reading $= 2.295 - 0.020$

$= 2.275$ m

$EE' = V = D \tan\theta$

$= 190 \times \tan 25$

$= 88.598$ m

RL of B $=$ RL of $A + h + V - r$

$= 50.000 + 1.350 + 88.598 - 2.295$

$= 137.653$ m.

The ruins of an old fort exist on a hill. It was required to determine the distance of the fort from the road and the height of its roof above the plinth with a tacheometer. observations were made on a 4 m staff held vertical on the entrance gate of the fort and on the roof from the road. constants of the instrument were 100 & 0.

Instrument station	Height of instrument	Staff station	Vertical angle	Staff readings (m)
Road	1.45 m	Plinth	$+10°30'$	2.150, 2.720, 3.290
		Roof	$+16°24'$	1.850, 2.400 & 3.040

Solution:-

Let the distance of the fort from the road be D.

$$S_1 = 3.290 - 2.150 = 1.140\text{ m}$$

$$\theta_1 = +10°30'$$

$$K = 100 \quad ; \quad C = 0$$

condition: The staff is held vertical

$$D = KS\cos^2\theta + C\cos\theta$$

$$= 100 \times 1.140 \times \cos^2 10°30' + 0 \times \cos 10°30'$$

$$\boxed{D = 110.214\text{ m}}$$

Let, V_1 = vertical height of plinth of the entrance gate

$$V_1 = \frac{KS_1 \sin 2\theta_1}{2} + C\cancel{\sin\theta_1}^{0}$$

$$= \frac{100 \times 1.140 \times \sin(2 \times 10°30')}{2} + 0$$

$$\boxed{V_1 = 20.427\text{ m}}$$

Roof

V_2 = vertical height of top roof

$$S_2 = 3.040 - 1.850 = 1.190\text{ m}$$

$\theta_2 = +16°24'$

$K = 100 \quad ; \quad C = 0$

$$V_2 = \frac{K S_2 \sin 2\theta_2}{2} + C \sin\theta_2$$

$$= \frac{100 \times 1.190 \times \sin 2 \times 16°24'}{2} + 0 \times \sin 16°24'$$

$V_2 = 32.232\ m$

∴ Height of top of roof above plinth

$= V_2 - V_1$

$= 32.232 - 20.427$

$= 11.805\ m$

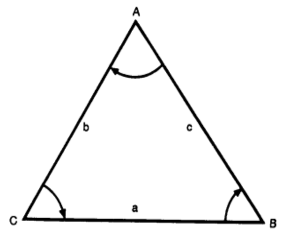
UNIT III
CONTROL SURVEYING

Geodetic Surveying:

- Geodetic or trigonometric surveying differs from plane surveying.
- It deals with long distances and large areas.
- In Geodetic surveying, the curvature of earth is taken into an account.
- It is very accurate method and highly refined instruments are used.
- Geodetic work is usually undertaken by the state agency in India, it is done by the Survey of India.

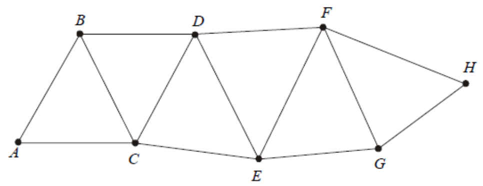
Triangulation – Basic Concept:

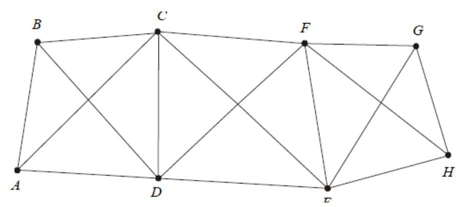
- In triangulation, one side and the three angles of a triangle is known or measured, the remaining sides can be computed by the application of the sine rule.



$$\frac{a}{\sin A} = \frac{b}{\sin B} = \frac{c}{\sin C}$$

- 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 chain of triangles or a chain of quadrilaterals.





- These stations from the vertices of a series of mutually connected triangles, the complete figure being called as ***triangulation system***.
- In this triangles, one side, say AB and all the angles are measured with the greatest care and the lengths of all the remaining lines in the system are then computed. The measured length AB is called a ***base line***.
- The triangulation stations at which the azimuth, latitude or longitude are directly determined by astronomical observations are called azimuth, latitude and longitude stations respectively. These stations are called ***Laplace stations***.

Objectives of triangulation:

Triangulation surveys are carried out

- To establish to establish accurate control for plane and geodetic surveys of large areas, by terrestrial methods,
- To establish accurate control for photogrammetric surveys of large areas,
- To assist in the determination of the size and shape of the earth by making observations for latitude, longitude and gravity.

Applications:

To determine accurate locations of points in engineering works such as:

- Fixing centre line and abutments of long bridges over large rivers.
- Fixing centre line, terminal points, and shafts for long tunnels.
- Transferring the control points across wide sea channels, large water bodies, etc.
- Detection of crustal movements, etc.
- Finding the direction of the movement of clouds.

Field work of triangulation:

It is carried out in the following well defined operations:

- Reconnaissance
- Station preparation (Erection of signals and towers)
- Base line measurement
- Measurement of angles (horizontal, vertical angles)
- Astronomical observations to determine the azimuth of the lines.

- Triangulation consists of the specifications, the design of stations and signals, the reduction and adjustment of the observations.

Horizontal Control:

- Horizontal control surveys co-ordinate horizontal positional data.
- These positions can be referenced by parallel or plane co-ordinate axis.
- Horizontal control in geodetic survey is established either by triangulation, trilateration, traversing, aerial photogrammetric methods, inertial and Doppler positioning systems and GPS.
- For relatively large topographical surveys, primary and secondary control are established by triangulation and trilateration.

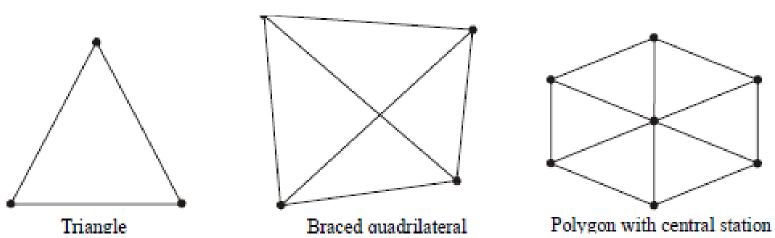
- These methods are also employed in areas of smaller extent when field conditions are appropriate (hilly, urban or rugged mountainous regions).
- Traversing with total station (a theodolite with EDM instrument) can also be used for establishing primary and secondary control.
- When the area is large and scale of mapping is small, establishment of horizontal control can be performed by aerial photogrammetric methods.
- These methods requires a basic frame work of horizontal control points which is established by triangulation and / or trilateration or GPS etc.,
- When the extent of the area is very large, it is establish primary control by inertial and Doppler or GPS methods.
- These methods can cover inaccessible regions or the regions requiring conduct of survey governed by special conditions.

Vertical Control:

- A vertical control surveys determines elevation with respect to sea level.
- These surveys are also used as a bench mark upon which other surveys are based ad high degree of accuracy is required.
- These surveys are useful for tidal boundary surveys, route survey, construction survey, and topographical surveys.
- In a vertical control system, at least two permanent bench marks should be used, but more may be required depending upon the needs and complexity of the project.
- These projects are needed for the construction of water and sewer systems, highway, bridges, drains and major town or city infrastructure.

Triangulation Figures or Systems or Layouts

- It is defined as a system considering of triangulation stations connected by chain of triangles. The complete figure is called triangulation figure of triangulation systems.
- The most common types of figures used in triangulation systems are the triangle, braced or geodetic quadrilateral, and the polygon with a central station.

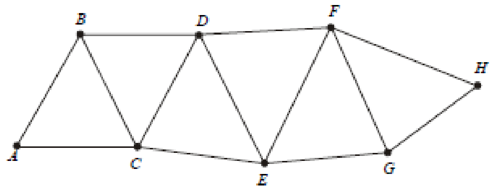


Basic triangulation figures

- The triangles in a triangulation system can be arranged in a number of ways.
    - Single chain of triangles
    - Double chain of triangles
    - Centre point figures (triangle & polygon)
    - Braced quadrilaterals
    - Centered triangles and polygons
    - A combination of above systems.

Single chain of triangles

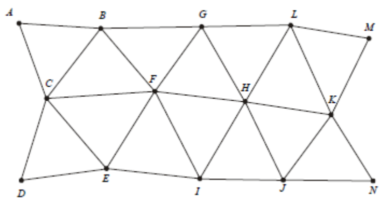
- When the control points are required to be established in a narrow strip of terrain such as a valley between ridges, a layout consisting of single chain of triangles.
- It is used to cover smaller area.
- It is rapid and economical (due to its simplicity of sighting only four other stations, and does not involve observations of long diagonals).



- Simple triangles of a triangulation system provide only one route through which distances can be computed.
- This system does not provide any check on the accuracy of observations.

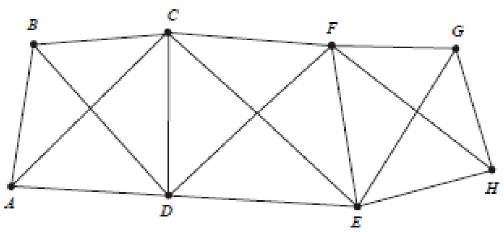
Double chain of triangles

- This arrangement is used for covering the larger width of a belt.
- This system also has disadvantages of single chain of triangles system.



Braced quadrilaterals

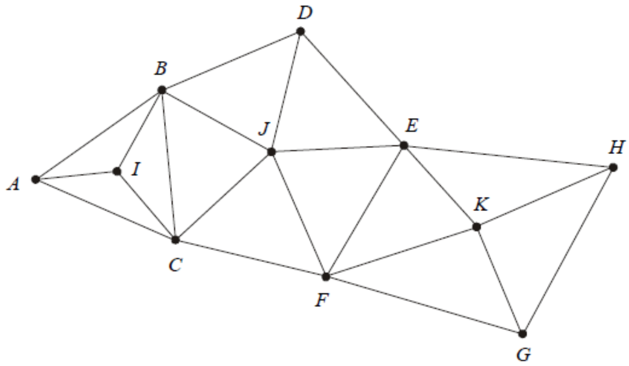
- These are best suited for hilly areas.
- It consists of figures containing four corner stations and observed diagonals are known as a layout of braced quadrilaterals.
- Braced quadrilaterals consist of overlapping triangles.
- This system is treated to be the strongest and the best arrangement of triangles.



- It provides a means of computing the lengths of the sides using different combinations of sides and angles.
- Most of the triangulation systems use this arrangement.

Centered triangles and polygons

- It is generally used vast area in all directions is required to be covered.
- It consists of figures containing interior stations in triangle and polygon as known as centered triangles and polygons.
- The centers figures generally are quadrilaterals, pentagons, or hexagons with central stations.
- This system provides checks on the accuracy of the work.
- Generally it is not as strong as the braced quadrilateral arrangement.



- The progress of work is quite slow due to the fact that more setting of the instrument are required.

<u>Combination of all above systems</u>

- Sometimes a combination of above systems may be used, which may be according to the shape of the area and the accuracy requirements.

<u>Classification of Triangulation System</u>

- Based on the extent and purpose of the survey, and consequently on the degree of accuracy desired.
- Triangulation surveys are classified as
  - *First-order (or) Primary triangulation,*
  - *Second-order (or) Secondary triangulation,*
  - *Third-order (or) Tertiary triangulation*.
- **First-order triangulation** is used to determine the shape and size of the earth or to cover a vast area like a whole country with control points to which a second-order triangulation system can be connected.
- **Second-order triangulation** system consists of a network within a first-order triangulation. It is used to cover areas of the order of a region, small country.
- **Third-order triangulation** is a framework fixed within and connected to a second-order triangulation system. It serves the purpose of furnishing the immediate control for detailed engineering and location surveys.

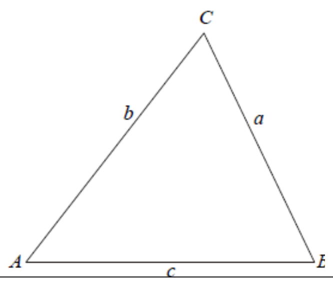
Sl. No	Characteristics	First-order triangulation	Second-order triangulation	Third-order triangulatio
1	Length of base line	8 to 12 Km	2 to 5 Km	100 to 500 m
2	Length of sides	16 to 150 Km	10 to 25 Km	2 to 10 Km
3	Average triangular error (after correction for spherical excess)	Less than 1”	3”	12”
4	Maximum station closure	Not more than 3”	8”	15”
5	Actual error of base	1 in 50,000	1 in 25,000	1 in 10,000
6	Probable error of base	1 in 10,00,000	1 in 5,00,000	1 in 2,50,000
7	Discrepancy between two measures (‘K’ is distance in	5√K mm	10√K mm	25√K mm
8	Probable error of the computed distance	1 in 50,000 to 1 in 2,50,000	1 in 20,000 to 1 in 50,000	1 in 5,000 to 1 in 20,000
9	Probable error astronomical azimuth	0.5”	5”	10”

- These are the general specifications for the triangulation system.

Strength of Figure:

Well conditioned triangles:

- There are various triangulation figures and accuracy attained in each figure depends upon
  - The magnitude of the angles in each individual triangle
  - The arrangement of the triangles
- The shape of the triangle should be such that any error in the measurement of angle shall have minimum effect upon the lengths of the calculated sides. Such a triangle is then called a well conditioned triangle.
- In a triangle, one side is known from the computations of the adjacent triangle.
- The errors in the other two sides will affect the rest of the triangulation figure.
- These two sides be equally accurate, they should be equal in length, which could be possible only by making the triangle isosceles.



- To find the magnitude of the angle of the triangle 'A', 'B' & 'C' be the three angles and 'a', 'b', & 'c' be the three opposite side of an isosceles triangle ABC.
- Let 'AB' be the known length or side and 'BC' & 'CA' be the sides of equal length to be computed. (a = b)

  i.e., ∟A = ∟B
- By sine formula, $\frac{a}{\text{SIN A}} = \frac{b}{\text{SIN B}} = \frac{c}{\text{SIN C}}$
- Applying sine rule to ΔABC, we have $\frac{a}{\sin A} = \frac{c}{\sin C}$

$$a = c\,\frac{\sin A}{\sin C} \qquad (1)$$

- *Let*

  δA = *error in the measurement of angle A*

  δa_1 = *corresponding error in the side a*
- *Differentiate equation 1 with respect to A*

  $(\delta a / \delta A)$ = $(c/\text{Sin } C)\,\text{Cos } A$

  δa_1 = $(c\,\text{Cos}A\;\delta A) / \text{Sin } C \qquad (2)$

  Equation 2 divided by equation 1

  $(\delta a_1/a)$ = $(c\;\text{Cos}A\;\delta A)\,\text{Sin } C / c \sin A\;\text{Sin } C$ =

  = $(\text{Cos } A\;\delta A) / \text{Sin } A$

  $(\delta a_1/a)$ = $\delta A\;\text{Cot } A \qquad (3)$

  Similarly,

  δC = *error in the measurement of angle C*

  δa_2 = *corresponding error in the side a*
- *Differentiate equation **2** with respect to **C***

  δa_2 = $-c\,(\text{Sin } A\;\text{Cos}C\;\delta c / \text{Sin}^2 C \qquad (4)$

  *Equation **4** divided by equation **1***

  $(\delta a_2/a)$ = $[-c\,(\text{Sin } A\;\text{Cos}C\;\delta c / \text{Sin}^2 C] / c \sin A\;\text{Sin } C$

  = $[-c\;\text{Sin } A\;\text{Cos}C\;\delta c\;\;\text{Sin } C] / [c \sin A\;\text{Sin}^2 C]$

  = $(-\text{Cos } C\;\delta c) / \text{Sin } C$

  $(\delta a_2/a)$ = $-\delta c\;\text{Cot } C \qquad (5)$
- *If* δ_A *and* δ_C = *Probable errors in angles,*

  ie., δ_A *and* δ_C = $\pm \beta$

  $\delta a / a$ = *Probable friction error in the side a*

  = $\pm \beta\sqrt{(\cot^2 A + \cot^2 C)}$ *is minimum*

  But C = $180 - A - B$ $\qquad [A=B]$

C = *180 – A – A* = *180 – 2A*

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

- *Differentiate* $\cot^2 A + \cot^2 2A$ *with respect to* ***A*** ***and*** *equating to zero, we get after reduction*

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

  From which,

  $$A = 56^o\ 14' \text{ (approximately)}$$

- Hence, the best shapes of an isosceles triangle with base angles are 56°14′ each.
- However, in practical considerations ($56^o\ 14' \approx 60^o\ 0'$), an equilateral triangle may be treated as a well-conditional triangle.
- In actual practice, the triangles having an angle less than 30° or more than 120° should not be considered.

Strength of Figure:

- The strength of figure is a factor to be considered in establishing a triangulation system to maintain the computations within a desired degree of precision.
- It plays also an important role in deciding the layout of a triangulation system.
- This method is based on an expression for the square of the probable error (L^2) that would occur in the sixth place of the logarithm of any side, if the computations are carried from a known side through a single chain of triangles after the net has been adjusted for the side and angle conditions.
- The expression for L^2 is

  $$L^2 = {}^4/_3\, d^2 R$$

- where

	d	=	probable error of an observed direction in seconds of arc
	R	=	the shape of figure
	R	=	$[(D-C)/D]\quad \Sigma\,[\delta A^2 + \delta A\,\delta B + \delta B^2]$
	R	=	$[(D-C).\, a\, /D]$
Therefore	L^2	=	${}^4/_3\, d^2 R$
	D	=	number of directions observed excluding the known side of the figure (forward & / or backward)
	δA	=	difference per second in the sixth place of logarithm of the sine of the distance angles *A*
	δB	=	difference per second in the sixth place of logarithm of the sine of the distance angles *B*

δC = difference per second in the sixth place of logarithm of the sine of the distance angles C (Distance angle is the angle in a triangle opposite to a side)

C = number of geometric conditions for side and angle.

- It is given by

$$C = (n' - S' + 1) + (n - 2S + 3)$$

- Where

n = total number of lines including the known side in a figure,

n' = number of lines observed in both directions (including known side)

S = total number of stations

S' = number of stations occupied

Problem:

1. Compute the value of $[(D - C)/D]$ for the following triangulation figures if all the stations have been occupied and all the lines have been observed in both directions :

(*i*) A single triangle

(*ii*) A braced quadrilateral

(*iii*) A four-sided central-point figure without diagonals

(*iv*) A four-sided central-point figure with one diagonal.

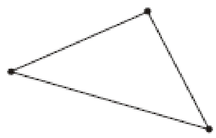
Solution:

(i) Single triangle

$C = (n' - S' + 1) + (n - 2S + 3)$

$n' = 3$ $\quad$ $n = 3$

$S = 3$ $\quad$ $S' = 3$



$C = (3 - 3 + 1) + (3 - 2 \times 3 + 3)$

$C = 1$

D = the number of directions observed excluding the known side.

= 2 (total number of lines – 1)

$= 2 \times (3 - 1)$

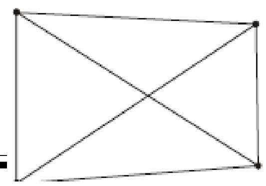
$D = 4$

$[(D - C)/D] = (4 - 1) / 4 = \mathbf{0.75.}$

(ii) Braced quadrilateral

$n = 6$ $\quad$ $n' = 6$

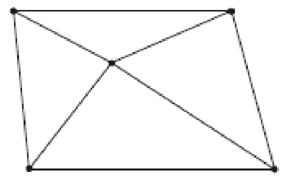
$S = 4$ $\quad$ $S' = 4$



$C' = (6 - 4 + 1) + (6 - 2 \times 4 + 3) = 4$

D	=	$2 \times (6 - 1)$	=	10
$(D-C)/D$	=	$(10-4) / 10$	=	**0.6**

(iii) Four-sided central-point figures without diagonals



$n = 8$ $\quad\quad n' = 8$

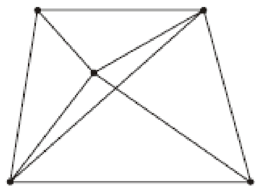
$S = 5$ $\quad\quad S' = 5$

$C = (8 - 5 + 1) + (8 - 2 \times 5 + 3) = 5$

$D = 2 \times (8 - 1) = 14$

$(D-C)/D = (14-5) / 14 =$ **0.64**

(iv) Four-sided central-point figure with one diagonal



$n = 9$ $\quad\quad n' = 9$

$S = 5$ $\quad\quad S' = 5$

$C = (9 - 5 + 1) + (9 - 2 \times 5 + 3) = 7$

$D = 2 \times (9 - 1) = 16$

$(D-C)/D = (16-7) / 14 =$ **0.56**

Routine of Triangulation Survey:

- The routine of triangulation survey, broadly consists of
  - **a.** field work,
  - **b.** computations
- The field work of triangulation is divided into the following operations :
  - *i.* Reconnaissance
  - *ii.* Erection of signals and towers
  - *iii.* Measurement of base line
  - *iv.* Measurement of horizontal angles
  - *v.* Measurement of vertical angles
  - *vi.* Astronomical observations to determine the azimuth of the lines.

Reconnaissance

- *Reconnaissance* is the preliminary field inspection of the entire area to be covered by triangulation, and collection of relevant data.
- The basic principle of survey is working from whole to the part, reconnaissance is very important in all types of surveys.
- It requires great skill, experience and judgement.
- The accuracy and economy of triangulation greatly depends upon proper reconnaissance survey. It includes the following operations:

- Examination of terrain to be surveyed.
- Selection of suitable sites for measurement of base lines.
- Selection of suitable positions for triangulation stations.
- Determination of intervisibility of triangulation stations.
- Selection of conspicuous well-defined natural points to be used as intersected points.
- Collection of miscellaneous information regarding:
    - Access to various triangulation stations
    - Transport facilities
    - Availability of food, water, etc.
    - Availability of labour
    - Camping ground.

- Reconnaissance may be effectively carried out if accurate topographical maps of the area are available.
- If maps and aerial photographs are not available, a rapid preliminary reconnaissance is undertaken to ascertain the general location of possible schemes of triangulation suitable for the topography.
- The main reconnaissance is a very rough triangulation.
- The plotting of the rough triangulation may be done by protracting the angles.
- The essential features of the topography are also sketched in.
- For reconnaissance the following instruments are generally employed:
    - Small theodolite and sextant for measurement of angles.
    - Prismatic compass for measurement of bearings.
    - Steel tape.
    - Aneroid barometer for ascertaining elevations.
    - Heliotropes for ascertaining intervisibility.
    - Binocular.
    - Drawing instruments and material.
    - A guyed ladder, creepers, ropes, etc., for climbing trees.

Selection of triangulation stations

- Triangulation stations should be intervisible. For this purpose the station points should be on the highest ground such as hill tops, house tops, etc.
- Stations should be easily accessible with instruments.
- Station should form well-conditioned triangles.

- Stations should be so located that the lengths of sights are neither too small nor too long.
- Small sights cause errors of bisection and centering. Long sights too cause direction error as the signals become too indistinct for accurate bisection.
- Stations should be at commanding positions so as to serve as control for subsidiary triangulation, and for possible extension of the main triangulation scheme.
- Stations should be useful for providing intersected points and also for detail survey.
- In wooded country, the stations should be selected such that the cost of clearing and cutting, and building towers, is minimum.
- Grazing line of sights should be avoided, and no line of sight should pass over the industrial areas to avoid irregular atmospheric refraction.

Erection of signals and towers

- A ***signal*** is a device erected to define the exact position of a triangulation station.
- It is placed at each station so that line of sight are established between triangulation stations.
- A ***tower*** is a structure over a station to support the instrument and the observer, and is provided when the station or the signal, or both are to be elevated.

Characteristics or Requirements of a Good Signal:

- It should be clearly visible against any background.
- It should be kept at least 75 cm above the station mark.
- It should be suitable for bisection from other stations.
- It should be free from phase, or should exhibit little phase
- In general, the diameter of the signals should be a range of 1.3 D to 1.9 D.

  Where

  D = Distance in Kilometer
- It should be capable of being accurately centered over the station mark.
- It should be symmetrical
- It should be easy to erect in minimum time.
- It should be sufficient height, capable being vertical and accurately centered over the station mark.
- In general, the height of the signal is a range of 13.3 D

  Where

  h = height of signal

  D = Distance in Kilometer

Classification of signals

i. Non-luminous, opaque or daylight signals

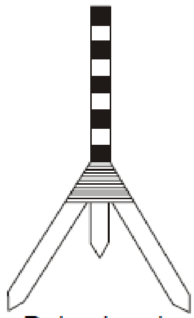
ii. Luminous signals.

(i) Non-luminous signals or daylight signals

- Non-luminous signals are used during day time and for short distances.
- Most commonly used for,

(a) Pole signal

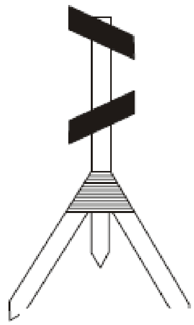
- It consists of a round pole painted black and white in alternate strips, and is supported vertically over the station mark, generally on a tripod.
- Pole signals are suitable up to a distance of about 6 km.



Pole signal

(b) Target signal

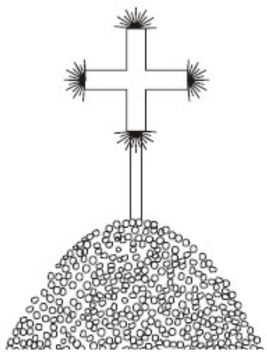
- It consists of a pole carrying two squares or rectangular targets placed at right angles to each other.
- The targets are generally made of cloth stretched on wooden frames.
- Target signals are suitable up to a distance of 30 km.



Target signal

(c) Pole and brush signal

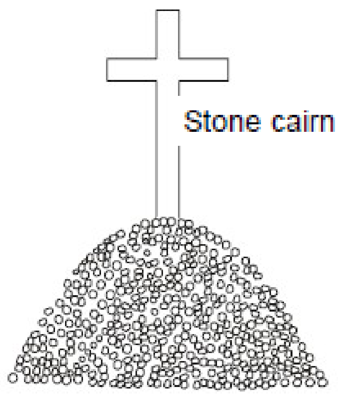
- It consists of a straight pole about 2.5 m long with a bunch of long grass tied symmetrically round the top making a cross.
- The signal is erected vertically over the station mark by heaping a pile of stones, up to 1.7 m round the pole.
- A rough coat of white wash is given to make it more conspicuous to be seen against black background.
- It must be erected over every station of observation during reconnaissance.



Pole and brush signal

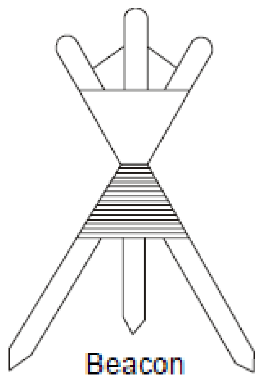
(d) Stone cairn

- A pile of stone heaped in a conical shape about 3 m high with a cross shape signal erected over the ston e heap, is stone cairn.
- White washed opaque signal is very useful in the dark background.



Stone cairn

(e) Beacons



- It consists of red and white cloth tied round the three straight poles.
- It can easily be centered over the station mark.

(ii) Luminous signals

- Luminous signals may be classified into two types :

  (a) Sun signals

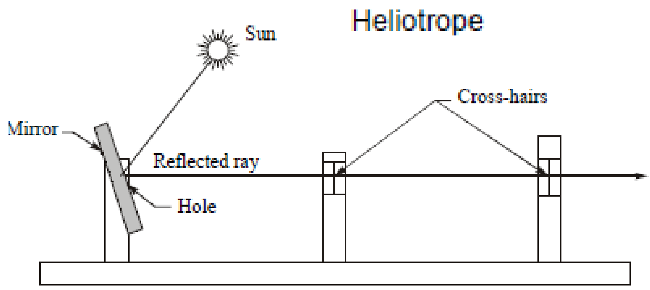
  (*b*) Night signals.

(a) Sun signals

- Sun signals reflect the rays of the sun towards the station of observation, and are also known as heliotropes.
- Such signals can be used only in day time in clear weather.

Heliotrope:

- It consists of a circular plane mirror with a small hole at its centre to reflect the sun rays, and a sight vane with an aperture carrying cross-hairs.

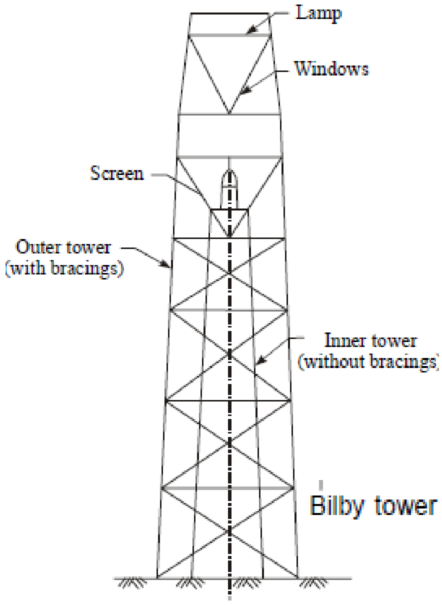


- The circular mirror can be rotated horizontally as well as vertically through 360°.
- The heliotrope is centered over the station mark, and the line of sight is directed towards the station of observation.
- The sight vane is adjusted looking through the hole till the flashes given from the station of observation fall at the centre of the cross of the sight vane.
- Once this is achieved, the heliotrope is disturbed.
- Now the heliotrope frame carrying the mirror is rotated in such a way that the black shadow of the small central hole of the plane mirror falls exactly at the cross of the sight vane.
- The reflected beam of rays will be seen at the station of observation.
- Due to motion of the sun, this small shadow also moves, and it should be constantly ensured that the shadow always remains at the cross till the observations are over.
- The heliotropes do not give better results compared to signals.

- These are useful when the signal station is in flat plane, and the station of observation is on elevated ground.
- The distance between the stations exceed 30 km, the heliotropes become very useful.

(b) Night signals:

- When the observations are required to be made at night, the night signals of following types may be used.
  - ✷ Various forms of oil lamps with parabolic reflectors for sights less than 80 km.
  - ✷ Acetylene lamp designed by Capt. McCaw for sights more than 80 km.
  - ✷ Magnesium lamp with parabolic reflectors for long sights.
  - ✷ Drummond's light consisting of a small ball of lime placed at the focus of the parabolic reflector, and raised to a very high temperature by impinging on it a stream of oxygen.
  - ✷ Electric lamps.



TOWERS

- ✷ A tower is erected at the triangulation station when the station or the signal or both are to be elevated to make the observations possible form other stations in case of problem of intervisibility.
- ✷ The height of tower depends upon the character of the terrain and the length of the sight.
- ✷ The towers generally have two independent structures.
- ✷ The outer structure is for supporting the observer and the signal whereas the inner one is for supporting the instrument only.
- ✷ The **two structures** are made entirely **independent of each** other so that the movement of the observer does not **disturb the instrument setting**.
- ✷ The two towers may be made of masonry, timber or steel.
- ✷ For small heights, masonry towers are most suitable.
- ✷ Timber scaffolds are most commonly used, and have been constructed to heights over 50 m.
- ✷ Steel towers made of light sections are very portable, and can be easily erected and dismantled.

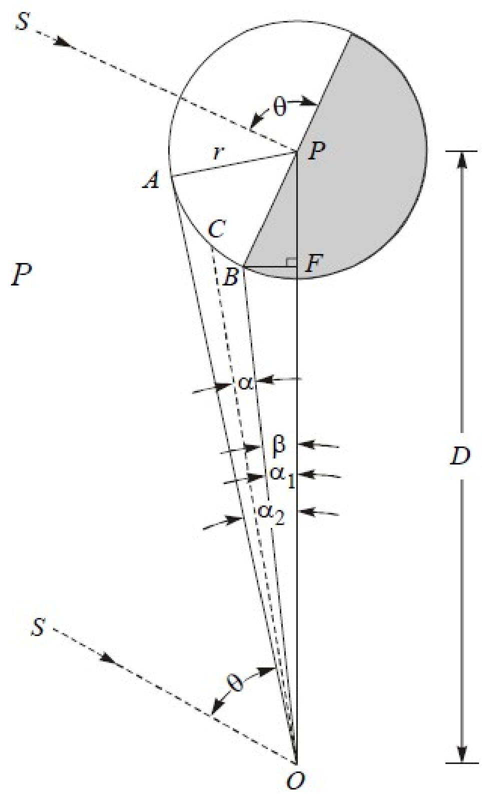
- ✸ Bilby towers patented by J.S. Bilby of the U.S. Coast and Geodetic Survey, are popular for heights ranging from 30 to 40 m.
- ✸ This tower weighing about 3 tones can be easily erected by five persons in just 5 hrs.

Phase of Signal:

- When the observations are made in the sun light on a signal of a circular shape due to lateral illumination.
- Some part of the signal is lighted up, while the other part is shade.
- A require correction in the observed horizontal angle due to an error is known as phase.
- The method of observation, phase correction is computed by the following two conditions.
    - Observations made on bright portion
    - Observations made on bright line.

(i) Observation made on bright portion

Let,



r = radius of cylindrical signal

P = centre of the signal

O = observer position

A & B = *observations made on a bright portion*

C = midpoint of *AB*

Θ = the angle between the sun and the line *OP*

α_1 = angle AOP

α_2 = the angles *BOP* and *AOP*

D = horizontal distance between *OP* *(observer position & signal)*

α = half of the angle *AOB*

$\boldsymbol{\alpha = (\alpha_2 - \alpha_1)/2}$

β = phase correction

$= \alpha_1 + \alpha \quad = \quad \alpha_1 + (\alpha_2 - \alpha_1)/2$

$= (2\alpha_1 + \alpha_2 - \alpha_1)/2$

$\boldsymbol{\beta = (\alpha_1 + \alpha_2)/2}$ **.. (1)**

From Δ*OAP*

$\tan \alpha_2 = (r / D)$

α_2 being small,

$$\alpha_2 = (r / D) \text{ radians} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

As the distance *PF* is very small compared to *OP*,

OF may be taken as *OP*.

From right angle ΔBFO,

$$\tan \alpha_1 = (BF / OF) = (BF / OP) = (BF / D)$$

$$\tan \alpha_1 = (BF / D) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

From Δ*PFB*,

$$BF = r \sin (90 - \theta) = r \cos \theta$$

Substituting the value of *BF* in Eq. (3), we get

$$\tan \alpha 1 = (BF / D) = (r \cos \theta / D)$$

α_1 being small

$$\alpha_1 = (r \cos\theta / D) \text{ radians} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Substituting the values of α_1 and α_2 in Eq. (1),

$$\beta = (\alpha_1 + \alpha_2) / 2 = (r \cos \theta / 2D) + (r / 2D)$$

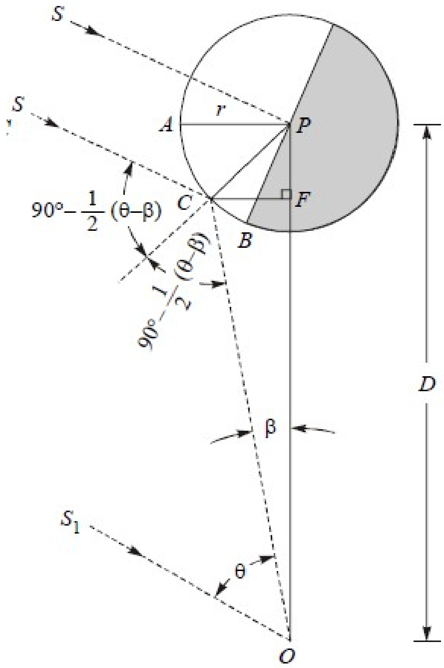
$$= (r / D) [(1 + \cos \theta) / 2]$$

$$= (r / D) \cos^2 (\theta/2) \text{ radians}$$

$$= (r / D \sin 1'') \cos^2 (\theta/2) \text{ seconds}$$

$$\beta = 206265 (r / D) \cos^2 (\theta/2) \quad \text{in seconds}$$

(ii) Observations made on the bright line



Let,

C	=	bright line
r	=	radius of cylindrical signal
CO	=	reflected ray of the sun from the bright line at *C*
β	=	phase correction
θ	=	angle between the sun and the line O*P*

The rays of the sun are always parallel to each other,

Therefore, *SC* is parallel to S_1O .

$$\angle SCO = 180° - (\theta - \beta)$$

$$\angle PCO = 180° - 1/2 \angle SCO$$

$$= 180 - (1/2) [180 - (\theta - \beta)]$$

$$\angle PCO = 90 + (1/2) (\theta - \beta) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Therefore,

$\angle CPO = 180° – (\beta + \angle PCO)$ .... (2)

Substituting the value of $\angle PCO$ from Eq. (1) in Eq. (2)

$$\angle CPO = 180° – (\beta + \angle PCO)$$
$$= 180° – (\beta + 90° + (\theta – \beta) /2$$
$$= 180° – \beta - 90° - (\theta/2) + (\beta /2)$$
$$= 90° – (\theta/2) - (\beta /2)$$
$$\angle CPO = 90° – (\beta + \theta)/2)$$

As β is very small compared to θ, it can be ignored,

Therefore,

$$\angle CPO = 90° – \theta/2$$

From the right angle Δ*CFP*

$$(CF/ CP) = \sin CPO = \sin (90° – \theta/2)$$
$$CF = r \sin (90° – \theta/2) \quad \text{.............................. (3)}$$

From the right angle Δ*CFO*

$$\tan \beta = (CF/ OF) \quad \text{.............................. (4)}$$

PF being very small compared to *OP*,

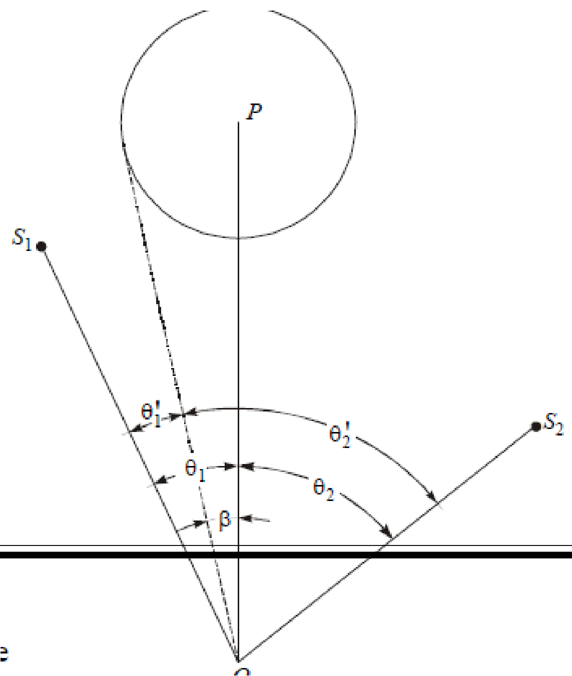
Therefore, *OF* may be taken as *OP*.

Substituting the value of *CF* from Eq. (3) and taking *OF* equal to *D*, we get the Eq. (4)

$$\tan \beta = (CF/ OF) = [r \sin (90° – \theta) /2] / OP$$
$$\tan \beta = [r \sin (90° – \theta) /2] / D$$

β being very small

$$\beta = (r / D) [\cos \theta / 2] \quad \text{radians}$$
$$= (r / D \sin 1") \cos (\theta/2) \text{ seconds}$$
$$\beta = 206265 (r / D) \cos (\theta/2) \quad \textit{in seconds}$$



Applying the phase correction to the measured horizontal angles

Let

$S1$, $S2$, P, and O are the four stations

O be the observer station

Measured angle

$S_1OP = \theta_1'$ and

$POS_2 = \theta'_2$

If the required corrected angles are θ1 and θ2 , then

$\theta_1 = \theta_1' + \beta$

$\theta_2 = \theta'_2 - \beta$

where,

β is the phase correction.

Problem:

A cylindrical signal of diameter 4 m, was erected at station *B*. Observations were made on the signal from station *A*. Calculate the phase corrections when the observations were made

(*i*) on the bright portion, and

(*ii*) on the bright line.

Take the distance *AB* as 6950 m, and the bearings of the sun and the station *B* as 315° and 35°, respectively.

Solution:

Given that

Dia = 4 m

Distance (D) = 6950 m

Bearing of sun = 315°

Bearing of B = 35°

Angle between sun and observer,

θ = ***Bearing of sun – bearing of B***

= 315° – 35°

θ = 280°

r = 2m

(i) Observation made on bright portion

$\beta = 206265\,(r / D)\cos^2(\theta/2)$ ***in seconds***

$= 206265\,(2 / 6950)\cos^2(280/2)$

= **34.83 seconds**.

(ii) Observation made on bright line

$\beta = 206265\,(r / D)\cos(\theta/2)$ ***in seconds***

= 206265 (2 / 6950) cos (280/2)

= **45.47 seconds**.

Problem: 2

The horizontal angle measured between two stations *P* and *Q* at station *R*, was 38°29′30″. The station *Q* is situated on the right of the line *RP*. The diameter the cylindrical signal erected at station *P,* was *3* m and the distance between *P* and *R* was 5180 m. The bearing of the sun and the station *P* were measured as 60° and 15°, respectively. If the observations were made on the bright line, compute the correct horizontal angle *PRQ*.

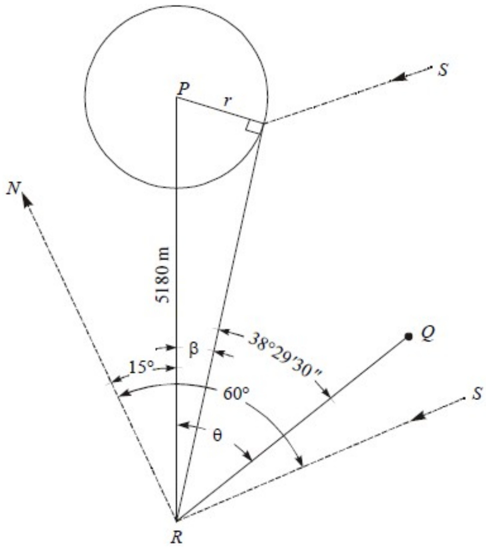
Solution:

Dia = 3 m

Distance (D) = 5180 m

Bearing of sun = *60°*

Bearing of B = 15°



Angle between sun and observer,

θ = Bearing of sun – bearing of B

= 60° – 15°

θ = 45°

r = 1.5 m

phase correction for observation made in bright line

β = 206265 (r / D) cos (θ/2) ***in seconds***

= 206265 (1.5 / 5180) cos (45/2)

= **55.18 seconds**.

The correct horizontal angle *PRQ* = 38° 29′ 30″ + β

= 38°29′30″ + 55.18″

= **38°30′25.18″**.

Base Line Measurement:

- The accuracy of an entire triangulation system depends on that attained in the measurement of the base line.
- Base line forms the most important part of the triangulation operations.
- The base line must be measured very accurately so that the other sides calculated from the base line and the angles are accurate.
- The length of baseline varies from a fraction of 1.5km to 15 km according to grades of triangulation.

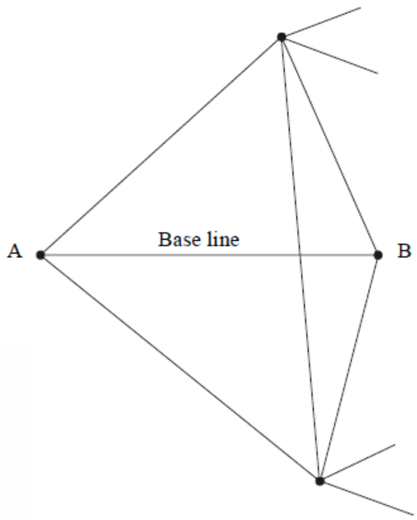
- It generally lies between 1/3rd and 2/3rd of the length of the average side of the triangulation system.
- In India, ten bases were used.
- The length of 9 bases varied from 10.7km to 13km and that of the tenth base was 2.83km.

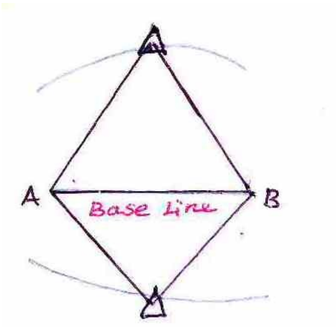
Selection of site for base line:

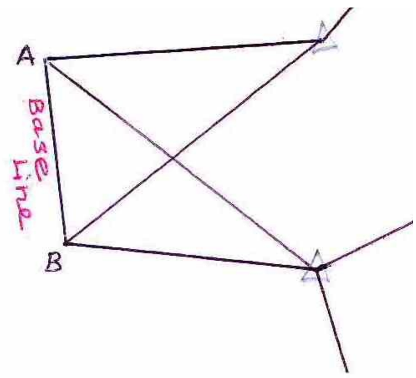
- The ground should be firm and level. (If the ground is sloping the slope should be uniform and gentle).
- The site should be free from obstructions throughout the length of the base line.
- The ground should be firm and smooth.
- It should be provide a system of well conditioned triangles.
- It should be passing through the centre of the area.

Base Net:

- A series of triangles connecting a base line to the main triangulation is called base net.
- The base should be expanded gradually by triangulation.







Equipment for base line measurements:

Flexible Apparatus: chain, wire and tape.

a) Standardised tapes:

- For measuring short bases in plain areas standardised tapes are generally used.
- After having measured the length, the correct length of the base is calculated by applying the required corrections.
- If the triangulation system is of Extensive nature, the corrected lengths of the base are reduced to the mean sea level.

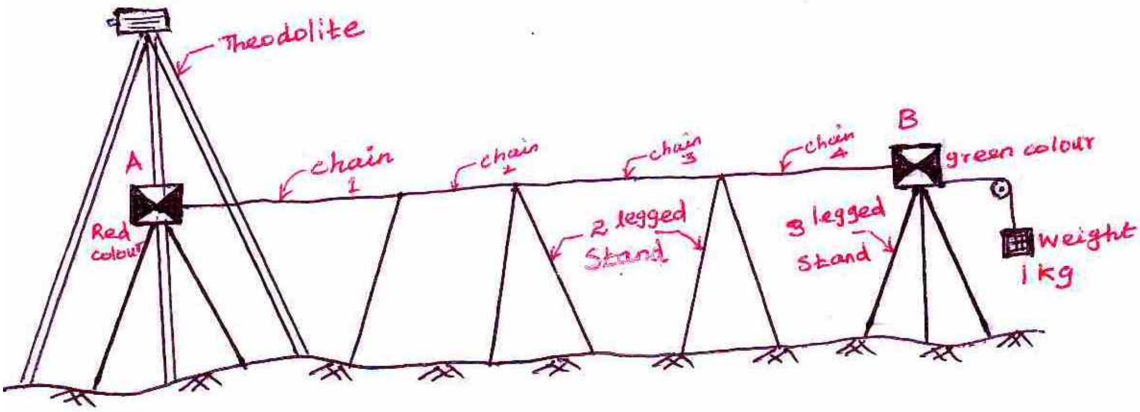
- There are two methods

  (i) wheeler's method

  (ii) Jaderin's method

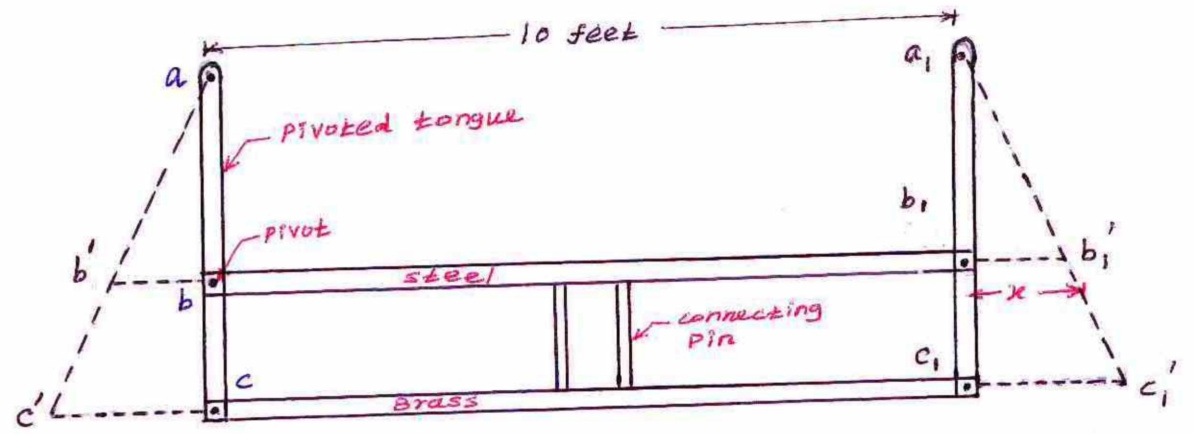
Hunter's short base method:

- Dr. Hunter who was a Director of Survey of India, designed an equipment to measure the base line which was named as hunter's short base.
- It consists of 4 chains, each of 20.117 m (66ft) linked together.
- There are 5 stands, 3 intermediate two legged stands, 2 three legged stands at e nds.



- A 1kg weight is suspended at the end of an arm, so that the chains remain straight during observations.
- The correct length of the individual chain is supplied by the manufacturer or is determined in the laboratory.
- The length of the joints between two chains at intermediate supports is measured directly with the help of graduated scale.
- To obtain correct length between the centres of the tangents used corrections such as temperature, sag, slope etc, are applied.
- To set the hunters short base, the stand at end A(marked on red colour) is centred on the ground mark and the target is fitted with a clip.
- The target 'A' is made truly vertical so that the notch on its tip side is centred on the ground mark.
- The end of the base is hooked with the plate A

Colby Apparatus:



- It is designed by Major general Colby
- All the ten bases of GTS (Great trigonometrically Survey) of India were measured with the Colby Apparatus
- It consists of an iron and a brass bar, each 10 ft 1½ inch long, fixed together at middle by means of two steel pins
- A flat steel tongue ,about 6 inches long, is pivoted at each end of the bar
- Each of the tongue carries one microscopic platinum dot ‘a’ and ‘a_1’ making the distance a a_1 exactly 10 feet.
- To secure compensation ,the ratio ab/ac is made equal to the ratio of coefficients of linear expansion of iron and brass i.e.,3/5
- The tongue is free to pivot, the position of the dot remains constant under the change of temperature.
- Due to change of temperature, the length bb_1 say be x
- The length cc_1 will change to c’ c_1’ by 5/3 x
- The positions of the dots ‘a’ and ‘a_1’ remain unchanged.
- The bar is held in a box at the middle of its length.
- A spirit level is placed on the bar, and is observed through a window in the top of the box.

- For measuring the bases in India, five such bars were simultaneously used with a gap of 6 inches between the forward mark of one bar and the rear mark of the next bar by means of a framework.
- Framework was equipped with two microscopes with their cross wires 6 in apart.
- A small telescope, parallel to the microscopes is fixed at the middle of this bar for sighting reference marks on the ground.

Tape Corrections

i) *Correction for temperature(C_t)*

$C_t = \alpha\,(T_m - T_o)\,L$

α is coefficient of thermal expansion

T_m is mean temperature during measurement

T_o is standardized temperature

L is the measured length

ii) *Correction for Absolute Length(C_a):*

$C_a = L\,C / l$

L is the measured length

l is nominal length of measuring unit

C is correction to measuring unit

iii) *Correction for pull or tension: C_p*

$C_p = (P - P_o) / AE$

L is the measured length

P_o is the standard pull

P is pull applied during measurement

A is cross sectional area of tape in cm^2

E is young's modulus of tape

iv) Sag Correction:

$$C_{sag} = W\ l/(24 P^2 n^2)$$

W is the total weight of tape

P is the pull applied in N

L is the length of tape

N is the number of equal span

v) Reduction to mean sea level:

$$C_r = hL/R$$

L is the measured length

H is the altitude

R is the radius of earth

vi) Slope Correction:

$$C_{sl} = L(1-\cos\theta)$$

L is the measured length

θ is the slope

Problem:

A tape of standard length 20 m at 85° F was used to measure a base line .the measured distance was 882.10 m. the following being the slopes for various segments of the line.

Segment	100 m	150m	50 m	200 m	300 m	882.10m
Slope	2° 20′	4°12′	1° 06′	7° 45′	3° 0′	5°10′

Find the true length of the base line, if the mean temperature during measurement was 63° F. The coefficient of the tape material is 6.5 F. the coefficient of the tape material is 6.5×10^{-6} per ° F.

Solution:

i) *Correction for temperature(C_t)*

$$C_t = \alpha\,(T_m - T_o)\,L$$

$$= 6.5 \times 10^{-6}\,(63-85) \times 882.10$$

$$= 0.126 \text{(negative)}$$

ii) Slope Correction

$$C_{sl} = L(1-\cos\theta)$$

$$= 100(1-\cos 2^\circ 20') + 150\,(1-\cos 4^\circ 12') + 50\,(1-\cos 1^\circ 06') + 200\,(1-\cos 7^\circ 45') + 882.10(1-\cos 5^\circ 10')$$

$$C_{sl} = 3.079 \text{ m}$$

Total Correction =3.205 m

Corrected Length = 882.10-3.205 = 878.895 m

Extension of base line

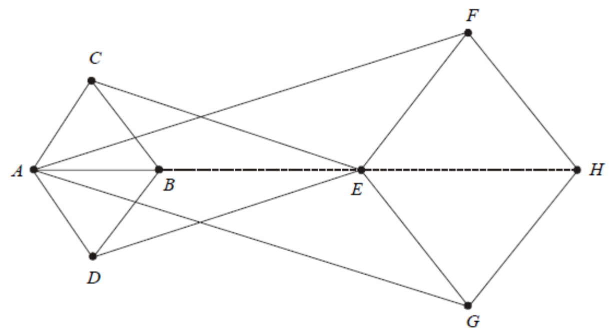
- Usually the length of the base lines is much shorter than the average length of the sides of the triangles.
- This is mainly due to the following reasons:
    - It is often not possible to get a suitable site for a longer base.
    - Measurement of a long base line is difficult and expensive.
- The extension of short base is done through forming a base net consisting of well-conditioned triangles.
- There are a great variety of the extension layouts but the following important points should be kept in mind in selecting the one.
    - ✓ Small angles opposite the known sides must be avoided.
    - ✓ The length of the base line should be as long as possible.
    - ✓ The length of the base line should be comparable with the mean side length of the triangulation net.

- ✓ A ratio of base length to the mean side length should be at least 0.5 so as to form well-conditioned triangles.
- ✓ The net should have sufficient redundant lines to provide three or four side equations within the figure.
- ✓ Subject to the above, it should provide the quickest extension with the fewest stations.
- ✓ There are two ways of connecting the selected base to the triangulation stations.

There are

- extension by prolongation, and
- extension by double sighting.

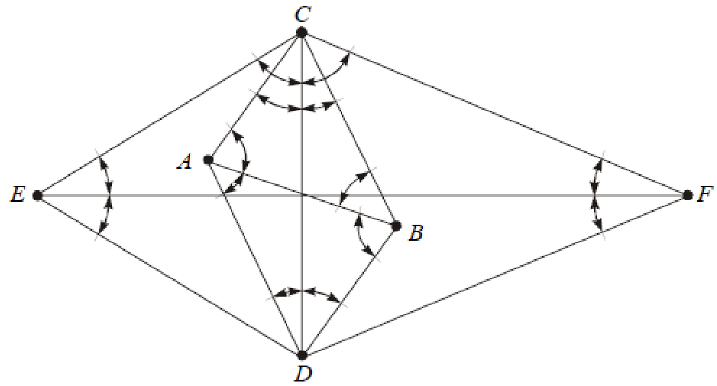
(a) Extension by prolongation



1. Let up suppose that *AB* is a short base line (in fig) which is required to be extended by four times.
2. The following steps are involved to extend *AB*.
3. Select *C* and *D* two points on either side of *AB* such that the triangles *BAC* and *BAD* are well conditioned.
4. Set up the theodolite over the station *A*, and prolong the line *AB* accurately to a point *E* which is visible from points *C* and *D*, ensuring that triangles *AEC* and *AED* are well-conditioned.
5. In triangle *ABC*, side *AB* is measured. The length of *AC* and *AD* are computed using the measured angles of the triangles *ABC* and *ABD*, respectively.
6. The length of *AE* is calculated using the measured angles of triangles *ACE* and *ADE,* and taking mean value.
7. Length of *BE* is also computed in similar manner using the measured angles of the triangles *BEC* and *BDE*.
8. The sum of lengths of *AB* and *BE* should agree with the length of *AE* obtained in step (6)
9. If found necessary, the base can be extended to *H* in the similar way.

(b) Extension by double sighting

1. Let *AB* be the base line (Fig. 1.38). To extend the base to the length of side *EF*, following steps are involved.
2. Chose intervisible points *C*, *D*, *E*, and *F*.
3. Measure all the angles marked in triangles *ABC* and *ABD*. The most probable values of these angles are found by the theory of least-squares.



4. Calculate the length of *CD* from these angles and the measured length *AB*, by applying the sine law to triangles *ACB* and *ADB* first, and then to triangles *ADC* and *BDC*.
5. The new base line *CD* can be further extended to the length *EF* following the same procedure as above.
6. The line *EF* may from a side of the triangulation system. If the base line *AB* is measured on a good site which is well located for extension and connection to the main triangulation system, the accuracy of the system is not much affected by the extension of the base line.

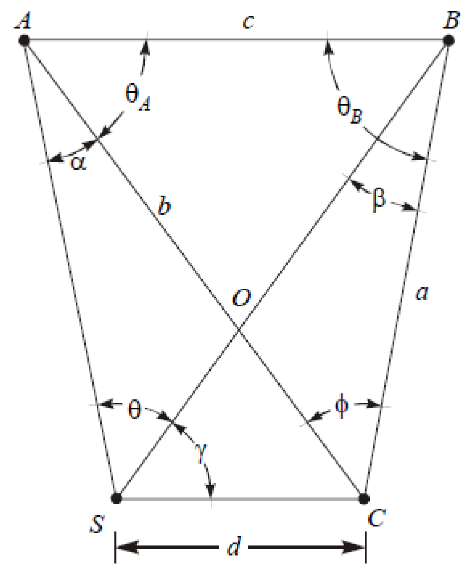
Satellite Station and Reduction to Centre

- To secure well-conditioned triangles or to have good visibility, objects such as chimneys, church spires, flat poles, towers, lighthouse, etc., are selected as triangulation stations.
- Such stations can be sighted from other stations but it is not possible to occupy the station directly below such excellent targets for making the observations by setting up the instrument over the station point.
- Signals are frequently blown out of position, and angles read on them have to be corrected to the true position of the triangulation station.
- Thus, there are two types of problems:

    1. When the instrument is not set up over the true station, and

2. When the target is out of position.

- In Fig. A, B, and C are the three triangulation stations.



- It is not possible to place instrument at C.
- To solve this problem another station S, in the vicinity of C, is selected where the instrument can be set up, and from where all the three stations are visible for making the angle observations.
- Such station is known as *satellite station.*
- As the observations from C are not possible, the observations form S are made on A, B, and, C from A and B on C.
- From the observations made, the required angle ACB is calculated. This is known as *reduction to centre.*
- In the other case, S is treated as the true station point, and the signal is considered to be shifted to the position C.
- This case may also be looked upon as a case of *eccentricity of signal.*
- Thus, the observations from S are made to the triangulation stations A and B, but from A and B the observations are made on the signal at the shifted position C.
- This causes errors in the measured values of the angles BAC and ABC.
- Both the problems discussed above are solved by reduction to centre.
- Let the measured
  - $\angle BAC = \theta A$
  - $\angle ABC = \theta B$

- $\angle ASB = \theta$

- $\angle BSC = \gamma$

- Eccentric distance $SC = d$

The distance AB is known by computations form preceding triangle of the triangular net.

- $\angle SAC = \alpha$

- $\angle SBC = \beta$

- $\angle ACB = \varphi$

- $AB = c$
- $AC = b$
- $BC = a$

As a first approximation in ΔABC the $\angle ACB$ may be taken as

$$= 180° - (\angle BAC + \angle ABC)$$

or $\varphi = 180° - (\theta_A + \theta_B)$ ------------------------------ (1)

In the triangle *ABC* we have

$$(c / \sin\varphi) = (a / \sin\theta_A) = (b / \sin\theta_B)$$

$$\mathbf{a} = (c / \sin\theta_A) / \sin\varphi \quad \text{------------------------------ (2)}$$

$$\mathbf{b} = (c / \sin\theta_B) / \sin\varphi \quad \text{------------------------------ (3)}$$

Compute the values of *a* and *b* by substituting the value of φ obtained from Eq. (1) in Eqs. (2) and (3), respectively.

Now, from Δ *SAC* and *SBC* we have

$$(d / \sin\alpha) = b / \sin(\theta + \gamma) = (b / \sin\theta_B)$$

$$(d / \sin\beta) = a / \sin\gamma$$

$$\sin\alpha = [d \sin(\theta + \gamma)] / b$$

$$\sin\beta = d \sin\gamma / a$$

As the satellite station *S* is chosen very close to the main station *C*, the angles α and β are extremely small.

Therefore, taking sin α = α, and sin β = β in radians, we get.

$$\alpha = [d \sin(\theta + \gamma)] / b \sin 1'' \quad \text{or}$$

$$= 206265\,[d \sin(\theta + \gamma)] / b \quad \text{in seconds} \quad \text{------------------(4)}$$

and $\beta = 206265\,[d \sin\gamma / a]$ in seconds ------------------(5)

In Eqs. (4) and (5), θ, γ, *d*, *b* and *a* are known quantities, therefore, the values of α and β can be computed.

Now a more correct value of the angle ∠*ACB* can be found. We have

$$\angle AOB = \theta + \alpha = \varphi + \beta \quad \text{or}$$

$$\varphi = \theta + \alpha - \beta \qquad \text{-------------------------------(6)}$$

Eq. (6) gives the value of φ when the satellite station *S* is to the left of the main station *C*. In the general, the following four cases as shown in Fig. Can occur depending on the field conditions.

Case I:

S towards the left of *C* (Fig. a)

$\varphi = \theta + \alpha - \beta$

Case II:

S towards the right of *C* (Fig. *b*), the position *S*2.

$\varphi = \theta - \alpha + \beta$

Case III:

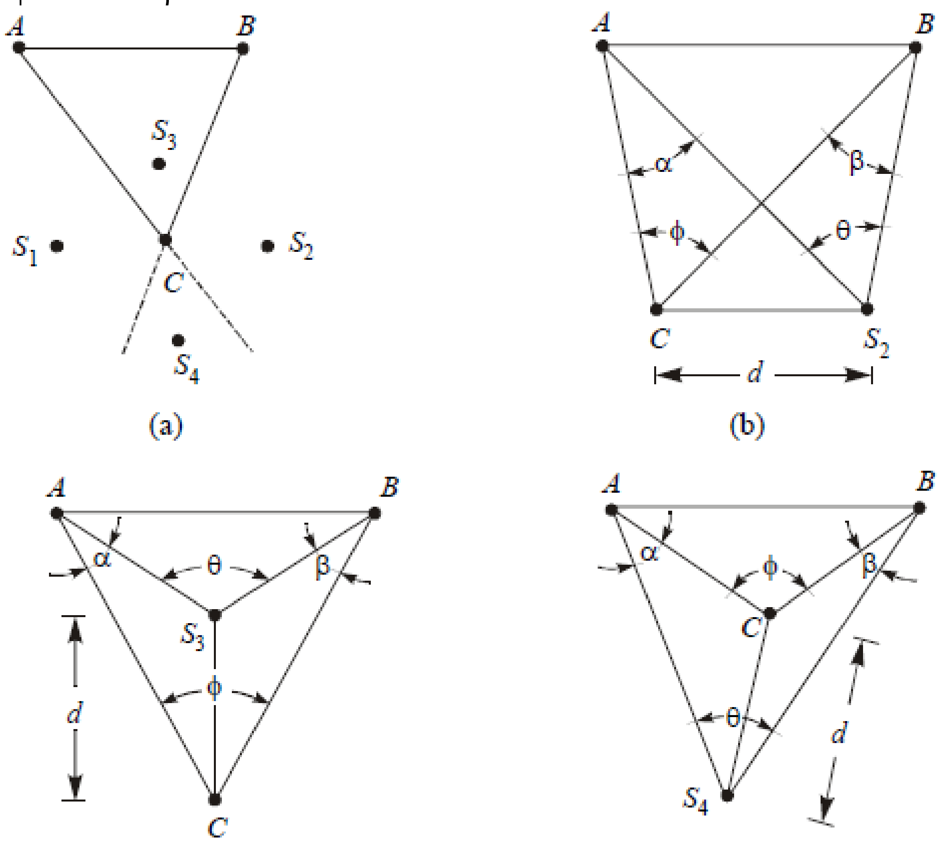
S inside the triangle *ABC* (Fig. *c*), the position *S*3.

$\varphi = \theta - \alpha - \beta$

Case IV:

S outside the triangle *ABC* (Fig. *d*), the position *S*4.

$\varphi = \theta + \alpha + \beta$



Geodetic observation

Curvature and refraction:

- The effect of curvature is to make the objects signted to appear lower in position than they are in real position.
- The effect of refraction is to make the objects to appear higher than they are in its position.
- The combined effect of curvature & refraction is that the objects appear lower than its position.
- In plane surveying where a graduated staff is observed with the horizontal or inclined line of sight the correction for curvature or refraction of combined correction is applied linearly to observed staff reading.
- In geodetic observations where the stations are widely distributed and at large distances the correction for curvature, refraction or combined is applied to the observed angles.

Co-efficient of Refraction :

- Co-efficient of refraction (m) is defined as the ratio of angle of refraction ® and the angle (θ) subtended at the centre of the earth by the distance over the observations.

i.e., $$m = \frac{r}{\theta} = \frac{\text{Angle of refraction}}{\text{Central angle of the earth}}$$

(Or)

$r = m\theta$

m -----→ varies from 0.6 to 0.9

Take the avg.value $m = 0.07$

- The co-efficient of refraction is determined for the following two cases.

(i) Distance d small and H large

- One angle α_1 is angle of elevation & the other angle β_1 is angle of depression.

$$\boxed{\beta_1 = \alpha_1 + \theta(1 - 2m)}$$

(or)

$$\boxed{r = \frac{\theta}{2} - \frac{\beta_1 - \alpha_1}{2}}$$

(ii) Distance d large and H small :

Both angles are angles of depression

Where,

θ ----> Central angle of each

α_1 ----> corrected angle of elevation for axis signal

β_1 ----> corrected angle of depression for axis signal

m ----> co-efficient of refraction

Correction for curvature (C_c)

$$Cc = \frac{\theta}{2} \qquad \theta = \frac{d}{R}$$

$$\boxed{Cc = \frac{\theta}{2} \times 206265 \quad \text{(or)} \quad \frac{d}{2R \sin 1''}}$$

Note :- If the θ is angle of elevation, correction $(+)^{ve}$

θ angle of depression, correction $(-)^{ve}$

Correction for refraction (Cr) :-

$$Cr = m\theta \qquad \theta = \frac{d}{R}$$

$$\boxed{Cr = \frac{md}{R \sin 1''} \quad \text{(or)} \quad \frac{md}{R} \times 206265}$$

Axis signal correction : () :-

- At the stations, the signals are erected at different heights. The signals may or may not be the height as that of the instrument.
- If the height of the signal is not the sameas that of the height of instrument axis but above the station, a correction known as axis signal correction or eye and object correction is to be applied.

$$\delta = \frac{s-h}{d} \times 206265$$

(or)

$$\delta = \frac{s-h}{d \sin 1''}$$

where,

----> central angle subtended at the centre of the earth.

s ----> height of signal

h ----> height of instrument

d ----> horizontal distance

R ----> Radius of the earth

m ----> co-efficient of refraction

m ----> surveyed over land, m = 0.07

surveyed over land m = 0.08

Note :- observed angle is angle of elevation, then the

Correction for curvature ----> (+)ve

Correction for refraction ----> (-v)^{ve}

Correction for axis signal ----> (-)^{ve}

If the observed angle is angle of depression, then the

Correction for curvature ----> (-)^{ve}

Correction for refraction ----> (+)^{ve}

Correction for axis signal ----> (+)^{ve}

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.

Error:

- Error is the numerical difference between the observed value of the quantity and its true value.

Types of Errors:

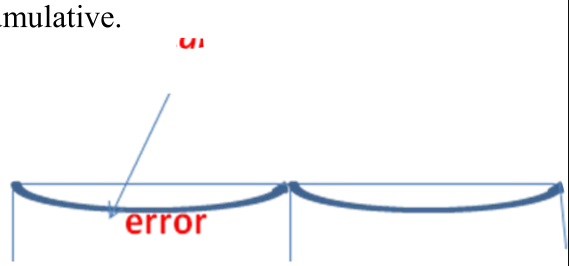
- Mistakes (or) gross Errors
- Systematic (or) Cumulative Errors
- Accidental (or) compensating (or) Random Errors

Mistakes (or) gross Errors:

- Mistakes are the errors that occur due to inexperience, carelessness and poor judgment, confusion in the minds of observer.

Systematic (or) Cumulative Errors:

- The systematic errors are the errors which always have some magnitude and same size and sign.
- Such errors generally (add up) positive or negative according with whether they make the result too small (or) too great. This effect is cumulative.
- It is simply due to the error in instrument.



Example:

- length of chain or tape – using measured incorrect chain length

Accidental (or) compensating (or) Random Errors:

- Accidental Errors occurs by a combination of reasons beyond the ability of the observer (surveyor) to control.
- They sometimes occur in one direction and sometimes in the other side.
- To make the apparent result too large or too small.
- The Accidental errors remain even after the observer quantity is corrected for mistakes and systematical errors.

Quantity:

- Quantity of a measurement made in correction with a survey.

Observed value of a Quantity:

- The observed value of a quantity is the true obtained as a result of an observation which is corrected for all errors.

Classification of Observed Quantity:

- An observer quantity may be classified as
  - Independent Quantity
  - Conditioned Quantity

Independent Quantity:

- The independent quantity is an observed quantity whose values does not depends upon any other quantity.

  Example: R.L of several B.M

Conditioned Quantity (or) Dependent Quantity

- The conditioned quantity or dependent quantity is an observed quantity whose value depends on one or more other quantities.
- ***Example:*** Sum of interior angles of the triangles = $\angle A + \angle B + \angle C = 180°0'$
- In-case triangle ABC, the value of any angle depends on the other two angles.

True value of Quantity:

- The true value of a quantity is the value which is absolutely free from all the errors.
- It is an intermediate since the true error is never known.

Observations:

- An observation is the numerical value of a measured quantity. There are classified as

Direct Observations:

- Direct observation is an observation which is made directly on the quantity to be observer.
- ***Example:*** measured length of a base line.

In-direct observation:

- In-direct observation is one in which in quantity to be observed then it is deduced from the measured value of a related quantity.
- ***Example:*** measurement of horizontal angle between any two lines by the method of repetition.

Weight of an observation:

- Weight of an observation is a measure of its relative worth (accuracy or precision) which may be indicated by a number.
- ***Example:*** if a certain observation is said to have weightage 5, it is meant to say that it is 5 times of as much as an observation of weight 5

Weighted observation:

- Observation are said to be weighted observations when different weights are assigned to them.
- Need for observation to be weighted occurs when unequal care and dissimilar conditions exist at the time of observation.
- Weights are assigned to the observations or quantities observed in direct proportion to the number of observations.

Observational Equation:

- An observational equation is the relation between the observed quantity and its numerical value.

True error:

- A true error is the difference between the true value of a quantity and its observed value.

Most probable error:

- It is the error which has added or subtracted from the most probable value of the measured quantity.

Residual error:

- It is the difference the most probable value of a quantity and its observed value.

Most probable value:

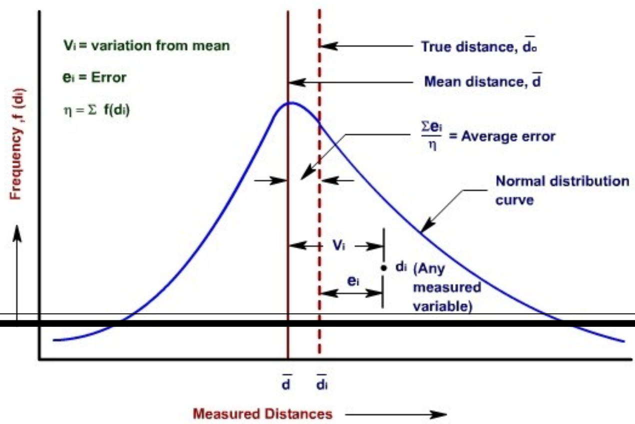
- The most probable value of a quantity is the value, which has more chances of being true than any other value.

Normal Equation:

- A normal equation is the one, which is formed by multiplying each equation by the coefficient of unknown, whose normal equation is to be found, and by adding the equations thus formed.
- The number of normal equations is the same as the number of unknowns.
- The most probable values of the unknowns are found out by using the normal equations.

Laws of Accidental Errors:

- Accidental errors are deals with based on the probability error curve.



This curve has been plotted between the size of the errors and their frequency of occurrence.

- Very small errors and very large errors (in magnitude) have a small chance of occurring.
- Positive and negative errors have equal chances of occurring. The curve is thus symmetrical about the mean error value.

Principles of least squares:

- It is found from the probability equation that the most probable values of a series of 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 a minimum.
- When a quantity is being deduced from a series of observations, the residual errors will be the difference between the adopted value and the several observed values,
- Let V1, V2, V3 etc. be the observed values
- x = most probable value

The laws of weights:

- From the method of least squares the following laws of weights are established:

1. ***The weight of the arithmetic mean of the measurements of unit weight is equal to the number of observations.***

- **For example,** let an angle A be measured six times, the following being the values:

<A	**Weight**	**<A**	**Weight**
30° 20′ 8”	1	30° 20′ 10”	1
30° 20′ 10”	1	30° 20′ 9”	1
30° 20′ 7”	1	30° 20′ 10”	1

- Arithmetic mean = 30° 20′ + 1/6 (8” + 10” + 7” + 10” + 9” + 10”)
  = 30° 20′ 9”.
- Weight of arithmetic mean = number of observations = 6.

2. ***The weight of the weighted arithmetic mean is equal to the sum of the individual weights***

- For example, let an angle A be measured six times, the following being the values:

<A	**Weight**	**<A**	**Weight**
30° 20′ 8”	2	30° 20′ 10”	3
30° 20′ 10”	3	30° 20′ 9”	4
30° 20′ 7”	2	30° 20′ 10”	2

- Sum of weights = 2 + 3 + 2 + 3 + 4 + 2 =16
- Arithmetic mean = 30° 20′ + 1/16 (8”X 2 + 10” X 3+ 7”X 2 + 10”X 3 + 9” X 4+ 10”X 2)

  = 30° 20′ 9”.
- Weight of arithmetic mean = 16.

3. ***The weight of algebraic sum of two or more quantities is equal to the reciprocals of the individual weights.***

- For example,

Let an angle	A	=	30° 20′ 8”	Weight 2
	B	=	15° 20′ 8”	Weight 3

- Sum of reciprocal individual weight = ¼ + ½ = ¾

Weight of A + B	=	(30° 20′ 8” + 15° 20′ 8”)	=	72° 50′ 30”
	=	1/ [(¼ +½)]	=	1/ (¾)
	=	4/3		
Weight of A - B	=	(30° 20′ 8” - 15° 20′ 8”)	=	11° 30′ 10”
	=	1/ [(¼ +½)]	=	1/ (¾)
	=	4/3		

4. ***If a quantity of given weight is multiplied by a factor, the weight of the result is obtained by dividing it's given weight by the square of the factor.***

- For example,

Let an angle A	=	42° 10′ 20”		Weight 6		
Then, weight of 3A	=	(126° 31′ 0”)				
	=	6/ [32]	=	6/ (9)	=	2/3

5. ***If a quantity of given weight is divided by a factor, the weight of the result is obtained by multiplying its given weight by the square of the factor.***

- For example,

Let an angle	A	=	42° 10′ 20”	Weight 6
Then, weight of	A/3	=	(14° 3′ 30”)	
		=	4 x [32]	
		=	4 x 9	
		=	36	

6. ***If a equation is multiplied by its own weight, the weight of the resulting equation is equal to the reciprocal of the weight of the equation.***
7. ***The weight of the equation remains unchanged, if all the signs of the equation are changed or if the equation is added or subtracted from a constant.***

Distribution of error of the field measurement:

- Whenever observations are made in the field, it is always necessary to check for the closing error, if any.
- The closing error should be distributed to the observed quantities.
- ***For examples***, the sum of the angles measured at a central angle should be 360°; the error should be distributed to the observed angles after giving proper weight age to the observations.
- The following rules should be applied for the distribution of errors:

- The correction to be applied to an observation is inversely proportional to the weight of the observation.

  The correction to be applied to an observation is directly proportional to the square of the probable error.
- In case of line of levels, the correction to be applied is proportional to the length.

Problem: 1

During a student's field exercise, one angle 'A' was measured by 12 students independently. The measured angles and the number of measurements are given below. Find the most probable value of the angle.

Angle	**Number of measurements**
48° 30′ 20"	3
48° 29′ 50"	4
48° 30′ 10"	3
48° 30′ 00"	2

Solution:

- The most probable value of an angle is equal to its weighted arithmetic mean

48° 30′ 20" x 3 = 145° 31′ 00"

48° 29′ 50" x 4 = 193° 59′ 20"

48° 30′ 10" x 3 = 145° 30′ 30"

48° 30′ 00" x 2 = 97° 00′ 00"

Sum = 582° 00′ 50"

Σ of weight = 3 + 4 + 3 + 2 = 12

Therefore,

Weighted arithmetic mean = 582° 00′ 50" / 12

= 48° 30′ 4.14"

Hence, most probable value of the angle = **48° 30′ 4.14"**

Method : 2 - Distribution of Error

- *Correction to be applied*
  - *Observation is inversely proportional to the weight of the observation*
  - *Observation is directly proportional to the square of the probable error*
  - *Proportional to the length*

Problem: 2

The angle of a triangle ABC were recorded as follows;

A	=	77° 14′ 20”	weight	=	4
B	=	49° 40′ 35”	weight	=	3
C	=	53° 04′ 53”	weight	=	2

Give the corrected values of the angles.

Solution:

Sum of observed angle = A + B+C

= 77° 14′ 20” + 49° 40′ 35” + 53° 04′ 53”

= 179º59’48”

Total correction E = 180º - (179º 59’ 48”)

= + 12”

Take C_1, C_2, C_3 are the (individual) corrections to the observed angle A, B, and C respectively

Therefore,

$$C_1 : C_2 : C_3 = (1/W_1) : (1/W_2) : (1/W_3)$$

$$C_1 : C_2 : C_3 = (1/4) : (1/3) : (1/2)$$

$$C_1 + C2 + C_3 = 12” \quad \ldots\ldots\ldots\ldots (1)$$

Take, $C_1 : C_2 = (1/4) : (1/3)$

$C_1 / C_2 = (1/4) / (1/3) = 0.25 / 0.333$

$$\boxed{C_2 = 1.3333\ C_1}$$

Take, $C_1 : C_3 = (1/4) : (1/2)$

$C_1 / C_3 = (1/4) / (1/2) = 0.25 / 0.50$

$$\boxed{C_3 = 2\ C_1}$$

Substituting the values **C_2 & C_3 in equation** (1)

$$C_1 + C2 + C_3 = 12”$$

$$C_1 + 1.3333\ C_1 + 2\ C_1 = 12”$$

$$C_1 = (12” / 4.333)$$

$$\boxed{C_1 = 2.77”}$$

$$C_2 = 1.3333\ C_1 = 1.3333 \times 2.77$$

$$\boxed{C_2 = 3.69”}$$

$$C_3 = 2\ C_1 = 2 \times 2.77$$

$$\boxed{C_3 = 5.54”}$$

Check:

$$C_1 + C2 + C_3 = 12”$$

$$2.77” + 3.69” + 5.54 = 12”$$

$$12” = 12”$$

Hence it is correct

Therefore, the corrected angle

$A = 77° 14' 20'' + 2.77'' = 77° 14' 22.77''$

$B = 49° 40' 35'' + 3.69'' = 49° 40' 38.69''$

$C = 53° 04' 53'' + 5.54'' = 53° 04' 58.54''$

Sum of corrected angle $= A + B + C$

$= 77° 14' 22.77'' + 49° 40' 38.69'' + 53° 04' 58.54''$

$= 180° 00' 00''$

Problem: 3

The following are the three angle of a triangle ABC was observed at a station X, the closing horizon with their probable errors of measurements. Determine their corrected values (find the error in the angle using the methods of distribution of errors);

$A = 78° 12' 10'' \quad \pm 2''$

$B = 136° 48' 32'' \quad \pm 3''$

$C = 144° 59' 08'' \quad \pm 5''$

Solution:

Sum of observed angle $= A + B + C$

$= 78° 12' 10'' + 136° 48' 32'' + 144° 59' 08''$

$= 359° 59' 50''$

Total correction $E = 360° - (359° 59' 50'')$

$= + 10''$

This error of 10" is to distributed by increasing in proportion to the square of the probable error.

Let C_1, C_2, C_3 are the (individual) corrections to the observed angle A, B, and C respectively

Therefore,

$$C_1 : C_2 : C_3 = (W_1)^2 : (W_2)^2 : (W_3)^2$$

$$C_1 : C_2 : C_3 = (2)^2 : (3)^2 : (5)^2$$

$$C_1 : C_2 : C_3 = 4 : 9 : 25$$

$$C_1 + C2 + C_3 = 10'' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Take, $C_1 : C_2 = 4 : 9$

$C_1 / C_2 = (4/9)$

$$\boxed{C_2 = 2.25\, C_1}$$

Take, $C_1 : C_3 = (4) : (25)$

$C_1 / C_3 = (4/25)$

$$\boxed{C_3 = 6.25\, C_1}$$

Substituting the values **C_2 & C_3 in equation (1)**

$C_1 + C2 + C_3 = 10''$

$C_1 + 2.25\,C_1 + 6.25\,C_1 = 10''$

$C_1 = (10'' / 9.5)$

$\boxed{C_1 = 1.05''}$

$C_2 = 2.25\,C_1 = 2.25 \times 1.05''$

$\boxed{C_2 = 2.37''}$

$C_3 = 6.25\,C_1 = 6.25 \times 1.05''$

$\boxed{C_3 = 6.58''}$

Check:

$C_1 + C2 + C_3 = 10''$

$1.05'' + 2.37'' + 6.58 = 10''$

$10'' = 10''$

Hence it is correct

Therefore, the corrected angle

$A = 78° 12' 10'' + 1.05'' = 78° 12' 11.05''$

$B = 136° 48' 32'' + 2.37'' = 136° 48' 34.37''$

$C = 144° 59' 08'' + 6.58'' = 144° 59' 14.58''$

Sum of corrected angle $= A + B + C$

$= 78° 12' 11.05'' + 136° 48' 34.37'' + 144° 59' 14.58''$

$= 360° 00' 00''$

Most Probable Values (MPV)

1. ***Direct observations of quantity of equal weights***

- Most probable value of directly observed quantity of equal weights is equal to the arithmetic mean of the observed values.
- $V_1, V_2, V_3, \ldots\ldots V_n$ are the observed values
- $M = (V_1 + V_2 + V_3 + \ldots\ldots + V_n) / n$

Where,

n = number of observations

M = Most probable value

2. ***Direct observations of quantities of unequal weights***

- Most probable value of directly observed quantity of unequal weights is equal to the weighted arithmetic mean of the observed values.
- $N = (W_1V_1 + W_2V_2 + W_3V_3 + \ldots\ldots + W_nV_n) / (W_1 + W_2 + W_3 + \ldots\ldots + W_n)$

Where,

- $V_1, V_2, V_3, \ldots\ldots V_n$ are the observed value of quantity

- W_1, W_2, W_3, ... W_n are the weight of observed values
- N = Most probable value of quantity

3. ***In-Direct observation of quantities involving equal or unequal weights***

- When the unknowns are independent of each other and their most probable values can be found by forming normal equations and solving of the unknowns.
- For example;

A	=	40° 00′ 10”
2A	=	80° 00′ 05”
6A	=	240° 00′ 00”

Forming normal equation,

(A x Coefficient 1)	=	A	=	40° 00′ 10”	x	1	=	40° 00′ 10”
(2A x Coefficient 2)	=	4A	=	80° 00′ 05”	x	2	=	160° 00′ 10”
(6A x Coefficient 6)	=	36A	=	240° 00′ 00”	x	6	=	1440° 00′ 00”

....

41 A = 1640° 00′ 20”

Therefore, the most probable value of ‘A’ = 1640° 00′ 20” / 41

$\boxed{A = 40° 00′ 0.49”}$

Problem: 4

Find the following most probable value of the angle Q from the following equations;

A	=	40° 28′ 32”
3A	=	120° 40′ 40”
4A	=	161° 05′ 28”

Solution:

Unknown = 1 i.e., = A

Weight = equal weight = 1

Therefore, multiplying these equations into the coefficients of each equation.

Forming normal equation,

(A x Coefficient 1)	=	A	=	40° 28′ 32”	x	1	=	40° 28′ 32”
(3A x Coefficient 3)	=	9A	=	120° 40′ 40”	x	3	=	362° 02′ 00”
(4A x Coefficient 4)	=	16A	=	161° 05′ 28”	x	4	=	644° 21′ 52”

....

26 ***A = 1046° 52′ 24”***

Therefore, the most probable value of ‘A’ = *1046° 52′ 24”* / 26

$\boxed{A = 40° 15′ 51.69”}$

The most probable value of ‘A’ = 40° 15′ 51.69”

Problem: 5

Find the following most probable value of the angle P from the following equations;

P	=	20° 20′ 20”	weight	2
3P	=	61° 10′ 20”	weight	3

Solution:

The observations are unequal weight

Unknown = 1 i.e., = P

Forming normal equation by multiplying each two observations by the corresponding weightage and the coefficient of ‘P’ and then adding them.

(P x Coefficient x weight) = P x 1 x 2 = 2P = 20°20′20” x 1 x 2 = 40° 40′ 40”

(3P x Coefficient x weight) = 3P x 3 x 3 = 27P = 61°10′20” x 3 x 3 = 550° 33′ 00”

....

29 *P* = *591° 13′40”*

Therefore, the most probable value of ‘P’ = *591° 13′ 40” / 29*

$\boxed{A = 20° 23' 13.79''}$

The most probable value of ‘P’ or normal equation for ‘P’ = 20° 23′ 13.79”

Method of differences

Problem: 6

A	=	42° 36′ 28”	weight =	2
B	=	28° 12′ 42”	weight =	2
C	=	65° 25′ 16”	weight =	1
A + B	=	70° 49′ 14”	weight =	2
B + C	=	93° 37′ 55”	weight =	1

Find the most probable value of A, B &C

Solution:

Let K_1, K_2 & K_3 be the most probable corrections to A, B & C respectively.

To find the values of K_1, K_2 & K_3

Let us assume the observed angle (value) of A, B & C as correct values.

(Hence $K_1 = 0$, $K_2 = 0$ & $K_3 = 0$)

Therefore,

$\boxed{\textit{Correction } K_1 = \textit{observed value of 'A' – correct value of A}}$

Correct value of ‘A’ = observed value of ‘A’ + correct ion K_1

A = **42° 36′ 28” + *k*1** ----------------------(1)

B = **28° 12′ 42” + *k*2** ----------------------(2)

$$C = 65° 25' 16'' + k3 \quad \text{-----------------------(3)}$$

$$A + B = 42° 36' 28'' + k_1 + 28° 12' 42'' + k_2$$

$$A + B = 70°49'10'' + k_1 + k_2 \quad \text{-----------------------(4)}$$

$$B + C = 28° 12' 42'' + k_2 + 65° 25' 16'' + k3$$

$$B + C = 93°37'58'' + k2 + k3 \quad \text{-------------------------(5)}$$

$k_1 + k_2$ = observed value of '(A+B)' – correct value of '(A+B)'

Equating Eq. (1) to the respective observed values, i.e.,

$$A = 42° 36' 28'' + k1$$

$$42° 36' 28'' = 42° 36' 28'' + k1$$

$$k1 = 0 \quad \text{---------------------------------------(a)}$$

Equating Eq. (2) to the respective observed values, i.e.,

$$B = 28° 12' 42'' + k2$$

$$28° 12' 42'' = 28° 12' 42'' + k2$$

$$k2 = 0 \quad \text{---------------------------------------(b)}$$

Equating Eq. (3) to the respective observed values, i.e.,

$$C = 65° 25' 16'' + k3$$

$$65° 25' 16'' = 65° 25' 16'' + k3$$

$$k3 = 0 \quad \text{---------------------------------------(c)}$$

Equating Eq. (4) to the respective observed values, i.e.,

$$A + B = 70°49'14'' + k_1 + k_2$$

$$70° 49' 14'' = 70°49'10'' + k_1 + k_2$$

$$k_1 + k_2 = 4'' \quad \text{---------------------------------------(d)}$$

Equating Eq. (5) to the respective observed values, i.e.,

$$B + C = 93°37'58'' + k2 + k3$$

$$93° 37' 55'' = 93°37'58'' + k2 + k3$$

$$k2 + k3 = -3'' \quad \text{---------------------------------------(e)}$$

Forming the normal equations for $k1$, $k2$ and $k3$, we get

$$k1 = 0 \quad \text{weight} = 2 \quad \text{---------------------------------------(a)}$$

$$k2 = 0 \quad \text{weight} = 2 \quad \text{---------------------------------------(b)}$$

$$k3 = 0 \quad \text{weight} = 1 \quad \text{---------------------------------------(c)}$$

$$k_1 + k_2 = 4'' \quad \text{weight} = 2 \quad \text{---------------------------------------(d)}$$

$$k2 + k3 = -3'' \quad \text{weight} = 1 \quad \text{---------------------------------------(e)}$$

Then the equation becomes

$$2k1 = 0 \quad \text{---------------------------------------(A)}$$

$$2k2 = 0 \quad \text{---------------------------------------(B)}$$

$$k3 = 0 \quad \text{---------------------------------------(C)}$$

$2k_1 + 2\,k_2 \quad = \quad 8''$ --------------------------------------(D)

$k2 + k3 \quad = \quad -3''$ --------------------------------------(E)

Forming the normal equations for k1

$2k1 \quad = \quad 0$

$2k_1 + 2\,k_2 \quad = \quad 8''$

$4k_1 + 2\,k_2 \quad = \quad 8''$ ------------------------------(I)

Forming the normal equations for k2

$2k2 \quad = \quad 0$

$2k_1 + 2\,k_2 \quad = \quad 8''$

$k2 + k3 \quad = \quad -3''$

$2k_1 + 5\,k_2 + k3 \quad = \quad 5''$ ------------------------------(II)

Forming the normal equations for k3

$k3 \quad = \quad 0$

$k2 + k3 \quad = \quad -3''$

$k_2 + 2k3 \quad = \quad -3''$ ------------------------------(III)

Hence the normal equations in $k1$, $k2$ and $k3$ are

$4k_1 + 2\,k_2 \quad = \quad 8''$ ------------------------------(I)

$2k_1 + 5\,k_2 + k3 \quad = \quad 5''$ ------------------------------(II)

$k_2 + 2k3 \quad = \quad -3''$ ------------------------------(III)

Solving these equations and we get

$k1 \quad = \quad 1.643''$

$k2 \quad = \quad 0.714''$

$k3 \quad = \quad -1.857''$

Hence the most probable values of A, B and C are,

$A = 42°\,36'\,28'' + k1 = 42°\,36'\,28'' + 1.643'' = 42°\,36'\,29.643''$

$B = 28°\,12'\,42'' + k2 = 28°\,12'\,42'' + 0.714'' = 28°12'\,42.714''$

$C = 65°\,25'\,16'' + k3 = 65°\,25'\,16'' - 1.857'' = 65°\,25'\,14.14''$

Problem: 7

Determine the adjusted values of the angles of the angles A, B and C from the following observed values by the method of differences.

A	=	39°14′15.3″	$A + B$	=	70°29′45.2″
B	=	31°15′26.4″	$B + C$	=	73°33′48.3″

$C \quad = \quad 42°18'18.4''$

Answer:

The solution of the above normal equations gives

$k1 = 0.88''$

$k2 = 1.75''$

$k3 = 0.88''$.

Therefore, the most probable values or the adjusted values of the angles are

$A = 39°14'15.3'' + 0.88'' =$ **$39°14'16.18''$**

$B = 31°15'26.4'' + 1.75'' =$ **$31°15'28.15''$**

$C = 42°18'18.4'' + 0.88'' =$ **$42°18'19.28''$**.

Indirect observation with conditional equation

Problem: 8

Determine the most probable values of angles *A, B* and *C of triangle ABC from the* following observed equations.

$A \quad = \quad 58°\ 46'\ 36''$

$B \quad = \quad 53°\ 12'\ 12''$

$C \quad = \quad 68°01'\ 18''$

Solution:

The conditional equation is

$$\mathbf{A + B + C \quad = \quad 180°\ 00'\ 00''}$$

i.e., $C \quad = \quad 180 - (A + B) \quad = \quad 68°\ 01'\ 18''$ -------- *(a)* *or*

$A + B \quad = \quad 180° - 68°\ 01'\ 18'' \quad = \quad 111°\ 58'\ 42''$

Forming normal equations

$A \quad = \quad 58°\ 46'\ 36''$

$B \quad = \quad 53°\ 12'\ 12''$

$A + B \quad = \quad 111°\ 58'\ 42''$

Normal equation for A

$A \quad = \quad 58°\ 46'\ 36''$

$A + B \quad = \quad 111°\ 58'\ 42''$

$2A + B = \quad 170°\ 45'\ 18''$ --(1)

Normal equation for B

$B \quad = \quad 53°\ 12'\ 12''$

$A + B$ = $111° 58′ 42″$

A + 2B= $165° 10′ 54″$ --(2)

Solving these equations (1) and (2), we get

A = $58° 46′ 34″$

B = $53° 12′ 10″$

Substituting these values in equation (a)

$C = 180 – (A + B) = 180 – (58° 46′ 34″ + 53° 12′ 10″)$

$\boxed{C = 68° 01′ 16″}$

Methods of correlates

- *Correlates are the unknown multiplies or independent constants are used finding the most probable value of unknowns.*

Problem: 9

The angles of triangles were recorded as follows;

A	=	77° 14′ 20”	weight	=	4
B	=	49° 40′ 35”	weight	=	3
C	=	53° 04′ 52”	weight	=	2

Determine the corrected values are the most probable value of the angles by methods of correlates.

Solution:

Sum of observed angle = A + B+C

= 77° 14′ 20” + 49° 40′ 35” + 53° 04′ 52”

= 179º59’47”

Total correction E = 180º - 179º59’47”

= + 13”

Let,

e_1, e_2 & e_3 are the individual corrections (Residual error)

$e_1 + e_2 + e_3 = 13”$ ……………………………..(1)

From the least squares principle, we have

ΣWe^2 should be a minimum

i.e., $W_1e_1^2 + W_2e_2^2 + W_3e_3^2 =$ a minimum ……………………(a)

Where,

W_1, W_2, W_3 are the weight of observations

Therefore, $4 e_1^2 + 3e_2^2 + 2e_3^2$ = a minimum ……………………………..(2)

Differentiating partially Eqs. (1) and (2), we get

(1) $e_1 + e_2 + e_3 = 13''$

$\partial e_1 + \partial e_2 + \partial e_3 = 0$ --(3)

(2) $4 e_1^2 + 3e_2^2 + 2e_3^2 =$ a minimum

$8 e_1 \partial e_1 + 6e_2 \partial e_2 + 4e_3 \partial e_3 = 0$

$2 [4 e_1 \partial e_1 + 3e_2 \partial e_2 + 2e_3 \partial e_3] = 0$

Therefore, $4 e_1 \partial e_1 + 3e_2 \partial e_2 + 2e_3 \partial e_3 = 0$ ------------------------------(4)

Multiplying Eq. (3) by $-\lambda$, and then adding the results to Eq. (4), we get

(3) $\partial e_1 + \partial e_2 + \partial e_3 = 0$

$-\lambda (\partial e_1 + \partial e_2 + \partial e_3) = 0$

$$\left.\begin{aligned} -\lambda\, \partial e_1 - \lambda\, \partial e_2 - \lambda\, \partial e_3 &= 0 \\ (4) \quad 4 e_1 \partial e_1 + 3e_2 \partial e_2 + 2e_3 \partial e_3 &= 0 \end{aligned}\right\}$$

--------------------------------------(5)

Adding both equations

$-\lambda\, \partial e_1\, 4\, e_1 \partial e_1 - \lambda\, \partial e_2\, 3e_2 \partial e_2 - \lambda\, \partial e_3\, 2e_3 \partial e_3 = 0$

$\partial e_1 [4 e_1 - \lambda] + \partial e_2 [3e_2 - \lambda] + \partial e_3 [2e_3 - \lambda] = 0$ ----------- (6)

For $\partial e_1, \partial e_2,$ *and* ∂e_3 are independent quantities, we have

$4 e_1 - \lambda = 0$ $\quad$ $3e_2 - \lambda = 0$ $\quad$ $2e_3 - \lambda = 0$

$e_1 = (\lambda/4)$ $\quad$ $e_2 = (\lambda/3)$ $\quad$ $e_3 = (\lambda/2)$

Substituting these values in equation (1)

(1) $e_1 + e_2 + e_3 = 13''$

$(\lambda/4) + (\lambda/3) + (\lambda/2) = 13''$

$0.25\,\lambda + 0.333\,\lambda + 0.50\,\lambda = 13''$

$$\boxed{\lambda = 12''}$$

Therefore

$e_1 = (\lambda/4) = (12/4) = 3''$

$e_2 = (\lambda/3) = (12/3) = 4''$

$e_3 = (\lambda/2) = (12/2) = 6''$

Therefore the corrected angles,

A = 77°14′ 20″ + 3″ = **77°14′ 23″**

B = 49°40′ 35″ + 4″ = **49°40′ 39″**

C = 53°04′ 52″ + 6″ = **53°04′ 58″**

Problem: 10

The following angles were measured at a station 'O' so as to close horizon.

AOB	=	83° 42′ 28.75"	weight	=	3
BOC	=	102° 15′ 43.26"	weight	=	2
COD	=	94° 38′ 27.22"	weight	=	4

DOA = 79° 23′ 23.77” weight = 2

Adjust the angles by methods of correlates.

Answer:

λ = *-1.895”*

e_1 = $(\lambda/3)$

e_2 = $(\lambda/2)$

e_3 = $(\lambda/4)$

e_4 = $(\lambda/2)$

corrected angles,

AOB = 83° 42′ 28.12”

BOC = 102° 15′ 42.31”

COD = 94° 38′ 28.75”

DOA = 79° 23′ 22.82”

Problem: 11

A surveyor carried out a levelling operations for a closed circuit ABCDA starting from ‘A’ and made the following observations.

B	was	8.164 m	above A	weight	=	2
C	was	6.284 m	above B	weight	=	2
D	was	5.626 m	above C	weight	=	3
D	was	19.964 m	above A	weight	=	3

Determine the probable heights of B, C, and D above ‘A’ by methods of correlates.

Solution:

Difference in elevation between A & D = 19.964 m

From the difference in elevation between the observation (A, B & C)

= 8.164 + 6.284 + 5.626

= 20.074 m

Correction E = 19.964 – 20.074

E = - 0.11 m

Let,

e_1, e_2, e_3 & e_4 are the individual corrections (Residual error)

$e_1 + e_2 + e_3 + e_4$ = -0.11 ……………………………..(1)

From the least squares principle, we have

ΣWe^2 should be a minimum

i.e., $W_1e_1^2 + W_2e_2^2 + W_3e_3^2 + W_4e_4^2 =$ a minimum ……………………(a)

Where,

W_1, W_2, W_3 & W_4 are the weight of observations

Therefore, $2e_1^2 + 2e_2^2 + 3e_3^2 + 3e_4^2$ = a minimum ……………………..(2)

Differentiating partially Eqs. (1) and (2), we get

(1) $e_1 + e_2 + e_3 + e_4 = -0.11$

$\partial e_1 + \partial e_2 + \partial e_3 + \partial e_4 = 0$ -----------------------------(3)

(2) $2e_1^2 + 2e_2^2 + 3e_3^2 + 3e_4^2$ = a minimum

$4e_1\partial e_1 + 4e_2\partial e_2 + 6e_3\partial e_3 + 6e_4\partial e_4 = 0$

$2[2e_1\partial e_1 + 2e_2\partial e_2 + 3e_3\partial e_3 + 3e_4\partial e_4] = 0$

Therefore, $2e_1\partial e_1 + 2e_2\partial e_2 + 3e_3\partial e_3 + 3e_4\partial e_4 = 0$ ------------------------------(4)

Multiplying Eq. (3) by –λ, and then adding the results to Eq. (4), we get

(3) $\partial e_1 + \partial e_2 + \partial e_3 + \partial e_4 = 0$

$-\lambda(\partial e_1 + \partial e_2 + \partial e_3 + \partial e_4) = 0$

$$\left.\begin{aligned} -\lambda\,\partial e_1 - \lambda\,\partial e_2 - \lambda\,\partial e_3 - \lambda\,\partial e_4 &= \\ 2e_1\partial e_1 + 2e_2\partial e_2 + 3e_3\partial e_3 + 3e_4\partial e_4 &= \end{aligned}\right\} \begin{aligned} &0 \quad \text{------------------------------(5)} \\ &0 \quad \textbf{Adding both equations} \end{aligned}$$

(4)

$$-\lambda\,\partial e_1\,2e_1\partial e_1 - \lambda\,\partial e_2\,2e_2\partial e_2 - \lambda\,\partial e_3\,3e_3\partial e_3 - \lambda\,\partial e_4\,3e_4\partial e_4 = 0$$

$$\partial e_1[2e_1 - \lambda] + \partial e_2[2e_2 - \lambda] + \partial e_3[3e_3 - \lambda] + \partial e_4[3e_4 - \lambda] = 0 \quad \text{----------- (6)}$$

For $\partial e_1, \partial e_2, \partial e_3$ *and* ∂e_4 are independent quantities, we have

$2e_1 - \lambda = 0$ $2e_2 - \lambda = 0$ $3e_3 - \lambda = 0$ $3e_4 - \lambda = 0$

$e_1 = (\lambda/2)$ $e_2 = (\lambda/2)$ $e_3 = (\lambda/3)$ $e_4 = (\lambda/3)$

Substituting these values in equation (1)

(1) $e_1 + e_2 + e_3 + e_4 = -0.11$

$(\lambda/2) + (\lambda/2) + (\lambda/3) + (\lambda/3) = -0.11$

$0.5\lambda + 0.5\lambda + 0.333\lambda + 0.333\lambda = -0.11$

$$\boxed{\lambda = -0.066\ m}$$

Therefore

$e_1 = (\lambda/2) = (-0.066/2) = -0.033\ m$

$e_2 = (\lambda/2) = (-0.066/2) = -0.033\ m$

$e_3 = (\lambda/3) = (-0.066/3) = -0.022\ m$

$e_4 = (\lambda/3) = (-0.066/3) = -0.022\ m$

Therefore the corrected levels are,

B = 8.164 – 0.033 = **8.131 m above 'A'**

C = 6.284 – 0.033 = **6.251 m above 'B'**

D = 5.626 – 0.033 = **5.604 m above 'C'**

Figure Adjustments:

- Figure adjustments are the determination of the most probable values of the angles involved in any geometrical figure. So as to fulfil the geometric requirements.
- The geometrical figures adopted in the triangulation systems are
    - *Triangles*
    - *Quadrilaterals*
    - *Polygons with central stations*

Rules for Figure Adjustments:

- Let us considered a triangle having an included angle A, B, and C.
- Take W_1, W_2, & W_3 be the weight of observed angle and also n_1, n_2 and n_3 be the number of observations for angles A, B, and C respectively.
- E_1, E_2, & E_3 are the most probable error in the angles A, B, and C.
- C_1, C_2, & C_3 be the corresponding corrections of A,B, & C.
- C be the total correction.

Rule: 1 – Equal weight correction

- If the observed angles of a triangle are equal weight, then the total error is equally distributed to the observed angles.
- $C_1 = C_2 = C_3 = (1/3)\ C$
- For example, if the total error is 6” then $C_1 = C_2 = C_3 = (6/3) = 2”$

Rule: 2 – Inverse weight correction

- If the observed angles of a triangle are unequal weight, then the total error is distributed to all the angles inverse proportion to the weights.
- $C_1 : C_2 : C_3 = (1/W_1) : (1/W_2) : (1/W_3)$
- $C_1 / (C_1 + C_2 + C_3) = (1/W_1) / [(1/W_1) + (1/W_2) + (1/W_3)]$
- $C_2 / (C_1 + C_2 + C_3) = (1/W_2) / [(1/W_1) + (1/W_2) + (1/W_3)]$
- $C_3 / (C_1 + C_2 + C_3) = (1/W_3) / [(1/W_1) + (1/W_2) + (1/W_3)]$

Rule: 3 – Inverse correction

- If the weight of observations are not given, then the error is distributed to all the angle is inverse proportion to their number of observations.
- $C_1 : C_2 : C_3 = (1/n_1) : (1/n_2) : (1/n_3)$
- $C_1 / (C_1 + C_2 + C_3) = (1/n_1) / [(1/n_1) + (1/n_2) + (1/n_3)]$
- $C_2 / (C_1 + C_2 + C_3) = (1/n_2) / [(1/n_1) + (1/n_2) + (1/n_3)]$
- $C_3 / (C_1 + C_2 + C_3) = (1/n_3) / [(1/n_1) + (1/n_2) + (1/n_3)]$

Rule: 4 – Inverse square correction

- If the error is distributed to all the angle is inverse proportion to the square of the number of observations.
- $C_1 : C_2 : C_3 = (1/n_1)^2 : (1/n_2)^2 : (1/n_3)^2$

- $C_1 / (C_1 + C_2 + C_3) = (1/n_1)^2 / [(1/n_1)^2 + (1/n_2)^2 + (1/n_3)^2]$
- $C_2 / (C_1 + C_2 + C_3) = (1/n_2)^2 / [(1/n_1)^2 + (1/n_2)^2 + (1/n_3)^2]$
- $C_3 / (C_1 + C_2 + C_3) = (1/n_3)^2 / [(1/n_1)^2 + (1/n_2)^2 + (1/n_3)^2]$

Rule: 5 – Probable error square correction

- If the probable errors of each angle of a triangles are known, then the error is distributed to all the angle in direct proportion to the squares of the probable error.
- $C_1 : C_2 : C_3 = E_1^2 : E_2^2 : E_3^2$
- $C_1 / (C_1 + C_2 + C_3) = (E_1^2) / [(E_1^2 + E_2^2 + E_3^2)]$
- $C_2 / (C_1 + C_2 + C_3) = E_2^2 / [(E_1^2 + E_2^2 + E_3^2)]$
- $C_3 / (C_1 + C_2 + C_3) = E_3^2 / [(E_1^2 + E_2^2 + E_3^2)]$

UNIT III TOTAL STATION SURVEYING 9

Basic Principle – Classifications -Electro-optical system: Measuring principle, Working principle, Sources of Error, Infrared and Laser Total Station instruments. Microwave system: Measuring principle, working principle, Sources of Error, Microwave Total Station instruments. Comparis on between Electro-optical and Microwave system. Care and maintenance of Total Station instruments. Modern positioning systems – Traversing and Trilateration.

Total Station:

- Total station is a combination of an electronic theodolite and an electronic distance meter.
- It is also integrated with microprocessor, electronic data collector and storage system.
- It is measured horizontal, vertical distance, angles and slope distances.

Measurement principles:

- *The combination of an electronic distance meter (EDM) and an electronic theodolite, it makes to determine the co-ordinates of a reflector by aligning the instruments cross hairs on the reflector and simultaneously measuring the vertical, horizontal angles and slope distances.*
- *A microprocessor in the instrument takes care of recoding, reading and the necessary computations.*
- *This data is easily transferred to a computer, where it can be used to generate a map.*
- *A total station fulfils several purposes (mine survey, cadastral survey, road/ rail/ canal survey).*
- *A total station involves the physics of making measurements, the geometry of calculations and statics for analysing the results of a traverse.*
- *In the field, it requires team work, planning and careful observations.*
- *It is equipped with data logger it also involves interfacing the data logger with a computer, transferring the data and working with the data on a computer.*

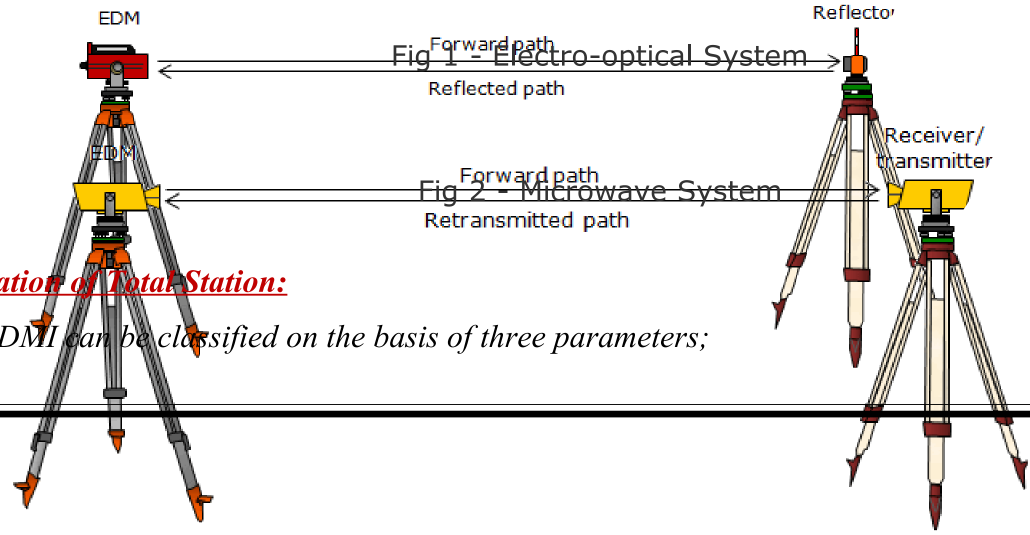


Fig 1 - Electro-optical System

Fig 2 - Microwave System

Classification of Total Station:

- *EDMI can be classified on the basis of three parameters;*

- ***Wave length used***
  - *Electronic optical system*
  - *Electronic or microwave system*
- ***Working range***
  - *Long range*
  - *Medium range*
  - *Short range*
- ***Achievable accuracy***

Classification based on wavelength used:

Present EDMI use the following types of wavelength;

- *Infrared*
- *Laser*

The above two types of systems are also knows as electro-optical system

- *Microwaves (or) Electronic System*

Electro optical System:

Infrared:

- *Systems employing these frequencies allow use of optical corner reflectors (special types of reflectors to return the signal) but need optically clean path between two stations.*
- *These systems use transmitter at one end of the line and a reflecting prism or target at other end.*

Laser:

- *These systems also use transmitter at one end of line and may or may not use a reflecting prism or tangent at the other end.*
- *The reflectors less laser instruments are used for short distances (100m to 350 m)*
- *These use light reflected off the future to be measured (say a wall)*

Electronic or Microwave Systems:

- *These systems have receiver / transmitter at both ends of measured line.*
- *Microwave instruments are often used for hydrographic surveys normally up to 100 km.*
- *Hydrographic EDMI have generally been replaced by global positioning system(GPS)*
- *These can be used in adverse weather conditions (such as fog and rain) unlike infrared and laser systems.*
- *However, uncertainties caused by varying humidity over measurement length may result in lower accuracy and prevent a more reliable estimate of probable accuracy,*

- *Existence of undesirable reflections and signal leakage from transmitter to the receiver requires the use of another transmitter at the remote station (or) slave station.*
- *The slave or remote station is operated at different carrier frequency in order to separate two signals.*
- *This additional transmitter and receiver add to weight of equipment.*
- *Multipath effects at microwave frequency also add to slight distance error which can be reduced by taking series of measurements using different frequency.*

Classification based on the range of EDMI:

- *Long range - Radio wave equipment's are used as the range of up to 100 km.*
- *Medium range - Microwave equipment with frequency modulation for ranges up to 25 km*
- *Short range - Electro-optical equipment using amplitude modulated infrared or visible light for ranges up to 5 km.*

Classification based on the accuracy of EDMI:

- *Accuracy of EDMI is generally stated in terms of constant instruments error and measuring error proportional to the distance being measured.*

  i.e; $\pm(a \text{ mm} + b \text{ mm})$

  - *The first part in the expression indicates a constant instrument error that is independent of the full length of the line measured.*
  - *The second component is the distance related error.*

Where,

a — Result of errors in phase measurements (θ) and zero error (Z)

b — Result from error in modulation frequency (f) and the group refractive index (n_g)

- *The term group index pertains to the refractive index for a combination of waves-carrier wave and multiple modulated waves in EDMI*
- *θ and Z are the independent of distance but f and n_g are functions of distance and are expressed as*

$$a = \sqrt{\sigma_\theta^2 + \sigma_Z^2}$$

$$b = \sqrt{\left(\frac{\sigma_f}{f}\right)^2 + \left(\frac{\sigma_{n_g}}{n_g}\right)^2}$$

Where,

σ indicates the standard error

- *Most of the EDMI have an accuracy levels from ± (3 mm + 1 ppm) to ±(10 mm +10 ppm)*
- *For short distances, part 'a' is more significant.*
- *For long distances, part 'b' will have large contribution.*

General classification of total station (available in market):

Mechanical / Manual Total Station:

- *The conventional multipurpose manual station are used for routine works with powerful built in applications program and are cheaper than the other type of Total Station.*

Motorized Total Station:

- *It is equipped with servo to allow for fast, smooth and accurate aiming.*
- *So it is increases the productivity by about 30 %*
- *The servo technology enables automated measurement.*
- *For example, during angle measurement one can simply aim the instrument at each end.*
- *The instrument can repeat the measurements automatically as many times are required.*
- *Servo equipped Total Station act as base for autolock and robotic surveying.*

Auto lock Total Station

- *It is allow for a semi- automatic measurement where measuring and recording takes place at the total station.*
- *In this case, the instrument searches for a active remote positioning target (RMT), locks to it and follows the target as it moves to different points.*
- *Autolock technology eliminates the need for time consuming error prone focussing and allows you to work effectively even in poor and low visibility environment.*
- *It improves the time efficiency by up to 50 %.*

Automatic/ Robotic Total Station:

- *This is a true one person surveying total station and is ideal for surveying and stake out operations.*

- *The control units can be taken to the prism to record measurements and collect other data.*
- *Generally a radio communication is used between Total Station and the prism. The control unit, battery, antenna and radio modem are integrated to allow full control over instrument and its operation.*
- *The prism used may be omni- directional (usually for short distance up to 500 m)*
- *Always aligned to the instrument or directional for longer distances.*
- *During stakeout, the control unit is used to move to point of interest.*
- *It improves the time efficiency by up to 80 %*

Field techniques with total station:

- *Various field operations in total station are in the form of wide variety of programs with microprocessor and implement with the help of data collector.*
- *All these programs need that the instrument station and atleast one reference station be identified, so that all subsequent stations can be identified interms of (X,Y,Z).*
- *Typical programs include the following functions,*
    - *Point location*
    - *Slope reduction*
    - *Missing Line Measurement (MLM)*
    - *Resection*
    - *Azimuth calculation*
    - *Remote distance and elevation measurement*
    - *Offset Measurements*
    - *Layout or setting out operation*
    - *Area computation*
    - *Tracking*
    - *Stakeout*

Classificatios:

- *Generally total station is classified in two categories, i.e,*
    - *Microwave System or Electronic System*

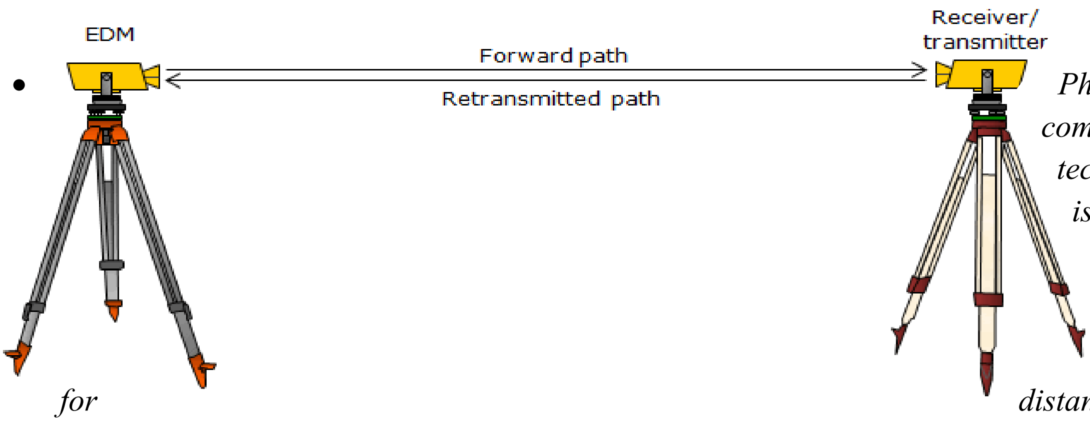
- *Electro Optical System*
    - *Infrared System*
    - *Visible light (or) laser light system*

Microwave system (or) Electronic system:

- *A microwave system is a system of equipment used for microwave data transmission.*
- *The typical microwave system includes radios located high a top microwave towers.*
- *It is used for the transmission of microwave communications using line of sight microwave radio technology.*
- *Frequency of wave is 1 GHz (1 GHz = 10^9Hz)*
- *Distance around 100 km is sunny weather conditions.*
- *Range of maximum for EDM microwave is 25 km- 30 km.*
- *Accuracy is with in +10 mm / +3 mm per km.*

  Example: Tellumat, Tellurometer.

Microwave instruments:

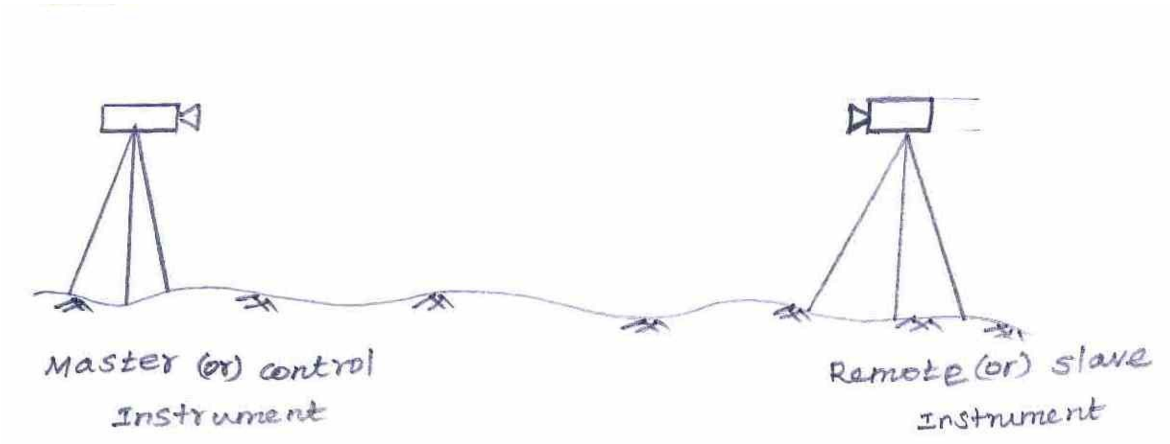
- *These are long range instruments.(Distance measured up to 100 km)*
- *Frequency range 3 to30 GHz (1 GHz = 10^9Hz)*
- 

  Phase comparison technique is used for distance measurement.
- *Erection of reflector (various form) at the remote end of line.*
- *Passive reflector is placed at other end.*
- *A weak signal would be available for phase comparison.*
- *Microwave EDM instruments require two instruments and two operators.*

- *Frequency modulation is used most of the microwave instruments.*
- *The method of varying the measuring wavelength in multiplies of 10 is used to obtain a correct measurement of distance.*
- *The microwave signals are radiated from small aerials (dipoles) mounted in front of each instrument.*
- *It producing directional signal with a beam of width varying from 2° to 20°. Hence the alignment of master and remote units is not critical.*
- *Maximum ranges for microwave instruments are from 30 to 80 km, with an accuracy of ±15 mm to ±5 mm/km.*

Tellurometer:

- *High frequency radio waves (or microwaves) are used instead of light waves.*
- *It can be worked with a light weight 12 or 24 volt battery. Hence it is portable.*
- *The observations are taken both during day as well as night.(but geodimeter observations are normally restricted in the night)*
- *The tellurometers are required; one is to be stationed at each end of the line, with two highly skilled persons, to take observations.*
- *One instrument is used as the master set or control and the other instrument is used as the remote set or slave set.*

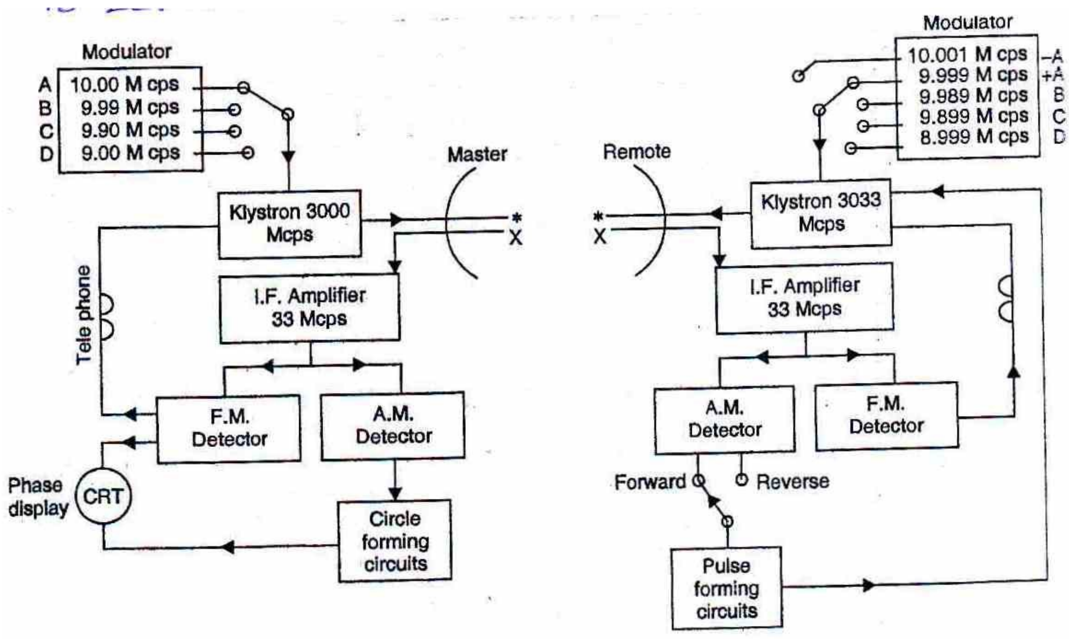


- *Model MRA-2 (manufactured by M/s. Cooke, Troughten and Simms Ltd)*

Block Diagram:

- *It was designed by T.L. Wadley, South America council for scientific and industrial research.*

- *Radio waves (microwave) are emitted by the master instrument at a frequency of 3000 Mc.s (3 $\times 10^9$ C.P.S) from a Klystron, and have super imposed on them a crystal controlled frequency of 10 Mc.s. The high frequency wave is termed as carrier wave.*



- *High frequency wave can be propagated in straight line paths other than long distance much more rapidly.*
- *The low frequency wave is known as the pattern wave and it is used for making accurate measurements.*
- *The light frequency pattern wave is said to be frequency modulated (F.M) by low frequency pattern wave.*
- *Modulated signal is received at the remote station where a second klystron is generating another carrier wave at 3033 Mc.s*
- *The difference between the two frequencies*
- *i.e, 3033-3000 = 33 Mc.s(intermediate frequency)*
- *It is obtained by an electrical mixer and is used to provide sensitivity in the internal detector circuits at each instrument.*
- *In addition to the carrier wave of 3033 Mc.s a crystal at the remote station is generating a frequency of 9.999 Mc.s*
- *This is heterodyned with the incoming 10 Mc.s to provide a 1 K.c.p.s signal.*
- *The 33 Mc.s intermediate frequency signal is amplitude modulated by 1 K.c.p.s signal.*
- *The amplitude modulated signal passes to the amplitude demodulator, which detects the 1 K.c.p.s frequency.*

- *The pulse forming circuit, a pulse with a repetition frequency of 1 K.c.p.s is obtained.*
- *Then the pulse is applied to the klystron and frequency modulates the signal emitted.ie, 3033 Mc.s modulated by 9.999 Mc.s and pulse of 1 K.c.p.s .*
- *The signal received at the master station.*
- *Further compound heterodyne processes takes place and here the two carrier frequencies subtracts to an intermediate frequency of 33 Mc.s*
- *The two pattern frequencies of 10 and 9.999Mc.s also substract to provide 1 K.c.p.s reference frequency as amplitude modulation.*
- *The change in the phase between this and the remote 1 k.c.p.s signal is measure of distance.*
- *The value of phase delay is expressed in time units and appear as a break in a circular trace on the oscilloscope cathode ray tube.*
- *Four low frequencies (A,B,C and D) of values 10.00,9.99, 9.90, and 9.00Mc.p.s are employed as the master station.*
- *The values of phase delays corresponding to each of these measured on the oscilloscope cathode ray tube.*
- *The phase of delay of B, C and D are subtracted from A in turn.*
- *The A values are termed as 'fine reading' and B, C, D values as coarse readings.*
- *The oscilloscope scale is divided into 100 parts*
- *The wavelength of 10 Mc.s pattern wave as approximately 100 ft(30m) and hence each division of the scale represents 1 foot on the two way journey of the waves or approximately 0.5 foot on the length of the line.*
- *The final readings of A, A-B, A-C and A-D readings are recorded in millimicro seconds (10^{-9}seconds) and are converted into distance readings by assuming that the velocity of wave propagations as 299, 792.5 km/sec.*
- *It should be noted that the success of the system depends on a property of the heterodyne process.*
- *The phase difference between two heterodyne signal is maintained in the signal, that results from the mixing.*

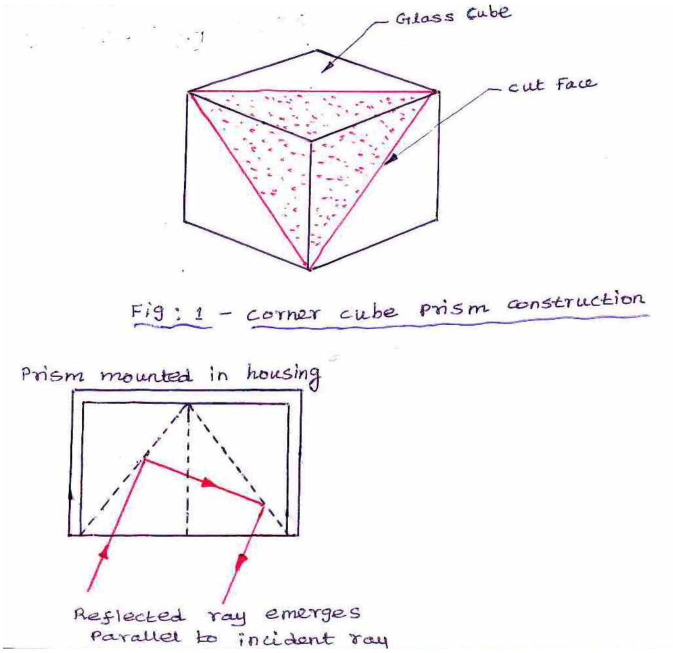
Electro optical system:

- *The use of infrared EDM equipment is a simple and easy method in which most of the tools used to work surveying.*

- *The use of infrared EDM equipment cause carrier wave is an infrared emitting diode arsenaid gallium (GaAs)*
- *Single prism limited to the range of 1 km but it can be added to the 2 or 3 km by using a reflector consisting of a sequent of 3 or 9 prisms.*
- *Accuracy is with in ± 10 mm.*

  Example: wild, geodimeter ,sokia, tapoor, leica and kern.

Visible light instruments:

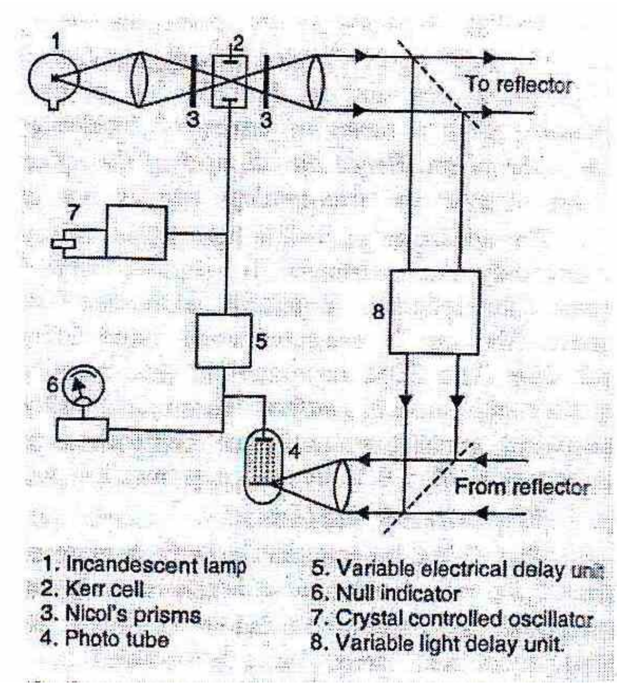


- *Prism mounted in housing*
- *Visible light is used as the carrier wave with a higher frequency of 5×10^{14}Hz.*
- *The transmitting power of carrier wave of such high frequency falls off rapidly with the distance, the range of EDM instruments in lesser than those microwave units.*
- *Example:* *Geodimeter.*
- *The carrier wave transmitted as light beam, is concentrated on a signal using lens or mirror system.(there is no loss of signal at that place).*
- *The beam divergence is less than 1°, accurate alignment of the instrument is necessary.*
- *Corner cube prism,(in fig) are used as reflectors at the remote end.*
- *These prisms are constructed from the corners of glass cubes which have been cut away in a plane making at an angle of 45° with the faces of the cube.*
- *The light wave directed into the cut face is reflected by highly silvered inner surfaces of the prism, resulting in the reflection of the light beam along parallel path.*
- *This is obtained over arrange of angles of incidence of about 20° to the normal of the front face of the prism.*
- *Hence the alignment of the reflecting prism towards the main EDM instrument at the receiver or (transmitting) end is not critical.*
- *The advantage of visible light EDM instruments, over the microwave EDM instruments is that only one instrument is required.*

- *Line is measured using three different wave lengths, using carrier or microwave in each case.*
- *In this type of instruments are measured at the range of 25 km, with an accuracy of*
- *±10 mm/ km to ±2 mm/km.*
- *The recent instruments use pulsed light sources and highly specialized modulation and phase comparison techniques, so it has produce a very high degree of accuracy of 0.2 mm/ km to ±1 mm/km with a range 2 to 3km.*

Geodimeter:

- *It is based on propagation of modulated light waves was developed by E. Bergestrand of the Swedish Geographical survey in collaboration with the manufacturer,(M/S. AGA of Sweden).*
- *Model 2-A can be used only for observations made at night.*
- *Model - 4 can be used for limited day time observations.*



Schematic Diagram of the Geodimeter

Working /Measuring Principles:

- *Figure shows, the photograph of the front panel of model -4 geodimeter mounted on the tripod.*
- *The main instrument is stationed at one end of the line (to be measured)with its back facing, the other end of the line , while a reflector (consisting either of a spherical mirror or a reflex prism system)is placed at the other end of the line.*
- *The light from an incandescent lamp (1) is focused by means of an achromatic condenser and passed through a kerr cell (2).*
- *The kerr cell consists of two closely spaced conducting plates, the space between which is filled with nitrobenzene.*
- *When high voltage is applied to the plates of the cell and a ray of light is focussed on it.*

- *The ray is split into two parts, each moving with different velocity.*
- *Two nicol's prisms (3) are placed on either side of the kerr cell.*
- *The light leaving the first nicol's prisms is plane polarised (divide into two groups with completely opposite views.)*
- *The light is split into two (having a phase difference) by the kerr cell. on leaving the kerr cell, the light is recombined.*
- *However because of phase difference, the resulting beam is elliptically polarised.*
- *Diverging light from the second polarised can be focused to the parallel beam by the transmitter objective, and then can be reflected from a mirror lens to a large spherical concave mirror.*
- *On the other end of the line being measured is put a reflex prism system or a spherical mirror, which reflects the beam of light back to the geodimeter.*
- *The receiver system of the geodimeter consists of spherical concave mirror, mirror lens and receiver objective.*
- *The light of variable intensity after reflection, have an effect on the cathode of the photo tube (4).*
- *In the photo tube, the light photons impact on the cathode causing a few primary electrons to leave and travel accelerated by a high frequency voltage, to the first dynode, where the secondary emission takes place.*
- *This is repeated through a further eight dynodes.*
- *The final electron current at the anode is some hundreds of thousand times greater than that at the cathode.*
- *The sensitivity of the photo tube is varied by applying the high frequency kerr cell voltage between the cathode and the first dynode.*
- *The low frequency vibrations are eliminated by a series of electrical chokes and condensers.*
- *The passage of this modulating voltage through the instrument is delayed by means of an adjustable electrical delay unit (5).*
- *The difference between the photo tube currents during the positive and negative bias period is measured on the null indicator (6) which is a sensitive D.C moving coil micro-ammeter.*
- *To make both positive and negative current intensifies equal (ie, to obtain null point), the phase of the high frequency voltage from the kerr cell must be adjusted ± 90° with respect to the voltage generated by light at the cathode.*

- *The light is focussed to a narrow beam from the geodimeter stationed at other end to the reflector stationed at the other end of the line.*
- *It is reflected back to the photo multiplier.*
- *The variation in the intensity of this reflected light causes the current from the photo multiplier to vary where the current is already being varied by the direct signal from the crystal controlled oscillator (7).*
- *The phase difference between the two pulses received by the cell are measure of the distance between geodimeter and reflector (ie, length of the line).*
- *The distance can be measured at different frequencies,*
    - *Model -2A ----- Three frequencies are available.*
    - *Model -4 ----- Four frequencies are available on phase position indicator.*
- *The polarity of the kerr cell terminals of high and low tension are reversed in turn.*
- *Fine and coarse delays switches control the setting of the electrical delay between the kerr cell and the photo multiplier.*
- *The power required is obtained from a mobile gasoline generator.*
- *Model -4A has a night range of 15 meters to 15 km,*

  Day light range of 15 to 800 meters

  Average error of ±10 mm ± five millionth of distance

  Weight about 36 kg without generators.

Infrared instruments:

- *Infrared radiation band of wavelength about 0.9 μm as carrier wave which is easily obtained from gallium arsenide (Ga.As) infrared emitting diode.*
- *These diodes can be very easily directly amplitude modulated at high frequencies.*
- *Modulated carrier wave is obtained by an inexpensive method.*
- *Example: wild distomats.*
- *Power output of the diode is low.*
- *The range of these instruments limited to 2 to 5 km.3*
- *It is mostly suitable for civil engineering works.*
- *These instruments are very light and compact and theodolite can be mounted.*

- *The angles and distances to be measured simultaneously at the site.*
- *A typical combination is*
    - *Wild DI 1000 infrared EDM*
    - *Wild T 1000 electronic theodolite (theomat)*
    - *Wild TC 2000 electronic tacheometer (tanchymat)*
- *Microprocessor controlled angle measurements give very high degree of accuracy, enabling horizontal and vertical angles and the distances (horizontal, vertical and inclined) to be automatically displaced and recorded.*

Advantages:

- *Rapid measurement* -----*0.8 second for detail survey*
- *Long range* ----- *6 km to 1 prism in average condition*

  14 km to 11 prisms in excellent condition.
- *High accuracy* ----- *5 mm +1 ppm standard deviation*

  ----- *10 mm + 1 ppm tracking mode*

  ----- *Temperature ranges -20°c to +60 ° c*
- *Measurement to moving tangent* ----- *operation to moving object*
- *Used for off shore surveys* ----- *measuring to ship, dredges, pipe line laying, oil rings etc.*
- *Controlling objects on rails* ----- *position of cranes, gantries, vehicle, rail etc.*
- *Positioning & monitoring Movements in deformation Survey ----- bridges, load test etc.*

Wild Distomats:

- *Wild heerbrug manufacture EDM equipment under the trade name 'distomat' having the following popular models:*
- *Distomat DI 1000*
- *Distomat DI 5S*
- *Distomat DI 3000*
- *Distomat DI or 3002*
- *Tachymat TC 2000(Electonic tacheometer)*

Distomat DI 1000:

- *It is very small, compact EDM*
- *Used for building construction, civil Engineering construction, cadastral and detail survey, particularly in populated areas(where 99 % of distance measurements are less than 500m)*
- *It has a range of 500 m to a single prism and 800 m to three prisms (1000m in favourable conditions) with an accuracy of 5 mm + 5 ppm.*
- *It can be filled to all wild theodolites (such as T 2000, T 2000 S, T 2 etc.)*
- *Infrared measuring beam is reflected by a prism at the other end of the line.*
- *In the 5 seconds that is takes the DI 1000 adjust the signal strength to optimum level makes 2048 measurements on two frequencies, carries out a full internal calibration, computers and displays the result.*
- *In tracking mode, 0.3 second updates follow the initial 3 second measurement.*
- *The whole sequence is automatic, one has to simply point to the reflector, touch a key and read the result.*
- *The wild modular system ensures full compatibility between theodolites and distomats.*
- *DI 1000 fits* T_1, T_{16}, T_2 *optical theodolites*
- *Optical keyboard can be used.*
- *It also combines with wild T 1000 Electronic theodolites and the wild 2000 informatics theodolite to form fully electronic total station.*
- *Measurement, reductions and calculations are carried out automatically.*
- *DI 1000 also connects to the GRE 3 data terminal*
- *GRE 3 is connected to an electronic theodolite with DI 1000 all information is transferred and recorded at the touch of a single key.*
- *GRE can be programmed to carry out field checks and computations.*
- *DI Distomat is used separately, it can be controlled from its own key board. Ie, three keys each with three functions.*
- *Colour coding and a logical operating sequence ensure that the instrument is easy to use.*
- *Key controls all the functions. There are no mechanical switches.*
- *Measured distances are presented clearly and accurately with appropriate symbols for slope, horizontal distance, height and setting out.*

- *In test mode, a full check is provided of the display battery power and return signal strength.*
- *To indicate return of signal scale (ppm) and additive constant (mm) settings are displayed at the start of each measurement.*
- *Input of ppm takes care of any atmospheric correction, reduction to sea level and projection scale factor.*
- *The main input correct for the prism type being used.*
- *Microprocessor permanently stores ppm amd mm values and applies them to every measurement.*
- *Displayed heights are corrected for earth curvature and mean refraction.*
- *DI is designed for use as the standard measuring tool in short range work.*

Distomat DI 5S:

- *It is a medium range infrared EDM controlled by a small powerful microprocessor. It is multipurpose EDM.*
- *The 2.5 km range to single prism covers all short range requirements; detail, cadastral, Engineering, topographic survey, setting out, mining, tunnelling etc.*
- *The 5 km range to 11 prisms, it is ideal for medium range control survey: traversing, trigonometric heighting, photogrammetric control, breakdown of triangulation and GPS networks etc.*
- *Finally turned opto- electronics, a stable oscillator, and a microprocessor that continuously evaluates the results, ensure the high measuring accuracy of 3 mm + 2ppm standard deviation is standard measuring mode and 10 m + 2ppm standard deviation in tracking measuring mode.*
- *It has three control keys; each with three functions.*
- *There are no mechanical switches.*
- *A powerful microprocessor controls the DI 5S.*
- *Simply touch the DIST key to measure.*
- *Signal attenuation is fully automatic.*
- *Typical measuring time is 4 seconds.*
- *In tracking mode the measurement repeats automatically every second.*
- *A break in the measuring beam due to traffic etc., does not affect the accuracy.*

- *Large, liquid- crystal display shows the measured distance clearly and throughout the entire measuring range of the instrument.*
- *Symbols indicate the displayed values.*
- *A series of dashes shows the progress of the measuring cycle.*
- *Prism constant from – 99 mm to +99 mm can be input for the prism type being used.*
- *Ppm values from -150 ppm to +150 ppm can be input for automatic compensation for atmospheric conditions, height above sea level and projection scale factor.*
- *These values are stored until replaced by the new values.*
- *Microprocessor corrects every measurement automatically.*
- *DI 5S fitted to wild electronic theodolites T 1000, T 2000 or to wild optical theodolites* T_1, T_{16}, T_2.
- *Infrared measuring beam is parallel to the line of signal.*
- *Only a single processing is needed for both angle and distance measurements.*
- *When fitted to an optical theodolite, an optical keyboard converts it to efficient low cost effective total station.*
- *The following parameters are directly obtained for the corresponding input values;*
  - *Input the vertical angle for*
  - *Horizontal distance*
  - *Height difference corrected for earth curvature and mean refraction.*
  - *Input the horizontal angle for co-ordinate differences* Δ *E and* Δ *N*
  - *Input the distance to be set out for* Δ*D, the amount by which the reflector has to be moved forward or back.*
  - *Fitted with an electronic theodolite (T 1000 or T 2000) DI 5S transfers the slope distance to the theodolite.*

Distomat DI 3000 and DI 3002:

- *It is a long range infrared EDM, in which infrared measuring beam is emitted from a laser diode.*
- *Class-1 laser products are inherently safe.*
- *Maximum permissible exposure cannot be exceeded under any condition, as defined by international Electro technical commission.*

- *It is the time pulsed EDM.*
- *The time needed for a pulse of infrared lights to travel from the instrument of the reflector and back is measured.*
- *The displayed result is mean of hundreds or even thousands of time- pulsed measurements.*
- *The pulse techniques has the following advantages,*

Rapid measurement:

- *It provides 0.8 second for detail surveys, tacheometry, setting out etc.,*

Long range:

Condition	***Range***
Average	*6 km to 1 prism*
Excellent	*14 km to 11 prism*

High accuracy:

- *Standard deviation, accuracy = 5 mm +1 ppm*
- *Tracking mode, accuracy = 10 mm +1 ppm*
- *For 1 ppm the temperature range is -20°c to +60°c*

Offshore surveys:

- *Mounted on electronic theodolite for measuring to ships, dredgers, pipe laying barges, positioning oil rigs, controlling docking manoeuvres etc.,*

Controlling objects on rails:

- *Connected on-line to computer for controlling the position of cranes, gantries, vehicles, machinery on rails trucked equipment etc.*

Monitoring movements in deformation surveys:

- *It can be connected with DI 3000 and GRE 3 or computer for continuous measurement rapidly deforming structure (such as bridges, undergoing load test)*

Positioning moving machinery:

- *DI 3000 can be mounted on a theodolite for continuous determination of the position of mobile equipment.*

For conventional measurements in Surveying and Engineering:

- *Control surveys, traversing, trigonometric heighting, breakdown of the GPS networks, cadastral, detail and topographic survey setting out etc.*

DISTOMAT DIOR 3002:

- *It is a special version of the DI 3000.*
- *It is designed specifically for distance measurement without reflector.*
- *DIOR 3002 is also time pulsed infra-red EDM.*
- *For without reflector ----- ranges varies from100 m to 250 m*

  Standard deviation = 5 mm to 10 mm

- *For with reflector ----- Range of 4 km to 1 prism*

  Range of 5 km to 3 prisms

  Range of 6 km to 11 prisms

- *DIOR 3002 can fitted on any of the main wild theodolite, T 1000 electronic theodolite is mostly suitable.*
- *For without reflector, it can carry the following operations,*

Profile and cross section:

- *DIOR 3002 with an electronic theodolite can be used for measuring tunnel profiles and cross-section surveying slopes, caverns, interior of storage tanks, domes etc.*

Surveying and monitoring buildings, large objects quarries, rock faces, stock piles:

- *DIOR 3002 with a theodolite and data recorder can be used for measuring and monitoring large objects, to which access is difficult, such as bridges, buildings, cooling towers, pylons , roofs, rock faces, towers, stock piles etc.*

Checking liquid levels, measuring to dangerous or touch sensitive surfaces:

- *DIOR 3002 on –line to a computer can be used for controlling the level of liquids in storage tanks.*
- *Determining water level in docks, harbours.*
- *Measuring the amplitude of waves around oil rigs etc., also for measuring to dangerous surfaces such as furnace lining, hot tubes, pipes, and rods.*

Landing and docking manoeuvres:

- *It can be used for measuring from helicopters to landing pads, and ships to piers and dock walls.*

Sources of errors:

Personal error

- *Centring*
- *Height measurement*
- *Atmospheric conditions determination.*

Instrumental error

- *Levelling bubbles*
- *Optical plummet*
- *Manufacturer's stated accuracy (MSA)*
- *Combined constant*
- *Prism height*

Natural errors

- *Atmospheric conditions*
- *Refraction and curves*
- *Atmospheric anomalies*

Personal error:

- *It has to be careful for*
    - *Precise centring at the master and slave station*
    - *Pointing/ sighting of reflector.*
    - *Entry of correct values of prevailing atmosphere conditions.*

Centering:

- *It involves how accurately the operator can centre the total station instrument (TSI) or tribrach vertically over the ground mark.*
- *It using a hand held prism held prism pole, how carefully the rod person holds the bubble centred.*

Height instrument:

- *If the TSI will be used for trigonometric levelling or topo data collection than the heights of the instrument and prism must be measured.*

Atmospheric condition determination:

- *Temperature and barometric pressure must be obtained for the time of measurement.*
- *If not available, then the operator should record the settings on the TST so later on they can be compensated.*

Instrumental error:

It consist of three components ie,

- *Scale zero*
- *Zero error* — *these are systematic in nature.*
- *Cyclic error*

Levelling Bubbles:

- *At the TSI, proper levelling techniques should be used to compensate for the plate bubble being out of adjustment.*
- *The prism may be mounted in a tribrach in which case the tribrach bubble can be checked as on the TSI*

Optical plummet:

- *Optical plummets on the TSI and prism tribrach are used to orient the instrument vertically over its ground mark.*
- *These should be checked and adjustment as necessary.*
- *On the TSI, with a plummet that rotates with the instrument, the plummet can be checked by using it to set up over a mark, then rotating the instrument 180°; the plummet should stay on the mark.*
- *If it moves off the mark, the TSI is actually set up over a point halfway between the mark and the rotated plummet position.*

Combined constant:

- *The points of signal orgin and signal reflection may not be on the vertical axes used to orient the equipment over the ground points.*
- *Most surveyors are familiar with a prism offset and how it is affected by the mounting system.*
- *Additionally, because glass is denser than atmosphere the light wave is slowed as it travels through the prism, increasing the measured distance.*
- *A manufacturer reports is combined effect as the prism, increasing the measured distance.*

- *The TSI is also subject to an offset.*
- *The signal is generated internally then optically made to coincide with the line of sight.*
- *The slope distance is the sum of measured distance and the instrument and prism constants.*

$$S = S' + (C_I - C_p)$$

Where

S' ----- *measured slope distance*

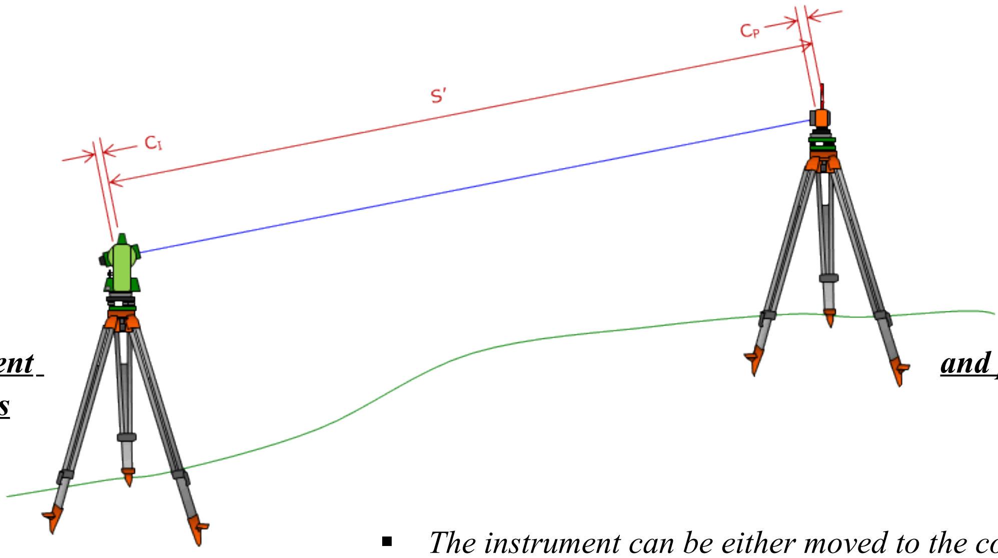
C_I ----- *Instrument offset*

C_p ----- *Prism offset*

S ------ *Correct slope Distance.*

- $(C_I + C_p)$ *is referred to as the combined constant.*

Instrument constants



and prism

- *The instrument can be either moved to the correct location or the optical plummet can be adjusted using the adjusting screws.*
- *On a tribrach the optical plummet is to use a plumb bob.*
- *Attach the tribrach to a tripod that has a plumb bob hanger.*
- *Center the tribrach circular bubble, attached a plumb bob, and set a mark directly below it.*
- *Because the plumb bob is used, the tribrach is correctly set up over the mark regardless the optical plummet's adjustment.*
- *Remove the plumb bob and sight through the optical plummet. If the plummet is in adjustment it will be centred on the mark.*

- *If not centred it shows how much the plummet is out of adjustment.*
- *To continue using the tribrach either use a plumb bob or adjust the plummet using its adjusting screws. This method can also be used to check a TSI with a rotating optical plummet.*

Manufacturer's stated accuracy (MSA):

- *Each TSI has an inherent random error in distance measurement*
- *This is MSA and is specified in the instrument manual.*
- *It is expressed as a two part uncertainity*
    - *Constant*
    - *A proportion based on distance.*
- *An example is an MSA of ± (2 mm + 3mm)*
- *Every distance measured with this TSI would have an expected error of*

$$Constant = \pm 2\text{ mm} \times \frac{39.37\ in}{1\ m} \times \frac{1\ m}{1000\ mm} \times \frac{1ft}{12\ in} = \pm 0.006\text{ ft}$$

- *The proportion error will increase for longer distances. In a 100 feet distance, the expected proportion error is ,*

$$Proportion = 100\text{ ft} \times \frac{\pm 3}{1000000} = \pm 0.0003\text{ ft}$$

Prism pole height:

- *Prism pole height does not affect horizontal distance determination.*
- *The TSI uses a zenith angle with a slope distance to compute the slope distance.*
- *Prism height doesn't matter since raising or lowering it will change both zenith angle and slope distance but still result in the same horizontal distance.*
- *Prism height can come into play when trignometrical levelling or topo mapping since both these require vertical distance.*

Natural errors:

- *Meteorological condition (temperature, pressure, humidity, etc.) have to be taken into account to correct for the systematic error arising due to this.*
- *These errors can be removed by applying an appropriate atmospheric correction model that takes care of different meteorological parameters from the standard (nominal) one.*

Atmospheric conditions:

- *Electro – optical EM signals are affected by atmospheric pressure and temperature.*
- *Total station instruments are generally standardised at a specific temperature and pressure.*
- *When measurement conditions deviate from either than a proportional correct must be applied.*
- *The equations for the proportional corrections are,*

English Units	Metric Units
Correction= 278.96- $\dfrac{10.5\, P_E}{1 + 0.002175\, T_E}$ Where, P_E = Pressure in inches of mercury T_E = temperature if ° F	Correction= 278.96- $\dfrac{0.3872 P_M}{1+0.003661\, T_M}$ Where, P_M = Pressure in mm of mercury T_M = temperature if °C

- *To apply the correction*

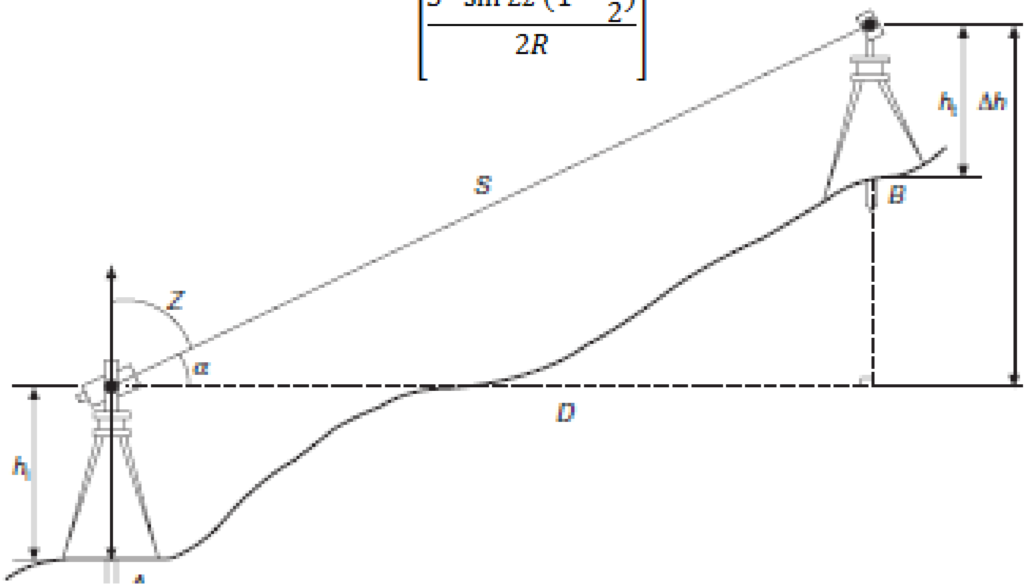
$$D = D'(1 + \text{correction in ppm})$$

Refraction and curvature:

- *The EM signal path is bent, refracted, as it moves through the atmosphere.*
- *The degree of refraction depends on atmospheric density and the signal's direction through it.*
- *The affects the zenith angle because it is measured from the vertical to a line tangent to the signal path at the TSI (dotted line)*
- *Also, over long distances earth's curvature must be taken into account.*
- *Vertical lines at the TSI and reflector are not parallel so that " horizontal " distance is actually a chord distance whose end points are the same elevation.*
- *This chord is not perpendicular to the vertical at each either the TSI or prism.*

- *The equations for reducing slope to horizontal and vertical components allowing for refraction and curvature are*

$$\left[\frac{S^2 \sin 2z\,(1-\frac{k}{2})}{2R}\right]$$



$$H = S \sin Z - \left[\frac{S^2 \sin 2z\,(1-\frac{k}{2})}{2R}\right]$$

$$V = S \cos Z + \left[\frac{S^2\, sin^2 Z(1-k)}{2R}\right]$$

Where,

H ---- Horizontal direction

V ---- Vertical direction

S ---- Slope distance

R ---- Radius of earth = 2.09 ×10⁷ feet

Z ---- Zenith angle

K ---- Refraction constant

For most elevations k =0.142

- *The refraction constant is a ratio of the earth radius.*
- *Most of the TSI provide the option to apply refraction and curvature corrections as measurements are made.*

Atmospheric anomalies:

- *The atmosphere immediately above as asphalt surface on a sunny day, the heat emitted by the surface causes a local atmospheric anomaly which affect the signal path.*

Problem:

1. ***A distance of 826.39 feet was measure without including atmospheric corrections. If the temperature and pressure at measurement were 70°F and 28.5" Hg. What is the corrected distance?***

Given Data:

Temperature T_E	=	*70° F*
Pressure energy in inches of mercury P_E	=	*28.5"*
Uncorrected distance D'	=	*826.39 feet*

To find:

Corrected distance =?

Solution:

The proportional correction in English units (inches and feet's)

$$\text{Correction} = 278.96 - \left[\frac{10.5\,P_E}{1+0.002175\,T_E}\right]$$

$$= 278.96 - \left[\frac{10.5 \times 28}{1+0.002175 \times 70}\right]$$

$$= +19.2 \text{ ppm}$$

Corrected distance D = *D' (1+ correction in ppm)*

= *826.39(19.2/1000000)*

Corrected distance D = *826.40 feet*

2. ***The surveyor measure the distances between a section and quarter section corners and records a slope distance of 2677.36 ft with a zenith angle of 81°10'25" corrected for atmospheric conditions.***

    Whatever is introduced in the horizontal distance is refraction and curvature are not taken into account?

Given Data:

Slope distance (S)	*= 2677.36 ft*
Zenith angle (Z)	*= 81°10'25"*
Assume, Radius of Earth(R)	*= 2.09×10^7 feet*

Assume, Refraction constant(k) = 0.142

To find:

What error is introduced in the horizontal distance if refraction and curvature =?

Solution:

Horizontal distance (H) = S Sin Z - $\left[\frac{S^2 \, Sin\, 2z\, (1-\frac{k}{2})}{2R}\right]$

$= 2677.36 \times Sin\ 81°10'\ 25'' - \left[\frac{S2677.36^2\, Sin\, 2\times 81°10'\ 25''\ (1-\frac{0.142}{2})}{2\times 2.09\times 10^4}\right]$

$= 2645.588\ ft$

Horizontal distance (H) not consider as a refraction and curvature correction

$H = S\ SinZ$

$= 2677.36\times sin81°10'\ 25''$

$= 2645.654\ ft$

The difference is (2645.654- 2645.588) = 0.096 feet

3. ***A horizontal distance of 985.37 ft is measured with a TSI having an (MSA) manufacturer's stated accuracy of ±(2 mm + 3mm). TSI centring error is estimated and hand held. Due to some wind the prism centring is assumed to be ±0.04 ft atmospheric conditions were accounted at the time of measurement. What is the expected error in the distance?***

Solution:

In this problem, these (MSA, centering, prism pole errors)are additive random errors. Since they affect parts of the line length.

Sum of the total error $E_{sum} = \sqrt{E_1^2 + E_2^2 + \cdots E_n^2}$

Contributing Error:

MSA constant $E_{const} = 2\ mm \times \frac{39.37\ in}{1\ m} \times \frac{1m}{1000\ mm} \times \frac{1\ ft}{12\ in}$

$= 0.00656\ ft$

MSA proportional, $E_{prop} = 985.378\ ft \times \frac{3}{1000000}$

$= 0.00296\ ft$

TSI centering , $E_{TSI} = 0.005\ ft$

Prism centering $E_{prism} = 0.04\ ft$

Sum of total error $E_{sum} = \sqrt{E_{cons}^2 + E_{prop}^2 + E_{TSI}^2 + E_{prism}^2}$

$E_{sum} = \sqrt{0.00656^2 + 0.00296^2 + 0.005^2 + 0.04^2}$

$E_{sum} = \pm 0.0504\ ft$

$= \pm 0.05\ ft$

Care and maintenance of the Total Station instruments:

Maintenance:

- *Do not leave the instrument in the direct sunlight or in a closed vehivle for prolonged periods.*
- *Overheating the instrument may reduce its efficiency.*
- *Some TSI has been used in wet conditions, immediately wipe off any moisture and dry the instrument completely before returning the instrument to the carrying case.*
- *The instrument contains sensitive electronic assemblies which have been well protected against dust and moisture. However, if dust or moisture gets into the instrument, severe damage could result.*
- *Sudden changes in temperature may could be lenses and drastically reduce the measurable distance, or cause an electrical system failure. If there has been a sudden change in temperature, leave the instrument in a closed carrying case in a warm location until the temperature of the instrument returns to room temperature.*
- *Do not store in hot or humid locations. In particular, you must store the battery pack in a dry location at a temperature of less than 30°c (86°F)*
- *High temperature or excessive humidity can cause mold to grow on the lenses. It can also cause the electronic assemblies to determine, and so lead to instrument failure.*
- *Store the battery pack with the battery discharged.*

- *When storing the instrument in areas subject to extremely low temperature leave the carrying case open.*
- *When adjusting the levelling screws, stay as close as possible to the centre of each screw's range. This centre is indicated by a line on the screw.*
- *If the tribrach will not be used for an extended period, lock down the tribrach clamp knob and tighter its safety screw.*
- *Do not use organic solvents (such as ether or paint thinner) to clean the non-metallic parts of the instrument (such as keyboard) or the painted or printed surfaces.*
- *Doing so could result in discoloration of the surface, or in peeling of printed characters.*
- *Clean these parts only with a soft cloth or a tissue, lightly moistened with water or a mild detergent.*
- *To clean the optical lenses, lightly wipe them with a soft cloth or a lens tissue that is moistened with alcohol.*
- *The reticle plate cover (near eye piece) has been correctly mounted. Do not release it or subject it to excessive force to make it water tight.*
- *Before attaching the battery pack, check that the contact surfaces on the battery and instrument are clean.*
- *Securely press the cap that covers the data output/ external power input connector terminal. The instrument is not watertight if the cap is not attached securely, or when the data output /external power input connector is used.*
- *The carrying case is designed to be water tight, but you should not leave it exposed to rain for an extended period. If exposure to rain is unavoidable, make sure that the carrying case is placed with the Nikon nameplate facing upward.*
- *The battery pack contains a lithium- ion- battery. When disposing of the battery pack, follow the laws or rules of your municipal waste system.*
- *The instrument can be damaged by static electricity from human body discharged through the data output/ external power input connector. Before handling the instrument, touch any other conductive material once to remove static electricity.*
- *Be careful not to pinch your finger between the telescope and trunnion of the instrument.*
- *Lightly tap the touch screen with the stylus otherwise you may damage the touch screen.*

Precautions:

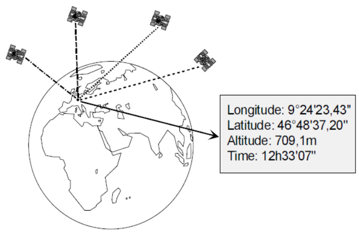
- *Do not carry tripod mounted instruments over the shoulder.*
- *Remove instruments from tripod when changing set up locations.*
- *Calibrate instruments daily per manufacturer's recommended procedures.*
- *Ensure the instrument prism offset value is set correctly for the prism in use.*
- *Ensure that the appropriate version of the instruments firmware is installed.*
- *Never point the telescope directly at the sun. the sun's rays may damage the electronic distance measuring(EDM) circuitry.*
- *If possible shade the instrument from direct sunlight as excess heat may reduce the range of the sender diodes in the EDM circuitry.*
- *To maintain maximum signal return at longer ranges shade prisms from direct sunlight.*
- *Avoid multiple unrelated prisms in the same field of view; this can cause blunders in distance observations.*
- *Most total stations are equipped to detect and correct various instrumental errors. If such errors exceed program limits, error codes will indicate the error. Consult the operator's manual for exact procedures and error code definitions.*

UNIT IV GPS SURVEYING 9

Basic Concepts - 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.

Introduction

- The first known surveyors are Egyptians who used distant control points to replace property corners destroyed by floods.
- The Greeks and Romans surveyed the settlements.
- French surveyors were probably the first to conduct surveys on large scales, by measuring the interior angles of a series of interconnecting triangles in combination with measured base lines.



- To determine the coordinates of selected points.
- Triangulation technique was used by surveyors to determine accurate distances.
- The use of triangulation was limited by the line of sight.
- The series of triangles were generally fixed by astronomical points by observing selected stars to determine the position of that point on the surface of the earth.
- These astronomical positions could be in error by hundreds of meters, the interrelationship between the coordinates cannot be precisely fixed. (then the optical global triangulation was developed)
- The worldwide satellite triangulation program often called BC-4 program was carried out to establish interrelationships of the major world datums.
- This method involves photographing special reflective satellites against a star background with a metric camera fitted with a chopping shutter.
- The main problem in using this optical technique was that clear sky was required simultaneously at a minimum of two observing sites separated by some 4000 km, and the equipment was massive and expensive.

Electromagnetic Technique:

- The electromagnetic ranging technique because of all-weather capability and greater accuracy.
- First attempts to (positional) connect the continents by electromagnetic techniques was by the use of an electronic High Ranging (HIRAN) system developed during World War II to position aircrafts.

- Today's modern positioning systems are Inertial Surveying System (ISS) and the Navy Navigational Satellite System (NNSS), also called TRANSIT system developed by USA.
- The TRANSIT system was composed of six satellites orbiting at altitudes of about 1100 km with nearly circular polar orbits.
- TRANSIT system was developed primarily to determine the coordinates of vessels and aircrafts.
- The positioning analysis technique used in the TRANSIT system utilized a ground receiver capable of noting the change in satellite frequency transmission as the satellite first approached and receded from the observer.
- The change in frequency was affected by the velocity of the satellite itself. The change in velocity of transmissions from the approaching and then receding satellite, known as the *Doppler effect*, is directly proportional to the shift in frequency of the transmitted signals, and is thus proportional to the change of distance between the satellite and the receiver over a given time interval.
- With the precise knowledge of the satellite orbit and that of the satellite position in that orbit, the position of the receiving station could be computed.
- Some of the TRANSIT experiments showed that accuracies of about 1 metre could be obtained by occupying a point for several days.
- The main problem with TRANSIT was the large time gaps in coverage. Since nominally a satellite would pass overhead every 90 minutes, users had to interpolate their position between *fixes* or *passes*.
- Unfortunately, the satellites that it uses are in very low orbit and there are not very many of them. So a user does not get a fix very often. Also, since the system is based on low frequency Doppler measurements, even small movements at the receiving end can cause significant errors in position.
- It was these shortcomings that led to the development of the US Global Positioning System (GPS), the Europian Satellite Based Navigation System (Galileo) and the Russian Global Navigation Satellite System (GLONASS).

GLONASS System:

- GLONASS is a radio-based satellite navigation system, developed by Russian Aerospace Defence Forces for the Russian Government.
- It was made operational in 1996. The first GLONASS satellite was launched and placed in the orbit on 12th October, 1982.

- Thereafter, numerous rocket launchers added satellites to the system. By 2010, GLONASS had achieved 100% coverage of Russian territory.
- The full orbital constellation of 24 satellites was restored in October 2011, enabling full
- global coverage.
- GLONASS satellite designers have undergone several upgrades, having three generations, from GLONASS to GLONASS-M to GLONASS-K.
- In November 2011, four more GLONASS-M was placed into final orbit.
- Originally GLONASS was designed to have an accuracy of 65 m, but in reality, it had an accuracy of 20 m in the civilian signal and 10 m in the military signal.
- GLONASS uses a coordinate datum named FZ-90.
- Its satellites transmit two types of signals:
    - *Standard precision (SP) Signal and*
    - *high precision (HP) signal.*
- GLONASS system is a counterpart and at par with the United States GPS system. Both the systems share the same principles in the transmission and positioning methods.
- The GLONASS system has both the precise positioning service and standard positioning service as in the GPS, but its datum and time reference system are different.
- GLONASS like GPS consists of three segments:
    - *The space,*
      The control, and
    - *The user segment.*
- The *operational space segment* of GLONASS consists of 21satellites in three orbital planes, with 3 on-orbit spares; making the total number of 24 satellites.
- The three orbital planes are separated by 120° and the satellites within the same orbit plane by 45°. Each orbital plane, therefore, has eight equally spaced satellites, operating at an altitude of 19,100 km at an inclination angle of 64.8° to the equator.
- Each satellite will complete an orbit in approximately 11 hr 15 min.
- The spacing of satellites is such that a minimum of five satellites are always in view round the globe.
- The geometric arrangement gives a considerable better coverage than GPS in Polar Regions above and below 50° latitude.
- The satellites work in GLONASS System Time, checked and updated twice daily, with a maximum time error of 15 ns.
- The ground control segment is entirely located within former Soviet Union Territory.

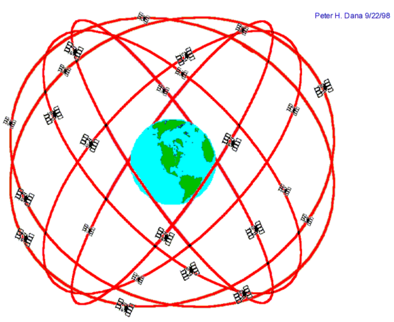
- The ground control station is located in Moscow.
- The user segment consists of antennas and receiver-processors that provide positioning, velocity and precise timing of the user.

What is GPS?

- GPS, which stands for Global Positioning System, is the only system today able to show you your exact position on the Earth anytime, in any weather, anywhere.
- The three parts of GPS are:
    - Satellites
    - Receivers
    - Software
- Location system based on a constellation of 24 satellites orbiting the earth at altitudes of approximately 11,000 miles.

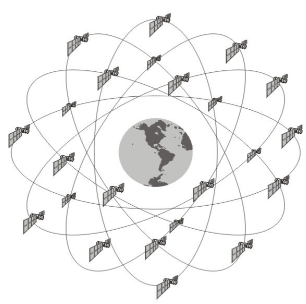
Satellites

- There are quite a number of satellites out there in space.
- They are used for a wide range of purposes: satellite TV, cellular phones, military purposes and etc.
- Satellites can also be used by GPS receivers.
- The GPS Operational Constellation consists of 24 satellites that orbit the Earth in very precise orbits twice a day.
- GPS satellites emit continuous navigation signals.



GPS Nominal Constellation
24 Satellites in 6 Orbital Planes
4 Satellites in each Plane
20,200 km Altitudes, 55 Degree Inclination

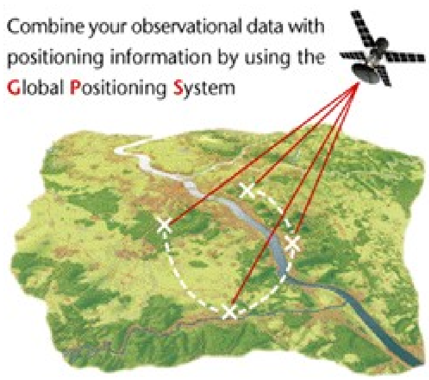
GPS CONSTELLATION



21 SATELLITES WITH 3 OPERATIONAL SPARES,
6 ORBITAL PLANES, 55 DEGREE INCLINATIONS
20,200 KILOMETER, 12 HOUR ORBITS

Receivers and Satellites:

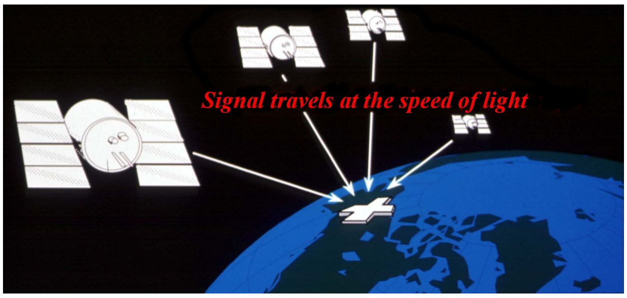
- GPS units are made to communicate with GPS satellites (which have a much better view of the Earth) to find out exactly where they are on the global scale of things.



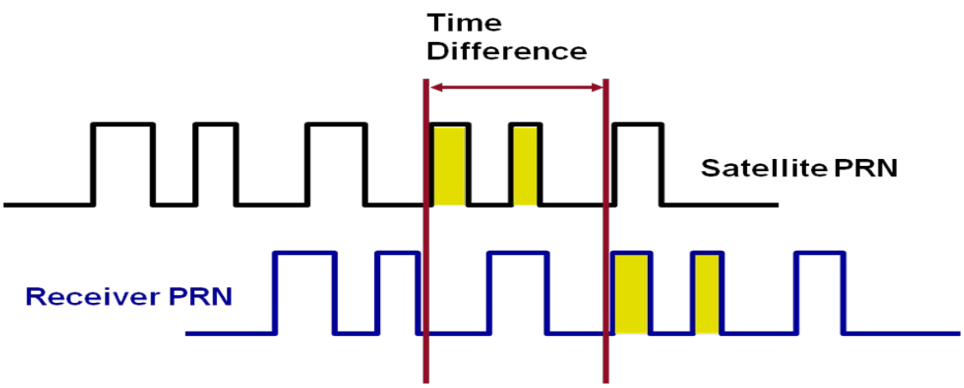
- The complex pattern ensures that the receiver does not accidentally synchronize up to some other signal or so the receiver won't accidentally pick up another satellite's signal.

<u>How a GPS Receiver determines its Position?</u>

- Each satellite transmits what's called a Navigation Message, which is downloaded by GPS receivers.
- GPS constellation status (all the satellites) satellite ephemeris and health data (individual satellites).
- The GPS currently uses two frequencies to accomplish data transmission, L1 and L2.
- The NAV Message and coarse acquisition information are provided on the L1 frequency.

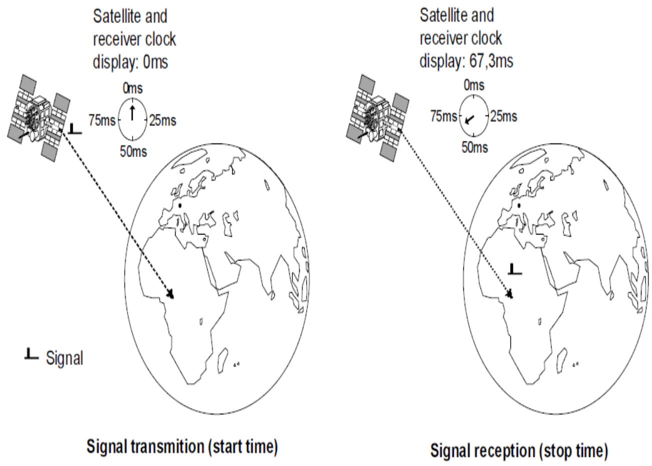


- Another frequency (L3) is planned for the next generation of satellites to enhance position and navigation precision of GPS receivers.
- The pseudo random noise (PRN) code is a fundamental part of the GPS.
- It's a very complicated digital code unique to each satellite.
- It's a complex sequence of "on" and "off" digital pulses.



- The signal looks like random electrical noise (similar to the "snow" you might see on a TV), but is actually a very precise code. Hence the name ***"pseudo-random noise."***
- When a GPS receiver acquires a GPS signal it examines the satellite's incoming PRN and begins generating a matching digital signal to mimic the satellite's signal.

- The receiver matches each satellite's PRN code with an identical copy of the code contained in the receiver's database.
- Its next task is to try and determine how long ago the signal was generated by the satellite.
- But there's a problem, then each satellite is equipped with atomic clocks.
- Clocks which are constantly monitored for accuracy by the Master Control Station.
- The GPS receiver on the other hand is equipped only with a single digital clock comparable to a cheap wrist watch.
- The only way for the receiver to calculate an accurate position is if it can accurately measure the precise travel time of the satellite radio signal.
- A discrepancy of just a few nanoseconds between satellite and receiver will translate into a large position error on the ground.



- So the GPS receiver uses a clever technique to calculate the precise time it took for the GPS signal to reach it.
- By shifting its own generated copy of the satellite's PRN code in a matching process, and by comparing this shift with its own internal clock, the receiver can calculate how long it took the signal to travel from the satellite to itself.
- By comparing the time difference between the two, and multiplying that time by the 186,000 miles per second travel speed of the signal, the receiver can roughly determine the distance separating it from the satellite.
- This process is repeated with every satellite signal the receiver locks on to.
- The distance between satellite and receiver derived from this method of computing distance is called a ***pseudo-range ("false range")*** because the receiver's clock is not synchronized with the satellites clocks.
- Pseudo-range is subject to several error sources, such as delays caused by the atmosphere, and multipath interference.
- For example, the GPS satellite PRN signal is a song being broadcast by a radio station.
- The GPS receiver is a record player which is playing the same song, but it's not synchronized to the broadcast song.

- Both songs are playing, but at different places in the song and at different speeds.
- By speeding up or slowing down the turntable, the two songs can be precisely matched. They become synchronized.
- Similarly, the GPS receiver synchronizes its digital signal to match that of each satellite's signal.

Time Difference:

- The GPS receiver compares the time a signal was transmitted by a satellite with the time it was received.
- The time difference tells the GPS receiver how far away the satellite is.

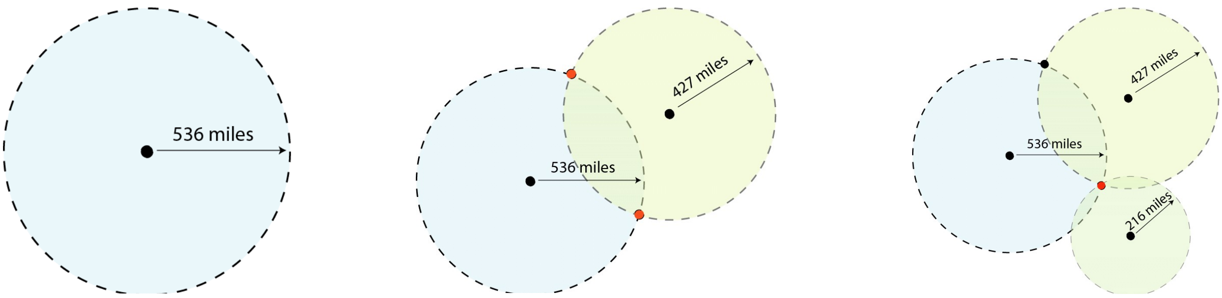
Velocity x Time = Distance

- Radio waves travel at the speed of light, roughly 186,000 miles per second (mps)

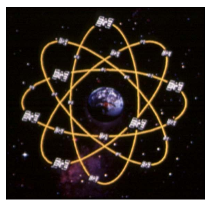
Triangulation:

Geometric Principle:

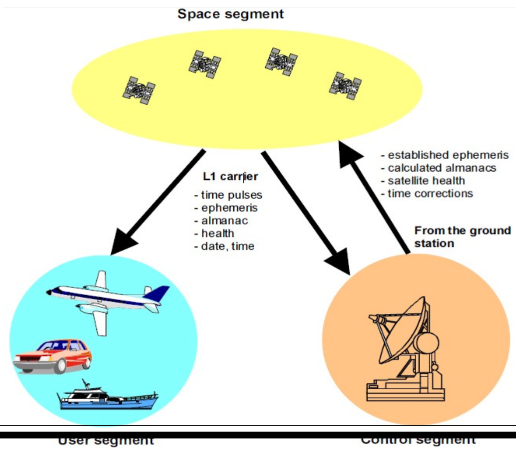
- You can find one location if you know its distance from other, already-known locations.



Segments or Components of GPS:



Space Segment:

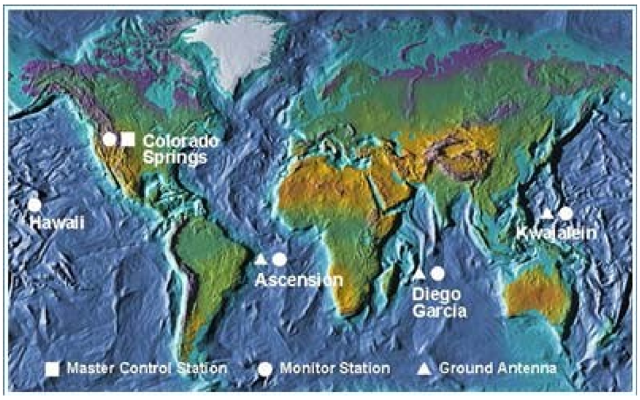


- Consists of 24 satellites in 6 orbits.
- Each satellite transmits low powered radio signals.
- The orbital position is constantly monitored and updated by ground stations.
- Each satellite is identified by number and broadcasts a unique signal.

- ✷ The signal travels at the speed of light.
- ✷ Each satellite has a very accurate clock, 3 x 10^{-9} Seconds.

Distance = Velocity x Time

- ✷ ***GPS Satellite***
  - Name : NAVSTAR
  - Altitude : 11,000 miles
  - Inclination : 55 Deg to the Equator
  - Weight : 863 Kg (in orbit)
  - Orbital Period :12 hrs



The Control Segment

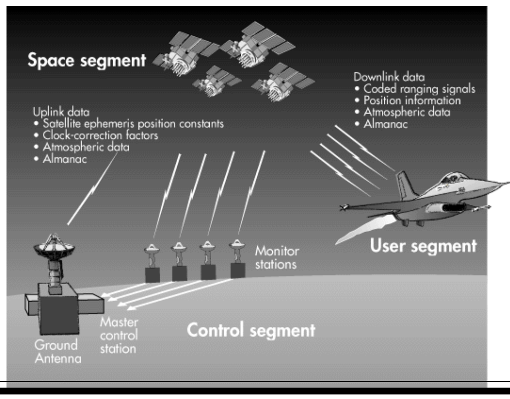
- ➢ ***A Master Control Station***
- ➢ ***Unmanned Monitor Stations***
- ➢ ***Large Ground-antenna Stations***

Global Positioning System (GPS) Master Control and Monitor Station Network

- The control segment or ground segment has one Master Control Station, one alternative Master Control station (Monitor station).
- 12 command and large ground or control antennas and 16 monitoring sites.

Most important tasks of the control segment

- Observing the movement of the satellites and computing orbital data
- Monitoring the satellite clocks and predicting their behavior
- Synchronizing on board satellite time



- Relaying precise orbital data received from satellites in communication
- Relaying further information, including satellite health, clock errors etc.

The User Segment

- ***Users-Military and Civilians***
- ***GPS Receivers***
    - ***Decodes the signals from Satellites.***
    - ***Calculate the distance.***
- GPS receivers are generally composed of an antenna, tuned to the frequencies transmitted by the satellites, receiver-processors, and a highly-stable clock, commonly a crystal oscillator).
- They can also include a display for showing location and speed information to the user.
- A receiver is often described by its number of channels this signifies how many satellites it can monitor simultaneously.
- As of recent, receivers usually have between twelve and twenty four channels.
- Using RTCM SC-104 format, GPS receivers may include an input for differential corrections.
- This is typically in the form of a RS-232 port at 4800 bps speed.
- Data is actually sent at much lower rate, which limits the accuracy of the signal sent using RTCM.
- Receivers with internal DGPS (differential GPS) receivers are able to outclass those using external RTCM data.

Modes of Operation

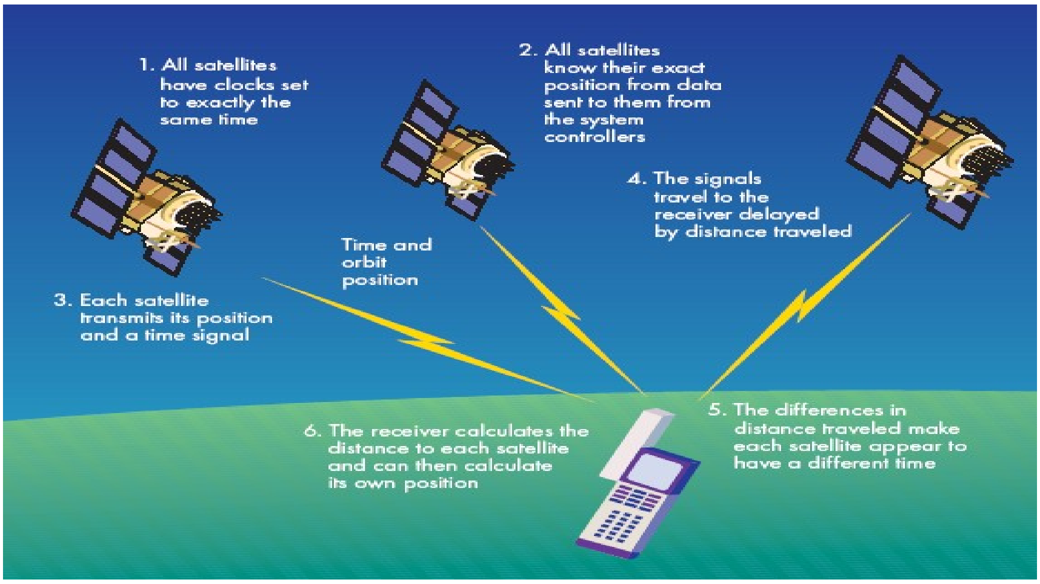
▪ Standard Positioning System	HA	=	100 m
▪ Data Transmitted on L1 Frequency	VA	=	156 m
▪ For civil users	TA	=	340 ns
▪ Accuracy is degraded	HA	=	22 m
▪ Precise Positioning System	VA	=	27.7 m

- Data Transmitted on L1 and L2 Frequencies
- For Military users
- Highly Accurate

Differential GPS

- The majority of data collected using GPS for GIS is differentially corrected to improve accuracy.
- The underlying premise of differential GPS (DGPS) is that any two receivers that are relatively close together will experience similar atmospheric errors.
- This GPS receiver is the base or reference station. The base station receiver calculates its position based on satellite signals and compares this location to the known location.
- The difference is applied to the GPS data recorded by the second GPS receiver, which is known as the roving receiver.
- The corrected information can be applied to data from the roving receiver in real time in the field using radio signals or through post processing.

Working of GPS:



How User Segment calculates the position?

Calculation of Position

- *Satellites Location*
    - *Almanac data*
    - *Ephemeris*
- *Time is the essence*
    - *Velocity * Time=distance*
- The GPS Almanac is a set of data to describe the orbits of the complete active fleet of Satellites.
- GPS receivers use the almanac to determine "approximately" where the satellites are relative to the local sky.

- It then uses this information to determine what satellites it should track (no point in devoting resources to satellites below the horizon)

Sources of Errors:

- Ionosphere Delays
- Troposphere Delays
- Clock Error
- Multi-path Error
- Relativity Error

Satellite clock errors:

- Caused by slight discrepancies in each satellite's four atomic clocks.
- Errors are monitored and corrected by the Master Control Station.

Orbit errors:

- Satellite orbit (referred to as "satellite ephemeris") pertains to the altitude, position and speed of the satellite.
- Satellite orbits vary due to gravitational pull and solar pressure fluctuations.
- Orbit errors are also monitored and corrected by the Master Control Station.

Ionospheric interference:

- The ionosphere is the layer of the atmosphere from 50 to 500 km altitude that consists primarily of ionized air.
- Ionospheric interference causes the GPS satellite radio signals to be refracted as they pass through the earth's atmosphere – causing the signals to slow down or speed up.
- This results in inaccurate position measurements by GPS receivers on the ground.
- Even though the satellite signals contain correction information for ionospheric interference, it can only remove about half of the possible 70 nanoseconds of delay, leaving potentially up to a ten meter horizontal error on the ground.
- GPS receivers also attempt to "average" the amount of signal speed reduction caused by the atmosphere when they calculate a position fix. But this works only to a point.
- Fortunately, error caused by atmospheric conditions is usually less than 10 meters. This source of error has been further reduced with the aid of the Wide Area Augmentation System (WAAS), a space and ground based augmentation to the GPS (to be covered later).

Tropospheric interference:

- The troposphere is the lower layer of the earth's atmosphere (below 13 km) that experiences the changes in temperature, pressure, and humidity associated with weather changes. GPS errors are largely due to water vapor in this layer of the atmosphere.
- Tropospheric interference is fairly insignificant to GPS.

Receiver noise:

- It is simply the electromagnetic field that the receiver's internal electronics generate when it's turned on.
- Electromagnetic fields tend to distort radio waves.
- This affects the travel time of the GPS signals before they can be processed by the receiver.
- Remote antennas can help to alleviate this noise.
- This error cannot be corrected by the GPS receiver.

Multipath interference:

- It is caused by reflected radio signals from surfaces near the GPS receiver that can either interfere with or be mistaken for the true signal that follows an uninterrupted path from a satellite.
- An example of multipath is the ghosting image that appears on a TV equipped with rabbit ear antennas.
- Multipath is difficult to detect and sometimes impossible for the user to avoid, or for the receiver to correct.
- Common sources of multipath include car bodies, buildings, power lines and water.
- When using GPS in a vehicle, placing an external antenna on the roof of the vehicle will eliminate most signal interference caused by the vehicle.
- Using a GPS receiver placed on the dashboard will always have some multipath interference.

Correction of Errors:

- Error Modeling
    - Mathematical Model

  Data Frequency Measurement
    - Compare the Delays of L1 and L2

Applications of GPS:

✓ *Industry*

- *Agriculture*
- *Mapping & GIS Data Collection*
- *Public safety*
- *Surveying*
- *Telecommunication*

✓ *Military*

- *Intelligence & Target Location*
- *Navigation*
- *Weapon Aiming &Guidance*

✓ *Transportation*

- *Aviation*
- *Fleet Tracking*
- *Marine*

✓ *Science*

- *Archaeology*
- *Atmospheric Science*
- *Environmental*
- *Geology & Geophysics*
- *Oceanography*
- *Wildlife*

Selective availability (SA):

- GPS was originally designed that real-time autonomous positioning and navigation with the civilian C/A code receivers would be less precise than military P-code receivers.
- Surprisingly, the obtained accuracy was almost the same from both receivers.
- Static positioning with P – code is accurate to 5 – 10m and is therefore denied access to civilian users by encryption of the code. This is referred as anti-spoofing (AS).
- It was anticipated that use of the S-code or, as it was originally called C/A (coarse acquisition) code, would result very much worse positional accuracies.
- This was not the case, and accuracies in the region of 30m were obtained.
- This gave the American Government cause for concern as to its use by an enemy in the time of war, and a decision was made to degrade Pseudo-range measurement. This process was called selective availability (SA).

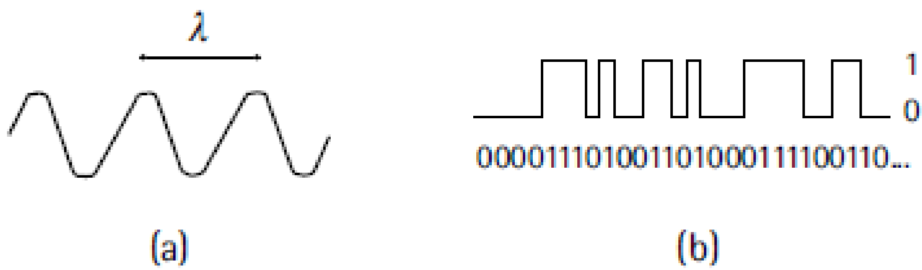
Components of SA:

1. ***Epsilon :***

- *It was a corruption of the broadcasts ephemeris on S-code, resulting in incorrect positioning of the satellite.*

2. ***Dither:***
   - *It was a corruption of the rate at which the satellite clocks function, resulting in further degrading of observed pseudo-ranges to accuracy no greater than 30m.*

GPS signal structure:

- GPS satellite transmits a microwave radio signal composed of two carrier frequencies (or sine waves) modulated by two digital codes and a navigation message.
- 

(a) A sinusoidal wave; and (b) a digital code.

The two carrier frequencies are generated at 1,575.42 MHz (referred to as the L1 carrier) and 1,227.60 MHz (referred to as the L2 carrier).
- The carrier wavelengths are approximately 19 cm and 24.4 cm, respectively.
- It results from the relation between the carrier frequency and the speed of light in space.
- The availability of the two carrier frequencies allows for correcting a major GPS error, known as the ionospheric delay.
- All of the GPS satellites transmit the same L1 and L2 carrier frequencies.
- The code modulation, however, is different for each satellite, which significantly minimizes the signal interference.
- The two GPS codes are called coarse acquisition (or C/A-code) and precision (or P-code).
- Each code consists of a stream of binary digits, zeros and ones, known as bits or chips.
- The codes are commonly known as PRN codes because they look like random signals (i.e., they are noise-like signals).
- But in reality, the codes are generated using a mathematical algorithm.
- Presently, the C/A-code is modulated onto the L1 carrier only, while the P-code is modulated onto both the L1 and the L2 carriers. This modulation is called ***biphase***

modulation, because the carrier phase is shifted by 180° when the code value changes from zero to one or from one to zero.

- The C/A-code is a stream of 1,023 binary digits (i.e., 1,023 zeros and ones) that repeats itself every millisecond.
- It means, the chipping rate of the C/A-code is 1.023 Mbps.
- The duration of one bit is approximately 1ms, or equivalently 300m.
- Each satellite is assigned a unique C/A-code, which enables the GPS receivers to identify which satellite is transmitting a particular code.
- The C/A-code range measurement is relatively less precise compared with that of the P-code.
- The P-code is a very long sequence of binary digits that repeats itself after 266 days.
- It is also 10 times faster than the C/A-code (i.e., its rate is 10.23 Mbps).
- Multiplying the time it takes the P-code to repeat itself, 266 days, by its rate, 10.23 Mbps, tells us that the P-code is a stream of about 2.35×10^{14} chips.
- The 266-day-long code is divided into 38 segments; each is 1 week long.
- 32 segments are assigned to the various GPS satellites.
- Each satellite transmits a unique 1-week segment of the P-code, which is initialized every Saturday/Sunday midnight crossing.
- The remaining six segments are reserved for other uses.
- The P-code is designed primarily for military purposes.
- It was available to all users until January 31, 1994.
- At that time, the P-code was encrypted by adding to it an unknown W-code.
- The resulting encrypted code is called the Y-code, which has the same chipping rate as the P-code. This encryption is known as the anti-spoofing (AS).

Differential Global Positioning Systems (DGPS):

- Increase accuracy dramatically.
- DGPS was used in the past, to overcome selective availability (SA) [100m to 4 – 5m].
- DGPS uses one stationary and one moving receiver to help overcome the various errors in the signal.
- By using two receivers that are nearby each other, within a few dozen Km, they are getting essentially the same errors (except receiver error).
- DGPS improve accuracy much more than disabling of SA.

Tasks of control segment :

- Observing the movement of the satellites and computing orbital data

- Monitoring the satellite clocks and predicting their behavior
- Synchronizing on board satellite time
- Relaying precise orbital data received from satellites in communication
- Relaying further information, including satellite health, clock errors etc.

Following points must be kept in mind while collecting the data and processing the same:

Data collection

- Arrive early
- Follow proper procedures for antenna setup (check level, antenna height and offset)
- Setup a complete station log including:
    - field log
    - satellite status, tracking problem
    - local condition, sketch of location
    - meteorological readings if required
    - watch the GDOP < or = 8
    - use STOP & GO indicator as a guide
    - be sure you have sufficient memory capacity

Data Processing:

- Establish a project name to store all data
- back-up raw data to diskettes/CDs
- ensure data quality and integrity before demobilizing
    - daily baseline processing
    - check all possible closures and repeated baselines
- verify single point compared to published coordinates
- build up network adjustment daily
- back-up processed data and result to disk
- Transformation to local system
    - use local control held "fixed" in adjustment transformation into a local data system
    - use given transformation parameters
    - apply geoid undulation to obtain optometric heights

Types of GPS receivers

Receivers can be classified in many ways;

Two basic types of GPS receivers are:

1. code phase receivers

- C/A code receivers
- P-code receivers

2. carrier phase receivers
    - Codeless receivers
    - Single frequency receivers
    - dual-frequency receivers
    - Receivers using cross-correlation or squaring or P-W techniques

Code dependent or code phase receivers

- These are also called code correlating receivers since they need access to the satellite navigation message of the P- or C/A-code signal for operation.
- A complete code dependent correlation channel produces following observables and information:
    - code phase
    - carrier phase
    - change of carrier phase (Doppler frequency)
    - satellite message

Carrier phase receivers

- Utilize the actual GPS signal itself to calculate a position.
- Two general types of such receivers are
    - single frequency
    - dual frequency

(a) Single frequency receiver

- Tracks L1 frequency signal only
- Cheaper than dual frequency receivers
- Used effectively to relative positioning mode for accurate baselines of less than 50 km or where ionosphere effects can generally be ignored.

(b) Dual frequency receiver

- Tracks both L1 and L2 frequency signal
- More expensive than a single frequency receiver
- Can more effectively resolve longer baselines of more than 50 km where ionosphere effects have a larger impact
- Eliminate almost all ionosphere effects by combining L1 and L2 observations.

Comparison of single and double frequency receivers

Single Frequency	Double frequency
Access to L1 only	Access to L1 and L2
Mostly civilian users	Mostly military users
Much cheaper	Very expensive
Used for short base lines	Used for both long and short base lines
Most receivers are coded	Most receivers with dual frequency are codeless
Corrupted by ionospheric delay	Almost independent of ionospheric delay
Modulated with C/A and P codes	It may not be possible for civilian users once Y code is there.

Receiver based on user community/application

- Receivers can be classified depending upon who is the user, e.g. Military, Civilian, Navigation, Timing, Geodetic/surveying, Handheld receiver

Geodetic receivers

These receivers are essentially used for geodetic/surveying applications with the following characteristics;

- carrier phase data as observables
- Availability of both frequencies (L1, L2)
- Access to the P code, at least for larger distances, and in geographical region with strong ionospheric disturbances (low and high latitudes).

Following factors should be kept in mind for such receivers

- Tracking all signals from each visible satellite at any time (GPS only system requires 12 dual frequency channels; GPS+GLONASS system needs 20 dual frequency channels)
- Both frequencies should be available
- Low phase and code noise
- High data rate (> 10 Hz) for kinematic applications
- High memory capacity
- Low power consumption and weight and small size
- Full operational capability under AS
- Capability to track weak signals (under foliage, and difficult environmental conditions)
- multipath mitigation, interference suppression, stable antenna phase centre (explained later)
- Good onboard and office software

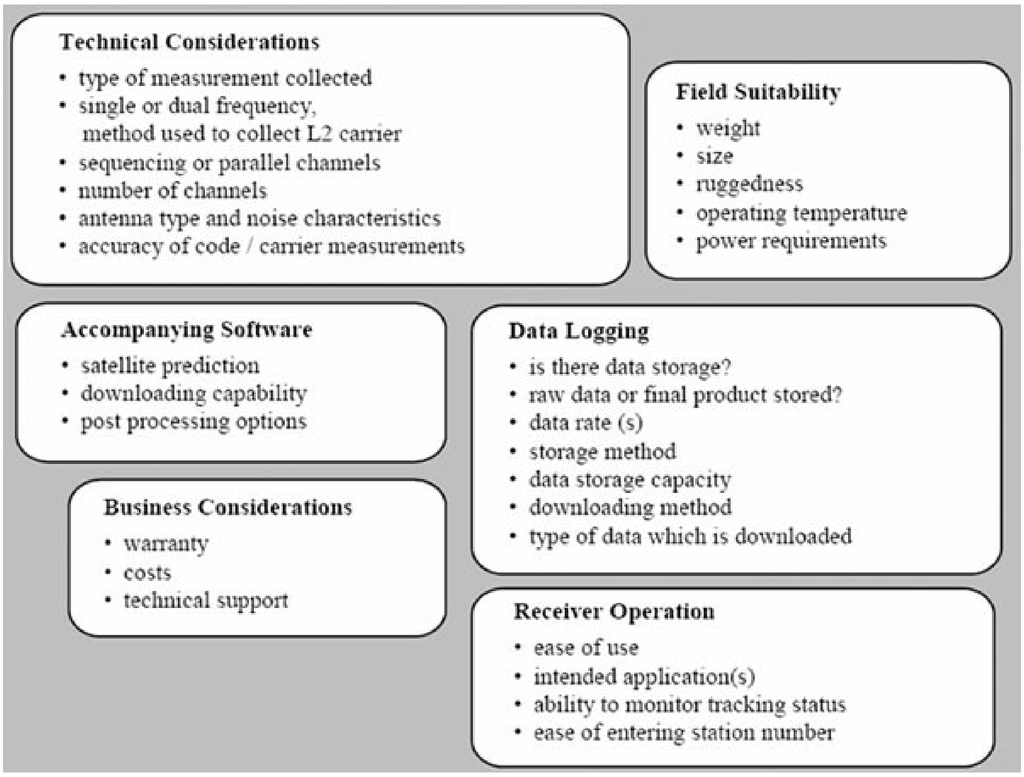
Other useful features for geodetic receivers

- A modern GPS survey system should measure accurately and reliably anywhere under any condition
- It should be useable for almost any application (geodetic, geodynamic, detailed GIS and topographic engineering survey, etc.) and may have the following features
    - 1 pps timing output
    - event marker (for marking special events or area of interest to the GPS use)
    - ability to accept external frequencies
    - fast data transfer to computer
    - few or no cable connection
    - radio modem
    - DGPS and RTK capability (explained later)
    - operate over difficult meteorological conditions
    - ease in interfacing to other systems and from other manufacturer
    - ease and flexibility of use (multi-purpose applications)
    - flexible set up (tripod, pole, pillar, vehicle)

Essential receiver requirements for geodetic/surveying applications:

- Leica
    - GS20
    - SR530
    - GPS1200
- Trimble
    - 4600
    - 5700
    - 5800
    - R8
- Topcon
    - Hiper
- Sokia
    - Stratus
    - GSR2650
    - Radian IS
- For a comprehensive survey of Geodetic quality receivers, refer to Key (2004)

SR530 - geodetic, real-time receiver
12L1 + 12 L2, C/A-code, P-code, RTK



Structure of GPS receiver

Functionality

- Functionally two groups of GPS receiver structures
- Application processing
- Signal processing

Application processing

- Time and frequency transfer
- Static and kinematic surveying
- Navigation
- Ionospheric Total Electron Content (TEC) monitoring
- Operation as differential GPS (DGPS) reference station
- GPS signal integrity monitoring

Signal processing

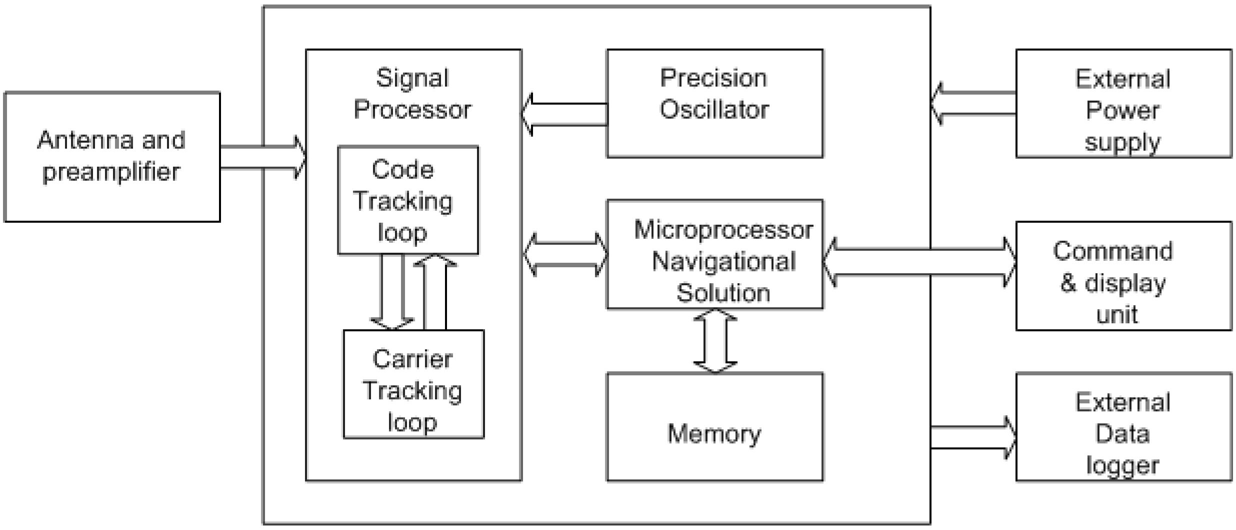
- Splitting of incoming signal into multiple satellite signals
- Generation of reference carrier

- Generation of reference PRN code
- Acquisition of satellite signal
- Tracking of code and carrier
- Demodulation and system data extraction
- Extraction of code phase measurements
- Extraction of carrier frequency and carrier phase
- Extraction of satellite Signal to Noise Ratio (SNR) information
- Relationship of GPS system time

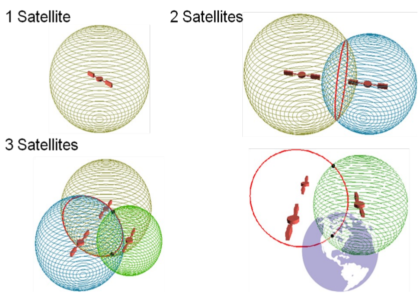
Components of GPS receiver

Main components

- Antenna with preamplifier
- Radio frequency (RF) and intermediate frequency (IF) Front end section
- Signal tracker and Code correlator section
- Reference oscillator
- Microprocessor (navigational solution unit)
- Other parts: memory, power supply, display and control



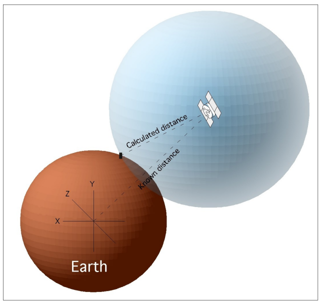
Major Components of a GPS Receiver



Triangulation:

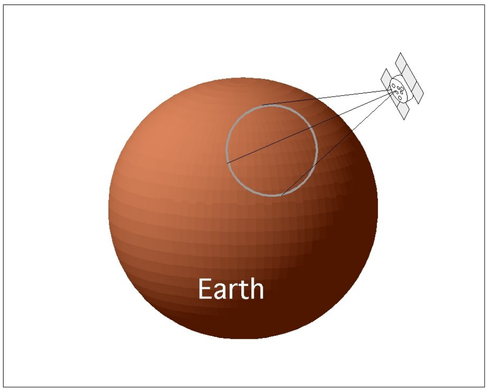
- Each satellite knows its position and its distance from the center of the earth.
- Each satellite constantly broadcasts this information.

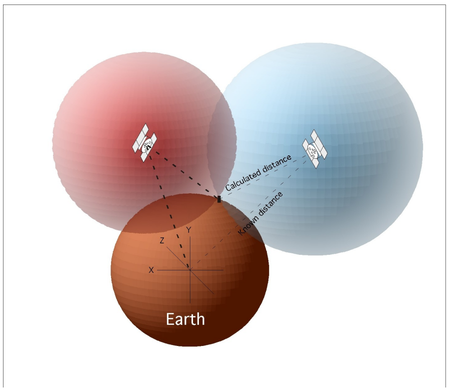
- With this information and the calculated distance, the receiver calculates its position.
- Just knowing the distance to one satellite doesn't provide enough information.

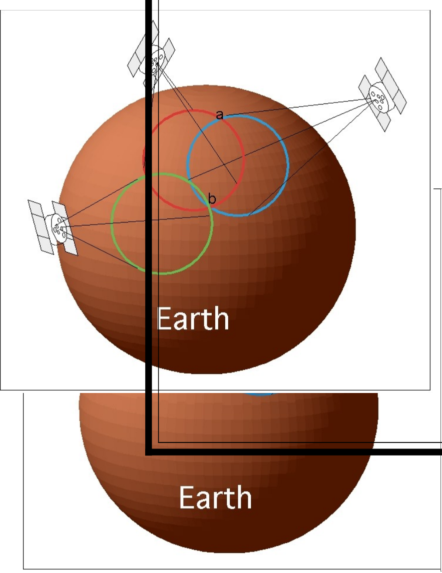


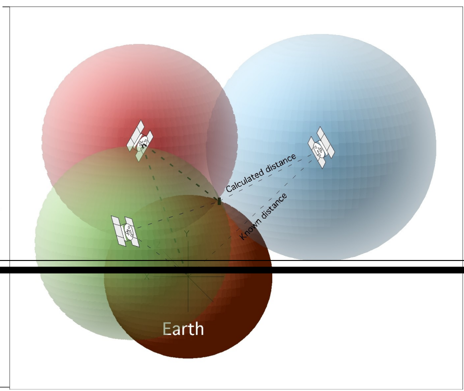
Trilateration

- *When the receiver knows its distance from only one satellite, its location could be anywhere on the earths surface that is an equal distance from the satellite.*
- *Represented by the circle in the illustration.*
- *The receiver must have additional information.*
- *With signals from two satellites, the receiver can narrow down its location to just two points on the earths surface.*
- *Were the two circles intersecting.*
- *Knowing its distance from three satellites, the receiver can determine its location because there is only two possible combinations and one of them is out in space.*
- *In this example, the receiver is located at b.*
- *The more satellite that are used, the greater the potential accuracy of the position location*









V V COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

CE8351 – SURVEYING

Anna University Solved Questions

(Last Five Years)

Staff Name : Mr. R. IYAPPAN **Semester / Year : III / II**

UNIT I

FUNDAMENTALS OF CONVENTIONAL SURVEYING AND LEVELLING

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.

April / May 2018

1. What are the sources of error in chaining? What precautions would you take to avoid them?

Errors in chaining may be classified as:

(i) Personal errors
(ii) Compensating errors, and
(iii) Cumulating errors.

Personal Errors

Wrong reading, wrong recording, reading from wrong end of chain etc., are personal errors. These errors are serious errors and cannot be detected easily. Care should be taken to avoid such errors.

Compensating Errors

These errors may be sometimes positive and sometimes negative. Hence they are likely to get compensated when large number of readings are taken. The magnitude of such errors can be estimated by theory of probability.

The following are the examples of such errors:

(i) Incorrect marking of the end of a chain.
(ii) Fractional part of chain may not be correct though total length is corrected.
(iii) Graduations in tape may not be exactly same throughout.
(iv) In the method of stepping while measuring sloping ground, plumbing may be crude.

Cumulative Errors

The errors, that occur always in the same direction are called cumulative errors. In each reading the error may be small, but when large number of measurements are made they may be considerable, since the error is always on one side. Examples of such errors are:

(i) Bad ranging
(ii) Bad straightening
(iii) Erroneous length of chain
(iv) Temperature variation
(v) Variation in applied pull
(vi) Non-horizontality
(vii) Sag in the chain, if suspended for measuring horizontal distance on a sloping ground.

Errors (i), (ii), (vi) and (vii) are always +ve since they make measured length more than actual.

Errors (iii), (iv) and (v) may be +ve or –ve.

2. The following are the observed fore and back bearings of the lines of a closed tracerse. Correct them necessary for local attraction.

Line	F.B	B.B
AB	**292° 15'**	**111° 45'**
BC	**221° 45'**	**41° 45'**
CD	**90° 05'**	**270° 00'**
DE	**80° 35'**	**261° 40'**
EA	**37° 00'**	**216° 30'**

Answer:

F.B difference B.B = 180° (Free from Local attraction)

Line	F.B	B.B	F.B. ≈ B.B
AB	292° 15'	**111° 45'**	180° 30'
BC	**221° 45'**	**41° 45'**	**180° 00'**
CD	**90° 05'**	270° 00'	179° 55'
DE	80° 35'	261° 40'	181° 05'
EA	37° 00'	216° 30'	179° 30'

The station B & C is free from Local attraction

The observed F.B of BC and B.B of BC is correct, and also B.B of AB & F.B of CD is correct

Line	Observed Bearing		Correction	Corrected Bearing	
	F.B	B.B		F.B	B.B
AB	292° 15'	**111° 45'**	A = -0° 30'	291° 45'	**111° 45'**
BC	**221° 45'**	**41° 45'**	B = 0	**221° 45'**	**41° 45'**
CD	**90° 05'**	270° 00'	C = 0	**90° 05'**	270° 05'
DE	80° 35'	261° 40'	D = +0° 5'	80° 40'	260° 40'
EA	37° 00'	216° 30'	E = -1° 0'	36° 0'	216° 0'

3. The following consecutive readings were taken with the help of a dumpy level. 1.904, 2.653, 3.906, 4.026, 1.964, 1.702, 1.592, 1.261, 2.542, 2.006, 3.145. The instrument was shifted after the forth and seventh readings, the first reading was taken on the staff held on the B.M. of R.L 100.000 meters. Rule out a page of level field book, enter the above readings there on. Calculate the R.Ls of the points and apply arithmetical check.

Station	B.S.	I.S.	F.S.	Rise	Fall	R.L	Remarks
1	1.904					**100.000**	B.M = 100.000 m
2		2.653			**0.749**	**99.251**	
3		3.906			**1.253**	**97.998**	
4	1.964		4.026		**0.120**	**97.878**	
5		1.702		**0.262**		**98.140**	
6	1.261		1.592	**0.110**		**98.250**	
7		2.542			**1.281**	**96.969**	
8		2.006		**0.536**		**97.505**	
9			3.145		**1.139**	**96.366**	
Total	**5.129**		**8.763**	**0.908**	**4.542**		

Arithmetical check:

∑ B.S ≈ ∑ F.S	=	∑ Rise ≈ ∑ Fall	=	Last R.L ≈ First R.L
3.634	=	3.634	=	3.634

Hence Ok.

(Or)

Station	B.S.	I.S.	F.S.	HOC	R.L	Remarks
1	1.904			**101.904**	**100.000**	B.M = 100.000 m
2		2.653			**99.251**	
3		3.906			**97.998**	
4	1.964		4.026	**99.842**	**97.878**	
5		1.702			**98.140**	
6	1.261		1.592	**99.511**	**98.250**	
7		2.542			**96.969**	
8		2.006			**97.505**	
9			3.145		**96.366**	
Total	**5.129**		**8.763**			

Arithmetical check:

∑ B.S ≈ ∑ F.S	=	Last R.L ≈ First R.L
3.634	=	3.634

Hence Ok.

4. A dumpy level was setup with its eye-piece vertically over a peg C. The height from the top of C to the centre of its eye-piece was measured and found to be 1.578 m. The reading on the staff held on the peg D was 1.008. The level was then moved and set up likewise at the peg D. The height of eye piece above D was 1.258 m and the reading on the staff held on the peg C was 1.812. Determine the true reduced level of peg D, if that of peg C was 163.373.

Answer:

When the peg is at C	When the peg is at D
Appearent difference in elevation between C & D = 1.008 – 1.578 = - 0.570 m (D higher)	Appearent difference in elevation between C & D = 1.258 – 1.812 = - 0.554 m (D higher)

True difference in elevation = [(-0.570) + (-0.554)] / 2

= -0.562 m (D higher)

True Reduced level of peg D = R.L. of C + 0.562 = 163.373 + 0.562

= 163.935 m

November / December 2017

1. Explain the principles adopted in the construction of vernier scales.

- A fractional part of one of the smallest division of a graduated scale can be measured with the help of vernier scale. **(2 Marks)**
- The principle of vernier is as that," eye can perceive without strain and with considerable precision when two graduations coincide to form one continuous straight line".
- This scale carries an index mark, which is the zero mark of the scale.
- used to read to a very small unit with great accuracy.
- It consists of two parts – a primary scale and a vernier.
- Primary scale is a plain scale fully divided into minor divisions

- difficult to sub-divide the minor divisions in ordinary way - done with the help of the vernier.
- Graduations on vernier are derived from the primary scale.
- Least count of the vernier = the difference between smallest division on the main division and smallest division on the vernier scale. **(6 Marks)**
- The types of vernier are:

  1. Direct vernier 2. Retrograde vernier

(i) Direct vernier:

- It is constructed (n-1) divisions of the main scale is equal to n division of the vernier.
- In direct vernier, vernier scale moves in same direction of main scale.

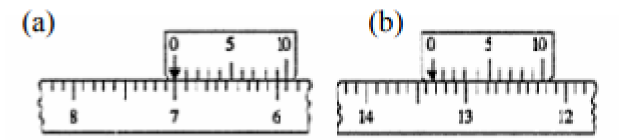
$$Least\,count = \frac{s}{n}$$

where, s = value of one smallest division of main scale
n = number of division on the vernier
v = value of one smallest division of vernier

also, **nv = (n-1) s**

(ii) Retrograde vernier:

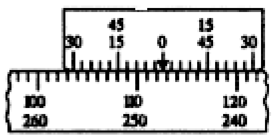
- It is so constructed that (n + 1) division of main scale is equal to n division of vernier. $Least\,count = \frac{s}{n}\ also\ nv = (n+1)s$
- In retrograde vernier, vernier scale moves in opposite direction of main scale.



Extended Vernier:

- This type of vernier is similar to the direct vernier scale except that every second division is omitted.
- extended vernier scale, (2n-1) divisions of the main scale are taken and they are divided into n equal parts.
- Let d = value of smallest division on the main scale v= value of smallest division on the vernier scale

Double Folded Vernier:

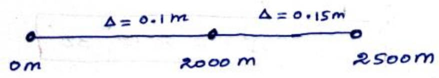


(5 Marks)

2. A distance of 2000 m was measured by 30 m chain, later on it was detected that the chain was 0.10 m too long. Another 500 m (i.e., total 2500 m) was measured and it was detected

that the chain was 0.15 m too long. If the length of the chain in the initial stage was correct, determine the exact length that was measured.

11 (b) Solution



For first 2000 m,

Average error $(e) = \frac{0 + 0.1}{2} = 0.05 m$

Incorrect chain length $(L') = L + e = 20.05 m$

True length $(T.L) = \frac{L'}{L} \times M.L$

$= \frac{20.05}{20} \times 2000$

$\boxed{T.L_1 = 2005 m}$ — (6 marks)

For next 500 m ie, (2500 − 2000 m)

Avg. error $(e) = \frac{0.1 + 0.15}{2} = 0.125 m.$

Incorrect chain length $(L') = L + e = 20 + 0.125 m$

True length $(T.L) = \frac{L'}{L} \times M.L = \frac{20.125}{20} \times 500$

$\boxed{T.L_2 = 503.125 m}$ — (6 marks)

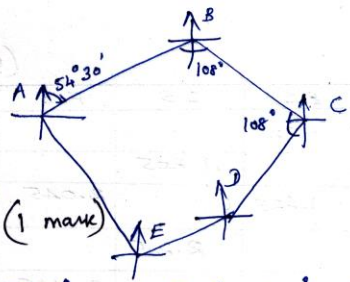
∴ Total True length (or) exact length $= T.L_1 + T.L_2$

$\boxed{T.L = 2508.125 m}$ — (1 marks)

3. A closed traverse with sides is almost that of a regular pentagon. One line of the pentagon has a bearing of 54º 30'. Compute the bearing of the remaining sides, taking the side in a clockwise order.

12 (a)

FB of AB = 54° 30'



For regular Pentagon,

Interior angle = 108°

∴ Traverse → clockwise direction — (1 mark)

∴ FB of BC = FB of AB − ∠A ± 180° = 54° 30' − 108° + 180°

$\boxed{\text{FB of BC} = 126° 30'}$ — (3 marks)

FB of CD = FB of BC − ∠B ± 180° = 126° 30' − 108° + 180°

$\boxed{\text{FB of CD} = 198° 30'}$ — (3 marks)

FB of DE = FB of CD − ∠C ± 180° = 198° 30' − 108° + 180°

$\boxed{\text{FB of DE} = 270° 30'}$ — (3 marks)

FB of EA = FB of DE − ∠D ± 180° = 270° 30' − 108° + 180°

$\boxed{\text{FB of EA} = 342° 30'}$ — (3 marks)

4. **In a fly level surveying, starting from bench mark A (R.L = 400.00) and ending with staff station, the following consecutive sights are taken 0.925, 1.205, 2.045, 1.625, 2.215, 2.415, 2.105 and 1.405. Find the RLs of point B.**

13 (a)

B.S	I.S	F.S	H.I	R.L	Remarks
0.925			400.925	400.00	RL = 400
	1.205			399.720	
	2.045			398.880	
	1.625			399.300	
	2.215			398.710	
	2.415			398.510	
	2.105			398.820	
		1.405		399.520	

Check

$\Sigma B.S \sim \Sigma F.S = 1^{st} RL \sim Last\ RL$

$0.925 \sim 1.405 = 400.00 \sim 399.52$

$0.48 = 0.48$

Hence ok.

Table forming – 2 marks R.L. find out - 9 marks Check – 2 marks

(or)

B.S	I.S	F.S	H.I	R.L	Remarks
0.925			400.925	400.00	RL = 400.00
	1.205			399.72	
1.625		2.045	400.505	398.88	
	2.215			398.29	
2.105		2.415	400.195	398.09	
		1.405		398.79	
$\Sigma = 4.655$		$\Sigma FS = 5.865$			

Check

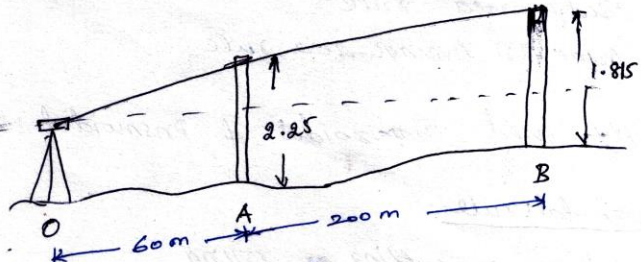
$\Sigma BS \sim \Sigma FS = 1^{st} RL \sim Last\ RL$

$1.21 = 1.21$

Table forming – 2 marks R.L. find out - 9 marks Check – 2 marks

5. **A level was set up at a point O and the distance to two staff stations A and B were 60 m and 200 m. The observed staff readings, on A and B were 2.25 and 1.815. Find the correct difference of level between stations A and B.**

13 (b)



Combined correction for staff A $\quad D = 60\,m = 60\times10^{-3}\,km$

$$C = 0.06728\,D^2 = 0.06728 \times (60\times10^{-3})^2$$

$$\boxed{C = 2.42\times10^{-4}\ m} \text{ ——— (3 marks)}$$

Combined correction for staff B

$$C = 0.06728\,D^2 = 0.06728 \times (200\times10^{-3})^2$$

$$\boxed{C = 2.69\times10^{-3}\ m.} \text{ ——— (3 marks)}$$

Corrected staff reading at A $= 2.25 - 0.00024$

$= 2.2498\ m.$ ——— (3 marks)

Corrected Staff reading at B $= 1.815 - 0.00269$

$= 1.8123\ m.$ ——— (3 marks)

$\therefore$ True (or) correct difference of level b/w A & B $= 2.2498 - 1.8123$

$= 0.4375\ m.$

(1 mark)

UNIT II
THEODOLITE 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

April May 2018

1. A reservoir of bottom size 35 m x 25 m is planned with a depth of 4 m. The side slope 1 ½ : 1 Calculate the quantity of earth to be excavated. Assume the surface of the ground to be level before excavation.

Given Data:

L = 35 m　　B = 25 m　　n = 1.5　　h = 4 m

Reservoir at Bottom

Length　L_{bot} = 35 m

Width　B_{bot} = 25 m

Reservoir at Top

Length　L_{top} = L + 2nh = 35 + (2 x 1.5 x 4) = 47 m

Width　B_{top} = B + 2nh = 25 + (2 x 1.5 x 4) = 37 m

Length at Mid height = $(L + L_{top}) / 2$ = (35 + 47) / 2 = 41 m

Width at Mid height = $(B + B_{top}) / 2$ = (25 + 37) / 2 = 31 m

Area of Reservor

A_{bottom} = $(L_{bot} \times B_{bot})$ = (35 x 25) = 875 m^2

A_{top} = $(L_{top} \times B_{top})$ = (47 x 37) = 1739 m^2

A_{mid} = $(L_{mid} \times B_{mid})$ = (41 x 31) = 1271 m^2

Volume of Reservor

Using Prismoidal formula

$V = (h/6) [A_1 + 4 A_m + A_2]$ = (4 / 6) [875 + (4 x 1739) + 1271] = 6068 m^3

2. A series of offsets were taken from a chain line to a curve boundary line at intervals of 20 m in the following order. 0, 7.2, 5.4, 6.0, 6.8, 7.4, 8.2, 0 metres. Find the area between the chain line, the curved boundary line and the offsets by Trapezoidal rule and Simpson's rules.

Given Data:

d = 20 m,　O_1 = 0,　O_2 = 7.2,　O_3 = 5.4,　O_4 = 6.0,

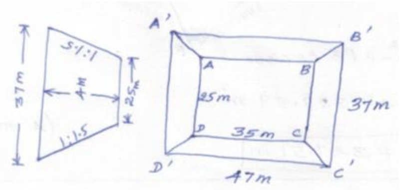
O_5 = 6.8,　O_6 = 7.4,　O_7 = 8.2,　O_n = 0

Trapezoidal Rule:

A = (d/2) [(First ordinate + Last ordinates) + 2 (Other ordinates)]

= (20/2) [(0 + 0) + 2 (7.2 + 5.4 + 6.0 + 6.8 + 7.4 + 8.2)]

A = 820 m^2



Simpson's Rule:

If the given ordinates are even. So take first seven readings except last one.

A = (d/3) [(First ordinate + Last ordinates) + 2 (Odd ordinates) + 4 (Even ordinates)]

= (20/3) [(O + 8.2) + 2 (5.4+6.8) + 4 (7.2 + 6.0 + 7.4)]

A_1 = 766.67 m^2

Take seveth and eighth readings, using trapezoidal rule

A_2 = (20/2) [(8.2 + 0)/ 2] = 41 m^2

Total Area = A1 + A2 = 807 m^2

3. A theodolite was set up at a distance of 200 m from a chimney and the angle of elevation to its top was 10°48'. The staff reading on a B.M. of R.L 70.25 m with the telescope horizontal was 0.977. Find the reduced level of the top of the chimney.

Given Data:

D = 200 m, $\alpha = 10^\circ 48'$ B.M. of R.L = 70.25 m S = 0.977 m

h = D tanα = 200 x tan 10° 48' = 38.152 m

RL of top of chimney = RL of B.M + S + h = 70.25 + 0.977 + 38.152

= 109.379 m

4. Two observations are taken upon a vertical staff by means of a theodolite, of which the R.L of the horizontal axis is 254.30 m. In case of the first, the line of sight is direct to give a staff reading of 1.00 and the angle of elevation is 4°58'. In the second observation, the staff reading is 3.66 m and the angle of elevations is 5°44'. Compute the R.L of staff station and the horizontal distance from the instrument.

Given Data:

$\alpha_1 = 5^\circ 44'$ $\alpha_2 = 4^\circ 58'$ R.L. of Instrument axis = 254.30 m

S = (3.66 – 1.00) = 2.66 m

Horizontal Distance = S / (tan α_1 – tan α_2)

= 197.06 m

Vertical Distance = D tan α_2 = 17.125m

R.L of Staff station = RL of instrument axis + v – r = 270.425 m

November / December 2017

1. Explain how will you determine the capacity of a reservoir using contour map.

* The Storage capacity of a reservoir is determined from contour map
* The contour line indicating the Full reservoir level (F.R.L) is drawn on the contour map.
* The area enclosed b/w sucessive contours are measured by planimeter. (2 marks)
* The volume of water b/w F.R.L and the river bed. is finally estimated by
* Computing of areas and volume is an important part of the office work involved in Surveying.
* For computation of the volume of earthwork, the sectional area of the map are taken to the longitudinal Section during Profile levelling are first calculated.
* After calculating the c/s areas, then, the volume of earth work is calculated by

(1) Trapezoidal Rule
(2) Prismoidal rule (or) simpson's rule
(3) End area rule

(4) Mid area rule
(5) Mean (or) Average area rule.

* Mostly used Trapezoidal & prismoidal rule. (4 marks)

Trapezoidal rule

Volume of cutting or filling

$$V = \frac{d}{2}\left[(A_1 + A_n) + 2(A_2 + A_3 + A_4 + \cdots A_{n-1})\right]$$

where, $d \rightarrow$ common distance/interval

(3 marks)

Prismoidal formula (or) Simpson's rule

$$V = \frac{d}{3}\left[(A_1 + A_n) + 2\underbrace{(A_3 + A_5 + A_5 + \cdots A_{n-2})}_{\text{odd ordinates}} + 4\underbrace{(A_2 + A_4 + \cdots A_{n-1})}_{\text{even ordinates}}\right]$$

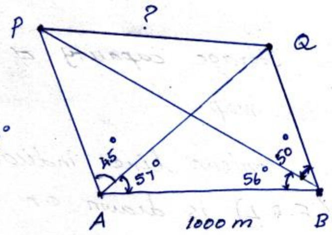
* Prismoidal formula is applicable when there are odd number of sections.
* If the number of sections are even, the end section is is treated separately and the area is calculated according to the trapezoidal rule.
* The volume of the remaining section is calculated in the usual manner by the prismoidal formula.
* Then both the result are added to obtain the total volume. (4 marks)

2. A reservoir of bottom size 35 m x 25 m is planned with a depth of 4 m. The side slope 1.5 : 1 Calculate the quantity of earth to be excavated. Assume the surface of the ground to be level before excavation.

Answer: Same as April May 2018

3. To find out the distance between two inaccessible points P and Q, the theodolite is set up at two stations A and B, 1000 m apart and the following angles were observed; PAQ = 45°, QAB = 57°, PBA = 56°, PBQ = 50°. Calculate the distance PQ.

15 (a)



In Δ^{le} PAB

$\angle APB = 180 - 45° - 57° - 56°$

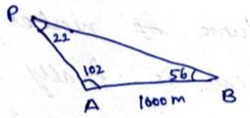
$= 22°$

In Δ^{le} QAB.

$\angle AQB = 180° - 57° - 56° - 50° = 17°$ (3 marks)

Using sine Rule,

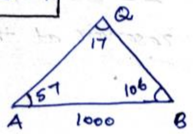
Δ^{le} PAB,



$$\frac{PB}{\sin 102^\circ} = \frac{1000}{\sin 22^\circ}$$

$\therefore \boxed{PB = 2611.13\ m}$ (3 marks)

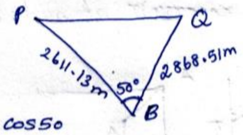
Δ^{le} QAB



$$\frac{QB}{\sin 57} = \frac{1000}{\sin 17^\circ}$$

$\therefore \boxed{QB = 2868.51\ m}$ (3 marks)

Δ^{le} BQP,



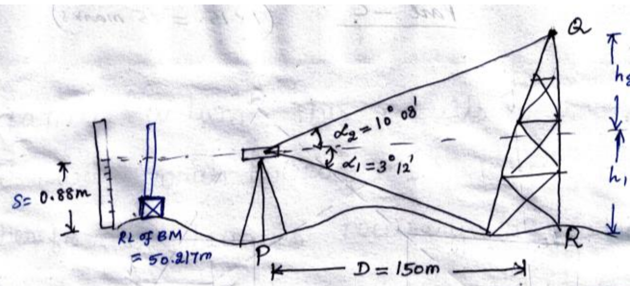
$$PQ^2 = PB^2 + QB^2 - 2PB \cdot QB \cos 50$$

$$PQ^2 = 541732 3.59\ m^2$$

$\therefore \boxed{PQ = 2327.51\ m}$ (4 marks)

4. A theodolite was set up at a distance of 150 m from a tower. The angle of elevation to the top of the tower was 10º 08', while the angle of depression to the foot of the tower was 3º12'. The staff reading on the B.M. of RL. 50.217 m with the telescope horizontal was 0.880 m. Find the height of the tower and reduced level of the top and foot of the tower.

15 (b)



$h_1 = D \tan \alpha_1 = 150 \times \tan 3^\circ 12'$

$h_1 = 8.386\ m$

$h_2 = D \tan \alpha_2 = 150 \times \tan 10^\circ 8'$ (3 marks)

$h_2 = 26.809\ m$ (2 marks)

ht of tower $(h) = h_1 + h_2 = 35.195\ m.$

RL of Inst. Axis = RL of BM + S = 50.217 + 0.88

$= 51.097\ m.$ (2 marks)

RL of top (Q) the tower at = Ht. of Inst. Axis + h_2

$= 51.097 + 26.809$ (3 marks)

$= 77.906\ m.$

RL of the tower at foot (R) = Ht. of Inst. Axis − h_1

$= 51.097 - 8.386\ m$ (3 marks)

$= 42.711\ m.$

5. **The following consequent readings where taken in a level and a 4 m leveling staff on a continuously sloping ground at common interval of 30 mthe readings are 0.855, 1.545, 2.335, 3.115, 3.825, 0.455, 1.380, 2.055, 2.855, 3.455, 0.585, 1.015, 1.850, 2.755, 3.845. R.L of A is 380.500 m the last reading taken point is B. Find the gradient between A and B.**

16 (b)

chainage	station	B.S	I.S	F.S	Rise	Fall	R.L	Remarks.
0 m	A	0.855					380.500	RL of A = 380.50 m
30 m	1		1.545			0.690	379.810	
60 m	2		2.335			0.790	379.02	
90 m	3		3.115			0.780	378.24	
120 m	4	0.455		3.825		0.710	377.530	
150 m	5		1.380			0.925	376.605	
180 m	6		2.055			0.675	375.930	
210 m	7		2.855			0.800	375.130	
240 m	8	0.585		3.455		0.600	374.530	
270 m	9		1.015			0.430	374.100	
300 m	10		1.850			0.835	373.265	
330 m	11		2.755			0.905	372.360	
360 m	B			3.845		1.090	371.270	RL of B
		$\Sigma BS = 1.895$		$\Sigma FS = 11.125$		$\Sigma Fall = 9.23$		(11 marks)

Check

$\Sigma BS \sim \Sigma F.S = \Sigma Rise \sim \Sigma Fall = 1^{st}\ RL \sim Last\ RL$

$1.895 \sim 11.125 = 0 \sim 9.23 = 380.500 \sim 371.270$

$9.23 = 9.23 = 9.23$

Hence ok. (2 marks)

Gradient

$$\text{Gradient b/w line } A \& B = \frac{1^{st}\ RL \sim Last\ RL}{\text{Total chainage length}} = \frac{9.23}{360}$$

$$= 0.0256$$

Gradient b/w line A&B = 1 in 39 (falling) (2 marks)

UNIT III

CONTROL SURVEYING AND ADJUSTMENT

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.

April / May 2018

1. ***What is mean by triangulation adjustment? Explain the different condition and cases with sketches.***

9.12. TRIANGULATION ADJUSTMENTS

In a triangulation system, all the measured angles should be corrected so that they satisfy :

(*i*) Conditions imposed by the station of observation, known as the *station adjustment;* and

(*ii*) Conditions imposed by the figure, known as the *figure adjustment.*

The most accurate method is that of least squares, and the most rigid application follows when the entire system is adjusted in one mass, all the angles being simultaneously involved. The process is exceedingly laborious, even in nets comprising few figures. As such, it is always convenient to break it into three parts which are each adjusted separately.

(*i*) Single angle adjustment. (*ii*) Station adjustment.

and (*iii*) Figure adjustment.

(1) Single Angle Adjustment

Generally, several observations are taken for a single angle. The corrections to be applied are inversely proportional to the weight and directly proportional to the square of probable errors. In the case of the measurement of the angle with equal weights, the most probable value is equal to the arithmetic mean of the observations. In the case of the weighted observations, the most probable value of the angle is equal to the weighted arithmetic mean of the observed angles. See examples 9.2, 9.3, 9.4 and 9.5.

(2) Station Adjustment

Station adjustment is the determination of the most probable values of two or more angles measured at a station so as to satisfy the condition of being geometrically consistent. There are three cases of station adjustment :

(*i*) when the horizon is closed with angles of equal weights

(*ii*) when the horizon is closed with angles from unequal weights

(*iii*) when several angles are measured at a station individually, and in combination.

Case 1. When the horizon is closed with angles of equal weights.

In Fig. 9.3, angles *A, B* and *C* have been measured and the horizon is closed. Hence $A+B+C$ should be equal to 360°. If this condition is not satisfied, the error is distributed *equally* to all the three angles.

Case 2. When the horizon is closed with angles of unequal weights.

If the angles observed are of unequal weight, discrepancy is distributed among the angles inversely as the respective weights.

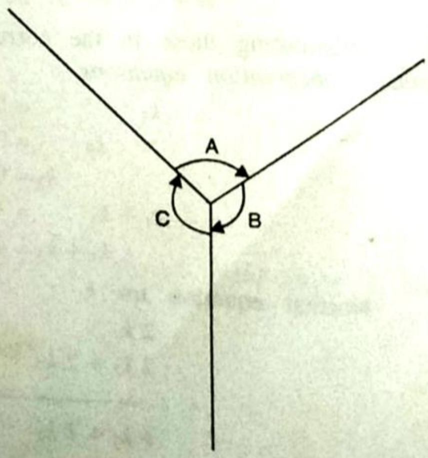


FIG. 9.3

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Case 3. When the several angles are measured at a station individually and also in combination.

In Fig. 9.4, the three angles A, B and C are measured individually. Also the summation angles $A + B$ and $A + B + C$ have been measured. As discussed earlier, the most probable value of the angles can be found by forming the normal equations for the unknowns and solving them simultaneously. See example 9.9, 9.10, 9.11, 9.21 and 9.22.

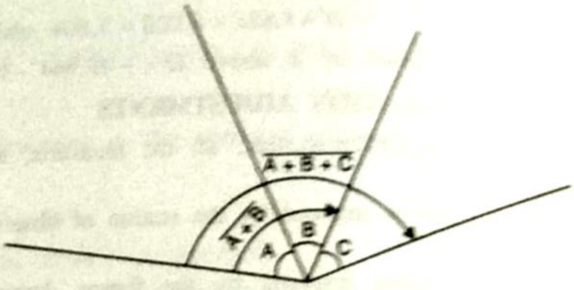

FIG. 9.4

2. ***A traverse ABCD was to be run but due to an obstruction between the stations A and B, it was not possible to measure the length and direction of the line AB. The following data could be only be obtained. Determine the length and the direction of BA. Also***

Line	***Length (m)***	***R.B.***
AD	***44.50***	***N50º20'E***
DC	***67.00***	***S69º45'E***
CB	***61.30***	***S30º10'W***

Answer:

Line	***Length (m)***	***R.B.***	***Northing (Latitude) (L Cos θ)***	***Southing (Latitude) (L Cos θ)***	***Easting (Departure) (L Sin θ)***	***Westing (Departure) (L Sin θ)***
AD	44.50	N50º20'E	28.405		34.25	
DC	67.00	S69º45'E		23.189	62.858	
CB	61.30	S30º10'W		52.997		30.804
Total			***28.405***	***76.186***	***97.108***	***30.804***

Algebraic sum of the latitude and departure should be equal to zero

28.405 – 76.186 + L Cos θ = 0   *i.e.,* ***L Cos θ = 47.781***

97.108 – 30.804 + L Sin θ = 0   *i.e.,* ***L Sin θ = -66.304***

The line AB lies in NW quadrant

tan θ = (departure / latitude) = (66.304 / 47.781) = 1.387

θ = N54º13'20"W

Length of line AB = ***√(lattitude² + departure²)***

= ***81.73 m***

3. ***Find the most probable value of angles A, B and C of triangle ABC from the following observation equations.***

A = 68° 12′ 36″

B = 53° 46′ 12″

C = 58°01′ 16″

Solution:

The conditional equation is

A + B + C = 180° 00′ 00″

i.e., C = 180 – (A + B) = 58° 01′ 16″ --------(a)

or

A + B = 180° - 58° 01′ 16″ = 121° 58′ 44″

Hence the new observation equations are

A = 68° 12′ 36″

B = 53° 46′ 12″
A + B = 121° 58′ 44″

Normal equation for A

A = 68° 12′ 36″
A + B = 121° 58′ 44″

2A + B = 190° 11′ 20″ --(1)

Normal equation for B

B = 53° 46′ 12″
A + B = 121° 58′ 44″

A + 2B = 175° 44′ 56″ --(2)

Solving these equations (1) and (2), we get

A = 68° 12′ 34.7″
B = 53° 46′ 10.6″

Substituting these values in equation (a)

C = 180 – (A + B) = 180 – (68° 12′ 34.7″ + 53° 46′ 10.6″)

C = 58° 01′ 14.7″

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4. ***Write the various rules that are adopted for corrections to the observed angles of triangles in figure adjustment.***
   - Figure adjustments are the determination of the most probable values of the angles involved in any geometrical figure. So as to fulfil the geometric requirements.
   - The geometrical figures adopted in the triangulation systems are
     - *Triangles*
     - *Quadrilaterals*
     - *Polygons with central stations*

Rules for Figure Adjustments:

- Let us considered a triangle having an included angle A, B, and C.
- Take W_1, W_2, & W_3 be the weight of observed angle and also n_1, n_2 and n_3 be the number of observations for angles A, B, and C respectively.
- E_1, E_2, & E_3 are the most probable error in the angles A, B, and C.
- C_1, C_2, & C_3 be the corresponding corrections of A,B, & C.
- C be the total correction.

Rule: 1 – Equal weight correction

- If the observed angles of a triangle are equal weight, then the total error is equally distributed to the observed angles.

$$C_1 = C_2 = C_3 = (1/3)\ C$$

For example, if the total error is 6” then $C_1 = C_2 = C_3 = (6/3) = 2”$

Rule: 2 – Inverse weight correction

- If the observed angles of a triangle are unequal weight, then the total error is distributed to all the angles inverse proportion to the weights.
- $C_1 : C_2 : C_3 = (1/W_1) : (1/W_2) : (1/W_3)$
- $C_1 / (C_1 + C_2 + C_3) = (1/W_1) / [(1/W_1) + (1/W_2) + (1/W_3)]$
- $C_2 / (C_1 + C_2 + C_3) = (1/W_2) / [(1/W_1) + (1/W_2) + (1/W_3)]$
- $C_3 / (C_1 + C_2 + C_3) = (1/W_3) / [(1/W_1) + (1/W_2) + (1/W_3)]$

Rule: 3 – Inverse correction

- If the weight of observations are not given, then the error is distributed to all the angle is inverse proportion to their number of observations.
- $C_1 : C_2 : C_3 = (1/n_1) : (1/n_2) : (1/n_3)$
- $C_1 / (C_1 + C_2 + C_3) = (1/n_1) / [(1/n_1) + (1/n_2) + (1/n_3)]$
- $C_2 / (C_1 + C_2 + C_3) = (1/n_2) / [(1/n_1) + (1/n_2) + (1/n_3)]$
- $C_3 / (C_1 + C_2 + C_3) = (1/n_3) / [(1/n_1) + (1/n_2) + (1/n_3)]$

Rule: 4 – Inverse square correction

- If the error is distributed to all the angle is inverse proportion to the square of the number of observations.
- $C1 : C2 : C3 = (1/n_1)^2 : (1/n_2)^2 : (1/n_3)^2$
- $C1 / (C1 + C2 + C3) = (1/n_1)^2 / [(1/n_1)2 + (1/n_2)^2 + (1/n_3)^2]$
- $C2 / (C1 + C2 + C3) = (1/n_2)^2 / [(1/n_1)^2 + (1/n_2)^2 + (1/n_3)^2]$
- $C3 / (C1 + C2 + C3) = (1/n_3)^2 / [(1/n_1)^2 + (1/n_2)^2 + (1/n_3)^2]$

Rule: 5 – Probable error square correction

- If the probable errors of each angle of a triangles are known, then the error is distributed to all the angle in direct proportion to the squares of the probable error.
- $C1 : C2 : C3 = E_1^2 : E_2^2 : E_3^2$
- $C1 / (C1 + C2 + C3) = (E_1^2) / [(E_1^2 : E_2^2 : E_3^2)]$
- $C2 / (C1 + C2 + C3) = E_2^2 / [(E_1^2 : E_2^2 : E_3^2)]$
- $C3 / (C1 + C2 + C3) = E_3^2 / [(E_1^2 : E_2^2 : E_3^2)]$

November / December 2017

1. ***A steel tape 20 m long standardized at 55° F with a pull of 10 Kg 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 16 Kg. Weight of 1 cubic metre of steel = 7.86g, weight of tape = 0.8 Kg and E = 2.1095 x 10⁶ Kg/cm². Coefficient of linear expansion of tape per 1° F = 6.2 x 10⁻⁶.***

Solution:

$L = 20$ m; $T_0 = 55°C$; $T_m = 80°C$; $P_o = 10$ Kg; $P = 16$ Kg; $\alpha = 6.2 \times 10^{-6}$;

Weight of steel = 7.86 g; Weight of tape = 0.8 Kg; $E = 2.109 \times 10^6$ Kg / cm²

i) Correction for Temperature:

$C_t = \alpha (T_m - T_0) L = 6.2 \times 10^{-6} (80 - 55) \times 20$; $\mathbf{C_t = 0.0031\ m}$

ii) Correction for Pull:

$$C_p = \left(\frac{P - Po}{AE}\right) L$$

Weight of tape = (Area x 1 x weight of steel) x length

0.80 = (A x 1 x 7.86) x 20

A = 5.1 mm²

$\mathbf{C_P = 0.00112\ m}$

iii) Sag Correction:

$$C_s = \frac{LW^2}{24n^2P^2}$$

Cs = 0.00208 m

Total correction = $C_t + C_P - C_s$ = 0.0031 + 0.00112 – 0.00208 = **0.00214 m**

True length = Length + correction = 20 + 0.00214 = **20.00214 m**

2. ***Observations were made from instrument station A to the signal at B. The sun makes an angle of 60º with the line BA. Calculate the phase correction if (i). the observation was made on the bright portion and (ii). The observation was made on the bright line. The distance AB is 9460 metres. The diameter of the signal is 12 cm.***

Given Data:

D = 9460 m; α = 60º; d = 12 cm; r = 6 cm = 0.06 m

ii) The observation is made on the bright portion:

$$\beta = \frac{206265 \; r \cos^2 \frac{\alpha}{2}}{D} \text{ seconds}$$

=

iii) Observation is made on the bright line:

$$\beta = \frac{206265 \; r \cos \frac{\alpha}{2}}{D} \text{ seconds}$$

=

3. ***Adjust the following angles closing the horizon at a station.***

A = 110º 20’ 48”	***weight 4***
B = 92º 30’ 12”	***weight 1***
C = 56º 12’ 00”	***weight 2***
D = 100º 57’ 04”	***weight 3.***

Solution:

Sum of observed angles = 110º 20’ 48” + 92º 30’ 12” + 56º 12’ 00” + 100º 57’ 04”

= 360º 0’ 4”

Error = + 4”

Total correction = - 4”

Let, C1, C2, C3 & C4 - corrections to the observed angles

A, B, C & D - error will be distributed to the angles in an inverse proportion to their weights.

A = 110º 20’ 48” + C1

B = 92º 30’ 12” + C2

C = 56º 12’ 00” + C3

D = 100º 57’ 04” + C4

$C_1 : C_2 : C_3 : C_4$ = $4^2 + 1^2 + 2^2 + 3^2$ = 16 : 1 : 4 :9 …………….. (1)

Also, $C_1 + C_2 + C_3 + C_4$ = 4” ……………. (2)

From (1) C_2 = $16C_1$

C_3 = $4C_1$

$9C_4$ = $16C_1$

Substituting these values of C2, C3 & C4 in (2), we get

$C_1 + 16C_1 + 4C_1 + (16/9)\, C_1$ = 4”

C_1 = 0.18”

C_2 = 2.88”

C_3 = 0.72”

$9C_4$ = 0.32”

Hence the corrected angles are

A	=	110^o 20’ 48”	- 0.18”	=	110^o 20’ 47.82”
B	=	92^o 30’ 12”	– 2.81”	=	92^o 30’ 9.19”
C	=	56^o 12’ 00”	– 0.70”	=	56^o 11’ 59.30”
D	=	100^o 57’ 04”	– 0.31”	=	100^o 56’ 33”
		Sum		=	**360^o 00’ 00’’**

4. The following observations of the three angles A, B, C were taken at one station.

A	=	***75° 32' 46.3"***	***Weight 3***
B	=	***55° 09' 53.2"***	***Weight 2***
C	=	***108° 01' 29"***	***Weight 2***
A+B	=	***130° 42' 4.6"***	***Weight 2***
B+C	=	***163° 19' 22.5"***	***Weight 1***
A + B+C	=	***238° 52' 9.8"***	***Weight 1***

Determine the most probable value of each angle.

Solution:

Normal equation of A:

3A	=	226° 38' 18.9"	
2A + 2B	=	261° 24' 9.2"	
A + B + C	=	238° 52' 9.8"	
6A + 3B + C	=	726° 54' 37.9"	……………. (1)

Normal equation of B:

B	=	55° 09' 53.2"	
2A + 2B	=	261° 24' 9.2"	
B + C	=	163° 19' 22.5"	
A + B + C	=	238° 52' 9.8"	
3A + 5B + 2C	=	718° 45' 34.7"	……………. (2)

Normal equation of C:

2C	=	216° 02' 58"	
B + C	=	163° 19' 22.5"	
A + B + C	=	238° 52' 9.8"	
A + 2B + 4C	=	618° 14' 30.3"	……………. (2)

The three normal equations are

6A + 3B + C	=	726° 54' 37.9"
3A + 5B + 2C	=	718° 45' 34.7"
A + 2B + 4C	=	618° 14' 30.3"

By solving above equations we get,

A	**=**	**75° 32' 25.82"**
B	**=**	**55° 11' 48.75"**
C	**=**	**108° 04' 36.74"**

April / May 2017

1. (i). ***What are signals? Classify them, Enumerate the requirements to be fulfilled by signals.***

- A ***signal*** is a device erected to define the exact position of a triangulation station.
- It is placed at each station so that line of sight are established between triangulation stations.

Characteristics or Requirements of a Good Signal:

- It should be clearly visible against any background.
- It should be kept at least 75 cm above the station mark.
- It should be suitable for bisection from other stations.
- It should be free from phase, or should exhibit little phase
- In general, the diameter of the signals should be a range of 1.3 D to 1.9 D.
  Where
    - D = Distance in Kilometer
- It should be capable of being accurately centered over the station mark.
- It should be symmetrical
- It should be easy to erect in minimum time.
- It should be sufficient height, capable being vertical and accurately centered over the station mark.
- In general, the height of the signal is a range of 13.3 D
  Where
    - h = height of signal
    - D = Distance in Kilometer

Classification of signals

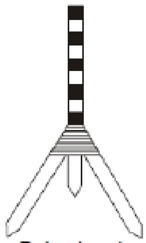
i. Non-luminous, opaque or daylight signals
ii. Luminous signals.

(i) Non-luminous signals or daylight signals

- Non-luminous signals are used during day time and for short distances.
- Most commonly used for,

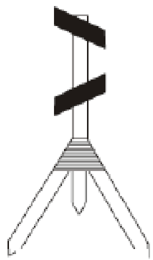
(a) Pole signal

- It consists of a round pole painted black and white in alternate strips, and is supported vertically over the station mark, generally on a tripod.
- Pole signals are suitable up to a distance of about 6 km.


Pole signal

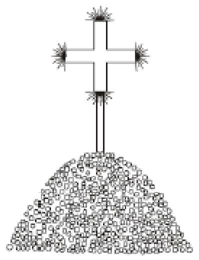
(b) Target signal

- It consists of a pole carrying two squares or rectangular targets placed at right angles to each other.
- The targets are generally made of cloth stretched on wooden frames.
- Target signals are suitable up to a distance of 30 km.


Target signal

(c) Pole and brush signal

- It consists of a straight pole about 2.5 m long with a bunch of long grass tied symmetrically round the top making a cross.
- The signal is erected vertically over the station mark by heaping a pile of stones, up to 1.7 m round the pole.
- A rough coat of white wash is given to make it more conspicuous to be seen against black background.


Pole and brush signal

- It must be erected over every station of observation during reconnaissance.

(d) Stone cairn

- A pile of stone heaped in a conical shape about 3 m high with a cross shape signal erected over the ston e heap, is stone cairn.
- White washed opaque signal is very useful in the dark background.

(e) Beacons

- It consists of red and white cloth tied round the three straight poles.
- It can easily be centered over the station mark.

(ii) Luminous signals

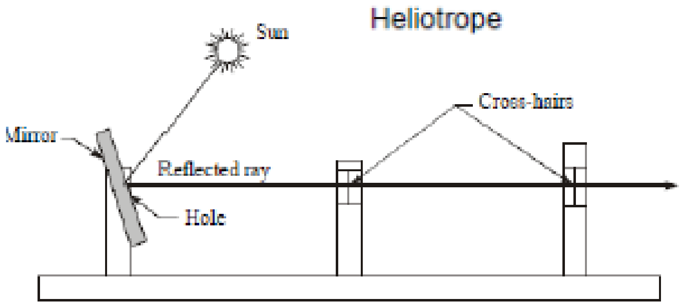
- Luminous signals may be classified into two types :
  (a) Sun signals
  (*b*) Night signals.

(a) Sun signals

- Sun signals reflect the rays of the sun towards the station of observation, and are also known as heliotropes.
- Such signals can be used only in day time in clear weather.

Heliotrope:

- It consists of a circular plane mirror with a small hole at its centre to reflect the sun rays, and a sight vane with an aperture carrying cross-hairs.
- The circular mirror can be rotated horizontally as well as vertically through 360°.



- The heliotrope is centered over the station mark, and the line of sight is directed towards the station of observation.
- The sight vane is adjusted looking through the hole till the flashes given from the station of observation fall at the centre of the cross of the sight vane.
- Once this is achieved, the heliotrope is disturbed.
- Now the heliotrope frame carrying the mirror is rotated in such a way that the black shadow of the small central hole of the plane mirror falls exactly at the cross of the sight vane.
- The reflected beam of rays will be seen at the station of observation.
- Due to motion of the sun, this small shadow also moves, and it should be constantly ensured that the shadow always remains at the cross till the observations are over.
- The heliotropes do not give better results compared to signals.
- These are useful when the signal station is in flat plane, and the station of observation is on elevated ground.
- The distance between the stations exceed 30 km, the heliotropes become very useful.

(b) Night signals:

- When the observations are required to be made at night, the night signals of following types may be used.
- Various forms of oil lamps with parabolic reflectors for sights less than 80 km.
- Acetylene lamp designed by Capt. McCaw for sights more than 80 km.
- Magnesium lamp with parabolic reflectors for long sights.
- Drummond’s light consisting of a small ball of lime placed at the focus of the parabolic reflector, and raised to a very high temperature by impinging on it a stream of oxygen.
- Electric lamps.

(ii). ***A steel tape of nominal length 30 m was suspended between two supports to measure the length on a slope of 4º 25' is 29.861 m. the mean temperature during measurement was 15ºC and pull applied was 120 N. if standard length of the tape was 30.008 m at 27ºC and the standard pull of 50 N, calculate the correct horizontal length. Take the weight of the tape as 0.16 N, its cross sectional area equal to 2.75 m² coefficient of linear thermal expansion = 1.2 x10⁻⁵ per degree Celsius and E = 2.05 x 10⁵ N /m².***

Solution:

L_t = 30 m; L_{sl} = 29.861 m; L_s = 30.008 m; T_0 = 27° C; T_m = 15° C; P_o = 50 N; P = 120 N;
α = 1.2 x 10^{-5}; Area = 2.75 mm^2; Weight of tape = 0.16 N/m; E = 2.05 x 10^5 N / mm^2

i) Correction for slope:

$$C = \frac{h^2}{2L}$$

Here h = L_{sl} sinθ = 29.861 x sin (4° 25') = 2.3 m

$$C = \frac{2.3^2}{2 \ X \ 29.861} \ \mathbf{= 0.0886 \ m}$$

ii) Correction for absolute length:

$$C_a = \frac{L_C}{l} = \frac{29.861 \ X \ (30.008 - 29.861)}{30.008}$$

C_a = 0.146 m

iii) Correction for Temperature:

C_t = α (T_m - T_0) L_{sl}
= 1.2 x 10^{-5} (15 – 27) x 29.861
C_t = - 0.0043 m

iv) Correction for Pull:

$$C_P = \left(\frac{P - Po}{AE}\right)L = \frac{120 - 50}{2.75 \ X \ 2.05 \ X \ 10^5} \ X \ 29.861$$

C_P = 0.0037 m

v) Sag Correction:

$$C_s = \frac{LW^2}{24n^2P^2} = \frac{29.861 \ X \ 0.16^2}{24 \ X \ 1^2 \ X \ 120^2}$$

C_s = 0.0000022 m

Total correction = - C + C_a + C_t + C_P - C_s
= - 0.0886 + 0.146 – 0.0043 + 0.0037 – 0.0000022

Total correction = 0.0568 m

True length = Length + correction
= 29.861 + 0.0568

True length = 29.92 m

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2. ***(i). Following are the observations made between two stations.***

Observation altitude	=	*+3º32'36"*
Height of Instrument	=	*1.15 m*
Height of signal	=	*4.85 m*
Horizontal distance	=	*4895 m*
Co-efficient of refraction	=	*0.07*
R sin 1"	=	*30.88 m.*

Correct the observed altitude for the height of signal – refraction and curvature.

Solution:

d = 6945 m;	α = +3º32'36"	s = 4.85m
h = 1.15 m	m = 0.07	R sin 1" = 30.88 m

Correction for Axis Signal (δ)

$$\delta = \frac{s - h}{d \sin 1''}$$

$\boxed{\delta = 155.91 \text{ sec.} = 0^o 2' 36''}$ Negative

Correction for refraction (r)

Refraction correction, $r = m\theta$

$$\theta = \frac{d}{R \sin 1''}$$

$\boxed{r = 11.096 \text{ sec.} = 0^o 0' 11''}$ Negative (angle is elevation)

Correction for curvature (c)

Curvature correction, $\frac{\theta}{2}$

$\boxed{c = 79.926 \text{ sec.} = 0^o 1' 19''}$ positive

Total Correction:

Total Correction = c – r – δ = **- 0°1'28"**

Corrected observed angle:

Observed angle or altitude = +3°32'36"

Corrected observed angle = α ± correction

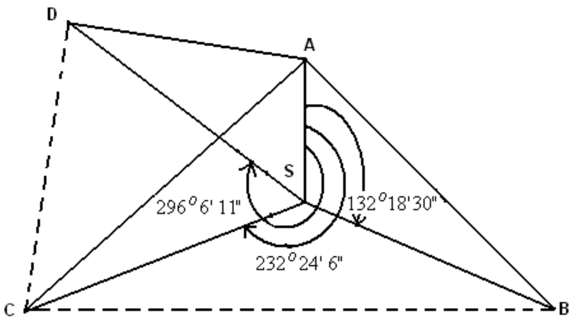
$\boxed{\alpha_1 = +3^o 31' 8''}$

(ii). From a satellite station S, 5.8 m from main triangulation station A, the following directions were measured. A = 0o 0' 0"; B = 132o 18' 30"; C = 232° 24' 06"; D = 296° 06' 11"; AB = 3265.5 m; AC = 4022.2 m; AD = 3086.4 m. determine the directions of AB, AC and AD.

Solution:

The correction to any direction is given by,

$$\beta = \frac{d \sin \theta}{D \sin 1''} \text{ seconds}$$



a) For the line AB:

$\theta = 132^{o}$ 18' 30" ; d = AS = 5.8 m; D = AB = 3265.5 m;

$$\beta = \frac{d \sin\theta}{D \sin 1''} = \frac{5.8\ X\ \sin\ (\ 132^{o}\ 18'\ 30''\ )}{3265.5\ X\ \sin 1''}$$

= 270.9" = 4' 30.9"

Direction of AB = direction of SB + β = 132^{o} 18' 30" + 4' 30.9"

= 132^{0} 23' 0.9"

b) For the line AC:

$\theta = 232^{o}$ 24' 6" ; d = AS = 5.8 m; D = AC = 4022.2 m;

$$\beta = \frac{d \sin\theta}{D \sin 1''} = \frac{5.8\ X\ \sin\ (\ 232^{o}\ 24'\ 6''\ )}{4022.2\ X\ \sin 1''}$$

= - 235.7" = - 3' 55.7"

Direction of AB = direction of SC + β = 232^{o} 24' 6" - 3' 55.7"

= 232^{0} 20' 10.3"

c) For the line AD:

$\theta = 296^{o}$ 6' 11" ; d = AS = 5.8 m; D = AD = 3086.4 m;

$$\beta = \frac{d \sin\theta}{D \sin 1''} = \frac{5.8\ X\ \sin\ (\ 296^{o}\ 6'\ 11''\ )}{3086.4\ X\ \sin 1''}$$

= - 348.1" = - 5' 48.1"

Direction of AB = direction of SD + β = 296^{o} 6' 11" - 5' 48.1"

= 296^{0} 0' 22.9"

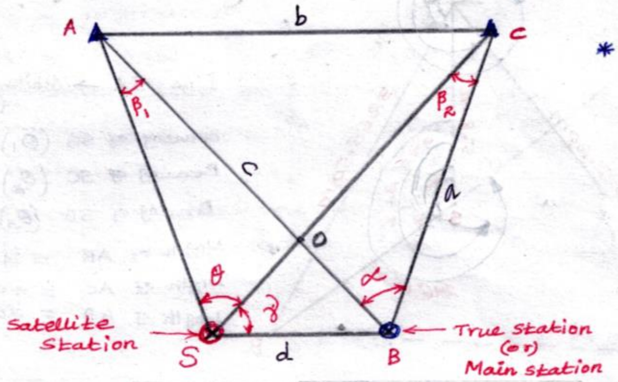
November December 2016

1. ***What is meant by a satellite station? Derive the expression for reducing the angles measured at the satellite station to centre.***

Satellite station:

Sometimes in order to form well-conditioned triangles of triangulation and also to have better visibility objects such as church spirals, towers of temples, flag poles, etc are selected. But the instrument cannot be set up over these true stations for the measurement of angles. In such cases, a subsidiary station called as satellite station or eccentric station or false station is selected as near as possible to the true station. From this station observations are taken to the other triangulation stations with the same precision.

Computation of True angle



* A subsidiary station is established as near the true (or) Principal station as possible, the station so established is called satellite (or) eccentric (or) false station.

A, B & C $\rightarrow$ Triangulation station

Assume B $\rightarrow$ True or main station (Tower/church spire)

$\alpha \rightarrow \angle ABC$; $\gamma \rightarrow$ observed angle at S $= \angle CSB$

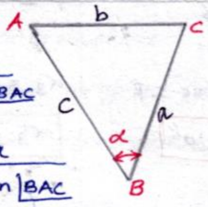
$\theta \rightarrow$ observed angle at S $= \angle ASC$

$\beta_1 \rightarrow \angle SAB$; $\beta_2 = \angle SCB$

$d \rightarrow$ eccentric distance b/w B & S

By applying sine rule

Δ^{le} ABC



$$\frac{AC}{\sin\alpha} = \frac{BC}{\sin\angle BAC}$$

$$\frac{b}{\sin\alpha} = \frac{a}{\sin\angle BAC}$$

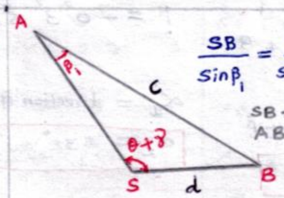
$$\therefore \boxed{a = \frac{b.\sin\angle BAC}{\sin\alpha}}$$

$$\frac{AB}{\sin\angle ACB} = \frac{AC}{\sin\alpha}$$

$$\frac{c}{\sin\angle ACB} = \frac{b}{\sin\alpha}$$

$$\therefore \boxed{c = \frac{b.\sin\angle ACB}{\sin\alpha}}$$

Δ^{le} ASB

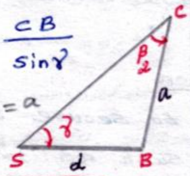


$$\frac{SB}{\sin\beta_1} = \frac{AB}{\sin(\theta+\gamma)}$$

SB = d
AB = C

$$\boxed{\sin\beta_1 = \frac{d\sin(\theta+\gamma)}{c}} \times \frac{180\times60\times60}{\pi} \text{ in sec.}$$

Δ^{le} CSB



$$\frac{SB}{\sin\beta_2} = \frac{CB}{\sin\gamma}$$

SB = d ; CB = a

$$\boxed{\sin\beta_2 = \frac{d\sin\gamma}{a}}$$

BS $\rightarrow$ Very small compared to AB & BC

$$\therefore \beta_1 = \frac{\sin\beta_1}{\sin 1''} \text{ in sec.}$$

$$\boxed{\beta_1 = \frac{d\sin(\theta+\gamma)}{c\sin 1''}}$$

(or)

$$\boxed{\beta_1 = \frac{d\sin(\theta+\gamma)}{c} \times 206265}$$

similarly

$$\beta_2 = \frac{\sin\beta_2}{\sin 1''}$$

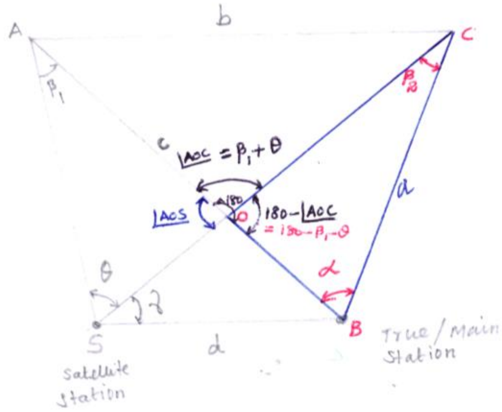
$$\boxed{\beta_2 = \frac{d\sin\gamma}{a\sin 1''}}$$

(or)

$$\boxed{\beta_2 = \frac{d\sin\gamma}{a} \times 206265}$$

True angle (α)

In Δ^{le} COB



$$\alpha = 180 - (\beta_2 + \angle COB) \quad \text{--- (1)}$$

But $\angle COB = 180 - \angle AOC$ --- (2)

$\therefore \angle AOC = 180 - \angle AOS$ --- (3)

$\therefore \angle AOS = 180 - \beta_1 - \theta$

$$\therefore \angle AOC = 180 - \angle AOS = 180 - (180 - \beta_1 - \theta)$$

$$= \cancel{180} - \cancel{180} + \beta_1 + \theta$$

$$\boxed{\angle AOC = \beta_1 + \theta}$$

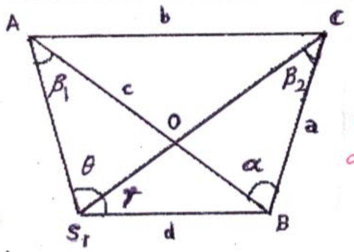
$$\therefore \angle COB = 180 - \angle AOC = 180 - (\beta_1 + \theta) = 180 - \beta_1 - \theta$$

$$\boxed{\angle COB = 180 - \beta_1 - \theta}$$

$$\therefore \alpha = 180 - (\beta_2 + \angle COB) = 180 - (\beta_2 + 180 - \beta_1 - \theta)$$

$$= \cancel{180} - \beta_2 - \cancel{180} + \beta_1 + \theta$$

$$\boxed{\alpha = \theta + \beta_1 - \beta_2}$$



True Angle

$\alpha = \theta + \beta_1 - \beta_2$

Fig. 1 – S_1 to the left of B

Positions of satellite stations

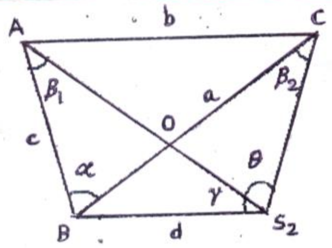


Fig. 2 – S_2 to the right of B

$\alpha = \theta - \beta_1 + \beta_2$

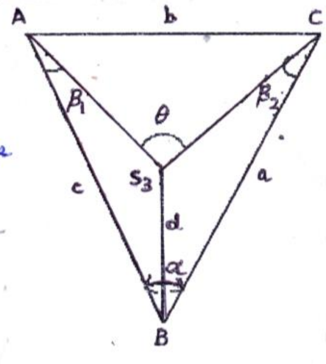


Fig. 3 – S_3 b/w AC & B

$\alpha = \theta - \beta_1 - \beta_2$

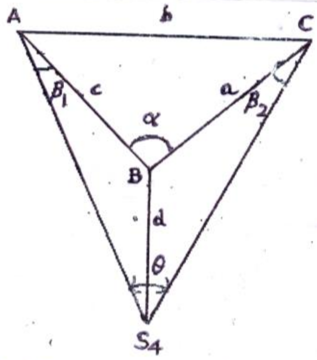


Fig. 4 – S_4 below B

$\alpha = \theta + \beta_1 + \beta_2$

(ii). From an eccentric station S, 12.25 m to the west of the main station B, the following angles were measured.

Angle of BSC = 76° 25' 32"

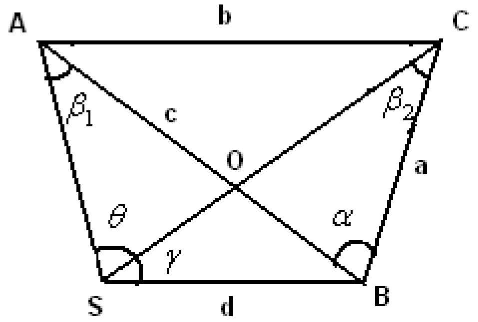
Angle of CSA = 54° 32' 20"

The stations S and C are to the oppose sides of the line AB. Calculate the correct angle. ABC if the length AB and BC are 5286.5 m and 4932.2 m respectively.

Solution:

BS = d = 12.25 m; AB = c = 5286.5 m; BC = a = 4932.2 m; $\theta = 54^{o}\ 32'\ 20''$;

$\gamma = 76^{o}\ 25'\ 32''$



Correct angle, $\alpha = \theta + \beta_1 - \beta_2$

$$\beta_1 = \frac{d\ \sin(\theta + \gamma)}{c} \text{ X } 206265$$

$$= \frac{12.25\ X\ \sin\ (54^{o}\ 32'\ 20'' + 76^{o}\ 25'\ 32'')}{5286.5} \text{ X } 206265$$

β₁ = 360.92 sec = 6' 0.92"

$$\beta_2 = \frac{d\ \sin\ \gamma}{b} \text{ X } 206265$$

$$= \frac{12.25\ X\ \sin\ (76^{o}\ 25'\ 32'')}{4932.2} \text{ X } 206265$$

β₂ = 497.98 sec = 8' 17.98"

$$\alpha = \theta + \beta_1 - \beta_2$$

$$= 54^{o}\ 32'\ 20'' + 6'\ 0.92'' - 8'\ 17.98''$$

$\alpha = 54^{o}\ 30'\ 2.94''$

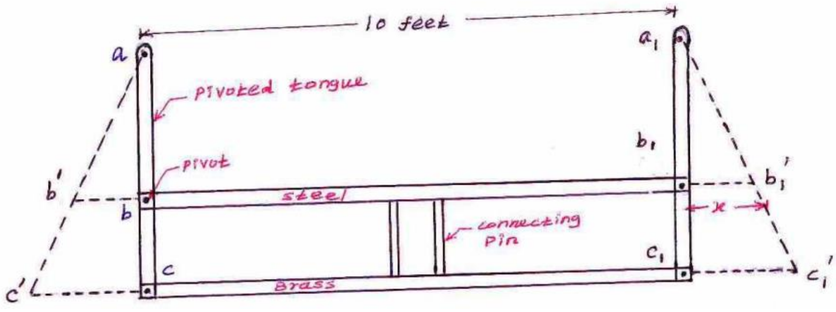
2. (i). What are the methods of measurement of base line and explain any one with neat sketch?

Baseline :

- The Base line is laid down with great accuracy of measurement & 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.

Methods used to measure baseline

- Rigid bar method
- Wheeler's method
- Jaderin's method
- Hunter's short base method
- Tacheometric method

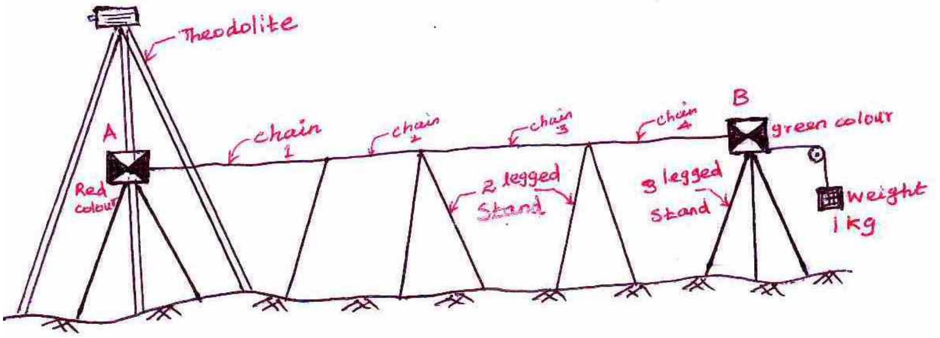


<u>Rigid bar method</u>

- It is designed by Major general Colby

- All the ten bases of GTS (Great trigonometrically Survey) of India were measured with the Colby Apparatus
- It consists of an iron and a brass bar, each 10 ft 1½ inch long, fixed together at middle by means of two steel pins
- A flat steel tongue ,about 6 inches long, is pivoted at each end of the bar
- Each of the tongue carries one microscopic platinum dot 'a' and 'a_1' making the distance a a_1 exactly 10 feet.
- To secure compensation ,the ratio ab/ac is made equal to the ratio of coefficients of linear expansion of iron and brass i.e.,3/5
- The tongue is free to pivot, the position of the dot remains constant under the change of temperature.
- Due to change of temperature, the length bb_1 say be x
- The length cc_1 will change to c' c_1' by 5/3 x
- The positions of the dots 'a' and 'a_1' remain unchanged.
- The bar is held in a box at the middle of its length.
- A spirit level is placed on the bar, and is observed through a window in the top of the box.
- For measuring the bases in India, five such bars were simultaneously used with a gap of 6 inches between the forward mark of one bar and the rear mark of the next bar by means of a framework.
- Framework was equipped with two microscopes with their cross wires 6 in apart.
- A small telescope, parallel to the microscopes is fixed at the middle of this bar for sighting reference marks on the ground.

Hunter's short base method:



- Dr. Hunter who was a Director of Survey of India, designed an equipment to measure the base line which was named as hunter's short base.
- It consists of 4 chains, each of 20.117 m (66ft) linked together.
- There are 5 stands, 3 intermediate two legged stands, 2 three legged stands at ends.
- A 1kg weight is suspended at the end of an arm, so that the chains remain straight during observations.
- The correct length of the individual chain is supplied by the manufacturer or is determined in the laboratory.
- The length of the joints between two chains at intermediate supports is measured directly with the help of graduated scale.
- To obtain correct length between the centres of the tangents used corrections such as temperature, sag, slope etc, are applied.
- To set the hunters short base, the stand at end A(marked on red colour) is centred on the ground mark and the target is fitted with a clip.
- The target 'A' is made truly vertical so that the notch on its tip side is centred on the ground mark.
- The end of the base is hooked with the plate A

(ii). A steel tape is 30 m long at a temperature of 15°C when lying horizontal on the ground. If c/s area is 0.08cm² and weight 18N and coefficient of expansion is 117 x10⁻⁷ per degree Celsius. The tape is stretched over 3 supports held at same level and at equal intervals. Calculate the actual length between and graduations at temperature = 25°C, pull 180kg, E = 2.1 x 10⁵ N / cm².

Given Data:

$\alpha = 117 \times 10^{-7}/^{o}C$,	L = 30m,	$T_m = 25^{o}C$,	$T_o = 15^{o}C$,
$A = 0.08cm^2$,	n = 3	P = 180 kg,	Po = 0 kg
w = 18N			(Note: 1Kg = 9.81N)

Correction for temperature(C_t)

$C_t = \alpha\,(Tm - To)\,L = \mathbf{0.00351m}$

Correction for pull or tension Cp

$Cp = (P - Po)L / AE = [(180 - 0) \times 9.81 \times 30] / 0.08 \times 2.1 \times 10^5$

$= \mathbf{3.153\ m}$

Sag Correction:

$C_{sag} = w^2\, l / (24 P^2 n^2) = (18^2 \times 30) / (24 \times 180^2 \times 3^2)$

$=$ **0.0014m** *Negative*

Total Correction = $C_t + Cp - C_{sag}$ = ***3.155m***

Actual length = ***30 + 3.155*** = ***33.155m***

April May 2016

1. (i). What is meant by triangulation and describe classification of triangluation?

Classification of Triangulation System

- Based on the extent and purpose of the survey, and consequently on the degree of accuracy desired.
- Triangulation surveys are classified as
  - *First-order (or) Primary triangulation,*
  - *Second-order (or) Secondary triangulation,*
  - *Third-order (or) Tertiary triangulation.*

First-order triangulation is used to determine the shape and size of the earth or to cover a vast area like a whole country with control points to which a second-order triangulation system can be connected.

Second-order triangulation system consists of a network within a first-order triangulation. It is used to cover areas of the order of a region, small country.

Third-order triangulation is a framework fixed within and connected to a second-order triangulation system. It serves the purpose of furnishing the immediate control for detailed engineering and location surveys.

Sl. No	Characteristics	First-order triangulation	Second-order triangulation	Third-order triangulation
1	Length of base line	8 to 12 Km	2 to 5 Km	100 to 500 m
2	Length of sides	16 to 150 Km	10 to 25 Km	2 to 10 Km
3	Average triangular error (after correction for spherical excess)	Less than 1"	3"	12"
4	Maximum station closure	Not more than 3"	8"	15"
5	Actual error of base	1 in 50,000	1 in 25,000	1 in 10,000
6	Probable error of base	1 in 10,00,000	1 in 5,00,000	1 in 2,50,000
7	Discrepancy between two measures ('K' is distance in	5√K mm	10√K mm	25√K mm
8	Probable error of the computed distance	1 in 50,000 to 1 in 2,50,000	1 in 20,000 to 1 in 50,000	1 in 5,000 to 1 in 20,000
9	Probable error astronomical azimuth	0.5"	5"	10"

- These are the general specifications for the triangulation system.

***(ii).* 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 105 N/mm2. Coefficient of linear expansion of tape per degree F = 6.2 x 10-6.**

Solution:

$L = 20$ m; $T_o = 55^oC$; $T_m = 80^oC$; $P_o = 98.1$ N;

$P = 156.96$ N; $\alpha = 6.2 \times 10^{-6}$; Weight of steel = 77107 N;

Weight of tape = 7.85 N; $E = 2.05 \times 10^5$ N /mm^2

i) Correction for Temperature:

$\boldsymbol{Ct} = \boldsymbol{\alpha (T_m - T_o) L} = 6.2 \times 10\text{-}6\ (80 – 55) \times 20$

Ct = 0.0031 m

ii) Correction for Pull:

$\boldsymbol{C_p} = \boldsymbol{(P\text{-}Po)L / AE}$

Here,

weight of tape = (Area x 1 x weight of steel) x length

7.85 = (A x 1 x 77107) x 20

A = (7.85) / (77107 x 20) = 5.1×10^{-6} m^2 = 5.1mm^2

CP = 0.00112 m

iii) Sag Correction:

$\boldsymbol{C_{sag}} = \boldsymbol{w^2\, l /(24P^2 n^2)} = (7.85^2 \times 20) / (24 \times 156.96^2 \times 1^2)$

$\mathbf{C_{sag}}$ = **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

2. *(i). From an eccentric station S, 12.25 m to the west of the main station B, the following angles were measured.*

Angle of BSC = 76° 25' 32"

Angle of CSA = 54° 32' 20"

The stations S and C are to the oppose sides of the line AB. Calculate the correct angle. ABC if the length AB and BC are 5286.5 m and 4932.2 m respectively.

Same as ***November December 2016; Question No:1(ii)***

(ii). Find the difference of 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	***=***	***1º 28'34"***
Height of signal of B	***=***	***3.886m***
Height of instrument at A	***=***	***1.497m***
Coefficient of refraction	***=***	***0.07***
Rsin1"	***=***	***30.876m***
R.L. of A	***=***	***1265.85m***

Given Data:

d = 5625.389 m   α = 1º*28'34"*   S = 3.886 m

h = 1.497 m   m = 0.07   Rsin1" = 30.876m

R.L. of A = 1265.85m

Axis signal correction:

$$\delta = \frac{s-h}{d\sin 1''}$$

= (3.866 – 1.497) / 5625.389 x Sin0º0'1")

= (+)^{ive}

Correction for curvature:

$$\theta = \frac{d}{R\sin 1''}$$

$$\text{Curvature correction, } \frac{\theta}{2}$$

= (-)^{ive}

Correction for refraction:

$$\text{Refraction correction, } r = m\theta$$

= (+)^{ive}

To fine H:

$$\beta_1 = \beta + \delta$$

$$H = \frac{d\sin\left(\beta_1 + m\theta - \frac{\theta}{2}\right)}{\cos\left(\beta_1 + m\theta - \theta\right)}$$

R.L. of B = R.L. of A + H

UNIT IV 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

April / May 2018

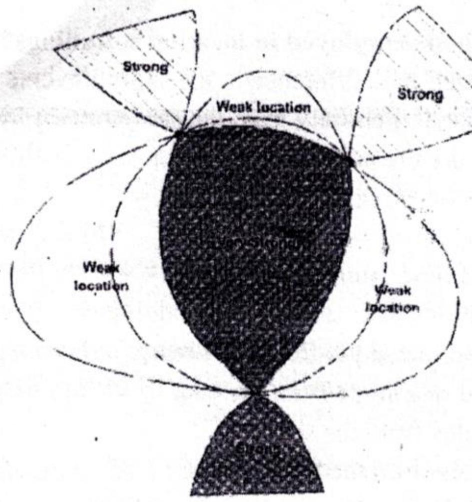
1. ***Explain the application of three point problem in hydrographic surveying and strength of fix in hydrographic surveying.***

Application of three point problem in hydrographic surveying: (7)

- The method of plotting the soundings depends upon the method used for locating the soundings.
- If the soundings have been taken along the range lines, the position of shore signals can be plotted and the sounding located on these in the plan.
- In the fixes by angular methods also, the plotting is quite simple, and requires the simple knowledge of geometry.
- However, if the sounding has been located by two angles from the boat by observations to three known points on the shore, the plotting can be done either by the mechanical, graphical or the analytical solution of the three-point problem.

Strength of fix in hydrographic surveying: (6)

- The accuracy with which a hydrographic 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 the station.
- Thus, the choice of plotted objects and location of table should be made to get a strong fix.



Qualitative presentation of strength of fix

2. What are the methods of employed in locating soundings?

The soundings are located with reference to the shore traverse by observations made (i) entirely from the boat, (ii) entirely from the shore or (iii) from both.

The following are the methods of location :

(a) By conning the survey vessel

1. By cross rope
2. By range and time intervals

(b) By observations with sextant or theodolite

3. By range and one angle from the shore
4. By range and one angle from the boat
5. By two angles from the shore
6. By two angles from the boat
7. By one angle from shore and one from boat
8. By intersecting ranges
9. By tacheometry.

3. Briefly explain Latitude by Prime Vertical transit and the effect of errors.

Latitude by Prime Vertical transit:

When the star is on the prime vertical of the observer, the astronomical triangle is evidently right-angled at Z. if the declination (δ) and the latitude (θ) of the place of observation are known. The altitude (α) and the hour angle (H) can be calculated by Napier's rule. The five parts taken in order are: the two sides (90^o - θ) and (90^o - α) and the complements of the rest of the three parts, i.e.,

$$(90^o - M),\ 90^o - (90^o - \delta) = \delta \text{ and } (90^o - H).$$

Now sine of middle part = product of cosine of opposite parts.

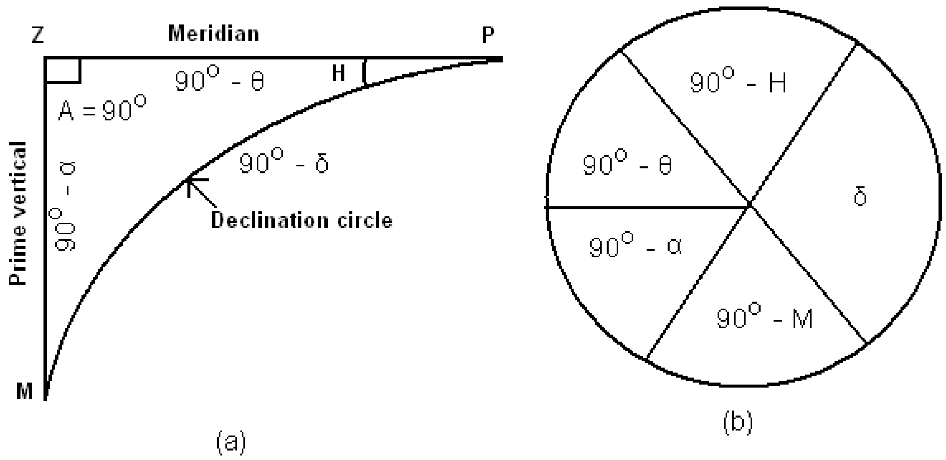


Fig. Star at Prime Vertical

$$\sin\delta = \cos(90^o - \theta)\cos(90^o - \alpha) = \sin\theta \sin\alpha$$

$$\sin\alpha = \frac{\sin\delta}{\sin\theta}\ \sin\delta\ \mathrm{cosec}\,\theta$$

And

$$\sin(90^o - H) = \tan(90^o - \theta)\tan\delta \ \text{(or)}$$

$$\cos H = \frac{\tan\delta}{\tan\theta} = \tan\delta\ \cot\theta$$

Effect of errors:

The error in the setting out of the direction of the prime vertical has very little effect in the latitude of the place for ordinary engineering purposes. If the eastern transit occurs earlier due to the wrong direction of the prime vertical, the western transit will also take place correspondingly earlier, though not exactly by the same amount. In a latitude of 30°, even if the prime vertical is set out by 1° out of its true position, the resulting error in latitude determination will be less then 1" for observations on a star having declination = 20°.

3. Write in detail about the methods of locating soundings.

The methods of locating soundings:

i) By cross rope.

ii) By range and time intervals.

iii) By range and one angle from the shore.

iv) By range and one angle from the boat.

v) By two angles from the shore.

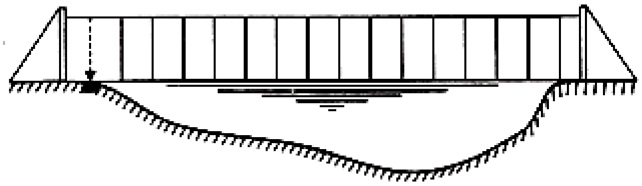
vi) By two angles from the boat.

vii) By one angle from shore and one from boat.

viii) By intersecting ranges.

ix) By tacheometry.

i) Location by Cross-Rope:



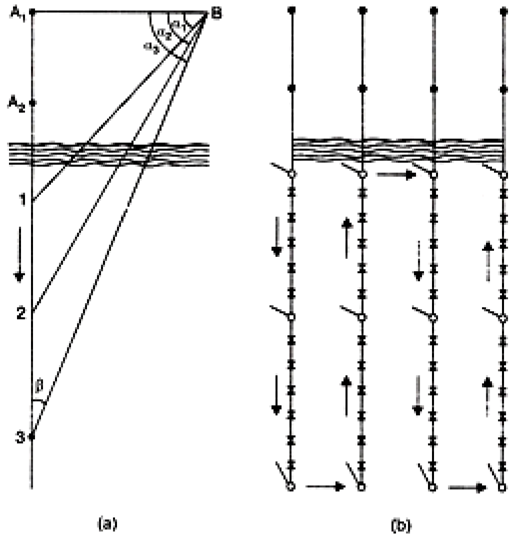
This is the most accurate method of locating the soundings and may be used for rivers, narrow lakes and harbours. It is also used to determine the quantity of materials removed by dredging the soundings being taken before and after the dredging work is done. A single wire or rope is stretched across the channel etc. and is marked by metal tags at appropriate known distance along the wire from a reference point or zero station on shore. The soundings are then taken by a weighted pole. The position of the pole during a sounding is given by the graduated rope or line.

ii) By range and time intervals:

In this method, the boat is kept in range with the two signals on the shore and is rowed along it at constant speed. Soundings are taken at different time intervals. Knowing the constant speed and the total time elapsed at the instant of sounding, the distance of the total point can be known along the range. The method is used when the width of channel is small and when great degree of accuracy is not required. However, the method is used in conjunction with other methods, in which case the first and the last soundings along a range are located by angles from the shore and the intermediate soundings are located by interpolation according to time intervals.

iii) By range and one angle from the shore:

In this method, the boat is ranged in line with the two shore signals and rowed along the ranges. The point where sounding is taken is fixed on the range by observation of the angle from the shore. As the boat proceeds along the shore, other soundings are also fixed by the observations of angles from the shore. Thus B is the instrument station, A1 A2 is the range along which the boat is rowed and α1, α2, α3 etc., are the angles measured at B from points 1, 2, 3 etc.



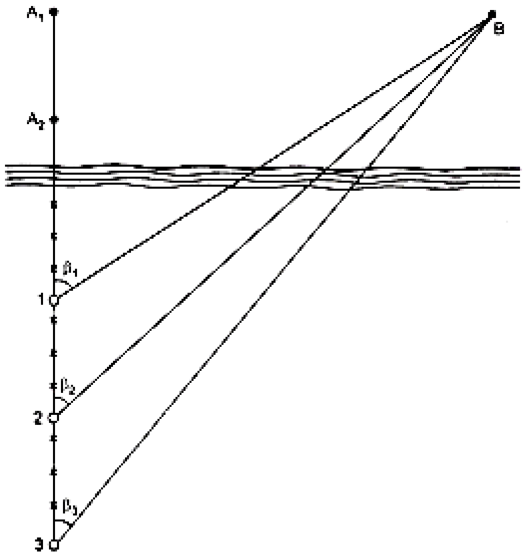
(a) (b)

To fix a point by observations from the shore, the instrument man at B orients his line of sight towards a shore signal or any other prominent point (known on the plan) when the reading is zero. He then directs the telescope towards the leadsman or the bow of the boat, and is kept continually pointing towards the boat as it moves. The surveyor on the boat holds a flag for a few seconds and on the fall of the flag, the sounding and the angle are observed simultaneously.

The angles are generally observed to the nearest 5 minutes. The time at which the flag falls is also recorded both by the instrument man as well as on the boat. In order to avoid acute intersections, the lines of soundings are previously drawn on the plan and suitable instrument stations are selected.

iv) By range and one angle from the boat:

The method is exactly similar to the previous one except that the angular fix is made by angular observation from the boat. The boat is kept in range with the two shore signals and is rowed along it. At the instant the sounding is taken, the angle, subtended at the point between the range and some prominent point B on the sore is measured with the help of sextant. The telescope is directed on the range signals, and the side object is brought into coincidence at the instant the sounding is taken. The accuracy and ease of plotting is the same as obtained in the previous method. Generally, the first and the last soundings, and some of the intermediate soundings are located by angular observations and the rest of the soundings are located by time intervals.

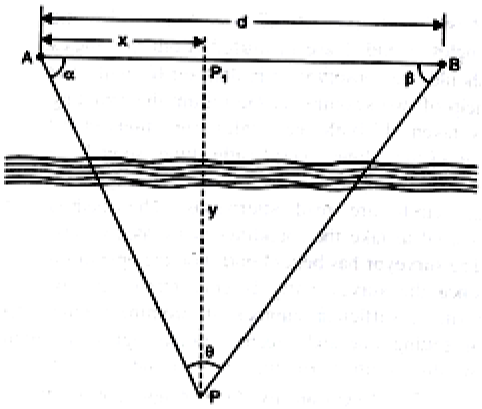


As compared to the previous methods, this method has the following **advantages**:

- Since all the observations are taken from the boat, the surveyor has better control over the operations.
- The mistakes in booking are reduced since the recorder books the readings directly as they are measured.
- On important fixes, check may be obtained by measuring a second angle towards some other signal on the shore.
- Obtain good intersections throughout; different shore objects may be used for reference to measure the angles.

v) By two angles from the shore:

In this method, a point is fixed independent of the range by angular observations from two points on the shore. The method is generally used to locate some isolated points. If this method is used on an extensive survey, the boat should be run on a series of approximate ranges. Two instruments and two instrument men are required. The position of instrument is selected in such a way that a strong fix is obtained. New instrument stations should be chosen when the intersection angle (θ) falls below 30°.



Thus A and B are the two instrument stations. The distance d between them is very accurately measured. The instrument stations A and B are precisely connected to the ground traverse or triangulation, and their positions on plan are known. With both the plates clamped to zero, the instrument man at A bisects B ; similarly with both the plates clamped to zero, the instrument man at B bisects A. Both the instrument men then direct the line of sight of the telescope towards the leadsman and continuously follow it as the boat moves.

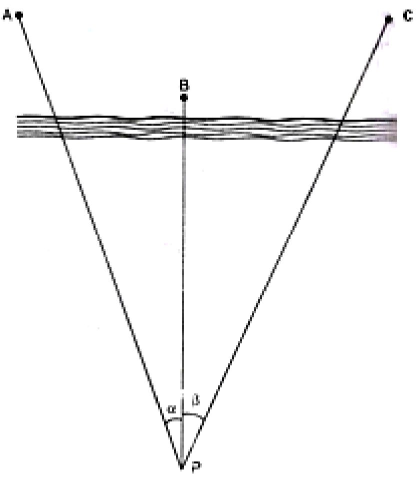
The surveyor on the boat holds a flag for a few seconds, and on the fall of the flag the sounding and the angles are observed simultaneously. The co-ordinates of the position P of the sounding may be computed from the relations:

$$x = \frac{d \tan\beta}{\tan\alpha + \tan\beta}; \quad y = \frac{d \tan\alpha \tan\beta}{\tan\alpha + \tan\beta}$$

Advantages:

- The preliminary work of setting out and erecting range signals is eliminated.
- It is useful when there are strong currents due to which it is difficult to row the boat along the range line.

vi) By two angles from the boat:



In this method, the position of the boat can be located by the solution of the three point problem by observing the two angles subtended at the boat by three suitable shore objects of known position. The three-shore points should be well-defined and clearly visible.

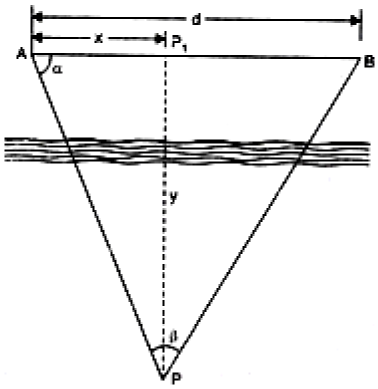
Prominent natural objects such as church spire, lighthouse, flagstaff, buoys etc., are selected for this purpose. If such points are not available, range poles or shore signals may be taken.

Thus A, B and C are the shore objects and P is the position of the boat from which the angles α and β are measured. Both the angles should be observed simultaneously with the help of two sextants; at the instant the sounding is taken. If both the angles are observed by surveyor alone, very little time should be lost in taking the observation. The angles on the circle are read afterwards. The method is used to take the soundings at isolated points. The surveyor has better control on the operations since the survey party is concentrated in one boat.

Advantages:

- Preliminary work setting out and erecting range signals is eliminated.
- The position of the boat is located by the solution of the three point problem either analytically or graphically.

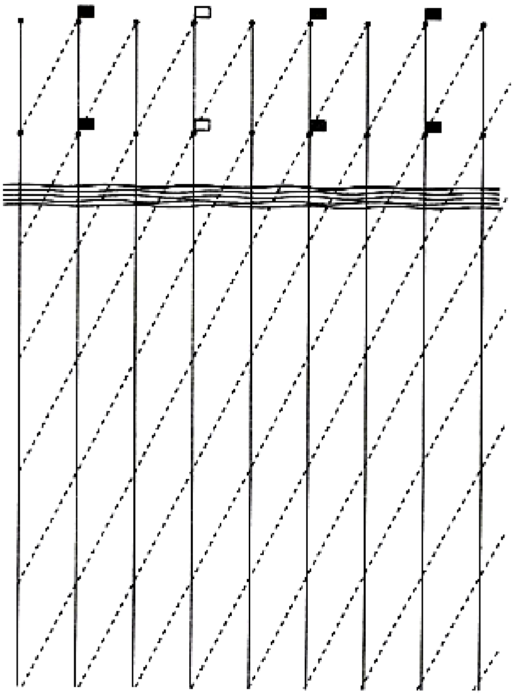
vii) By one angle from shore and one from boat:



This method is the combination of methods 5 and 6 described above and is used to locate the isolated points where soundings are taken. Two points A and B are chosen on the shore, one of the points (say A) is the instrument station where a theodolite is set up, and the other (say B) is a shore signal or any other prominent object. At the instant the sounding is taken at P, the angle α at A is measured with the help of a sextant. Knowing the distance d between the two points A and B by ground survey, the position of P can be located by calculating the two co-ordinates x and y.

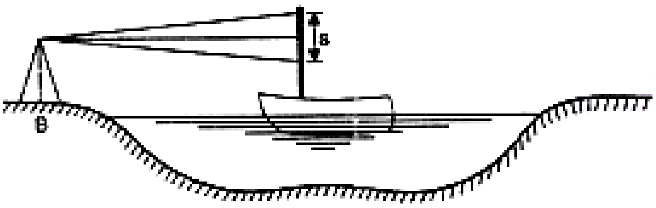
viii) By intersecting ranges:

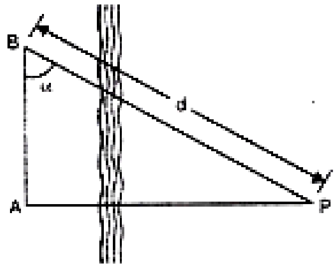
This method is used when it is required to determine by periodical sounding at the same points, the rate at which silting or scouring is taking place. This is very essential on the harbors and reservoirs. The position of sounding is located by the intersection of two ranges, thus completely avoiding the angular observations. Suitable signals are erected at the shore. The boat is rowed along a range perpendicular to the shore and soundings are taken at the points in which inclined ranges intersect the range, as illustrated in figure. However, in order to avoid the confusion, a definite system of flagging the range poles is necessary. The position of the range poles is determined very accurately by ground survey.



ix) By tacheometry:

The method is very much useful in smooth waters. The position of the boat is located by tacheometric observations from the shore on a staff kept vertically on the boat. Observing the staff intercept s at the instant the sounding is taken, the horizontal distance between the instrument stations and the boat is calculated.





The direction of the boat (P) is established by observing the angle (α) at the instrument station B with reference to any prominent object A The transit station should be near the water level so that there will be no need to read vertical angles. The method is unsuitable when soundings are taken far from shore.

4. What is a three point problem in hydrographic surveying? What are the various solutions for the problem? Explain in detail.

Given the three shore signals A, B and C, and the angles α and β subtended by AP, BP and CP at the boat P, it is required to plot the position of P.

1. Mechanical Solution

(i) By Tracing Paper

Protract angles α and β between three radiating lines from any point on a piece of tracing paper. Plot the positions of signals A, B, C on the plan. Applying the tracing paper to the plan, move it about until all the three rays simultaneously pass through A, B and C. The apex of the angles is then the position of P which can be pricked through.

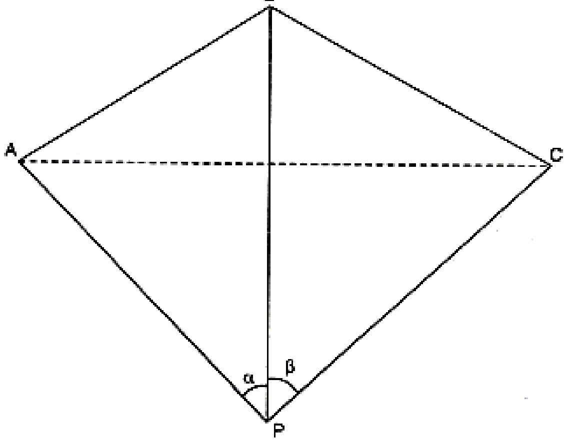


FIG. THE THREE-POINT PROBLEM

(ii) By Station Pointer:

The station pointer is a three-armed protractor and consists of a graduated circle with fixed arm and two movable arms to the either side of the fixed arm. All the three arms have beveled or fiducial edges. The fiducial edge of the central fixed arm corresponds to the zero of the circle. The fiducial edges of the two moving arms can be set to any desired reading and can be clamped in position. They are also provided with verniers and slow motion screws to set the angle very precisely. To plot position of P, the movable arms are clamped to read the angles α and β very precisely. The station pointer is then moved on the plan till the three fiducial edges simultaneously touch A, B and C. The centre of the pointer then represents the position of P which can be recorded by a prick mark.

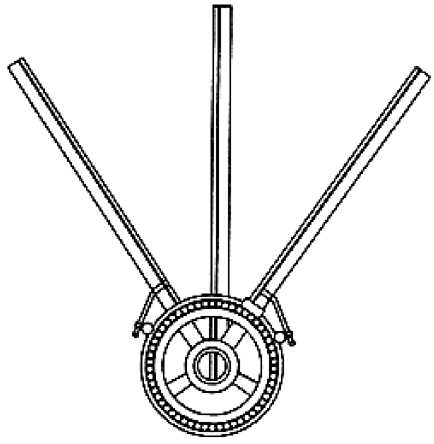
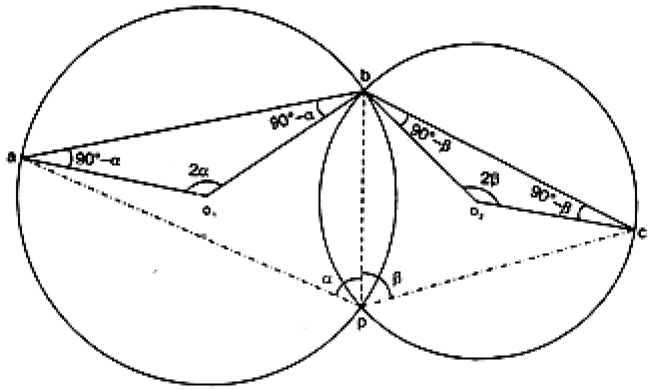


FIG. STATION POINTER

2. Graphical Solutions

(a) First Method:

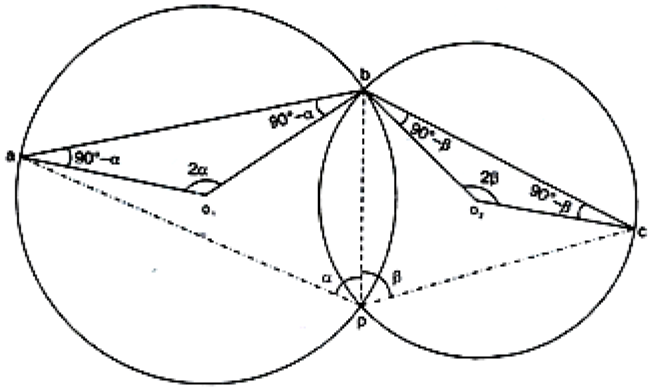
Let a, b and c be the plotted positions of the shore signals A, B and C respectively and let α and β be the angles subtended at the boat. The point p of the boat position p can be obtained as under:



- Join a and c.
- At a, draw ad making an angle β with ac. At c, draw cd making an angle α with ca. Let both these lines meet at d.
- Draw a circle passing through the points a, d and c.
- Join d and b, and prolong it to meet the circle at the point p which is the required position of the boat.

Proof: From the properties of a circle,
Angle apd = acd = α and cpd = cad = β
which is the required condition for the solution.

(b) Second Method:

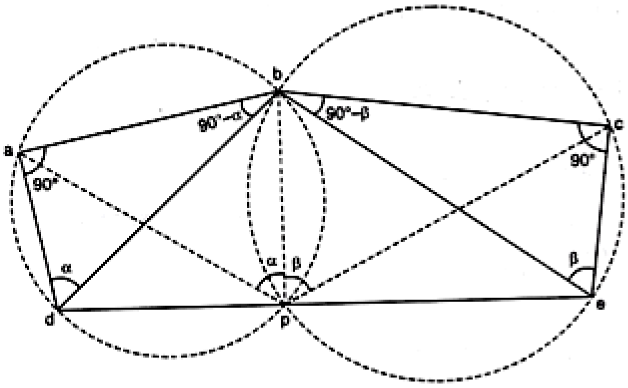


- Join ab and bc.
- From a and b, draw lines ao1 and bo1 each making an angle (90° - α) with ab on the side towards p. Let them intersect at 01.
- Similarly, from b and c, draw lines each making an angle (90° - β) with ab on the side towards p. Let them intersect.
- With – as the centre, draw a circle to pass through a and b. Similarly, with – as the centre draw a circle to pass through b and c. Let both the circles intersect each other at a point p. p is then the required position of the boat.
  Proof: ao1b = 180° - 2 (90° - α) = 2α

	Angle	apb = ½ ao1b = α
Similarly,	Angle	bo2c = 180° - 2 (90° - β) = 2β
and	Angle	bpc = ½ bo2c = β.

The above method is sometimes known as the method of two intersecting circles.

(c) Third Method:



- Join ab and bc.
- At a and c, erect perpendiculars ad and ce.
- At b, draw a line bd subtending angle (90° - α) with ba, to meet the perpendicular through a in d.
- Similarly, draw a line be subtending an angle (90° - β) with bc, to meet the perpendicular through c in e.
- Join d and e.
- Drop a perpendicular on de from b. The foot of the perpendicular (i.e. p) is then the required position of the boat.

5. Explain briefly the different methods of prediction of tides. (AUC May/June 2009)

i) Age of tide

ii) Lunitidal interval

iii) Mean establishment

iv) Vulgar establishment

i) Age of tide:

This condition is fulfilled only in southern ocean extending southwards from about 40O S latitude. It is the only portion of ocean where equilibrium figure may be developed. Primary tide waves are generated and secondary waves are propagated into pacific, Atlantic and Indian oceans. The velocity of wave travel may exceed 1000 km per hour, though it is less in shallow water. The amplitude of vertical range from crest to trough is not more than 60 to 90 cm. due to direction of propagation of tide wave, high or low water occurs at different times at various places on the same meridian. "The time which elapse between the generation of spring tide and its arrival at the place is called Age of tide".

ii) Lunitidal interval:

"It is the time interval that elapses between the moon's transits and occurrence of next high water". The value is found to vary because of existence of priming and lagging. The values can be observed and plotted for a fortnight against the times of moon's transits, a curve is obtained. A curve has approximately same for each fortnight and used for rough prediction of time of tide. The time of transit of moon at Greenwich is given in nautical almanac. The time of transit can be derived by adding 2 m for every hour of west longitude and subtracting 2 m for every hour of east longitude.

iii) Mean establishment:

The average value of Lunitidal at a place is known as mean establishment as shown by dotted line. If the value is known and Lunitidal interval and the time of high water can be estimated. The procedure of determination are

- Find from charts, the age of tide and mean establishment for the place.
- Knowing the hour of moon's transit, determine the time of moon's transit on the day of generation of tide.

Day of generation = day in question – age of tide

- Corresponding to time of transit of moon on the day of generation of tide, find out the amount of priming or lagging correction.
- Add the priming or lagging correction to mean establishment to get Lunitidal interval for day in question.
- Add the Lunitidal interval to the time of moon's transit on the day in question to get approximate time of high water.

Hour of moon's transit	0	1	2	3	4	5	6	7	8	9	10	11	12
Correction in minutes	0	-16	-31	-41	-44	-31	0	31	44	41	31	16	0

iv) Vulgar establishment:

"It is defined as the value of Lunitidal interval on the day of full moon or change of moon". Its value is always more than establishment since lagging correction in second or fourth quadrant is positive. The difference between vulgar establishment and mean establishment depends upon age of tide. The value of vulgar establishment is approximately equal to clock time at which high water occurs on day of full moon or change of moon.

Mean establishment = vulgar establishment – lagging correction

Height of tide:

The approximate height of tide of known rise at any time between high and low water can be expressed

$$H = h + \frac{1}{2} r \cos\theta$$

H = required height of tide above datum

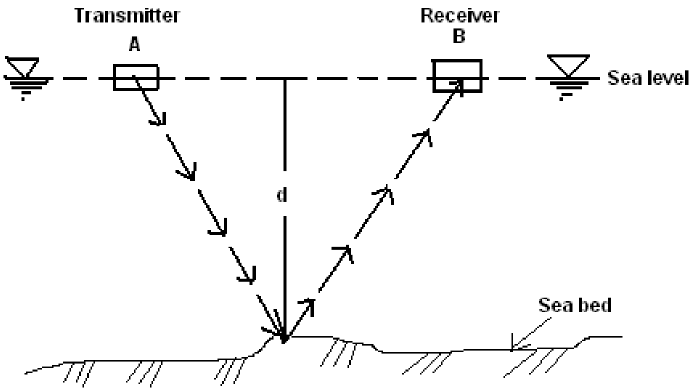
h = height of mean tide level above datum

r = range of tide

$$\theta = \frac{\text{interval from high water}}{\text{interval between high and low water}} \times 180^\circ$$

6. Explain the procedure to use fathometer in ocean sounding.

A Fathometer is used in ocean sounding where the depth of water is too much, and to make a continuous and accurate record of the depth of water below the boat or ship at which it is installed. It is an *echo-sounding* instrument in which water depths are obtained be determining the time required for the sound waves to travel from a point near the surface of the water to the bottom and back. It is adjusted to read depth on accordance with the velocity of sound in the type of water in which it is being used. A fathometer may indicate the depth visually or indicate graphically on a roll which continuously goes on revolving and provide a virtual profile of the lake or sea.



7. Explain the different types of tides in detail.

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.

Types of tides:

i) Lunar tides

ii) Solar tides

iii) Spring and neap tide (combined effect)

iv) Other effects

i) Lunar tides:

The figure shows the earth and the moon, with their centres of masses *O*1 and *O*2 respectively. Since moon is very near to the earth, it is the major tide producing force. To start with, we will ignore the daily rotation of the earth on its axis. Both earth and moon attract each other, and the force of attraction would act along *O*1 *O*2. Let *O* be the common centre of gravity of earth and moon. The earth and moon revolve monthly about *O*, and due to this revolution their separate positions are maintained. The distribution of force is not uniform, but it is more for the points facing the moon and less for remote points. Due to the revolution of earth about the common centre of gravity *O*, centrifugal force of uniform intensity is exerted on all the particles of the earth. The direction of this centrifugal force is parallel to *O*1*O*2 and acts outward. Thus, the total force of attraction due to moon is counter-balanced by the total centrifugal force, and the earth maintains its position relative to the moon. However, since the fore of attraction is not uniform, the resultant force will very all along. The resultant forces are the tide producing forces. Assuming that water has no inertia and viscosity, the ocean enveloping the earth's surface will adjust itself to the unbalanced resultant forces, giving rise to the equilibrium. Thus, there are two lunar tides at A and B, and two low water positions at C and D. The tide at A is called the superior lunar tide or tide of moon's upper transit, while tide at B is called inferior or antilunar tide.

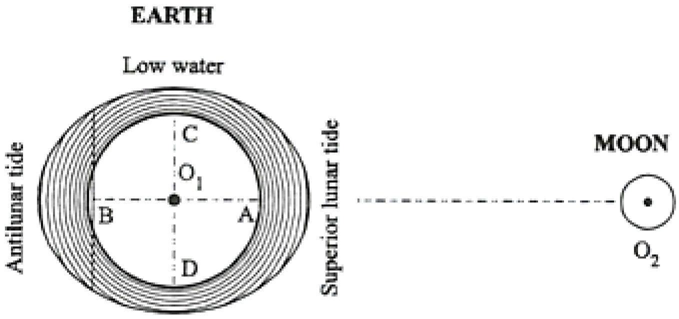


Fig. 6.1 Equilibrium figure

Now let us consider the earth's rotation on its axis. Assuming the moon to remain stationary, the major axis of lunar tidal equilibrium figure would maintain a constant position. Due to rotation of earth about its axis from west to east, once in 24 hours, point A would occupy successive position C, B and D at intervals of 6 h. Thus, point A would experience regular variation in the level of water. It will experience high water (tide) at intervals of 12 h and low water midway between. This interval of 6 h variation is true only if moon is assumed stationary. However, in a lunation of 29.53 days the moon makes one revolution relative to sun from the new moon to new moon. This revolution is in the same direction as the diurnal rotation of earth, and hence there are 29.53 transits of moon across a meridian in 29.53 mean solar days. This is on the assumption that the moon does this revolution in a plane passing through the equator. Thus, the interval between successive transits of moon or any meridian will be 24 h, 50.5 m. Thus, the average interval between successive high waters would be about 12 h 25 m. The interval of 24 h 50.5 m between two successive transits of moon over a meridian is called the tidal day.

ii) 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

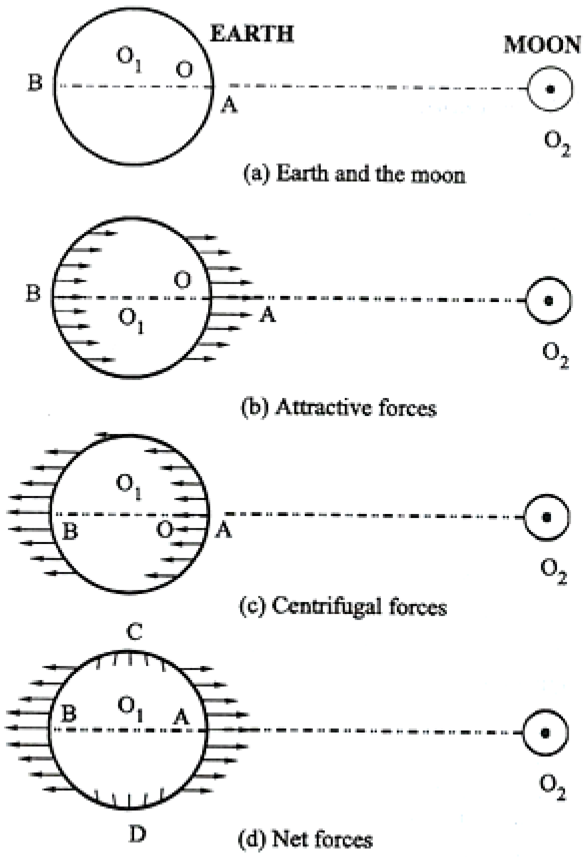


Fig. 6.2 Forces acting between earth and moon

iii) 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, especially at the new moon when both the sun and moon have the same celestial longitude, they cross a meridian at the same instant.

Assuming that both the sun and moon lie in the same horizontal plane passing through the equator, the effects of both the tides are added, giving rise to maximum or spring tide of new moon. The term ‘spring’ does not refer to the season, but to the springing or waxing of the moon. After the new moon, the moon falls behind the sun and crosses each meridian 50 minutes later each day. In after 7 ½ days, the difference between longitude of the moon and that of sun becomes 90°, and the moon is in quadrature. The crest of moon tide coincides with the trough of the solar tide, giving rise to the neap tide of the first quarter. During the neap tide, the high water level is below the average while the low water level is above the average. After about 15 days of the start of lunation, when full moon occurs, the difference between moon’s longitude and of sun’s longitude is 180°, and the moon is in opposition. However, the crests of both the tides coincide, giving rise to spring tide of full moon. In about 22 days after the start of lunation, the difference in longitudes of the moon and the sun becomes 270° and neap tide of third quarter is formed. Finally, when the moon reaches to its new moon position, after about 29 ½ days of the previous new moon, both of them have the same celestial longitude and the spring tide of new moon is again formed making the beginning of another cycle of spring and neap tides.

iv) Other effects:

The length of the tidal day, assumed to be 24 hours and 50.5 minutes is not constant because of (i) varying relative positions of the sun and moon,

(ii) Relative attraction of the sun and moon,
(iii) Ellipticity of the orbit of the moon (assumed circular earlier) and earth,
(iv) Declination (or deviation from the plane of equator) of the sun and the moon,
(v) Effects of the land masses and
(vi) Deviation of the shape of the earth from the spheroid.

Due to these, the high water at a place may not occur exactly at the moon’s upper or lower transit. The effect of varying relative positions of the sun and moon gives rise to what are known as priming of tide and lagging of tide.

At the new moon position, the crest of the composite tide is under the moon and normal tide is formed. For the positions of the moon between new moon and first quarter, the high water at any place occurs before the moon’s transit, the interval between successive high water is less than the average of 12 hours 25 minutes and the tide is said to prime. For positions of moon between the first quarter and the full moon, the high water at any place occurs after the moon transits, the interval between successive high water is more than the average, and tide is said to lag. Similarly, between full moon and 3rd quarter position, the tide primes while between the 3rd quarter and full moon position, the tide lags. At first quarter, full moon and third quarter position of moon, normal tide occurs.

Due to the several assumptions made in the equilibrium theory, and due to several other factors affecting the magnitude and period of tides, close agreement between the results of the theory, and the actual field observations is not available. Due to obstruction of land masses, tide may be heaped up at some places. Due to inertia and viscosity of sea water, equilibrium figure is not achieved instantaneously. Hence prediction of the tides at a place must be based largely on observations.

8. At a point in latitude 55^{O} 46' 12'' N, the altitude of sun's centre was found to be 23^{O} 17' 32'' at 5^{h} 17^{m}, P.M. (G.M.T.) The horizontal angle at the R.M. and Sun's centre was 68^{O} 24' 30''. Find the azimuth of the sun.

Data:

i) Sun's declination of G.A.N. on day of observation = 17^{O} 46' 52'' N

ii) Variation of declination per hour = – 37''

iii) Refraction of altitude 23^{O} 20' 00'' = 0^{o} 2' 12''

iv) Parallax for altitude = 0^{o} 0' 8''

v) Equation of time (App. – Mean) = 6^{m} 0^{s}

Solution:

i) Calculation of declination:

G.M.T. of observation = 5h 17m P.M.
Add equation of time = 0h 6m 0s
G.A.T. of observation = 5h 23m 0s P.M.
Now declination at G.A.T. = 17^{O} 46' 52'' N
Apparent time interval,
G.A.N. = 5h 23m 0s
Variation in the declination in this time interval at the rate of 37" per hour = 3' 39" (decrease).
Declination at G.A.T. of observation = 17o 46' 52'' - 3' 39" = 17o 43' 13''

ii) Calculation of altitude:

Observed altitude of sun's centre	=	23^{O} 17' 32''
Subtract refraction correction	=	0^{O} 2' 12''
	=	**23^{O} 15' 20''**
Add parallax correction	=	0' 8"
Correct altitude	=	23^{O} 15' 28''

Now, co-latitude = c = 90^{O} - θ = 90^{O} - 55^{O} 46' 12" = 34^{O} 13' 48"

co-declination = p = 90^{O} - δ = 90^{O} - 17^{O} 43' 13" = 72^{O} 16' 47"

co-altitude = z = 90^{O} - α = 90^{O} - 23^{O} 15' 28" = 66^{O} 44' 32"

2s = 173^{O} 15' 7"

S = 86^{O} 37' 33.5"

S – c = 52^{O} 23' 45.5" ; S – p = 14^{O} 20' 46.5" ; S – z = 19^{O} 53' 1.5"

Azimuth of sun is given by,

$$\tan\frac{A}{2} = \sqrt{\frac{\sin\,(s-z)\ \sin\,(s-c)}{\sin s\ \sin\,(s-p)}} = \sqrt{\frac{\sin\,(19^{O}\,53'\,1.5")\ \sin\,(52^{O}\,23'\,45.5")}{\sin\,(86^{O}\,37'\,33.5")\ \sin\,(14^{O}\,20'\,46.5")}} = 1.0437$$

$$\frac{A}{2} = 46^{O}\ 13'\ 29.84"$$

A = 23^{O} 6' 44.92'

9. Determine the hour angle and declination of star from the following data:
Altitude of star = 22° 30'
Azimuth of the star = 145° E
Latitude of the observer = 49° N. (AUC Apr/May 2010)

Solution:

The azimuth of the star is 145° E, the star is in the eastern hemisphere.
In the astronomical triangle ZPM, we have

$$\text{Co-altitude, } ZM = 90^o - \alpha = 90^o - 22^o 30' = 67^o 30'$$

$$\text{Co-latitude, } ZP = 90^o - \theta = 90^o - 49^o = 41^o$$

$$A = 145^o$$

Using cosine formula,

$$\text{Cos } PM = \cos ZM \cos ZP + \sin ZM \sin ZP \cos A$$

$$= \cos(67^o 30')\cos(41^o) + \sin(67^o 30')\sin(41^o)\cos(145^o)$$

$$\text{Cos } PM = -0.2077$$

$$\mathbf{PM = 101^O\ 59'\ 15.36''}$$

Declination of star, $\delta = 90^O - PM = 90^O - 101^O\ 59'\ 15.36'' = 11^O\ 59'\ 15.36''$

$$\mathbf{\delta = -11^O\ 59'\ 15.36''\ S}$$

Using cosine formula,

$$\cos H = \frac{\cos ZM - \cos PZ \cos PM}{\sin PZ \sin PM}$$

$$\cos H = \frac{\cos(67^o 30') - \cos(41^o)\cos(101^o 59' 15.36'')}{\sin(41^o)\sin(101^o 59' 15.36'')}$$

$$\text{Cos } H = 0.8406$$

$$\cos(360^\circ - H) = 0.8406$$

$$(360^\circ - H) = 32^\circ 47' 47.28''$$

$$H = 360^\circ - 32^\circ 47' 47.28''$$

Hour angle, H = 327^O 12' 12.72"

10. What are parallax and refraction and how do they affect the measurements of vertical angles in astronomical work?

1. Correction for Parallax

When the sun or star is viewed from different points, change in the direction of the body is observed due to parallax. The parallax in altitude is called diurnal parallax.

This is due to the difference in direction of a heavenly body as seen from the centre of the earth and from the place of observation on the surface of the earth.

The stars are very far off and hence the parallax error is insignificant. However, in case of sun or moon necessary correction should be applied.

An example of sun's parallax is illustrated in Fig.7.16

Let O be the centre of the earth.

A be the plane of observation.

S be the position of the sun at the time of observation.

S' be the position of the sun at horizon.

OC be the true horizon.

AB be the sensible horizon.

$\angle SAB = \alpha'$ be the observed altitude.

$\angle SOC = \alpha$ be the true altitude, corrected for parallax.

$\angle ASB = p_a$ be the parallax correction.

$\angle ASO = p_h$ be the sun's horizontal parallax.

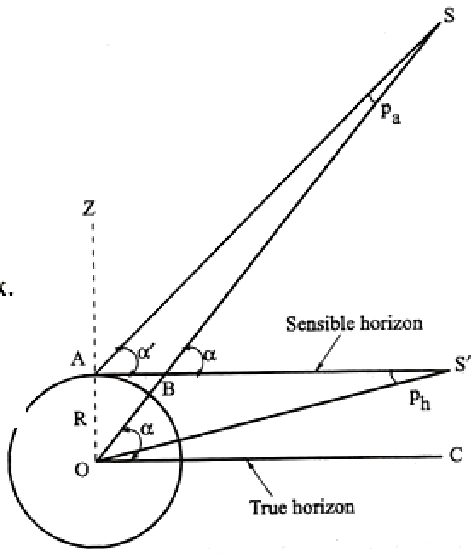


Fig. 7.16 Sun's parallax

When the sun is on the horizon, its apparent altitude is zero. For this condition the angle p_h, subtended at the centre of the sun is known as sun's horizontal parallax.

Thus, $\sin p_h = \dfrac{R}{OS'}$

Sun's horizontal parallax varies inversely with its distance from the centre of the earth. It varies from 8.95″ in the early part of January to 8.66″ during early in July. This variation is provided in the Nautical Almanac for every tenth day of the year.

$$\text{True altitude} = \alpha = \angle SOC = \angle SBS'$$
$$= \angle SAB + \angle ASB$$
$$= \alpha' + p_a \qquad (7.32)$$

Hence parallax correction $= (\alpha - \alpha') = p_a$

From ΔAOS

$$\sin \angle SO = \sin \angle OAS \frac{OA}{OS}$$

$$\text{or } \sin p_a = \sin (90° + \alpha') \frac{OA}{OS} = \cos \alpha' \frac{OA}{OS}$$

$$\text{But } \frac{OA}{OS} = \frac{OA}{OS'} = \sin p_h$$

$$\therefore \sin p_a = \sin p_h \cos \alpha' \qquad ...(7.33)$$

As p_a and p_h are very small, then

$$p_a = p_h \cos \alpha' \qquad ...(7.34)$$

That is,

$$\left.\begin{array}{l}\text{Correction for}\\ \text{parallax}\end{array}\right\} = \begin{array}{l}\text{(horizontal parallax)} \times\\ \text{cos (apparent altitude)}\end{array} \qquad (7.35)$$

$$= +\,8.8'' \cos \alpha' \qquad (7.36)$$

This correction for parallax is always additive.

The correction is maximum when the sun is at horizon.

2. Correction for Refraction

The layers of atmospheric air surrounding the earth becomes thinner as its distance from the surface increases. Because of variations of atmospheric density, the ray of light emanating from the celestial body passes through the atmosphere of the earth, the rays are bent (Fig.7.17) downwards. Because of this, body appears to be nearer to the zenith than it

The deviation angle of the ray from its direction on entering the earth's atmosphere to its direction at the surface of the earth is referred to as the refraction angle of correction.

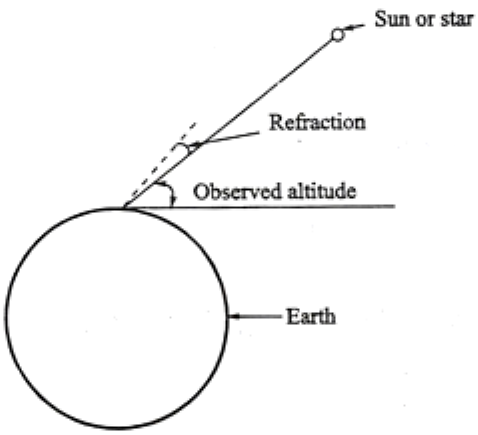


Fig. 7.17 Refraction

1. ***The following observations of the sun were taken for azimuth of a line in connection with a survey.***

   Mean time = 16h 30m
   Mean horizontal angle between the sun and the referring object = 18°20'30"
   Mean corrected altitude = 33°35'10"
   Declination of the sun from N.A. = ±22°05'36"
   Latitude of place = 52°30'20"
   Determine azimuth of line.

Solution:

Considering astronomical triangle, the hour angle ZPM = H,

Zenith distance, ZM = z = $90^o - \alpha$ = $90^o - 33^o\,35'\,10''$ = $56^o\,24'\,50''$

Polar distance, PM = $90^o - \delta = 90^o - 22^o\,5'\,36'' = 67^o\,54'\,24''$

Co-latitude, ZP = $90^o - \theta = 90^o - 52^o\,30'\,20'' = 37^o\,29'\,40''$

Using cosine rule,

$$\text{Cos PM} = \cos \text{ZM} \cos \text{ZP} + \sin \text{ZM} \sin \text{ZP} \cos A$$

$$\text{Cos A} = \frac{\text{Cos PM} - \text{Cos ZP Cos ZM}}{\text{Sin ZP Sin ZM}} = -0.1238$$

Azimuth of sun, A = 97° 6' 41.27"

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.

April / May 2018

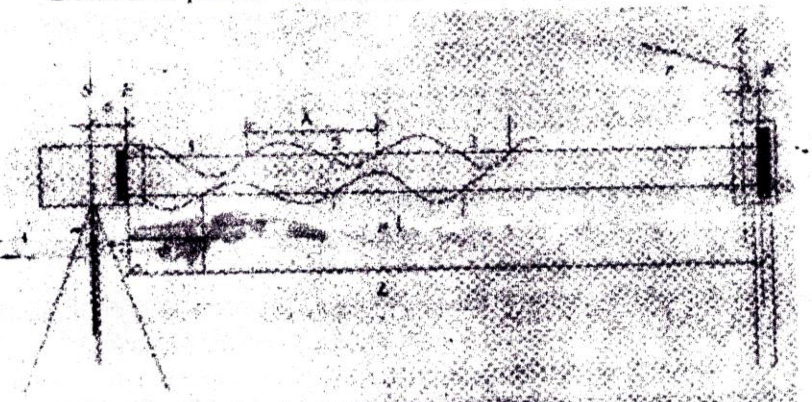
1. Explain briefly about the working principles of Total Station.

The principle of the measurement device in EDM, which is currently used in a total station and used along with electronic/optic theodolites, is that it calculates the distance by measuring the phase shift during the radiated electromagnetic wave (such as an infrared light or laser light or microwave) from the EDM's main unit, which returns by being reflected through the reflector, which is positioned at a measurement point.

This phase shift can be regarded as a part of the frequency that appears as the unit of time or length under a specific condition.

When the slope distance L and the slope angle ϕ are measured by EDM, if the elevation of point A is the reference point, we can find the elevation of point B by the following formula

Elevation of point B = Elevation of Point A + HI $\pm$ L sin ϕ - HR



Note: The components ... and ... are only auxiliary quantities.

Principle of electronic distance measurement

(7)

2. ***Explain the pulse method and phase difference method used in EDM's.***

Methods of measurements

i. ***pulse method***

ii. ***phase difference method***

Pulse method: (6)

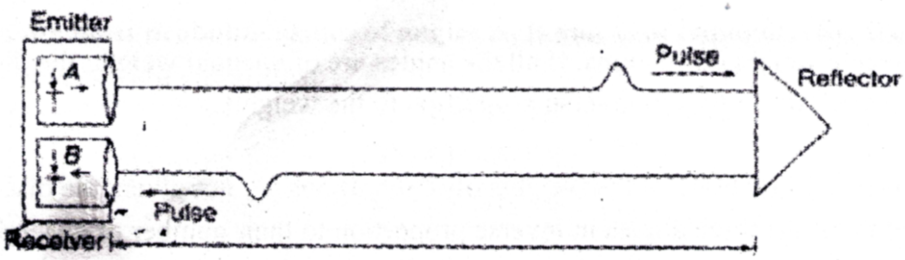
All the equipment used work on the principle that the distance D is equal to the product of velocity v and time t. This is the essence the pulse method. The speed of light in vacuum is well known. However, the measurements surveyors take are not in vacuum and thus corrections for atmospheric conditions must be applied. Also, because of great speed of light it is not possible to directly and precisely measure the time interval when the light beam travels from instrument to the reflector and back.

EDM instruments measure the phase difference between the transmitted and received signals. Light beams of different wavelengths are used to determine the distances. This forms the basis of difference method.

Figure shows a schematic diagram of pulse method. A short, intensive pulse of radiation is transmitted to a reflector target, which immediately transmits it back, along a parallel path, to the receiver. The measured distance is computed from the velocity of the signal multiplied by the time it took to complete its path, i.e.,

$$2D = c.\ \Delta t$$
$$D = c.\ \Delta t/2$$

c = velocity of the light
D = distance between instrument and target



Principle of pulse distance meter

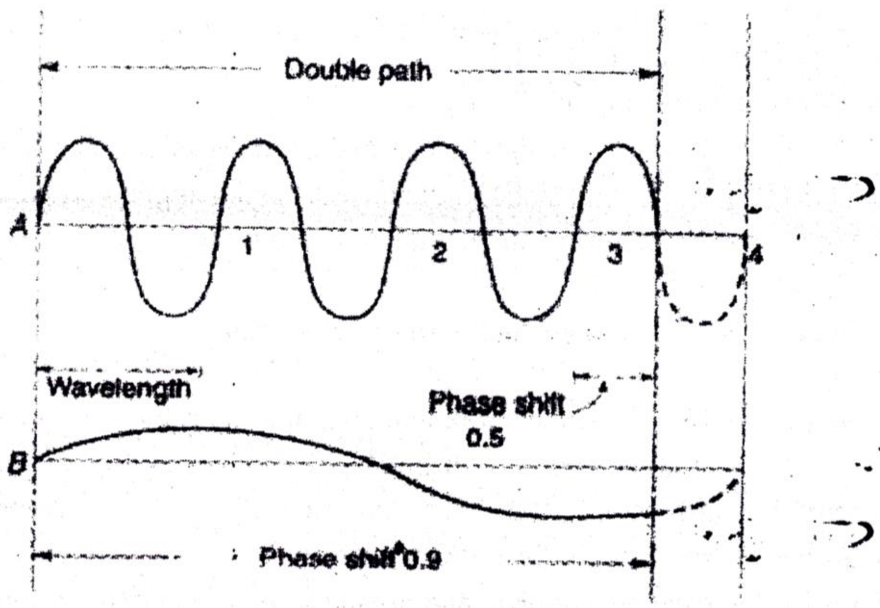
(7)

Phase difference method:

The majority of EDM instruments, whether infra-red, light or microwave, use this form of measurement.
The basic Eqn. used in this method is

$$2D = M\,\lambda + \delta\,\lambda$$

where, M = integer part of wavelength
$\delta\,\lambda$ = fraction part of wave length
λ = wave length



The electromagnetic waves are transmitted to a retroreflector (single or multiple prisms) which instantly returns them to the transmitting instrument. The instrument measures the phase shift. By comparison of the phase shift between the transmitted and the reflected signals, the time and thus the distance can be determined.

13.b.i. **Describe the steps involved in the initial setting of total station of a field.** (6)

The following are the steps for the initial setting of a total station:

1. Turn on the total station.
2. Release both horizontal and vertical locks.
3. Some total stations require rotating the telescope through 360° along the vertical and horizontal circles to initialize angles.
4. Adjust the telescope to best fit to the observer's eye. Using the inner ring of the eyepiece, make the image of the cross-hair sharp and clear.
5. Rotate the alidade until the Hz angle reading is equal to the azimuth to the back sight measured by the compass (for Sokkia models only). Push the HOLD key once. The Hz angle will not change until the next hold.
6. Aim at the very center of prism at the back sight. For the coarse aiming, rotate the alidade and the telescope by hand using optical sight. Adjust focus using the outer ring of the eyepiece. When the prism comes into the sight and close to the center, lock the horizontal and vertical drives. Then use dials to aim at the exact center of prism.
7. For Sokkia models, push HOLD button again. The horizontal reading will now change according to the rotation of the telescope in the horizontal direction. For Leica models, input the azimuth of the back sight manually in the measurement setup window.
8. If a station ID and back sight ID are required, use a 2-or 3-digit serial number like 101, 102 ... for each reference point. Use a 4-digit number for unknown points.
9. Input station parameters like hi (height of the instrument), E0, N0, and H0 (easting, northing and RL of the point where the instrument is set up). Use 1000, 1000, and 1000 for E0, N0 and H0 to avoid negative figures. If the coordinates are known, manually input the data.
10. Input the target height (hr).
11. Check the pointing of prism again
12. Using the distance calculation key, make the backsight measurement. From the LCD display of the total station note the horizontal angle, vertical angle, slope distance, easting, northing and height and record them in a field book.

13.b.ii. **How traversing is performed using total station:** (7)

When it is not possible to view the entire mapping area from the first station, we traverse to a new station and repeat radial shooting. Adjusting the coordinates and orientation of the second station, measured coordinates from multiple stations will be in a unique system. Most total stations have a programme for traverse.

1. Set up a prism on a tripod, tribrach, and prism carrier after centering on a mark on the ground. The back sight point may be used as a new station.
2. Measure the new station and record the E1, N1, H1 and horizontal angle, record the angular value in the memory and in a notebook. Turn off the total station.
3. Leaving the tribrach on the tripod, exchange the total station above the tribrach with the prism on the prism carrier.
4. The exchanged total station and prism should be levelled and centered. Carefully apply small adjustments for fine levelling and centering.
5. Turn on the total station at the new station and point at the prism.
6. Run a traverse programme.
7. Input the station coordinate (El, NI, HI) and the new height of the instrument (previous height of the prism)
8. Pointing the center of the prism, set Hz0 (horizontal angle zero) as Hz1 +180 or Hzl − -180 (Hzl > 180). Use the previous station as the new back sight.
9. Input the new hr (height of the reflector) and measure. The coordinate of the first station must be (E0, N0, H0). The error must be less than a few millimeters.
10. To define errors and evaluate accuracy, follow the standard procedures for surveyors.

14.a.Explain the pseudo range method and carrier phase measurement method.

In GPS there are two types of observables: the pseudo range and the carrier phase or carrier beat phase.

PSEUDO-RANGE MEASUREMENTS: (6)

- The pseudo-range observable, is calculated from observations recorded during a GPS survey.
- The pseudo-range observable is the difference between the time of signal transmission from the satellite, measured in the satellite time scale, and the time of signal arrival at the receiver antenna, measured in the receiver time scale.
- When the differences between the satellite and the receiver clocks are reconciled and applied to the pseudo-range observables, the resulting values are corrected pseudo-range values.
    - The value found by multiplying this time difference by the speed of light is an approximation of the true range between the satellite and the receiver, or a true pseudo-range.
    - A more exact approximation of true range between the satellite and receiver can be obtained if ionosphere and troposphere delays, ephemeris errors, measurement noise, and unmodeled influences are taken into account while pseudo-ranging calculations are performed.
    - The pseudo-range can be obtained from either the C/A-code or the more precise P-code.

CARRIER BEAT PHASE MEASUREMENTS: (1)

- The carrier beat phase observable is the phase of the signal remaining after the internal oscillator or frequency generated in the receiver is differenced from the incoming carrier signal of the satellite.
- The carrier beat phase observable can be calculated from the incoming signal or from observations recorded during a GPS survey.
- By differencing the signal over a period or epoch of time, one can count the number of wavelength cycles through the receiver during any given specific duration of time.
- The unknown cycle count passing through the receiver over a specific duration of time is known as the cycle ambiguity.
- There is one cycle ambiguity value per satellite-receiver pair as long as the receiver maintains continuous phase lock during the observation period.
- The value found by measuring the number of cycles going through a receiver during a specific time period, given the definition of the transmitted signal in terms of cycles per second, can be used to develop a time measurement for transmission of the signal.
- Once again, the time of transmission of the signal can be multiplied by the speed of light to yield an approximation of the range between the satellite and receiver.
- The biases for carrier beat phase measurement are the same as for pseudo-ranges although a higher accuracy can be obtained using the carrier.
- A more exact range between the satellite and receiver can be formulated when the biases are taken into account during derivation of the approximate range between the satellite and receiver.

14.b. (i) **Distinguish between single frequency receivers and dual frequency receivers.**

(i) Single frequency receivers: (3)

- A single-frequency receiver tracks the Ll frequency signal.
- It generally has a lower price than the dual-frequency receiver because it has fewer components and is in greater demand.
- A single-frequency receiver can be used effectively to develop relative positions that are accurate over baselines of less than 50 km or where ionosphere effects can generally be ignored.

(ii) **Dual-frequency receivers** (4)

- A dual-frequency receiver tracks both the L1 and L2 frequency signals and is generally more expensive than a single-frequency receiver.
- A dual-frequency receiver will more effectively resolve longer baselines of more than 50 km where ionosphere effects have a larger impact on calculations.
- Dual-frequency receivers eliminate almost all ionosphere effects by combining L1 and L2 observations.
- Most manufacturers of dual-frequency receivers utilize codeless techniques, which allow the use of the L2 during anti-spoofing.
- These codeless techniques are squaring, cross-correlation, and P-W correlation

14.b.(ii) List and discuss the sources of error in GPS.

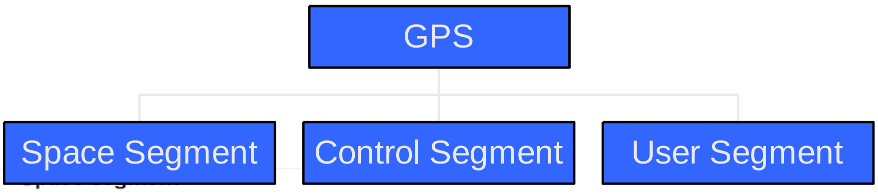
The major sources of errors are

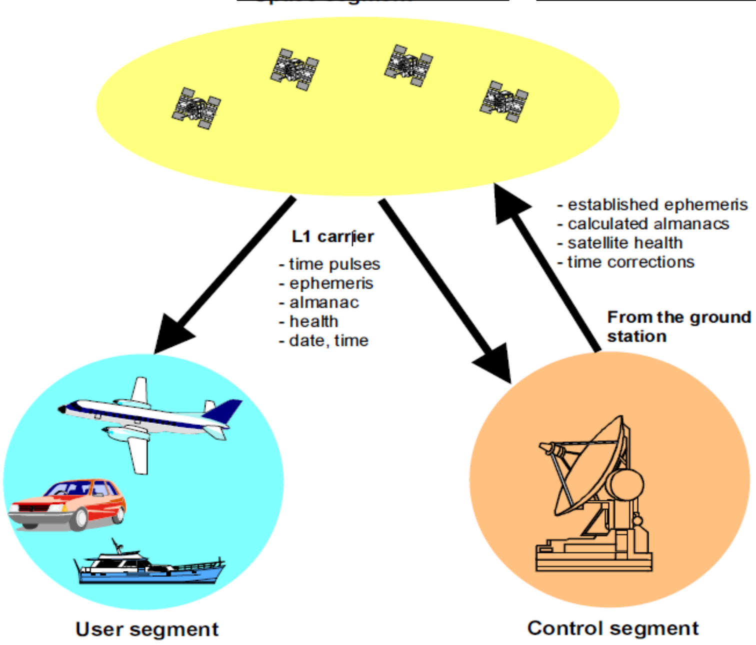
i. Satellite dependent error - Satellite clock error, satellite orbital error, satellite geometry, (2)
ii. Receiver dependent error - Receiver clock error, Antenna (2)
iii. Signal dependent error - Ionospheric and tropospheric delays, Multipath, Cycle slip, Selective availability (2)

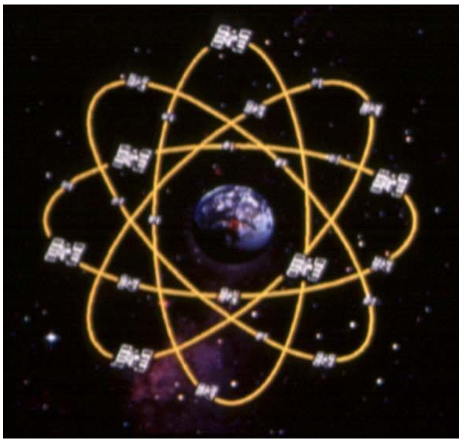
November / December 2017

1. Explain various segments of GPS.

Segments or Components of GPS:





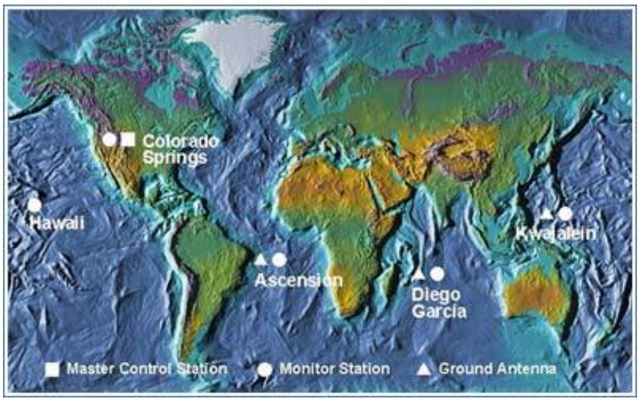


Space Segment:

- Consists of 24 satellites in 6 orbits.
- Each satellite transmits low powered radio signals.
- The orbital position is constantly monitored and updated by ground stations.
- Each satellite is identified by number and broadcasts a unique signal.
- The signal travels at the speed of light.
- Each satellite has a very accurate clock, 3×10^{-9} Seconds.

Distance = Velocity x Time

- ***GPS Satellite***
  - Name : NAVSTAR
  - Altitude : 11,000 miles
  - Inclination : 55 Deg to the Equator
  - Weight : 863 Kg (in orbit)
  - Orbital Period :12 hrs



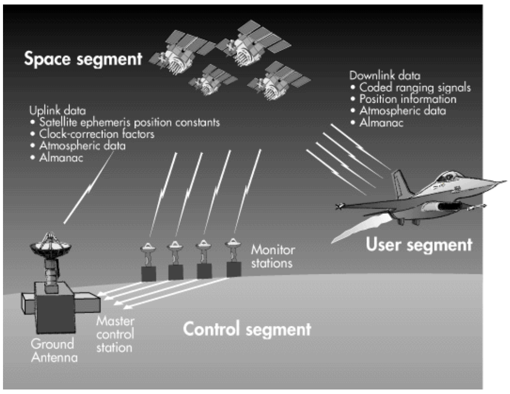
The Control Segment

- ***A Master Control Station***
- ***Unmanned Monitor Stations***
- ***Large Ground-antenna Stations***

Global Positioning System (GPS) Master Control and Monitor Station Network

- The control segment or ground segment has one Master Control Station, one alternative Master Control station (Monitor station).
- 12 command and large ground or control antennas and 16 monitoring sites.

Most important tasks of the control segment



- Observing the movement of the satellites and computing orbital data
- Monitoring the satellite clocks and predicting their behavior
- Synchronizing on board satellite time
- Relaying precise orbital data received from satellites in communication
- Relaying further information, including satellite health, clock errors etc.

The User Segment

- ***Users-Military and Civilians***
- ***GPS Receivers***
  - ***Decodes the signals from Satellites.***
  - ***Calculate the distance.***
- GPS receivers are generally composed of an antenna, tuned to the frequencies transmitted by the satellites, receiver-processors, and a highly-stable clock, commonly a crystal oscillator).
- They can also include a display for showing location and speed information to the user.
- A receiver is often described by its number of channels this signifies how many satellites it can monitor simultaneously.
- As of recent, receivers usually have between twelve and twenty four channels.
- Using RTCM SC-104 format, GPS receivers may include an input for differential corrections.
- This is typically in the form of a RS-232 port at 4800 bps speed.
- Data is actually sent at much lower rate, which limits the accuracy of the signal sent using RTCM.
- Receivers with internal DGPS (differential GPS) receivers are able to outclass those using external RTCM data.

Modes of Operation

- Standard Positioning System HA = 100 m
- Data Transmitted on L1 Frequency VA = 156 m

- For civil users TA = 340 ns
- Accuracy is degraded HA = 22 m
- Precise Positioning System VA = 27.7 m
- Data Transmitted on L1 and L2 Frequencies
- For Military users
- Highly Accurate

2. Discuss the types of GPS receivers.

Types of GPS receivers

Receivers can be classified in many ways;

Two basic types of GPS receivers are:

1. code phase receivers
   - C/A code receivers
   - P-code receivers
2. carrier phase receivers
   - Codeless receivers
   - Single frequency receivers
   - dual-frequency receivers
   - Receivers using cross-correlation or squaring or P-W techniques

Code dependent or code phase receivers

- These are also called code correlating receivers since they need access to the satellite navigation message of the P- or C/A-code signal for operation.
- A complete code dependent correlation channel produces following observables and information:
  - code phase
  - carrier phase
  - change of carrier phase (Doppler frequency)
  - satellite message

Carrier phase receivers

- Utilize the actual GPS signal itself to calculate a position.
- Two general types of such receivers are
  - single frequency
  - dual frequency

(a) Single frequency receiver

- Tracks L1 frequency signal only
- Cheaper than dual frequency receivers
- Used effectively to relative positioning mode for accurate baselines of less than 50 km or where ionosphere effects can generally be ignored.

(b) Dual frequency receiver

- Tracks both L1 and L2 frequency signal
- More expensive than a single frequency receiver
- Can more effectively resolve longer baselines of more than 50 km where ionosphere effects have a larger impact
- Eliminate almost all ionosphere effects by combining L1 and L2 observations.

Comparison of single and double frequency receivers

Single Frequency	Double frequency
Access to L1 only	Access to L1 and L2
Mostly civilian users	Mostly military users
Much cheaper	Very expensive
Used for short base lines	Used for both long and short base lines
Most receivers are coded	Most receivers with dual frequency are codeless
Corrupted by ionospheric delay	Almost independent of ionospheric delay
Modulated with C/A and P codes	It may not be possible for civilian users once Y code is there.

Receiver based on user community/application

- Receivers can be classified depending upon who is the user, e.g. Military, Civilian, Navigation, Timing, Geodetic/surveying, Handheld receiver

Geodetic receivers

These receivers are essentially used for geodetic/surveying applications with the following characteristics;

- carrier phase data as observables
- Availability of both frequencies (L1, L2)
- Access to the P code, at least for larger distances, and in geographical region with strong ionospheric disturbances (low and high latitudes).

Following factors should be kept in mind for such receivers

- Tracking all signals from each visible satellite at any time (GPS only system requires 12 dual frequency channels; GPS+GLONASS system needs 20 dual frequency channels)
- Both frequencies should be available
- Low phase and code noise
- High data rate (> 10 Hz) for kinematic applications
- High memory capacity
- Low power consumption and weight and small size
- Full operational capability under AS
- Capability to track weak signals (under foliage, and difficult environmental conditions)
- multipath mitigation, interference suppression, stable antenna phase centre (explained later)
- Good onboard and office software

Other useful features for geodetic receivers

- A modern GPS survey system should measure accurately and reliably anywhere under any condition
- It should be useable for almost any application (geodetic, geodynamic, detailed GIS and topographic engineering survey, etc.) and may have the following features
    - 1 pps timing output
    - event marker (for marking special events or area of interest to the GPS use)
    - ability to accept external frequencies
    - fast data transfer to computer
    - few or no cable connection
    - radio modem

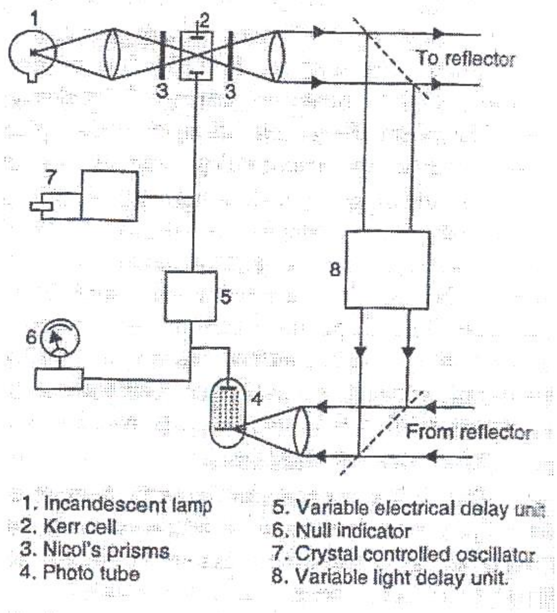
- DGPS and RTK capability (explained later)
- operate over difficult meteorological conditions
- ease in interfacing to other systems and from other manufacturer
- ease and flexibility of use (multi-purpose applications)
- flexible set up (tripod, pole, pillar, vehicle)

3. Discuss about the working principles of Geodimeter in total station.

Geodimeter:

- It is based on propagation of modulated light waves was developed by E. Bergestrand of the Swedish Geographical survey in collaboration with the manufacturer,(M/S. AGA of Sweden).
- Model 2-A can be used only for observations made at night.
- Model - 4 can be used for limited day time observations

Working /Measuring Principles:



- Figure shows, the photograph of the front panel of model -4 geodimeter mounted on the tripod.
- The main instrument is stationed at one end of the line (to be measured) with its back facing, the other end of the line , while a reflector (consisting either of a spherical mirror or a reflex prism system)is placed at the other end of the line.
- The light from an incandescent lamp (1) is focused by means of an achromatic condenser and passed through a kerr cell (2).
- The kerr cell consists of two closely spaced conducting plates, the space between which is filled with nitrobenzene.
- When high voltage is applied to the plates of the cell and a ray of light is focussed on it.
- The ray is split into two parts, each moving with different velocity.
- Two nicol's prisms (3) are placed on either side of the kerr cell.
- The light leaving the first nicol's prisms is plane polarised (divide into two groups with completely opposite views.)
- The light is split into two (having a phase difference) by the kerr cell. on leaving the kerr cell, the light is recombined.
- However because of phase difference, the resulting beam is elliptically polarised.
- Diverging light from the second polarised can be focused to the parallel beam by the transmitter objective, and then can be reflected from a mirror lens to a large spherical concave mirror.
- On the other end of the line being measured is put a reflex prism system or a spherical mirror, which reflects the beam of light back to the geodimeter.
- The receiver system of the geodimeter consists of spherical concave mirror, mirror lens and receiver objective.
- The light of variable intensity after reflection, have an effect on the cathode of the photo tube (4).

- In the photo tube, the light photons impact on the cathode causing a few primary electrons to leave and travel accelerated by a high frequency voltage, to the first dynode, where the secondary emission takes place.
- This is repeated through a further eight dynodes.
- The final electron current at the anode is some hundreds of thousand times greater than that at the cathode.
- The sensitivity of the photo tube is varied by applying the high frequency kerr cell voltage between the cathode and the first dynode.
- The low frequency vibrations are eliminated by a series of electrical chokes and condensers.
- The passage of this modulating voltage through the instrument is delayed by means of an adjustable electrical delay unit (5).
- The difference between the photo tube currents during the positive and negative bias period is measured on the null indicator (6) which is a sensitive D.C moving coil micro-ammeter.
- To make both positive and negative current intensifies equal (ie, to obtain null point), the phase of the high frequency voltage from the kerr cell must be adjusted ± 90° with respect to the voltage generated by light at the cathode.
- The light is focussed to a narrow beam from the geodimeter stationed at other end to the reflector stationed at the other end of the line.
- It is reflected back to the photo multiplier.
- The variation in the intensity of this reflected light causes the current from the photo multiplier to vary where the current is already being varied by the direct signal from the crystal controlled oscillator (7).
- The phase difference between the two pulses received by the cell are measure of the distance between geodimeter and reflector (ie, length of the line).
- The distance can be measured at different frequencies,
    - Model -2A ----- Three frequencies are available.
    - Model -4 ----- Four frequencies are available on phase position indicator.
- The polarity of the kerr cell terminals of high and low tension are reversed in turn.
- Fine and coarse delays switches control the setting of the electrical delay between the kerr cell and the photo multiplier.
- The power required is obtained from a mobile gasoline generator.
    - *Model -4A has a night range of 15 meters to 15 km,*
    - *Day light range of 15 to 800 meters*
    - *Average error of ±10 mm ± five millionth of distance*
    - *Weight about 36 kg without generators.*

2. Explain in detail about the route surveys for highway project.

- **In a highway reconnaissance survey, the following details are collected**:

    i. Obstructions along the route
    ii. Gradients and Length of curves
    iii. Cross drainage works
    iv. Soil type along the route.
    v. Sources of construction materials.
    vi. Type of terrain

- **The preliminary survey in a highway project is done with the main objective of**

    i. Various alternate arrangements
    ii. Estimate the quantity of earth work material and other construction aspects.
    iii. Compare different proposals

- **The following surveys are constructed**:
  i. Primary traverse
  ii. Topographical surveys
  iii. Levelling work
  iv. Hydrological data
  v. Soil surveys.
- **Detailed survey involves**
  i. fixing temporary bench marks along the route for every 300m
  ii. The C/S details are taken for 30m on either side of the central line.
  iii. All details of cross-drainage works are taken.
  iv. Topographical details are taken
  v. Detailed soil survey is carried out.

Part B

3

11) a) Correction for temperature $= C_t = \alpha(T_m - T_0)L$

$= 6.2\times10^{-6}(80-55)\,20$

$= 0.0031 \text{ m}$ (additive)

Correction for pull $= \dfrac{(P-P_0)L}{AE}$

Wt. of tape $= A(20\times100)(7.86\times10^{-3})\,kg = 0.8\,kg$

$A = \dfrac{0.8}{7.86\times2} = 0.051$ sq.cm.

$P = \dfrac{(16-10)\,20}{0.051\times2.109\times10^{6}} = 0.00112$ (additive)

Correction for sag $= \dfrac{l_1(wl_1)^2}{24P^2}$

$= \dfrac{20(0.8)^2}{24(16)^2} = 0.00208 \text{ m}$ (subtractive)

$\therefore$ Total correction $= +0.0031 + 0.00112 - 0.00208 = +0.00214\text{ m}$

(or)

11) b) i) observation made on the bright portion.

$$\beta = \frac{206265\, r \cos^2 \frac{1}{2}\alpha}{D}$$

$\alpha = 60'$ $r = 6$ cm $D = 9460$ m $= 9460 \times 10^2$ cm

$$\beta = \frac{206265 \times 6 \times \cos^2 30}{946000} = \underline{\underline{0.98 \text{ Seconds}}}$$

ii) observation made on the bright line

$$\beta = \frac{206265\, r \cos \frac{1}{2}\alpha}{D}$$

$$= \frac{206265 \times 6 \times \cos 30^\circ}{946000} = \underline{\underline{1.13 \text{ Seconds}}}$$

12) a) Sum of the observed angles $= 360^\circ\, 00'\, 04''$

Error $= +4''$

Total correction $= -4''$

This error of $4''$ will be distributed to the angles in an inverse proportion to their weights.

Let C_1, C_2, C_3 & C_4 be the corrections to the observed angles A, B, C & D respectively

$$\therefore C_1 : C_2 : C_3 : C_4 = \frac{1}{4} : \frac{1}{1} : \frac{1}{2} : \frac{1}{3}$$

(4)

or $C_1 : C_2 : C_3 : C_4 = 1 : 4 : 2 : \frac{4}{3}$

Also $C_1 + C_2 + C_3 + C_4 = 4''$

From (1) we have

$$C_2 = 4C_1 \quad C_3 = 2C_1 \quad \& \quad C_4 = \frac{4}{3} C_1$$

Substituting these values of C_2, C_3 and C_4 in (2)

$$C_1 + 4C_1 + 2C_1 + \frac{4}{3} C_1 = 4$$

$$C_1 \left[1 + 4 + 2 + \frac{4}{3} \right] = 4$$

$$C_1 = 0''.48$$

$$C_2 = 1''.92$$

$$C_3 = 0''.96$$

$$C_4 = 0''.64$$

Hence the corrected angles are

$A = 110^\circ 20' 48'' - 0''.48 = 110^\circ 20' 47''.52$

$B = 92^\circ 30' 12'' - 1''.92 = 92^\circ 30' 10''.08$

$C = 56^\circ 12' 00'' - 0''.96 = 56^\circ 11' 59''.04$

$D = 100^\circ 57' 04'' - 0''.64 = 100^\circ 57' 03''.36$

sum $360^\circ 00' 00''.00$

$6A + 4B = 517$ ~~23 01~~

12) b) Let k_1, k_2, k_3 be (or) the most probable correction to A, B and C are the most probable values of A, B, and C. Then

$k_1 = 0$ wt 3

$k_2 = 0$ wt 2

$k_3 = 0$ wt 2

$k_1 + k_2 = +2''.1$ wt 2

$k_2 + k_3 = +0''.5$ wt 1

$k_1 + k_2 + k_3 = +1''.5$ wt 1

Normal equation of K_1

$$3k_1 = 0$$
$$2k_1 + 2k_2 = +4.2$$
$$k_1 + k_2 + k_3 = +1.5$$
$$6k_1 + 3k_2 + k_3 = +5.7 \quad -(1)$$

Normal equation of k_2

$$2k_2 = 0$$
$$2k_1 + 2k_2 = +4.2$$
$$k_2 + k_3 = +0.5$$
$$k_1 + k_2 + k_3 = +1.5$$
$$3k_1 + 6k_2 + 2k_3 = +6.2 \quad -(2)$$

Normal equation for k_3:

$$2k_3 = 0$$
$$k_2 + k_3 = +0.5$$
$$k_1 + k_2 + k_3 = +1.5$$
$$k_1 + 2k_2 + 4k_3 = +2.0 \quad -(3)$$

Solving simultaneously 1, 2, 3 for k_1, k_2, k_3 we get

$k_1 = +0''.58$ $\quad k_2 = +0''.75$ $\quad k_3 = -0''.02$

Hence most probable values of A, B, & C are

$A = 75°\,32'\,46''.3 + 0''.58 = 75°\,32'\,46''.88$

$B = 55°\,09'\,53''.2 + 0''.75 = 55°\,09'\,53''.95$

$C = 108°\,09'\,28''.8 - 0''.02 = 108°\,09'\,28''.78$

9

13 a) **Types of Electronic Distance Measurement Instrument**

EDM instruments are classified based on the type of carrier wave as

1. Microwave instruments
2. Infrared wave instruments
3. Light wave instruments.

1. Microwave Instruments

These instruments make use of microwaves. Such instruments were invented as early as 1950 in South Africa by Dr. T.L. Wadley and named them as Tellurometers. The instrument needs only 12 to 24 V batteries. Hence they are light and highly portable. Tellurometers can be used in day as well as in night.

The range of these instruments is up to 100 km. It consists of two identical units. One unit is used as master unit and the other as remote unit. Just by pressing a button, a master unit can be converted into a remote unit and a remote unit into a master unit. It needs two skilled persons to operate. A speech facility is provided to each operator to interact during measurements.

2. Infrared Wave Instruments

In this instrument amplitude modulated infrared waves are used. Prism reflectors are used at the end of line to be measured. These instruments are light and economical and can be mounted on theodolite. With these instruments accuracy achieved is ± 10 mm. The range of these instruments is up to 3 km.

These instruments are useful for most of the civil engineering works. These instruments are available in the trade names DISTOMAT DI 1000 and DISTOMAT DI 55.

3. Visible Light Wave Instruments

These instruments rely on propagation of modulated light waves. This type of instrument was first developed in Sweden and was named as Geodimeter. During night its range is up to 2.5 km while in day its range is up to 3 km. Accuracy of these instruments varies from 0.5 mm to 5 mm/km distance. These instruments are also very useful for civil engineering projects.

Operations of Electronic Distance Measurement Instruments

It is essential to know the fundamental principle behind EDM to work with it. The electromagnetic waves propagate through the atmosphere based on the equation

$$V = f.\lambda = (\frac{1}{T}.\lambda)$$

$f = 1/T$: *(T=Time in seconds)*

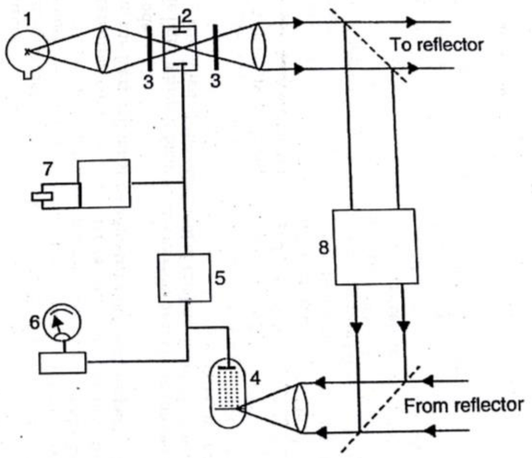
Where 'v' is the velocity of electromagnetic energy in meters per second(m/sec); f is the modulated frequency in hertz (Hz) and λ is, the wavelength measured in meters.

13) b)

THE GEODIMETER

The method, based on the propagation of *modulated light waves*, was developed by E. Bergestrand of the Swedish Geographical Survey in collaboration with the manufacturer, M/s AGA of Sweden. Of the several models of the geodimeter manufactured by them, model 2-A can be used only for observations made at night while model-4 can be used for limited day time observations.

Fig. 15.6 shows the schematic diagram of the geodimeter. Fig. 15.7 shows the photograph of the front panel of model-4 geodimeter mounted on the tripod. The main instrument is stationed at one end of the line (to be measured) with its back facing the other end of the line, while a reflector (consisting either of a spherical mirror or a reflex prism system) is placed at the other end of the line.



1. Incandescent lamp
2. Kerr cell
3. Nicol's prisms
4. Photo tube
5. Variable electrical delay unit
6. Null indicator
7. Crystal controlled oscillator
8. Variable light delay unit.

Fig. 15.6 Schematic Diagram of the Geodimeter.

The light from an incandescent lamp (1) is focused by means of an achromatic condenser and passed through a Kerr cell (2). The Kerr cell consist of two closely spaced conducting plates, the space between which is filled with nitrobenzene. When high voltage is applied to the plates of the cell and a ray of light is focused on it, the ray is split into two parts, each moving with different velocity. Two Nicol's prisms (3) are placed on either side of the Kerr cell. The light leaving the first Nicol's prisms is plane polarised. The light is split into two (having a phase difference) by the Kerr cell. On leaving the Kerr cell, the light is recombined. However, because of phase difference, the resulting beam is elliptically polarised. Diverging light from the second polariser can be focused to a parallel beam by the transmitter objective, and can then be reflected from a mirror lens to a large spherical concave mirror.

On the other end of the line being measured is put a reflex prism system or a spherical mirror, which reflects the beam of light back to the geodimeter. The receiver system of the geodimeter consists of spherical concave mirror, mirror lens and receiver objective. The light of variable intensity after reflection, impinges on the cathode of the photo tube (4). In the photo tube, the light photons impinge on the cathode causing a few primary electrons to leave and travel, accelerated by a high frequency voltage, to the first dynode, where the secondary emission takes place. This is repeated through a further eight dynodes. The final electron current at the anode is some hundreds of thousand times greater than that at the cathode. The sensitivity of the photo tube is varied by applying the high frequency-Kerr cell voltage between the cathode and the first dynode. The low frequency vibrations are eliminated by a series of electrical chokes and condensers. The passages of this modulating voltage through the instrument is delayed by means of an adjustable electrical delay unit (5). The difference between the photo tube currents during the positive and negative bias period is measured on the *null indicator* (6) which is a sensitive D.C. moving coil micro-ammeter. In order to make both the negative and positive current intensities equal (*i.e.* in order to obtain null-point), the phase of the high frequency voltage from the Kerr cell must be adjusted ± 90° with respect to the voltage generated by light at the cathode.

Thus, the light which is focused to a narrow beam from the geodimeter stationed at one end to the reflector stationed at the other end of the line, is reflected back to the photo multiplier. The variation in the intensity of this reflected light causes the current from the photo multiplier to vary where the current is already being varied by the direct signal from the crystal controlled oscillator (7). The phase difference between the two pulses received by the cell are a measure of the distance between geodimeter and the reflector (*i.e.*, length of the line).

The distance can be measured at different frequencies. On Model-2*A* of the geodimeter, three frequencies are available. Model-4 has four frequencies. Four phase positions are available on the *phase position indicator*. Changing phase indicates that the polarity of the Kerr cell terminals of high and low tension are reversed in turn. The 'fine' and 'coarse' delay switches control the setting of the electrical delay between the Kerr cell and the photo multiplier. The power required is obtained from a mobile gasoline generator. Model-4 has a night range of 15 metres to 15 km, a daylight range of 15 to 800 metres and an average error of ± 10 mm ± five millionth of the distance. It weighs about 36 kg without the generator.

14) a) **GPS Segments**

The Global Positioning System basically consists of three segments: the Space Segment, The Control Segment and the User Segment.

Space Segment

The Space Segment contains 24 satellites, in 12-hour near-circular orbits at altitude of about 20000 km, with inclination of orbit 55°. The constellation ensures at least 4 satellites in view from any point on the earth at any time for 3-D positioning and navigation on world-wide basis. The three axis controlled, earth-pointing satellites continuously transmit navigation and system data comprising predicted satellite ephemeris, clock error etc., on dual frequency L1 and L2 bands

Control Segment

This has a Master Control Station (MCS), few Monitor Stations (MSs) and an Up Load Station (ULS). The MSs are transportable shelters with receivers and computers; all located in U.S.A., which passively track satellites, accumulating ranging data from navigation signals. This is transferred to MCS for processing by computer, to provide best estimates of satellite position, velocity and clock drift relative to system time. The data thus processed generates refined information of gravity field influencing the satellite motion, solar pressure parameters, position, clock bias and electronic delay characteristics of ground stations and other observable system influences. Future navigation messages are generated from this and loaded into satellite memory once a day via ULS which has a parabolic antenna, a transmitter and a computer. Thus, role of Control Segment is: - To estimate satellite [space vehicle (SV)] ephemerides and atomic clock behaviour. - To predict SV positions and clock drifts. - To upload this data to SVs.

User Segment

The user equipment consists of an antenna, a receiver, a data-processor with software and a control/display unit. The GPS receiver measures the pseudo range, phase and other data using navigation signals from minimum 4 satellites and computes the 3-D position, velocity and system time. The position is in geocentric coordinates in the basic reference coordinate system: World Geodetic reference System 1984 (WGS 84), which are converted and displayed as geographic, UTM, grid, or any other type of coordinates. Corrections like delay due to ionospheric and tropospheric refraction, clock errors, etc. are also computed and applied by the user equipment / processing software..

14) b) **GPS Receivers**

A wide variety of GPS receivers are commercially available today. Depending upon the type of application, accuracy requirements and cost factor, the user can select the type of GPS receiver which best suits his demands. The receivers available cover a wide range from the high-precision Rouge receivers developed by the Jet Propulsion Laboratories, (JPL), of the National Aeronautics and Space Administration (NASA), with built-in atomic clock, to the hand-held navigation receivers used by Army personnel, mountaineers, etc., which can give the position to few-metres accuracy. Even wrist-watches with built-in GPS receivers are now commercially available (e.g.: the Casio GPS watch).

Navigation Receivers

These receivers are normally single-frequency, C/A code, hand-held light weight receivers, which can yield the position with a few-metres to few tens of metres accuracy. Single channel receivers, which can track 4 or more satellites by either sequential or multiplexing technique, which were more common in this category, are now being replaced by two or five channel receivers. These receivers are very much portable, weighing only few hundred grams, and are fairly inexpensive, being in the few hundred U.S. dollars price range. Examples of such receivers are the Magellan 5000 GPS receiver marketed in India by ROLTA (India), the NAVSTAR GPS PC card that can be fitted in personnel computer, marketed in India By Micronics Ltd., the Casio portable GPS receiver in a watch, etc. The accuracies in positioning obtained by these type of receivers are in the range of few tens of metres in absolute positioning 10 (in the absence of SA), and few tens of cm in relative positioning, over short baselines of few km.

Surveying Receivers

The surveying type of receivers are single frequency, multi-channel receivers, which are useful for most surveying applications, including cadastral mapping applications, providing tertiary survey control, engineering surveys, etc. These are more expensive than the navigation type of receivers, and more versatile. The data from many of these receivers can be directly imported in to most commonly used GIS software packages / formats. Most of these receivers can also be used in DGPS mode. Examples of surveying receivers are the PRO-XR model of Trimble Navigation Ltd., the SR 100 model of Leica Ag., etc.

Geodetic Receivers

The Geodetic receivers are multi-channel, dual-frequency receivers, generally with the capability of receiving and decoding the P-code. They are heavier and more expensive than the navigation and surveying receivers, ranging from the Rouge receivers installed at the GPS tracking stations, to the portable geodetic survey control receivers. They are capable of giving accuracies of the order of few cm-level in absolute positioning with precise post-processed satellite orbit information and of few mm-level in relative positioning. Examples of such receivers are the 4000 SSE of Trimble Navigation Ltd., the WILD 200 of Leica, and ASHTECH.

16) b) In the astronomical triangle (or) ZPM

$ZM = z = 90° - a = 90° - 33°35'10'' = 56°24'50''$

$PM = 90° - \delta = 90° - 22°05'36'' = 67°54'24''$

$ZP = 90° - 52°30'20'' = 37°29'40''$

By cosine rule.

$$\cos A = \frac{\cos PM - \cos ZP \cdot \cos ZM}{\sin ZP \cdot \sin ZM}$$

$$= \frac{\cos 67°54'24'' - \cos 37°29'40'' \cdot \cos 56°24'50''}{\sin 37°29'40'' \cdot \sin 56°24'50''}$$

From which $A = 97°6'48''$

Azimuth of the Sun $= 97°6'48''$

Since the Sun is to the west (or left) of the R.O, the

true bearing of R.O

$=$ Azimuth of Sun $+$ horizontal angle

$= 97°6'48'' + 18°20'30''$

$= 115°27'18''$ (Clockwise from North)

Part – B

1) a) i) Signals:-

A Signal is a device erected to define the exact Position of an observed Station.

They are classified as

1) Daylight (or) Non luminous [Opaque] Signal
2) Sun (or) luminous Signal
3) Night Signal

Requirements:-

1) It Should be Conspicuous [clearly Visible against background]
2) It Should be capable of being accurately center over the Station mark.
3) It Should be Suitable for accurate bisection.
4) It Should be free from phase or Should exhibit little phase.

11)(a)ii) Given:

All . Formula – 2 marks.

$$\text{Slope correction} = \Sigma l (1 - \cos\theta)$$
$$= 29.861 (1 - \cos 4^\circ 25') = 0.0887\ m \ (-Ve)$$

– 1 mark

$$\text{Temperature correction} = L\alpha (T_m - T_0)$$
$$= 29.861 \times 1.2 \times 10^{-5} (27^\circ - 15^\circ)$$
$$= 4.299 \times 10^{-3} m \ (-Ve)$$

– 1 mark

$$\text{Pull correction} = \frac{(P - P_0)L}{AE} = \frac{(130 - 50) \times 29.861}{2.75 \times 2.05 \times 10^5} = 3.71 \times 10^{-3} m \ (+Ve)$$

– 1 mark

$$\text{Sag correction} = \frac{n l W^2}{24P^2} = \frac{1 \times 30 \times 0.16^2}{24 \times 120^2} = 2.22 \times 10^{-6} m \ (+Ve)$$

– 1 mark

Total correction $= -0.0887 - 4.299\times10^{-3} + 3.71\times10^{-3} + 2.22\times10^{-6}$

$= -0.0893$ m — 1 marks

Correct Length $= 29.861 - 0.0893 = 29.772$ m — 1 mark

(11)(b)(i) Given:

$\alpha = +3^\circ 32' 36''$; $h = 1.15$ m ; $s = 4.85$ m ; $d = 4895$

$m = 0.07$ m ; $R \sin 1'' = 30.88$ m

$$\delta = \frac{s-h}{d \sin 1''} = \frac{4.85-1.15}{4895 \sin 1''} = \frac{3.7\times206265}{4895}$$

$= 155''.91 = 2'\ 35''.91$ (−Ve) — 1 marks

$$\text{Central Angle}, \theta = \frac{d}{R\sin 1''} = \frac{4895}{30.88} = 158''.52$$ — 1 marks

Curvature correction $= \frac{\theta}{2} = 79''.26$ (+Ve) — 1 marks

Refraction " , $r = m\theta = 0.07 \times 158.52$ — 1 marks

$= 11''.1$ (−Ve)

Total correction $= \frac{\theta}{2} - \delta - r = 79''.26 - 155''.91 - 11''.1$

$= -87''.75$

$= 1'\ 27''.75$ (−Ve)

Correct Altitude $= 3^\circ\ 32'\ 36'' - 1'\ 27''.75$

$= 3^\circ\ 31'\ 8.25''$ — 1 marks

ii) b) ii)

$$\beta = \frac{d \sin \alpha}{D \sin 1''} \text{ Seconds}$$

a) For the line AB :-

$\alpha = 132° 18' 30''$

$d = AS = 5.8 \text{ m}$

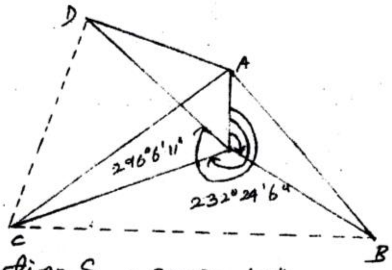
$D = AB = 3265.5$

$$\beta = \frac{5.8 \sin 132° 18' 30''}{3265.5} \times 206265 \text{ Seconds}$$

$= +270''.9 = +4' 30''.9$

$\therefore$ Direction of AB $= 132° 18' 30'' + 4' 30''.9$

$= 132° 23' 0''.9$



b) For the line AC :-

α = Angle Reduced to the direction S $= 232° 24' 6''$

$$\beta = \frac{5.8 \sin 232° 24' 6''}{4022.2} \times 206265 \text{ Seconds}$$

$= -235.7$ Seconds

$= -3' 55''.7$.

$\therefore$ Direction of AC = Direction of SC $+ \beta$

$= 232° 24' 6'' - 3' 55''.7$

$= 232° 20' 44''.3$

c) For the line AD :-

α = Angle Reduced to the direction SA $= 296° 6' 11''$

$D = AD = 3086.4 \text{ m}$

$$\beta = \frac{5.8 \sin 296° 6' 11''}{3086.4} \times 206265 \text{ Seconds}$$

$= -348.1$ Seconds

$= -5' 48''.1$

$\therefore$ Direction of AD $= 296° 6' 11'' - 5' 48''.1$

$= 296° 0' 22''.9$

13) a) Enumerate the Features of Total Stations:-

A Total Station is a Combination of EDM and electronic Theodolite.

Features:-

Horizontal Angle Measurement
Vertical Angle Measurement
Slope distance Measurement
Vertical distance Measurement
Horizontal distance Measurement
Zenith angle Measurement
Instrument Height Measurement
Reflector Height Measurement
Ground elevation of Total Station
Ground elevation of Reflector
Co-ordinate Measurement
Co-ordinate calculation.
Setting out works
Statistics for Analysing the Result of a Traverse
Interacts with Computer
Transfers the data
Works with data on Computer

13) b) Sources of Errors in Total station:

① Circle Eccentricity – Theoretical Centre of mechanical axis of the TS. does Coincide exactly with the Centre of the measuring Circle.

② Horizontal Collimation – optical axis of TS is not exactly $\perp^r$ to the Telescope axis.

③ Height of Standard – Telescope axis must be $\perp^r$ to vertical plane.

④ Circle Graduation Error – Not Clearly visible.

⑤ Vertical Circle Error –

⑥ Pointing Error – Due to Human & Environmental Condition

6

7. Uneven heating of Instrument
8. Vibration
9. Collimation Errors
10. Vertical Angles & Elevation – Tilt Sensor Error.
11. Atmospheric corrections. ...
12. optical Plummet Errors
13. Adjustment of prism poles
14. Angles
15. Slope to grid & Sea Level EDM Error.
16. Calibration Errors.

12 (a) Given :

$A = 32° \ 15' \ 3.62''$; $B = 40° \ 16' \ 18.4''$; $C = 35° \ 12' \ 26.6''$

Wt = 2   Wt = 1   Wt = 1

$A+B = 72° \ 31' \ 50.2''$; $A+B+C = 107° \ 44' \ 25.5''$

wt = 1   wt = 2

Normal Eqns:

For A

$$2A = 64° \ 30' \ 7.24''$$
$$A+B = 72° \ 31' \ 50.2''$$
$$2A+2B+2C = 215° \ 28' \ 51''$$
$$5A+3B+2C = 352° \ 30' \ 48.4''$$

4marks

For B

$$B = 40° \ 16' \ 18.4''$$
$$A+B = 72° \ 31' \ 50.2''$$
$$2A+2B+2C = 215° \ 28' \ 51''$$
$$3A+4B+2C = 328° \ 16' \ 59.6''$$

4marks

For C

$$2A+2B+2C = 215° \ 28' \ 51''$$

4marks

Normal Eqns are

$$5A+3B+2C = 352° \ 30' \ 48.4''$$
$$3A+4B+2C = 328° \ 16' \ 54.6''$$
$$2A+2B+2C = 215° \ 28' \ 51''$$

Solving abv. Normal Eqns simultaneously we get

$$A = 32° \ 15' \ 9.14''$$
$$B = 40° \ 16' \ 29.68''$$
$$C = 35° \ 12' \ 46.58''$$

1marks

7

(12)(b)

$POQ = 83^\circ\ 42'\ 28.75''$ wt = 3
$QOR = 102^\circ\ 15'\ 43.26''$ wt = 2
$ROS = 94^\circ\ 38'\ 27.22''$ wt = 4
$SOP = 79^\circ\ 23'\ 23.77''$ wt = 2

$\text{Sum} = 360^\circ\ 0'\ 3''$

Total correction $E = 360^\circ - 360^\circ 0' 3'' = -3''$ — 1 marks

$E = e_1 + e_2 + e_3 + e_4 = -3''$ — ①

$\Sigma we^2 = \text{minimum} \Rightarrow 3e_1^2 + 2e_2^2 + 4e_3^2 + 2e_4^2 = \text{min}$ — ② — 1 mark

Differentiating ①&② we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ — ③ — 1 marks

& $3e_1\delta e_1 + 2e_2\delta e_2 + 4e_3\delta e_3 + 2e_4\delta e_4 = 0$ — ④ — 1 marks

Multiply ③ by $-\lambda$ and add to ④ we get.

$\delta e_1(3e_1 - \lambda) + \delta e_2(2e_2 - \lambda) + \delta e_3(4e_3 - \lambda) + \delta e_4(2e_4 - \lambda) = 0$ — 1 marks

$3e_1 - \lambda = 0$ or $e_1 = \frac{\lambda}{3}$
$2e_2 - \lambda = 0$ or $e_2 = \frac{\lambda}{2}$
$4e_3 - \lambda = 0$ or $e_3 = \frac{\lambda}{4}$
$2e_4 - \lambda = 0$ or $e_4 = \frac{\lambda}{2}$

2 marks.

Substituting above values in ① we get

$\frac{\lambda}{3} + \frac{\lambda}{2} + \frac{\lambda}{4} + \frac{\lambda}{2} = -3''$

or $\lambda\left(\frac{19}{12}\right) = -3''$

$\Rightarrow \lambda = -\frac{3 \times 12}{19} = -1.8947$ — 2 marks

$e_1 = \frac{-1.8947}{3} = -0.63''$

$e_2 = \frac{-1.8947}{2} = -0.95''$

$e_3 = \frac{-1.8947}{4} = 0.47''$

$e_4 = \frac{-1.8947}{2} = -0.95''$

1 marks

Sum $= -3''$

$POQ = 83^\circ 42' 28.75'' - 0.63''$
$= 83^\circ 42' 28.12''$

$QOR = 102^\circ 15' 43.26'' - 0.95''$
$= 102^\circ 15' 42.31''$

$ROS = 94^\circ 38' 27.22'' - 0.47''$
$= 94^\circ 38' 26.75''$

$SOP = 79^\circ 23' 23.77'' - 0.95''$
$= 79^\circ 23' 22.82''$

3 marks

$\text{Sum} = 360^\circ 0' 0''$

14) a) Explain the different Segments of GPS

It Consists of 3 Segments

a) Space Segment

b) Control Segment

c) User Segment

a) Space Segment

Developed By US department of Defence

Maintained by US Air Force and Space Based Positioning

24 Operational Satellites orbiting at Height of 20180 kms on 6 different Orbits

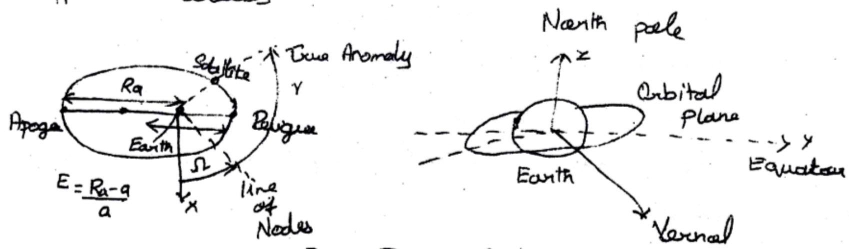
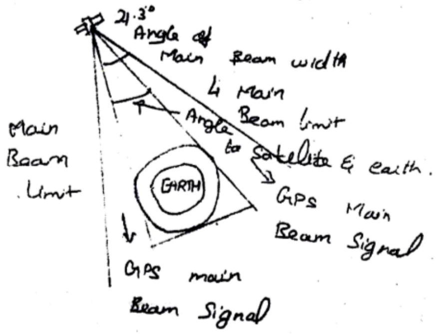


Fig :- Parameters of Keplerian Orbit

In Improvement Constellation 6 Satellites users have Instead of 4

GPS Satellites Can be Indentified Using, Space Vehicle Number or NAVSTAR number or PRN or Space Vehicle Identification Number



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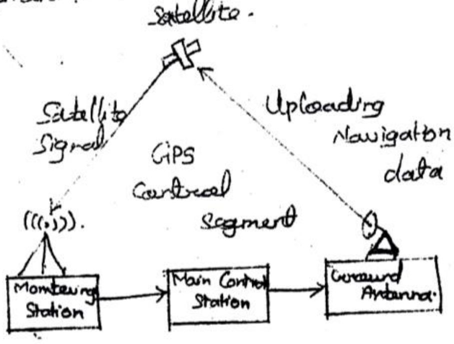
Control Segment:-



* Moniters the GPS Satellite Constellation
* Control of orbital Parameters
* Controlling Selective Availability & Anti Spoofing.

Components of Control Segments

* Master Control Station [MCS]
* Monitering Station [MS]
* Ground Antenna
* Operational Control Segment

User Segment

Each and every Satellite Transmit Signal to Control Segment and user Segment all the Time [24 × 7]

The GPS Satellite Signals consists of Three Components such as

Pseudo Random Noise Code [P- Code or C/A Code]

Carrier Signal [L1 or L2]

Data Signal

14) b) ii) Explain the Task of Control Segment in GPS ?

* Continuously Monitor the GPS Satellite constellation
* Control of Satellite orbital parameters
* Determine the GPS System Time
* Predict the ephemerides
* Update the Navigation data on periodic Basis
* Resolving Satellite anomalies.
* Controlling Selective Availability & Antispoofing
* Monitor Health of Satellites
* Spares to Substitute an unhealthy Satellite to maintain the GPS Satellite configuration.

14) (b) i)

Hand held Receivers	Geodetic Receivers.
① Simple to operate	① Difficult to operate
② Easy to Traverse	② Difficult
③ Signal Range - Normal	③ Signal Range - High
④ Less Accurate - Height	④ More Accurate.
⑤ Used for simple works	⑤ Used in High precision work.
⑥ Cheap	⑥ Costly
⑦ Simply held in Hand	⑦ Needs Tripod.

15 a) ii) How Reconnaissance Survey for Railway projected is conducted ?

Reconnaissance Survey Furnish Following details

Topographical Features of the area

Existing water Resources along with their discharge details

Geological and Soil classification

Natural features like Ridges, Valley, Forests, etc

Existing Survey maps and Aerial photographs

a few Tentative alignment are considered

Equipment used in Reconnaissance :

1) Barometer ; 2) Abney level ; 3) Prismatic compass

4) Binoculars ; 5) Pedometer.

15) b) Explain the Various Sounding Methods?

The Soundings are located with Reference to the Shore Traverse by observations made

i) entirely from the boat ii) Entirely from the Shore iii) both

The Following are the Methods

a) By Conning the Survey Vessel

1. By cross rope

2. By Range and Time intervals

13

b) By observations with Sextant or Theodolite

3. By Range and one angle from the Shore

4. By Range and one angle from the Boat

5. By Two angles from the Shore

6. By Two angles from the Boat

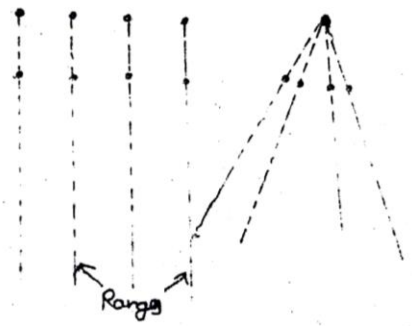
7. By one angle from Shore and one From Boat

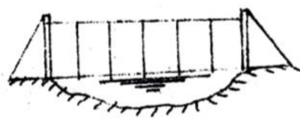
8. By Intersecting Ranges

a. By Tachecametry

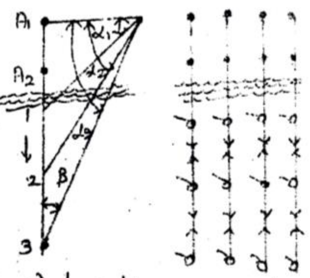
c) By Theodolite angles and EDM distances from the Shore

d) By microwave Systems.



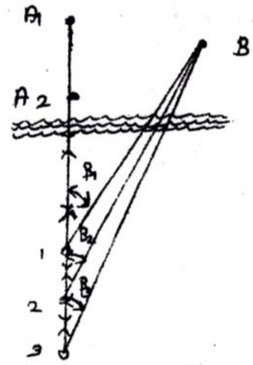


1. Location by Cross Ropes.

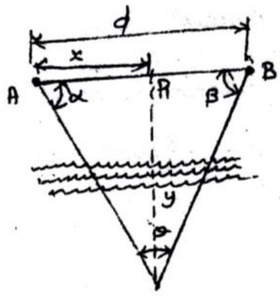


3) Location by Range and one angle from Shore

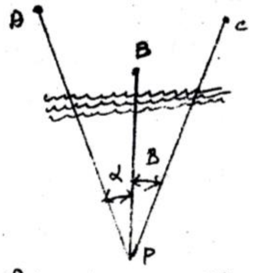
a) By Conning the Survey Vessel



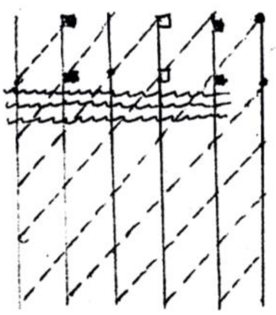
4) Location by Range and One angle from Boat



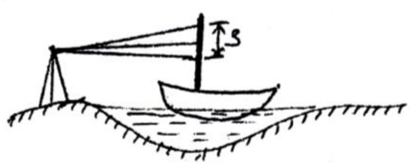
5) Location by Two angles from Shore



6) Location of Two Angles from Boat



8) Location by Intersecting Ranges



9) Location by Tacheometric observation

16)
I) Discuss the Various Steps in Triangulation Survey?

On basis of Triangulation figures

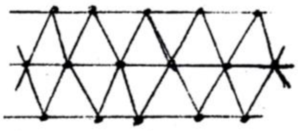
i) First Order (or) Primary Triangulation

ii) Second order (or) Secondary Triangulation

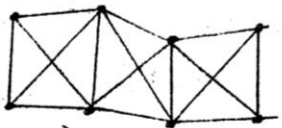
iii) Third order (or) Tertiary Triangulation

Triangulation Figures

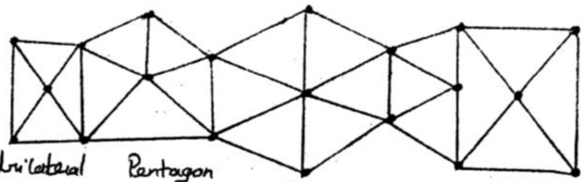
a) Single chain of Triangles



b) Double chain of Triangles



d) Quadrilaterals.

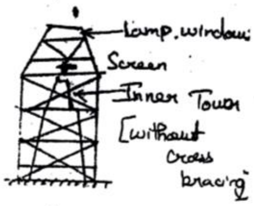


c) Centred Figure

Routine of Triangulation Survey

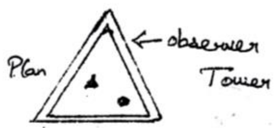
1) Reconnaissance

* Selection of Triangulation Stations
* Intervisibility and Height of Stations
* Profile of Intervening Ground



2) Erection of Signals and towers

1) Daylight (or) Nonluminous (opaque) Signal
2) Sun (or) Luminous Signal
3) Night Signal



3) Base line Measurement

* Selection of Base line
* Calculation of length of Base : Tape Corrections

4) Measurement of Horizontal angles

5) Astronomical observation at Laplace Stations

6) Computations

16) b) Briefly explain the application of Remote Sensing ?

1. Agriculture :-

Early Season estimation of Total cropped Area

Crop Yield modelling

2. Forestry :-

Forest stock mapping

Wild life habit assessment

3. Land use and Soils :-

4. Geology :-

5. Urban land Use :-

6. Water Resources Management

7. Coastal Environment

8. Ocean Resources

9. Watershed Management

10. Environment

11. Street network - based applications

12. Land parcel- Based applications

13. Natural Resources based applications

14. Facilities Management

15. Disasters

16. Digital elevation models

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Key for CE 6404 Surveying II

Part A

1. Triangulation is a surveying method that measures the angles in a triangle formed by three survey control points. Using trigonometry and the measured length of just one side, the other distances in the triangle are calculated.

2. In order to secure well-conditioned triangle or better visibility, objects such as church spires, flag poles, towers etc. are sometimes selected as the triangulation stations. When the observations are to taken from such a station, it is impossible to set up an instrument over it. In such a case, a subsidiary station, known as a Satellite station or false station is selected as near to the mains

3. The weight of an observation is a number giving an indication of its precision and trustworthiness when making a comparison between several quantities of different worth. Thus, if a certain observation is of wt. 4 it means that it is four times as much reliable as an observation of wt. 1.

4. ~~Answer~~ A normal equation is the one which is formed by multiplying each equation by the co-efficient of the unknown whose normal equation is to be found and by adding the equations thus formed. As the number of normal equations is the same as the number of unknowns, the most probable values of the unknown can be found from these equations.

10) 1) It is more accurate as a truly vertical sounding is obtained.
2) The speed of sounding and plotting is increased.
3.) It is more sensitive than the lead line.
4) The speed of sounding and plotting is increased.

5) The vertical angle is measured relative to the local vertical (plumb) direction. The vertical angle is usually measured as a zenith angle (0° is vertically up, 90° is horizontal, and 180° is vertically down), although one is also given the option of making 0° horizontal. The zenith angle is generally easier to work with.

6) A total station is a combination of an electronic theodolite and an electronic distance meter. This combination makes it possible to determine the coordinates of a reflector by aligning the instruments cross-hairs on the reflector and simultaneously measuring the vertical and horizontal angles and slope distances.

7) Anti-spoofing of the GPS system is designed for an anti potential spoofer. 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.

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

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Question Paper Code : 71559

CE6404 – Surveying – II

(R-2013)

1. What is meant by phase of Signal?

Phase of Signal is error of bisection, which arises under Lateral illumination

The Signal partly in light and partly in Shade. The observer Sees Only the eliminator Portion and Bisects it.

2. What is a Base Net?

Base Net – Extension of Base

The group of Triangle meet for extending the Base is Known as Base Net.

3. What are the Kinds of errors Possible in Survey works?

a) Mistakes
b) Systemmatic errors [cumulative errors]
c) Accidental errors [compensating errors].

4. Distinguish b/w True error and Residual error?

True error	Residual error
Difference between Measurement and True Value of quantity Measured	Difference between observed Reading and Predicted Reading

5. Compare the microwave and the electro-optical Systems adopted in Total Station.

Wave length :- Electro optical

Visible light = 0.4 – 0.7

Infra Red = 0.7 – 1.2

$S = 0.5 Vt$

Short distance Measurement

More Atmospheric effect

Microwave

Radio wave

$D = V/t$

Wave length is High

Long distance Measurement

Free from atmospheric effect

6) What is Total Station?

Total Station is a Surveying equipment Combination of EDM and electronic Theodolite.

Vhat do you Understand From the Satellite Configuration

Satellite Configuration is a System of Constellation of
[s]atellites placed at 6 different orbits Such that It Consist
.. 24 Satellites orbiting around the earth at an
[i]nclination of 55° which ensures that any of ground Statio[n]
[c]an Recieve Signals from atleast 4 Number of Satellite at
[a]n instance so that absolute position of a earth featur[e]
[c]an be obtained then and there.

[W]hat is GPS:- [Global Positioning System]

GPS is Simple EDM device which doesnot Requir[e]
[d]irect line of Sight between Survey Stations as in
[c]onventional Surveying. Inturn it uses atleast 4 or more
[G]PS Satellites unobstructed line of Sight and Tracking
[w]hich provides us absolute co-ordinates of features tha[t]
[e]xists in earth.

[W]hat are the Functions of Transition Curve?

To accomplish gradually the Transition from the
[t]angent to the circular Curve ViceVersa. so that the
[c]urvature is increased gradually from Zero to a
[s]pecified Value

To provide a medium for a gradual
[i]ntroduction (or) change of the Required Super-elevation

[De]fine Hydrographic Surveying?

Hydrographic Survey is that branch of Survey
[w]hich deals with measurement of Bodies of water.

It is the art of delineating the Submarine
[l]evels, Contours and features of Seas, gulfs, River
...