Name : P. Janaki Ramudu

Dept : CIVIL

SUB : SURVEYING

: Note book

UNIT- 1 :-

* Objects of Surveying :-

→ What is surveying.

* Determining relative positions of various points above or below the surface the surface of the earth.

* To mark the positions of proposed structure on ground.

* To determine area, volumes, and other related quantities

→ primary divisions of surveying :-

* primary division is based whether the curvature of earth is considered or not.

* The actual shape of earth is an oblate spheroid.

* The polar axis (12713.168 km) is shorter than the equitorial axis (12756.602 km) by about.

* 43.434 km (0.34) less than equitorial axis.

* The average radius of earth is taken as 6370 km for all calculations point of view.

→ plane surveying :-

The surveying in which the curvature of the earth is neglected and is assumed to be a flat surface.

* plane survey can safely be used when the extent of area is less than 250 sq.km.

⇒ Geodetic surveying :-

* It is the type of surveying in which the spherical shape of the earth is taken in to account.

* All survey lines are considered curved and all triangles are considered as spherical triangles.

* principles of surveying :-

* To work from whole to part.

* To fix positions of a new points by atleast two independent processes.

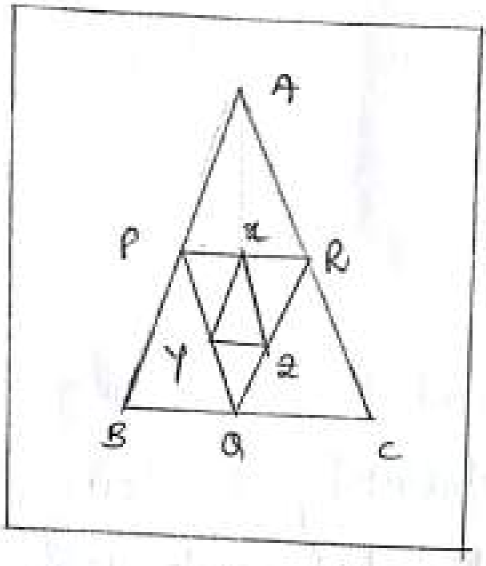
* To work from whole to part :-

This principal states that it is essential to first establish central points with high precision they are further subdivided in to smaller areas, with slightly less precision.

The object of this system is the prevent the accumulations of errors.

* According to this principle, first of all, control point A, B, C are fixed with great care and the frame work

ABC is prepared.



* A, B, C = control points.

  P, Q, R, x, y, z = minor control points.

  Δ ABC = Main frame work.

  Δ xyz = subsidiary frame work.

* The main frame work is subdivided in to small triangles PQR and xyz, by the method of triangulation.

* The details within these triangles are surveyed with less accuracy.

* classification of survey :-

⇝ Functional classification of surveying :

1. Control surveying.
2. Topographical surveying

3. Cadastral survey.
4. Engineering survey.
5. Mine survey.
6. Hydrographic survey.
7. Astronomical survey.
8. Geological survey.

* Control surveying :≡ Establishing the horizontal and vertical position of widely spaced control points using principles of Geodetic surveying.

In India, Control survey is done by "Survey of India" is located in 'uppal.'

* Topographical survey :≡ To show natural features of the country such as Rivers, hills, lakes etc.

* Cadastral survey :≡ Fixings of property lines, to show boundaries of fields, buildings etc. This is to be done by a Revenue Engineer.

* Engineering survey :≡ To obtain data for designing any type of project such as roads, railways, water supply systems etc. An engineer is interested in this survey works.

* Mine survey :— Underground works such as mines, shafts, pits, bore holes etc.

* Hydrographic survey :— surveying under water bodies.
→ Ex: Determination of channel depth etc.

* Astronomic survey :— To determine absolute location of a point on earth by taking latitude, longitude, azimuth, local time etc.

* Geological survey :— Determine different strata inside the earth's crust.
Geologist will be done the survey.

→ Classification based on the instruments :—

* Chain survey :— Only line-measurements are taken with a chain or tape.
Angular measurements are not taken.

* Generally used when high accuracy is not required.

* Compass survey :— Horizontal angles are measured with a compass in a addition to linear measurements with a chain or tape.

* Magnetic compass is not a precise instrument

* More precise than chain survey.

* Levelling :- Levelling instrument is used - for finding differences.

In elevations and determining elevations with reference to a datum more precise than compass survey.

* plane table survey :-

Both plotting and taking measurements are simultan-
-eous.

Linear measurements with a chain or tape. Most suitable in areas with magnetic material whether other surveys using with magnetic needle may get affected. Less accurate.

* Theodolite Survey :- It is very precise instruments for measurement for measuring horizontal and vertical angles.

* It is useful for the traverse survey and triangulation. Base linear are located used for triangulation.

* Tacheometric survey :- Theodolite with a stadia diaphragm having too horizontal cross-hairs.

In addition to central horizontal hair is used is used.

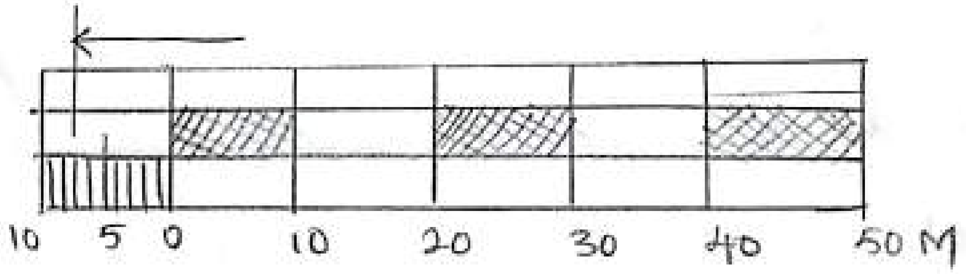
* Example :- Construct a plain scale 1cm = 3m and shown on it 47m.

Draw a line 20cm long and divide it in to 6 equal parts. This line will represent 20×3 = 60cm.

⟹ Each part will represent 10m.

⟹ Sub-divide the first left hand division in to 10 equal parts each reading 1m.

⟹ place zero of the scale between the sub-divided parts and the undivided part.



Scale,

1cm = 3m

$$\boxed{R.F = \frac{1}{300}}$$

* To take 47m, place one leg to the divider of 40M and the other of 7m.

Scanned by CamScanner

* Not very accurate – Extremely convenient for topographical survey details.

* photogrammetry :– Using photographs — Vast areas — area difficult to reach.

* EDM survey :– Based on triangulation — Where all the three sides of a triangle are measured with EDM survey.

* Scales :–

→ Types of scales :

1. plain scale.
2. Diagonal scale.
3. Vernier scale.
4. chord scale.

1. plain scale :– A plain scale is one on which it is possible to measure two dimensions only.

For example, meters and decimeters.

Hundred and tenths.

units and tenths, etc.

2. Diagonal scale :– On a diagonal. scale, it is possible to measure three dimensions such as metres, decimeters, and centimeters.

Units, tenths and hundreds etc.

* A diagonal scale is made on the principle of similar triangles.

* For example,

suppose it is required to divided a short distance PQ in to ten equal parts as shown in fig.

* At Q draw a line QR, perpendicular to PQ and of any convenient length.

* Divide it in to 10 equal parts. Join the diagonal PR.
* From each of the divisions 1, 2, 3 etc draw lines parallel to PQ, thus dividing the diagonal in to 10 equal parts.

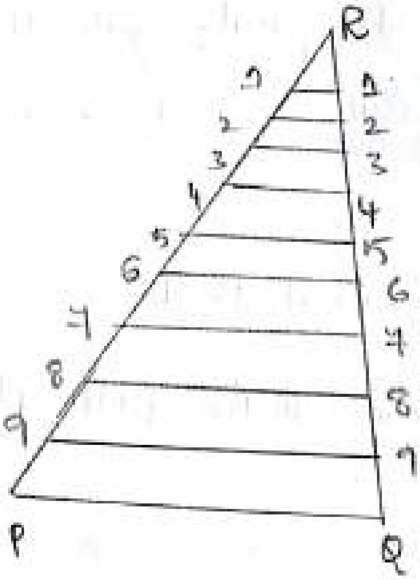
Thus,

$1-1 = \frac{1}{10}$ of PQ

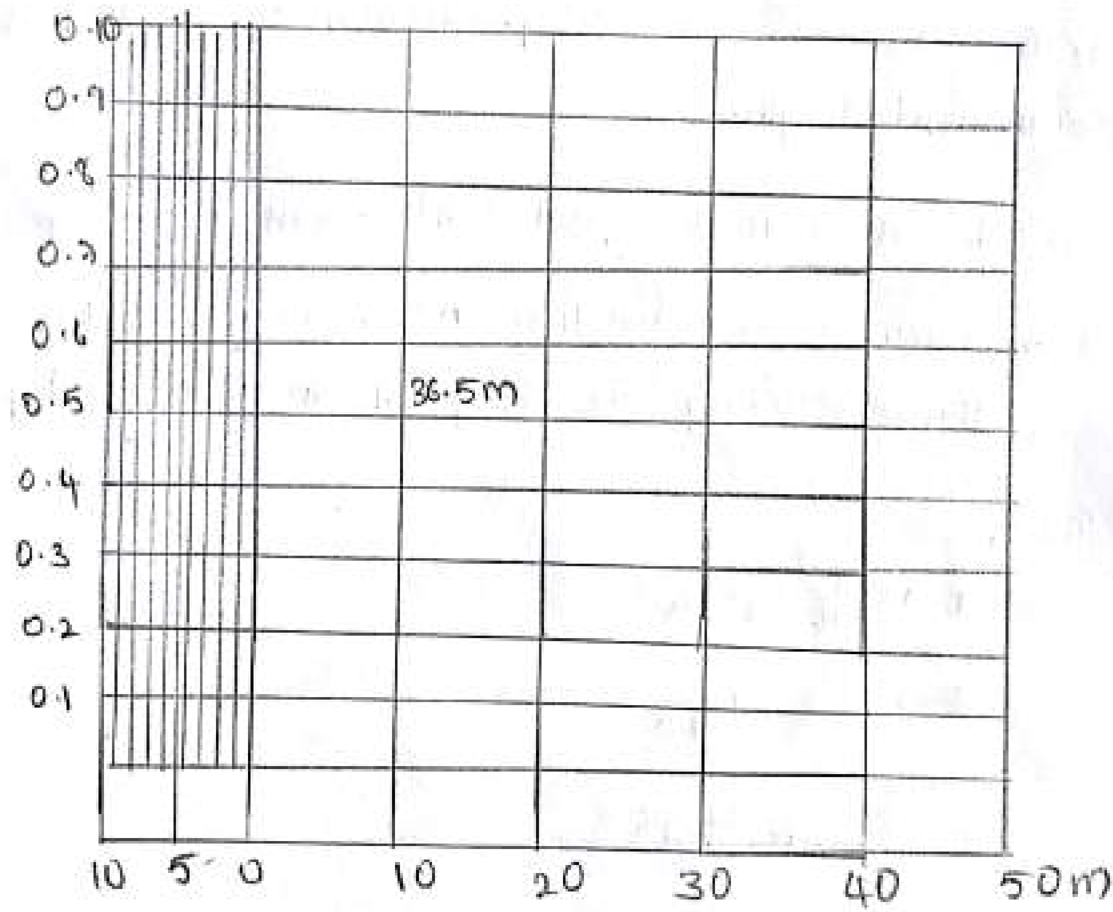
$2-2 = \frac{2}{3}$ of PQ

$3-3 = \frac{3}{10}$ of PQ

$4-4 = \frac{3}{10}$ of PQ



* Example :- Construct a diagonal scale 1cm = 4 m, to read metres and decimeters.



Construction steps :-

* Take 15cm length and divide it in to 6 equal parts. scale is 1cm = 4m.

  Therefore, this line represents 15x4 = 60m.

* Sub-divide the first left hand part in-to 10 divisions, each respresenting 1 metre.

* At the end of the sub-division erred a perpendicular of any division suitable length (say 5cm) and divide in to 10 equal parts through these parts draw horizontal lines.

* Draw vertical lines from 0, 10, 20, 30m------ divisions.
* Mark 36.5m on the scale as shown in figure.

chord scale :-

A chord scale is used to measure angles or to set off angles, it is marked either on a rectangular protractor or on an ordinary box wooden scale.

Vernier scale :-

Vernier scale was inverted by pierre vernier (1631), to measure a fractional part of a graduated scale it consists of two approxemating scales, it

Consists of two one primary scale (fixed) and the other vernier scale (movable).

⇒ Let, P = value of the smallest division of the primary scale.

V = value of the smallest division of the vernier scale.

n = number of divisions of the primary scale of a specified length.

n+1 = Number of divisions of the vernier scale of the small length.

⇒ Scales :-

* Full size scale.
* Reduced scale.
* Enlarged scale.
* Representative Fraction (R.F).

* Scale :- The distance measured on ground are plotted on paper. In such a way that a fixed ratio is maintained btwn the distance on ground to the corresponding distance on paper.

* The scale of a map or drawing is the ratio of distance on the map or drawing the corresponding distance on

on the ground.

$$\text{map scale} = \frac{\text{Distance on map}}{\text{Distance on ground}}$$

If the scale of the map is 1cm = 10m.

It means that 1cm on paper represents 10m on the ground.

* Representation of scale :=

Scale can be represented by three methods.

1. Engineer's scale.
2. Representative scale. Fraction.
3. Graphical scale.

* Engineer's scale :=

In this method, scale may be represented by a statement. for example,

1cm = 10m.

It means that 1 cm on paper represents 10m on the ground.

* Representative scale := (Representative Fraction)

In this case, the ratio of distance on the map to the distance on the ground is worked out in such a way that numerator is unity and denominator is fraction.

In the same unit of measurement as the numerator.

ms R.F = $\frac{\text{Distance b/w two points on the map}}{\text{distance b/w two points on the ground}}$

Both distance are taken in the same unit. The R.F can be very easily found for a given engineer's scale.

For example,

If the scale is 1cm = 50m.

$$\therefore R.F = \frac{1\,cm}{50}$$

$$= \frac{1}{50 \times 100}$$

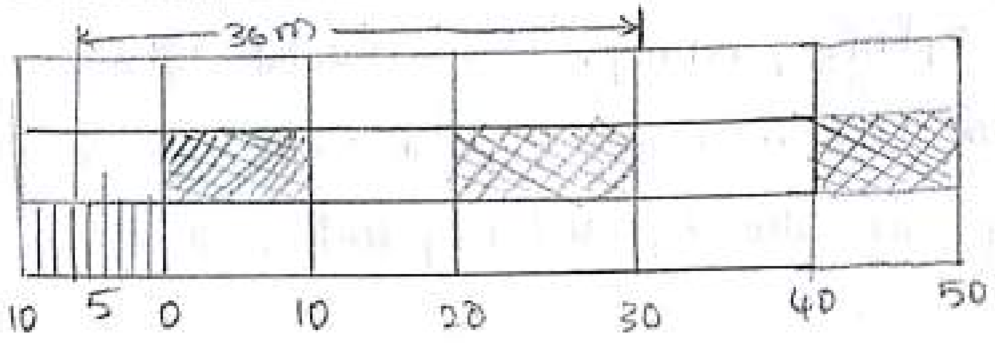
$$= \frac{1}{5000}$$

$\boxed{\therefore 1\,m = 100\,cm}$

R.F is unit less.

* Graphical scale :- In the graphical scale is drawn on plan itself.

* A graphical scale is a line sub-divided in to plan distance corresponding to convinient units of length on the ground.



my scale : 1cm = 5m

$$R.F = \frac{1}{500} = \frac{1\,cm}{5 \times 100\,cm}$$

* To prepare a graphical scale, draw a line 12cm long on the map. Divide it 6 equal parts. Each part will be 2cm each part will represent 2x5=10m of ground distance.

* Divide the first part in to 10 equal sub-parts. Each sub-part will be equal to 1m of ground distance.

* A distance of 36m is shown in the graphical scale.

* plans and maps :-

When the figure is drawn to a small scale then it is known as 'map' and when the figure is drawn to large scale then it is known as 'plan.'

For complete example a map of India (or) a state etc and a plan of a building, culvert etc.

On a plan, generally, only horizontal distance and direction are shown on a graphical map, however, the vertical distance are also represented by contour lines etc.

* Shrinkage of map :-

It is necessary to draw a scale on a map because as the map shrinks (or) expands the scale line also shrinks (or) expands with it and thus the measurements made from the map are not affected.

The ratio of the shrunk length to the actual length is known as the shrinkage ratio (S.R) or the shrinkage factor (S.F)

$$SF = \frac{\text{shrunk length}}{\text{original length}} = \frac{\text{shrunk scale}}{\text{original scale}}$$

$$= \frac{\text{shrunk R.F}}{\text{original R.F}}$$

Shrunk scale = S.F x Original scale.

Thus, Correct distance = $\frac{\text{Measured distance}}{S.F}$

and correct area = $\frac{\text{measured area}}{(S.F)^2}$

Scanned by CamScanner

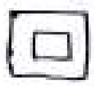
If a wrong measuring scale is used to measure the length of a line already drawn on a plan (or) map, the measured length will be erroneous. Then,

$$\text{Correct length} = \frac{\text{R.F of the wrong scale}}{\text{R.F of the correct scale}} \times M.L$$

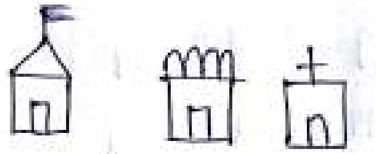
$$\text{and correct area} = \left[\frac{\text{R.F of the wrong scale}}{\text{R.F of the correct scale}}\right]^2 \times M.A$$

* The various types of graphical scale may be plain, diagonal or vernier scales. For details of these scales, refrence may be made to any book on engineering.

* Conventional symbols used in surveying :-

Name	Symbol
1. North direction	
2. Main station	
3. chain line	
4. Traverse direction	
5. Bench mark	
6. Building	

Scanned by CamScanner

7. Temple, mosque, church 

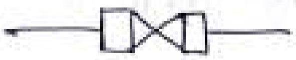
8. Barbed wire fencing 

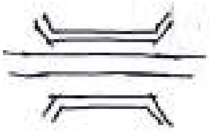
9. Hedge 

10. pipe Railing 

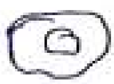
11. stone fencing 

12. Telescope line 

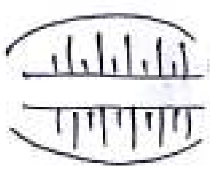
13. Wall well Gate 

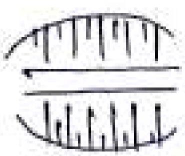
14. Bridge (or) culvert 

15. River (or) canal 

16. pond 

17. Well 

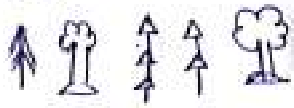
18. Embankment 

19. cutting 

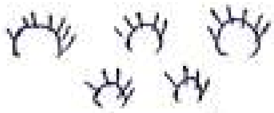
20. Railway line (single line) 

21. Double line 

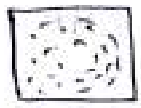
22. Tree 

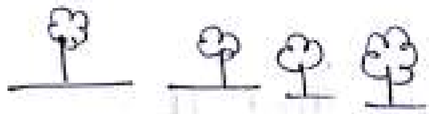
Forest 

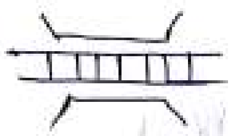
23. Marshy land 

24. Grass land garden 

25. Cultivated land 

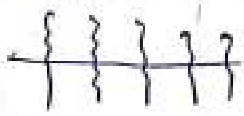
26. Barren land 

27. Orchard 

28. Railway bridge 

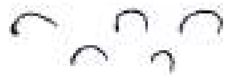
29. Tunnel 

30. Boundary line 

31. Dam 

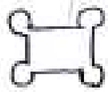
32. Counter line 

33. stone mine 

34. shrine 

35. Road 

36. pipe line 

37. Fort 

* Code of signal for Ranging :-

Signal by surveyor	Action by Assistant
* Rapid sweep with right hand	* Move preferably to the right.
* slow sweep with right hand	* Move slow to the right.
* Right arm extend	* Continue to move to the right.
* Rapid sweep with left hand	* Move considerably to the left.
* slow sweep with left hand	* Move slowly to the left.

* Left arm extended	* Move slowly to move to the left.
* Left arm up & moved to the left	* plumb the rod to the left.
* Both hand above head & then brought down.	* Correct.
* Both arms extend forward horizontally & the hands depressed briskly.	* Fix the rod.

* phases of survey work :-

A survey work has the following phases :

planning, analysis and decision making, it involves the following.

1. selection of appropriate method of surveying.
2. selection of instruments and other equipments and
3. selection and fixing of survey stations.

* Surveying accessories :-

* Books - survey of level.
* Detail - poles
* GNSS / GPS accessories
* Height meters
* Laser scanning.
* Measuring tapes
* Moisture meters.
* Ranging poles.

* Measurements :-

⇝ Direct measurements :- Distances is actually in measured in the field using "chain or tape' approximate method' used in Reconnaissance survey.

Are pacing, passometer, pedometer, odometer, measuring wheel and speedometer (speed and distance).

* chaining is the most accurate method of making direct measurements.

→ Chains :- Made of galvanised mild steel links.

* Length of chain is measured from the outside of one handle to outside of other hands.

1. Metric chains - 20m and 30m

→ 20m chain - 100 links

→ 30m chain - 150 links.

2. Gunters chain - 66 feet - 100 links

3. Revenue chain - 33 feet - 16 links

4. Engineer's chain - 100 feets - 100 links.

5. Steels - made of blue steel - 20 (or) 30 cm.

→ Each link 20cm - accurate than chain.

→ Arrows :- 40 cm long (IS code)

* 4 mm dia.

* Minimum 10 arrows with a chain.

→ Wooden pegs :- To make the position of stations on terminal points of a survey line.

→ Ranging rods :- 2m to 3m - to range some intermediate points - 3cm Nominal dia.

Scanned by CamScanner

⇒ Ranging poles rods :— length 4 to 8m - dia 60mm to 80 mm - Generally of steel conduct pipes.

⇒ Offset rods :— Measuring Rough offsets - 3m length - wooden.

⇒ Butt rod :— to measure offsets - used by building surveyors or Architects.

⇒ plumb bob :— Centering - to transfer points to the ground - ranging poles vertical.

* linear measurements :—

Linear measurements are carried out for finding out measurements.

In horizontal plane, various methods of making linear measurement are,

1. Direct methods.
2. optical methods.
3. Electronic methods.

⇒ Direct methods :— In the case of direct measurements, the distances are actually measured on the ground with the help of a chain or tape.

ms Various method discussed below are used for direct measurement of distance.

* Pacing :- This method for approximate distance measuring. In this method the distance between two points can be obtained by counting the number of the paces and multiplying it with average pace length.

Average value of pace length may be taken as 75cm to 80cm.

* Passometer :- It is a small instrument which is fixed to the leg of a person. It counts the number of paces the person has moved its mechanism is operated by the motion of the body.

The distance b/w two points can be obtained as explained in the ~~bel~~ previous method.

* Pedometer :- It is an instrument similar to passometer, it is adjusted according to the pace length of the person wearing it.

It recording the distance travelled by the person directly.

* Odometer :- It is a small instrument which can be attached to the wheel of any vehicle, it registers the number of revolutions of the wheel.

The distance traversed can be obtained by multiplying the number of revolution with the circumferece of the wheel.

* Speedometer :- It is an instrument fixed with automobiles.

It shows the speed and distance travelled by the vehicle. It gives better result than pacing.

* Time measurement :- This is rough method of determining distance time interval of travel of a person or an animal can be multiplied with the average speed of the person or animal to get approximate distance travelled.

* perambulator :- It resembles a single bicylcle wheel provided with forks and a handle.

The distance traversed One the dial.

* Problems :- (chain).

1. The length of a survey line measured with a 20m chain was found to be 250m. If the chain was 10cm too long, then true length of the line

Given,

L = designated length of the chain

$(L) = 20\ m$

L' = incorrect length of the chain.

$L' = 20 + 0.10 = 20.10\ m$

l = True length of the chain.

l' = measured length of a line = 250m

$$l = l' \times \left(\frac{L'}{L}\right)$$

$$= 250 \times \left(\frac{20.10}{20}\right)$$

$$= 251.25\ m$$

3. A 20m chain was found to be 20.10 m at the beginning and 20.30m at the end of the work. The Area of the field draw to a scale 1cm = 8m was found to be 32.56 cm² by planimeters. Find the true are of the field in hectares.

Scale of map : $1cm = 8m$

$\therefore 1cm^2 = 8m \times 8m = 64 \, m^2$

$\therefore$ Measured area of field $= A'$

$= 32.56 \, cm^2 = 32.56 \times 64$

$= 2083.84 \, m^2$.

Average length of the chain $= L'$

$= \dfrac{20.10 + 20.30}{2}$

$= 20.20 \, m \rightarrow L = 20m$.

$\therefore$ True area of the field $= A$.

$A = A' \times \left(\dfrac{L'}{L}\right)^2$

$= 2083.84 \times \left(\dfrac{20.20}{20}\right)^2$

$A = 2125.72 \, m^2$

$\therefore$ True area $= \dfrac{2125.72}{10^4} = 0.2125$ hectares.

9. A 20m chain found to be 15cm too long at the beginning of work and it was found to be 5cm too short at the end of the work. If the distance measured by this chain, the distance was 1km. Find the true distance.

Initial length of the chain = 20.15 m

Final length of the chain = 19.95 m

Average length of chain = L'

$$L' = \frac{20.15 + 19.95}{2}$$

$$= 20.05\ m$$

$L = 20\ m$, $L' = 20.05\ m$

$l' = 1km = 1000m.$

$$\therefore\ l = l' \times \left(\frac{L'}{L}\right)$$

$$= 1000 \times \left(\frac{20.05}{20}\right)$$

$$l = 1002.5\ m$$

4. A line was measured with a steel tape which is exactly 30m long at 18°c and found to be 452.343 m. The temperature during measurement was 32°c. Find the true length of line. Take co-efficient of expansion of tape per $\alpha = 0.0000117$.

Given data,

length of tape $l = 30m$

co-efficient of thermal expansion

$\alpha = 0.0000117$.

Temperature at which the tape is standardized $T_0 = 18°c$.

Mean temperature in to the field.

During measurement mean temperature $T_m = 30°c$

co-efficient for temperature

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

$$= 0.0000117 \times (32-18) \times 30$$

$$= 0.004914\ m$$

length of the tape at 32°c

$= 30 + Ct$

Measured length of line $= 452.43\ m$.

$\therefore$ True length $= \frac{L'}{L} \times ML = \frac{30.004914}{30} \times 452.43 = 452.4190\ m$.

* Error due to incorrect chain :-

⇒ If chain is too long, measured distance will be less (negative error).

⇒ Therefore positive correction.

⇒ If chain is too short, measured distance will be more (positive error).

⇒ Therefore negative correction.

Let, L = True length of chain

L' = Incorrect length of chain.

l' = measured length of line.

l = Actual or true length of line.

* Prismatic compass :-

A branch of surveying which directions of survey lines are determined with a compass and length of lines are measured with a tape or chain.

⇒ Types of meridians :

* Meridians :- Fixed line of Reference about which directions or angles are measured.

⇝ True meridian :- The line joining true north and true south established by Astronomical observations.

⇝ Magnetic meridians :- Directions shown by a freely floating and balanced magnetic needle free from all other other attractive forces (line passing through magnetic north and south) established by magnetic compass.

⇝ Grid meridian :- For survey of a state the true meridian of central place is taken as a reference meridian for whole state and is called grid meridian.

⇝ Arbitrary meridian :- Meridian taken in any convenient directions towards a permanent and prominent mark or signal.

Used to determine relative directions of various lines in as small traverse or small areas.

* Types of Bearings :-

Bearing is of a line is the angle b/w a meridian and survey line.

⇝ True meridian :- True bearing of a line is horizontal angle b/w true meridian and the line. Also known as Azimuth.

Does not change with time, it is a constant.

⇝ Magnetic bearing :≡ It is a line is the horizontal angle which the line makes with magnetic north changes with time used for small areas measured with a magnetic compass.

⇝ Grid bearing :≡ Grid bearing of a line is the horizon--tal angle with grid meridian.

⇝ Arbitrary bearing :≡ It is of line is the horizontal angle with "Arbitrary meridian."

* Systems of bearings :≡

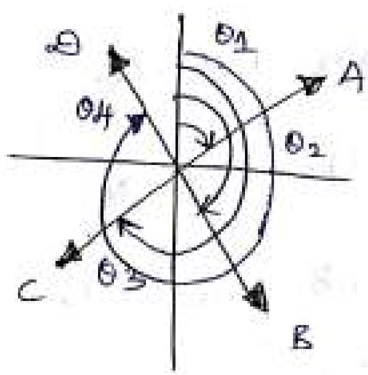
(a) Whole circle bearing system (WCB) :≡

⇝ Bearing of line $OA = \theta_1$

$OB = \theta_2$

$OC = \theta_3$

$OD = \theta_4$



Scanned by CamScanner

* Bearing of a line is measured always in clockwise from 0° to 360°.

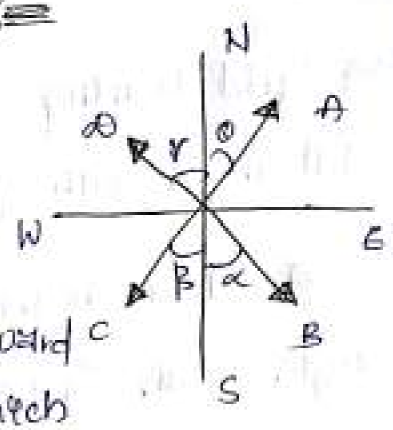
* prismatic compass is graduated in this system.

* Also called "Azimuthal system."

⇝ Quadrantal Bearing (QB) system :-

* Bearing OA = N θ E , OB = S α E

OC = S β W , OD = N γ W



* Bearing of a line is measured eastward or westward from north or south, which ever is nearer.

* varies from 0° to 90°. Observed by surveyor's compass

* Also called "Reduced bearings.".

* Conversion of WCB in to RB :-

Line	WCB b/w	Rule for RB	Quadrant.
AB	0° to 90°	RB = WCB	NE
AC	90° to 180°	RB = 180° - WCB	SE
AD	180° to 270°	RB = WCB - 180°	SW
AF	270° to 360°	RB = 360° - WCB	NW

* Conversion of RB in to W.C.B :-

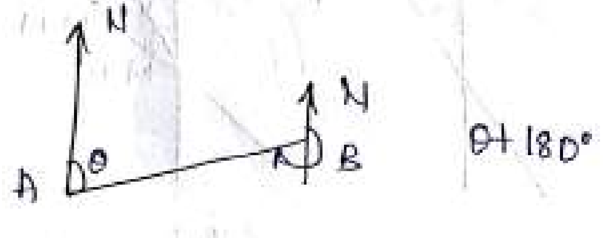
Line	R.B	Rule for WCB
AB	$N\theta_1 E$	WCB = RB
AC	$S\theta_2 E$	WCB = 180 - RB
AD	$S\theta_3 W$	WCB = 180° + RB
AE	$N\theta_4 W$	WCB = 360° - RB.

* Fore bearing and Back bearing :-

⇝ the bearing of a line in in the direction of progress of survey indicated by an arrow is called fore bearing (F.B).

⇝ The bearing in an opposite direction to F.B (or) in the direction opposite to the survey is Back Bearing (B.B)

Ex :-



If FB of AB = 60°
BB of AB = 240°

→ Determination of B.B for F.B :—

T.B and B.B differ by 90°

(i) If F.B is given as W.C.B

$B.B = F.B \pm 180°$ if $F.B < 180°$

and $B.B = F.B - 180°$ if $F.B \geq 180°$

(ii) If F.B of a line is given as quandrantal bearing :—

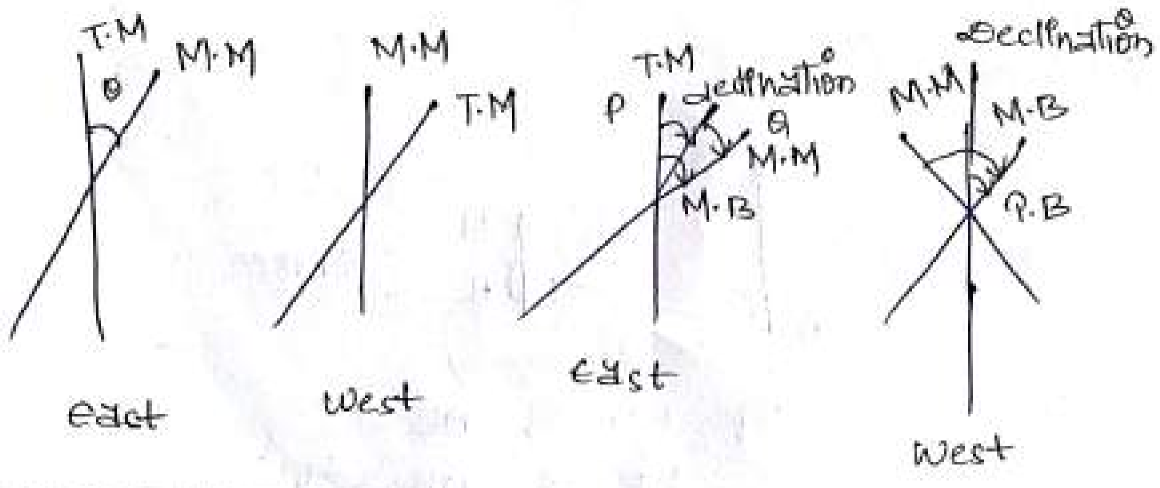
B.B is numerically equal to F.B

change 'N' for 'S' and vice-versa

change 'E' for 'W' and vice versa.

* Magnetic Declination :—

The horizontal angle b/w Two T.M and magnetic meridian.



True Bearing = M.B ± declination

East → + sign (or)

West → - sign.

Magnetic bearing = T.B ± declination

East → - sign

West → + sign.

* problems :-

1. AB 12° 24′ = 192° 24′

2. BC 119° 48′ = 299° 48′

3. CD 266° 30′ = 86° 30′

4. DE 354° 18′ = 174° 30′

⇝ W.C.B.

1. 50° 30′ → WCB = RB

2. 190° 15′ → N 50° 30′ E

3. 100° 30′ → 190° 15° - 180° = 10° 15′

LEVELLING

Levelling:-

Levelling is an art of determining the relatives heigts (or) Elevation of points or objects on earth surface. It deals with measurement in vertical plane.

Levelling is very important branch of Surveying. It forms basics for planning, desiging, Estimating and executing various civil engineering structures, such as roads, railways, canal, dams etc...

Purpose of levelling:-

1. knowing the topography of area and thus to prepare the contour maps.
2. In designing and aligning of highways, railways, Canals, sewers etc...
3. Laying out construction projects, such as water supply, Sanitary (or) drainage schemes.
4. Calculating the quantity of Earthwork, capacity of reservoirs etc... required at various construction projects with longitudinal and cross sections.
5. Fixing the locations of sites of reservoirs, dams, barrages etc.
6. To determine the reduced levels of various components

of civil Engg. Structures and to confirm the levels during construction.

7. To fix up the bench marks on the ground.

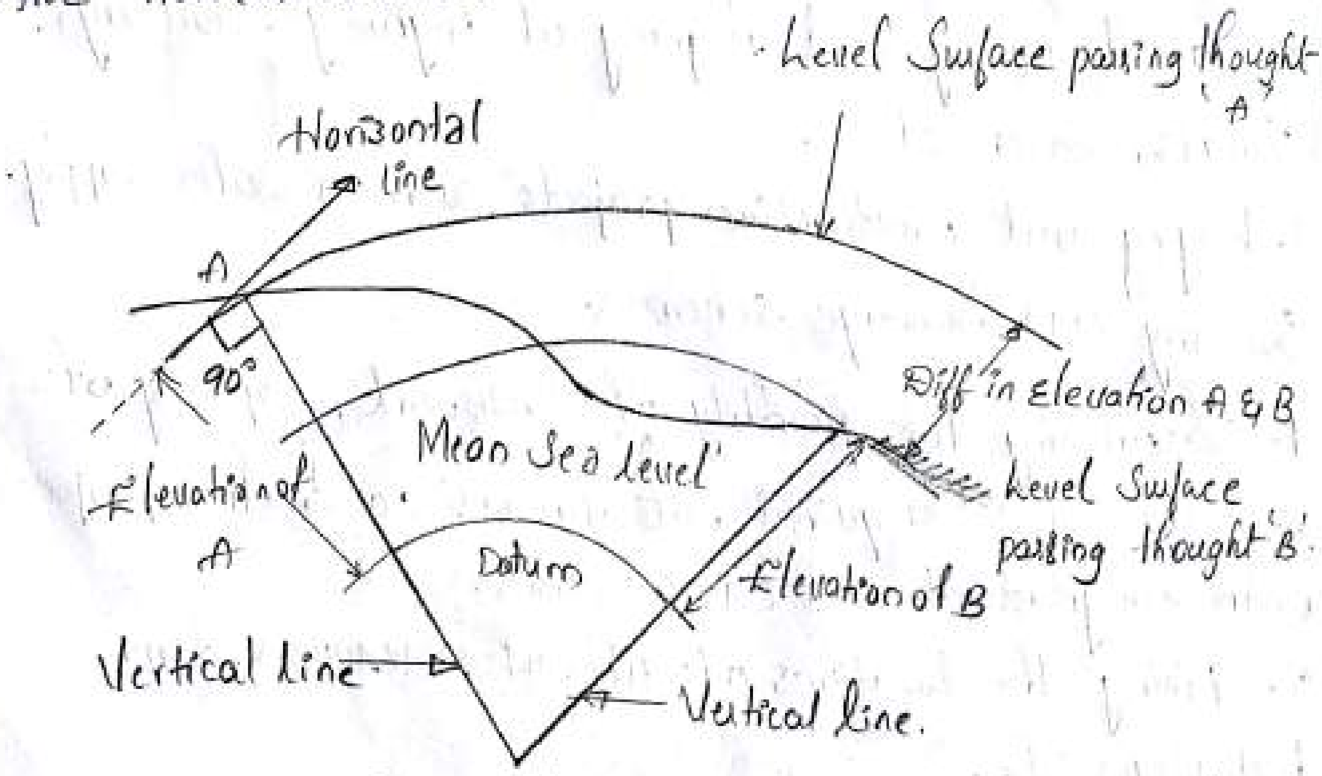
8. To set up the depth of Excavation in foundations of buildings and other structures.

9. To find the Elevation of given points with respect to given or assumed datum.

10. To Establish the elevation of given points at different elevations with respect to given or assumed datum.

Definitions

1. Vertical line:- It follows the direction of gravity at any point on the earth's Surface. It is perpendicular to the horizontal line.



Basic terms of levelling.

2. Horizontal line:- Any Straight line tangential to the level Surface is perpendicular to the vertical line at that point.

3. Level Surface:- It is continuous surface that is perpendicular to the plumb line at that point. It is normal to the plumb line at all points. A large body of still water, unaffected by tidal waves is best example of level surface. For small areas level surfaces is taken to the plane surface.

4. Mean sea level:- The average height of the sea surface for all stages of the tide over a very period (usually 19 years).

5. Datum:- Any level surface to which to which elevations are refered (mean sea level.).

6. Bench mark:- It is a point of known Elevation above or below the datum. It is usually a permanent object.

7. Elevation:- The Elevation of a point is its vertical distance above (or) below datum. It is also known as reduced level. (R.L.). The Elevation of point is +ve (or) -ve according to as the point is above or below the datum.

The difference in Elevation in Elevation 'H' b/w the two points is the vertical distance b/w the level Surface passing through the two points.

Fundamental lines of levelling Instrument.

The fundamental lines of levelling instrument are:-

1. Line of collimation.
2. Axis of bubble tube.
3. Axis telescope.
4. Vertical axis.

1. Line of collimation:- It is an imaginary straight line joining the inter section of crosshairs to the optical centre of the object glass and its continuation. It is also called the line of site.

2. Axis of bubble tube:- It is an imaginary line tangential to the longitudinal curve to the tube at its middle point. It is also known as bubble line. It is horizontal when the bubble is centered.

3. Axis of telescope:- It is an imaginary line joining the centre of the eye piece and the optical centre of the object glass.

4. Vertical axis:- The axis about which the telescope can be turned in a horizontal plane is known as the vertical axis of the instrument. It is the cetre line of the axis of rotation.

Conditions of Adjustment b/w Fundamental lines:-

A define fixed relation must be ensured b/w the fundamental lines of the instrument before an attempt is made to take any staff readings.

the fundamental relations are as follows:-

1. The Axis of bubble tube perpendicular to vertical axis.

2. The line of collimation is parallel to the axis of bubble tube.

Backsight, Intermediate Sight and Fore Sight.

Backsight:- It is the reading taken on a staff held at a point of known elevation.

If the reading is added to the R.L. of the point or staff, the R.L. of the height of the instrument i.e., height of collimation will be obtained.

Hence back sight is considered to be +ve. and it is the first reading taken after the level is set up.

If R.L. of station A is 100.00 and the backsight reading 2.340, then the height of collimation is 100+2.340 = 102.34.

Fore Sight:-

Fore Sight is the staff reading taken on a point whose elevation is not known and has to be determined.

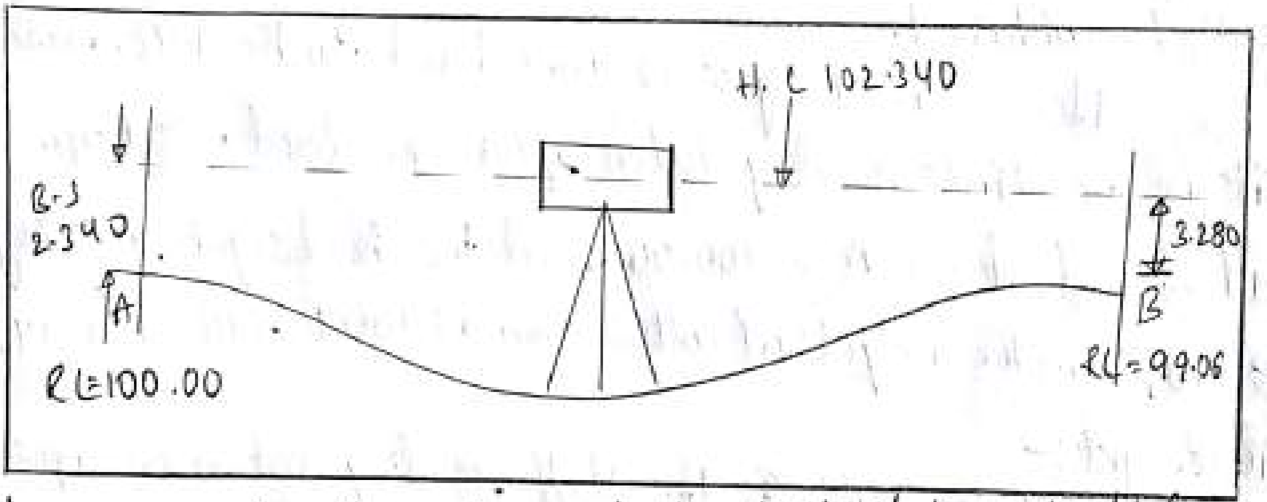
It is the last staff reading before shifting the level to another position.

If the foresight reading is substracted from the H.C the R.L. of that point po is obtained. therefore it is known as (-) sight.

Example:- If the H.C is 102.340 and foresight (F.S) reading on staff held at b is 3.21, then the R.L. of the station 'B' is 102.34 − 3.21 = 99.13.

Intermediate Sight:- Any staff reading other than the B.S and F.S taken on a point where R.L is not known is called I.S.

Example:- If the reading on staff placed at an intermediate station 'c' is 3.80 and the H.C is 102.340 and the R.L. of the station C is 102.340 − 3.280 = 99.060.



Change point:- An intermediate staff station at which both B.S and F.S are taken with the purpose of changing the position of the instrument is called as change point.

The elevation of a C.P. should be accurately determined as it is used for future reference and the error affect every succeeding R.L.

Height of Instrument:- When the levelling instrument is properly levelled, the R.L of the line of collimation is known as the height of the instrument. It is obtained by adding the back sight reading to the R.L of B.M. or C.P. on which the staff reading was taken.

Station:- A point whose Elevation is to be determined is called Station. the staff is kept on this point i.e.: A.B.etc.

Parallax:- The apparent movement of the image relative to the cross hair is known as parallax. thus occurs due to the imperfect focusing i.e., when the image is formed by the objective it does not fall in the plane of the diaphragm.

Bench mark (B.M).

A bench mark is a reference point of known R.L. There are four types of bench marks.

1. GTS bench marks. 2. Permanent bench mark.
3. Arbitrary bench mark 4. Temporary bench mark.

1. GTS Bench mark:- These bench marks are established in the course of the great trigonometric survey conducted by Survey of India department. they are established all over the country. their locations and R.L are with reference to the mean sea level at Karachi.

2. Permanent Bench mark:- These bench marks are established by state government department such as irrigation and power, R and B, P.R etc.

On well defined points such as parapet wall of bridge or culvert, corner of building, plinth etc. these are connected to GTS (BMS) and their levels are marked on the points.

3. Arbitrary bench marks:- These are points reference with any assumed level. these are used only for limited purpose.

4. Temporary bench mark:- In a continuous program of levelling work, It is necessary to close a days work on a reference point taken on a permanent location and continue the work for the nextday. Such points of reference for levelling are known as temporary bench mark.

Types of Bench marks (B.M)

GTS B.M | permanent B.M | Arbitary B.M | Temporary B.M.

Principles of levelling

1. Simple levelling:-

(a) Finding the height of the instrument is R.L of line of sight by taking a sight on known bench mark.

(b) Finding how much the next point is below or above the line of sight

Height of instrument = Elevation of A + bench mark.

= 110.240 + 2.450 = 112.690.

R.L of B = Height of instrument − F.S

= 112.690 − 3.380 = 109.310

2. Differential levelling:-

When the difference in Elevation of points far apart is required, then this cannot be found in one setting of the instrument. this is determined by differential levelling.

Differential levelling is done by dividing the distance into stages by changing the points on which the staff is held and difference of levels b/w successive pair of change points is found.

Let A and B two points are very far whose difference in Elevation is required. the distance b/w the points has been divided into 3 parts by choosing two change points.

Let R.L of point A be 100.00.

the height first setting of instrument $= 100 + 2.015$
$= 102.015$.

of the F.S on 1st change point is 1.790.

the R.L of $Cp_1 = 102.015 - 1.790 = 100.225$

Similarly,

RL of $Cp_2 = 100.225 + 2.150 - 2.325 = 100.050$

RL of $B = 100.050 + 1.990 - 1.105 = 100.935$.

Equalizing Backsight and ForeSight lengths.

It is Essential in spirit levelling that the line of collimation is perfectly horizontal when this condition is achieved the true difference of elevation b/o points sighted can be obtained whatever may be the distance of the staff from the instrument.

However, when the telescope is slightly tilted due to the correct adjustment of the instrument by Equalizing the distance of the B.S and F.S from change points intermediate sights get affects.

Level Book:- The levelling field work is entered in small note book called the level book. the pages are ruled with table forms to note the readings and reduced levels.

Instruments used in levelling:

The instruments generally used in direct levelling are:-

(i) A level (levelling instrument) and

(ii) A levelling staff.

Level:-

A level provides a horizontal line of sight from which the heights of different points are determined. A level consists of the following Essential parts.

(a) A telescope to provide the line of sight.

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

(c) A levelling head to bring the bubble in its centre of run.

(d) A tripod to Support the instrument.

Telescopes in levelling Instruments:-

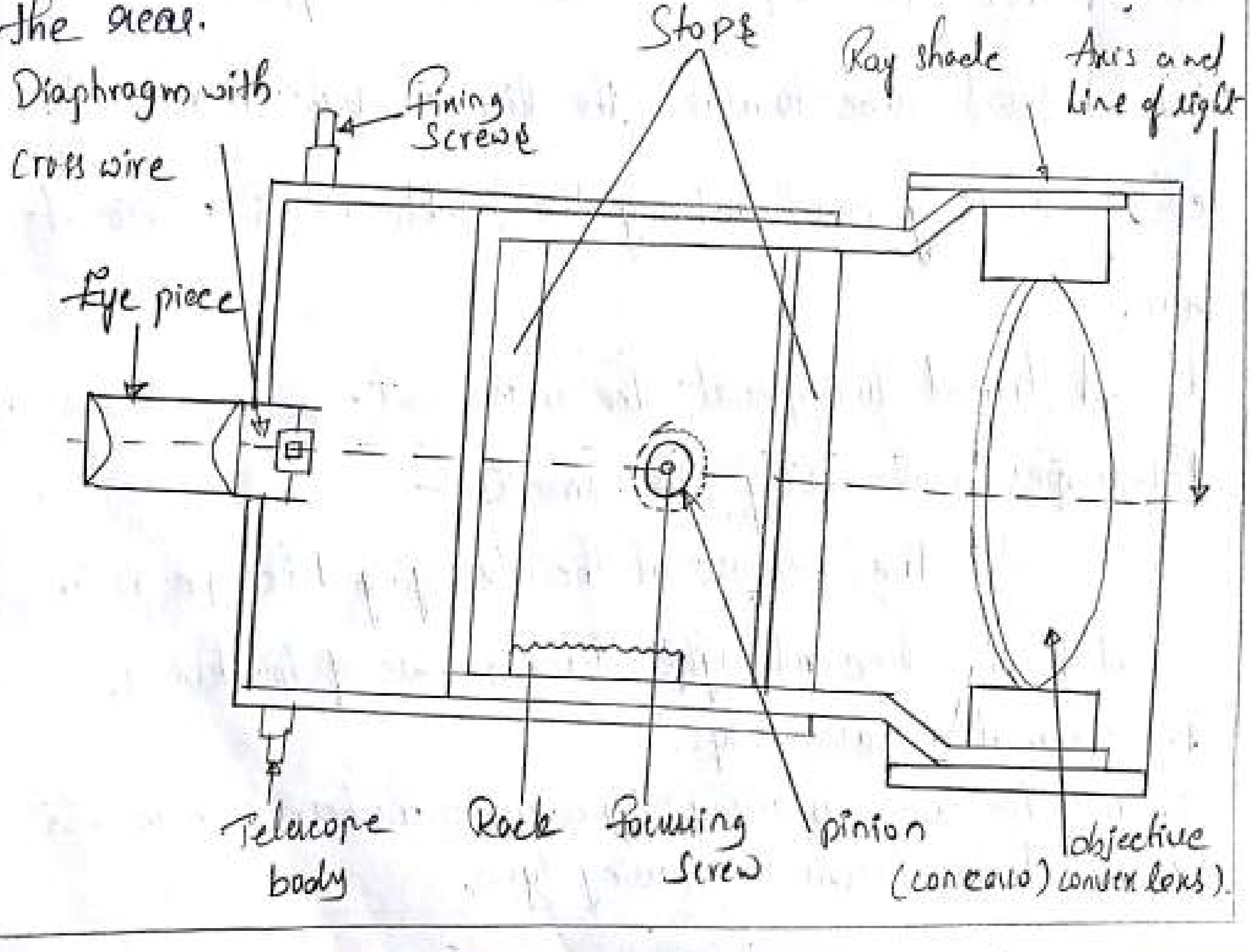
The purpose of the Surveying telescope is to Establish the line of sight. telescope are of two kinds.

1. External focusing type.

2. The internal focusing type. most modern instruments are used the internal focusing type.

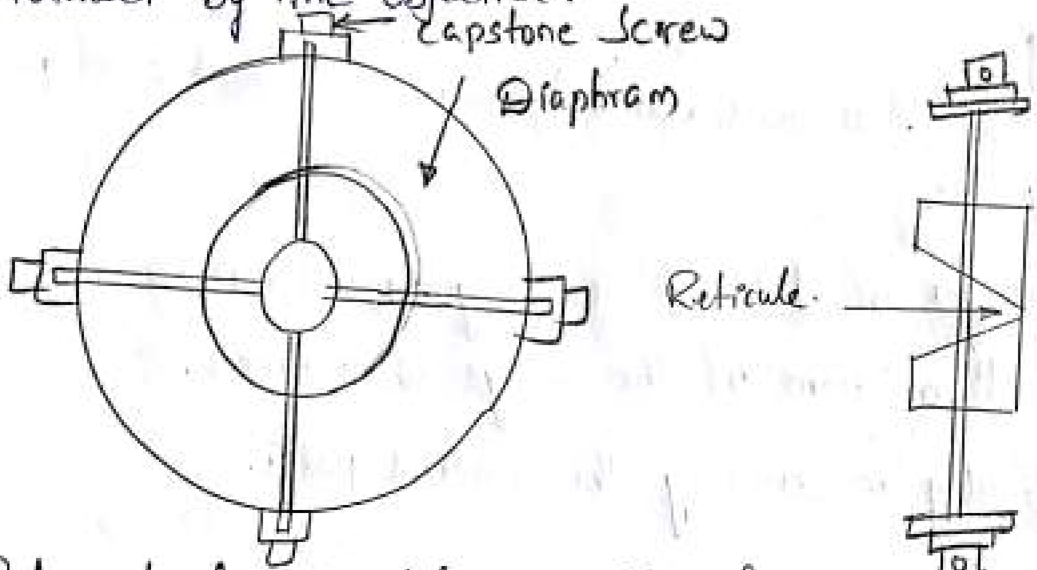
1. External focussing telescopes- A cross section of the External telescope is shown. the telescope consists of a diaphragm ring, an objective lens and an Eye piece. the objective provides a real inverted image of the object sighted at a distance less than its focal length. the Eye piece magnifies it.

The Objective is held in a long metal tube provided with a ray shade and a dust cap. In order to reduce the chromatic and spherical aberration of the objective, it is made of a double convex crown glass lens in the front connected to a concave convex flint glass lens in the rear.



The Eye piece is held in a metal body and consists of two plane convex lenses of Equal focal length with the convex sides inside at a distance of 2/3 focal length.

The diaphorgm ring is circular and made of gun metal. It carries vertical and horizontal cross wires made of silk, Spider thread or platinum wires. these are places in the plane where the vertical image of the object is formed by the objective.



2. Internal focussing telescopes- In this type, the Eye piece and the objective are kept fixed in their respective housings but a double concave lens is mounted on a short tube. which can be moved to and fro b/w the diaphragm and objective by a rack and pinion. Focussing is done with the help of this double concave lens.

Scanned by CamScanner

3. Advantages of Internal focussing telescope:-

(a) As the total length of the telescope remaining the same during focussing, there is no scope for the bubble to get displaced

(b) the telescope gets better protection from dust and moisture.

(c) the line of collimation is less likely to get effected by focussing.

(d) There will less wear and tear on the rack and pinion arrangement.

4. Disadvantages of Internal focussing type telescope:-

(a) The illumination of the image is reduced.

(b) Difficulty in servicing the interior parts.

The level tube:-

The level tube enables Establishing the horizontal plane at the instrument Station. It is also called as the bubble tube or spirit level. It consists of a glass tube Sealed at both ends in the front of an arc of a circle. It is filled with a sensitive liquid like spirit or either and a Small air bubble is Entrapped. It is fixed to the telescope by means of Capstans headed Screws.

The shape of the upper surface of the tube must be a perfect arc of a circle. It is graduated and the divisions make equal angles at the centre at the arc. the zero is placed at the midpoint of the arc. the bubble moves sideways do to change in temperature and inclination of the telescope. when the axis of the bubble is horizontal, the bubble will occupy the central position.

The levelling Head

The levelling head consists of an upper triangular base plate called tribrach with two spirit levels mounted on it. It has three foot screws for levelling it. By manipulating these foot, Screws, the bubble can be made to remain center for all positions of the telescope in the horizontal plane.

Levels and levelling staffs:-

(a) Dumpy level.

(b) Wye level.

(c) Reversible level.

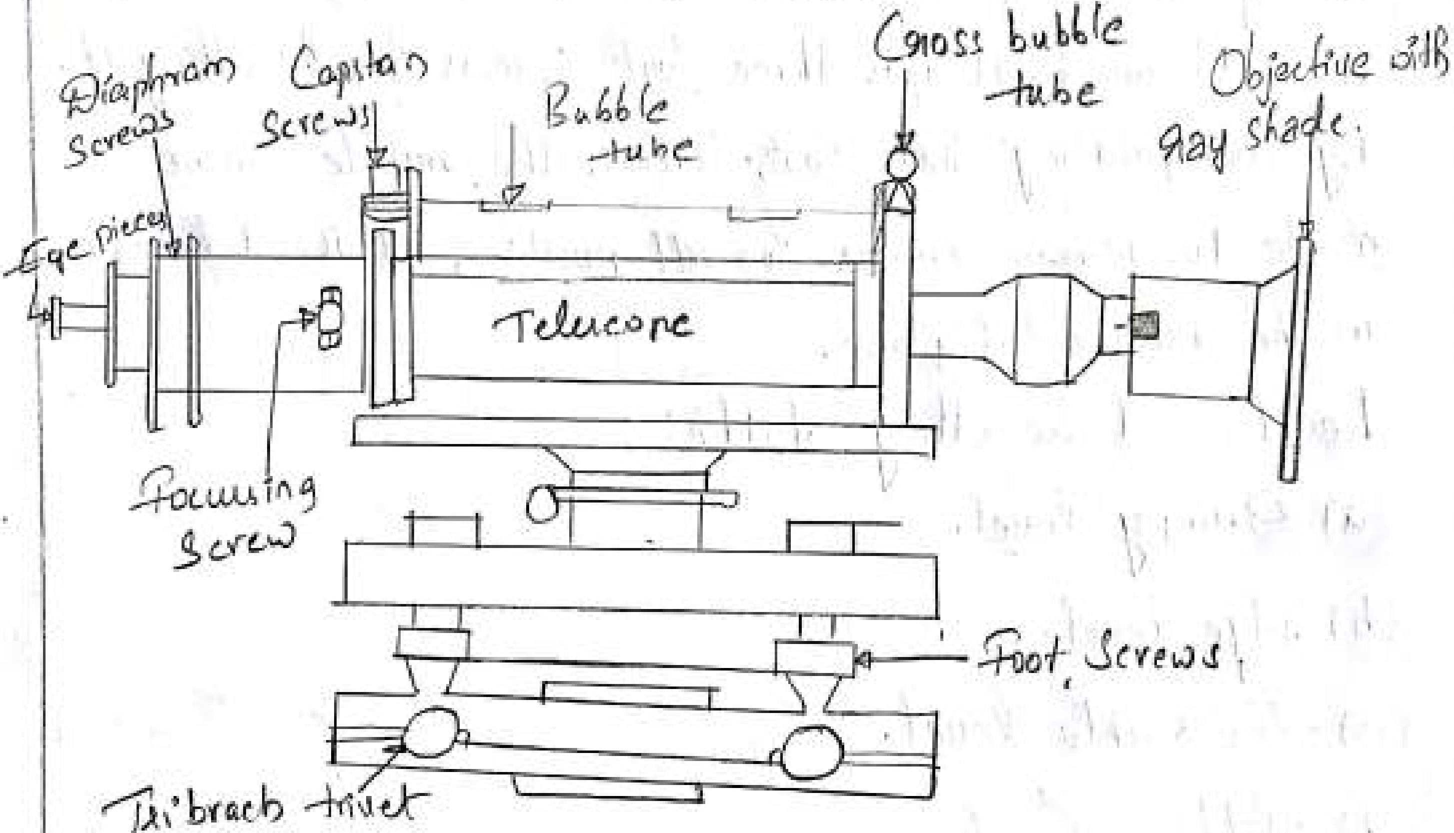
(d) Tilting Level.

(e) Automatic (or) adjusting level.

Dumpy level:-

The dumpy level consists of a telescope supported by a tripod which rotates about a vertical axis and a vertical Spindle. Both are cast iron piece. A long bubble tube is attached to the top of the telescope.

The levelling head consists of two parallel plates with three foot screws. the upper plate is known as a tribrach. the lower plate known as trivet can be screwed on to the tripod. At one end of the telescope is the object glass surrounded by a ray shade. At the other end is eye piece is a brass diaphragm held in the telescope by two brass capstan screws. the other parts are.



Components parts of Dump level :- The tripod stand :- consists of three legs which may be solid or framed. the legs are made of lights and hard wood. the lower ends of the legs are fitted with steel shoes.

2. Levelling head :- The levelling head consists of two parallel triangular plates having three grooves to support the foot screws.

Foot Screws:- three foot screws are provided between the trivet and tribench. By turning the foot screws the tribench can be raised or lowered to bring the bubble to the centre of its run.

Telescope:- The telescope consists of two metal tubes, one moving with in the other. It also consists of an object glass and an eye piece on opposite ends. A diaphragm is fixed with the telescope just in front of the Eyepiece. the diaphragm carries cross hairs. the telescope is focussed by means of the focussing screw and may have either External focussing or internal focussing.

5. Bubble tubes:- Two bubble tubes, one called the longitudinal bubble tube and other the cross bubble tube, are placed at right angles to each other. these tubes contain spirit bubble. the bubble is brought to the center of its run with the help of foot screws. the bubble tubes are fixed on top of telescope.

6. Compass :- A compass is provided just below the telescope for taking the magnetic bearing of a line when required.

Relation between the fundamental lines of a dumpy level:-

1. Line of collimation
2. Axis of bubble tube.
3. Axis of telescope
4. Vertical axis.

1. Line of collimation:- It is an imaginary straight line joining the intersection of the cross hairs at diaphragm to the optical centre of the object glass and its continuation. It is also called the line of sight. When the bubble in the center of its run, the line of collimation will be horizontal.

2. Axis of bubble tube:- It is an imaginary line tangential to the curved surface of the bubble tube at its middle point, it is also known as bubble line. When the bubble is in the centre of its run. the bubble line will be horizontal.

3. Axis of telescope:- It is the imaginary line joining the center of the eye piece and the optical centre of the object glass.

Relationship between the fundamental lines of dumpy level.

1. Axis of bubble tube should be perpendicular to vertical axis.
2. The line of collimation of telescope is parallel to the axis of bubble tube.

Wye level:- The Essential difference b/w the dumpy level and wye level is that in a dumpy level, the telescope is fixed to the spindle while in the wye level, the telescope is carried in two vertical wye supports. These supports

Scanned by CamScanner

consist of curved clips. If the clips are raised, the telescope can be rotated, removed or turned end-for end. when the clips are fastened, the telescope cannot be turned. the bubble tube is attached to the tube or to the stage carrying the wyres.

Reversible Level:- A reversible level combines the good features of both the dumpy level and the wye level. the telescope is supported on two rigid sockets. the telescope is introduced from either end and then fixed in position by means of a screw. the sockets are connected rigidly to the spindle through a stage for testing purposes or for making adjustments.

Tilting level:- The Essential features of a tilting level. in dumpy or wyre levels. when the instrument is levelled, the line of sight becomes truely vertical.

Automatic Level:- It is also known as the self aligning level. its special feature over a dumpy level is the provision of the attachment stabilizer or tilt compensator.

Advantages of Automatic Level:-

1. High speed of levelling and great accuracy.
2. All the disadvantages of non central bubble are eliminated

Levelling staff.

—A levelling staff consists of a straight rectangular piece of well seasoned wood on which graduations are painted. Reading on the staff with a levelling shows height of the station above or below the line of sight of the level.

Levelling staffs are of two classes:-

(i) Self reading staffs (ii) Target staffs.

In the self reading staff, the level-man observes the staff reading where the horizontal wire appears to intersect the face of the rod through the level. the level-man records the readings. the target staff is provided with a vernier which is adjusted by the staffman under the direction of the level man diaphragm coincides with the centre of the vernier.

Self reading staffs are available in three forms:-

1. Sop with telescopic staff.

2. Folding metric staff.

3. Solid staff.

Levelling staves made of aluminium are also used. these are weather resistant & lighter than wooden staves.

1. Sop with telescopic Staff:- It is 4m long when fully Extended. the top length 1.25m is solid and slides into the central box of 1.25m which again slides into the bottom box 1.5m long. the smallest division in this staff is 5mm.

2. Folding metric staff:- It is 75mm wide, 18mm thick and 4m long. It has two lengths of 2m each and connected at the middle by means of a locking device. It can be detached into two pieces or folded into two or can be attached together to form a rigid & Straight staff.

3. Solid staff:- It has only one length and is usually 3m long. It is graduated in divisions of 5mm in the same way as the telescopic metric staff.

4. Target Staff:- This is much used in America. It is graduated like a sop with staff, but is fitted with a sliding vane or target of circular shape. the quadrants are painted black and white alternately round a square central hole through which the staff graduations are visible. the sighting vane is moved up or down until the centre of the target coincides with the horizontal cross hair in the telescope of the level.

Temporary Adjustments of a dumpy level.

Adjustment of levels (levelling instruments).

there are two kinds of adjustment of levels:-

(a) Temporary Adjustment:- The adjustment which are required to be done at each set up of the level are called temporary adjustments such as setting and levelling up the level & focusing the Eye-piece and object glass so, that parallax is eliminated.

(b) Permanent Adjustments:-

There are required to establish a fixed relationship between the fundamental lines of level. These are as under.

(i) The line of collimation should be parallel to the bubble axis.

(ii) The line of collimation and the axis of the telescope should coincide with one another.

(iii) The bubble axis should be perpendicular to the vertical axis, so that the bubble remains in the curve for all directions of the telescope.

* RISE AND FALL METHOD :

station	B.S	I.S	F.S	Rise	Fall	RL	Remarks
1	2.225	—	—			135.750	BM
2	—	1.605	—	0.620		136.370	
3	2.090	—	0.995	0.610		136.980	CP_1
4	—	2.865	—		0.775	136.205	
5	0.600	—	1.265	1.600		137.805	CP_2
6	1.405	—	1.985		1.385	136.420	
7	—	—	2.685		1.280	135.140	
Sum	6.320		6.930	2.830	3.440		

Rise at station-2 = B.S at station ① − I.S at station ②

= 2.225 − 1.605 = 0.620

Rise at station-3 = B.S at station-② − F.S at station ③

= 1.605 − 0.995 = 0.610

Fall at station-4 = B.S at station ③ − I.S at station ④

= 2.090 − 2.865 = 0.775

Rise at station-5 = I.S at station ④ − F.S at station ⑤

= 2.865 − 1.265 = 1.600

Fall at station-6 = B.S at station 5 − F.S at station 6

= 0.600 − 1.985 = 1.385

Fall at station − 7 = B.S at station 6 − F.S at station 7

= 1.405 − 2.685 = 1.280

RL of station−2 = RL of station 1 + Rise at station 2

= 135.75 + 0.62 = 136.37

RL of station−3 = RL of station 2 + Rise at station 3

= 136.37 + 0.61 = 136.980

RL of station − 4 = RL of station 3 − Fall at station 4

= 136.980 − 0.775 = 136.205

RL of station−5 = RL of station 4 + Rise at station 5

= 136.205 + 1.600 = 137.805

RL of station − 6 = RL of station 5 − Fall at station 6

= 137.805 − 1.385 = 136.420

RL of station−7 = RL of station 6 − Fall at station 7

= 135.14 − 1.28 = 135.140

Check:

$\Sigma B.S - \Sigma F.S$ = Last RL − First RL = ΣRise − ΣFall

6.32 − 6.93 = 135.14 − 135.75 = 2.83 − 3.44

−0.61 = −0.61 = −0.61

HEIGHT OF INSTRUMENT METHOD:

st	B.S	I.S	F.S	HI	RL	Remarks
1	0.215	—	—	100.215	100.00	T.B.M
2	—	1.100	—		99.115	
3	—	1.750	—		98.465	
4	0.950	—	2.385	98.780	97.830	CP_1
5	—	1.425	—		97.355	
6	—	1.950	—		96.830	
7	1.105	—	2.555	97.330	96.225	CP_2
8	—	2.055	—		95.275	
9	—	2.730	—		94.600	
10	—	3.325	—		94.005	
11	—	—	3.950		93.380	
sum	2.270		8.890			

H.I at station ① = B.M of RL + B.S at station ①

= 100 + 0.215 = 100.215

RL of station ② = H.I at station ① − I.S at station ②

= 100.215 − 1.100 = 99.115

RL of station ③ = H.I at station ① − I.S at station ③

= 100.215 − 1.750 = 98.465

RL of station ④ = H.I at station ① − F.S at station ④

= 100.215 − 2.385

= 97.830

H.I at station 4 = RL of station 4 + B.S at station 4

= 97.830 + 0.950 = 98.780

RL of station 5 = H.I at station 4 − I.S at station 5

= 98.780 − 1.425 = 97.355

RL of station 6 = HI at station 4 − I.S at station 6

= 98.780 − 1.950 = 96.830

RL of station 7 = H.I at station 4 − F.S at station 7

= 98.780 − 2.555 = 96.225

H.I station 7 = RL of station 7 + B.S at station 7

= 96.225 + 1.105 = 97.330

RL of station 8 = H.I at station − IS at station 8

= 97.330 − 2.055 = 95.275

RL of station 9 = HI at station − IS at station 9

= 97.330 − 2.730 = 94.600

RL of station 10 = HI at station − IS at station 10

= 97.330 − 3.325 = 94.005

RL of station 11 = HI at station − F.S at station 11

= 97.330 − 3.950 = 93.380

<u>check:</u>

$\Sigma B.S - \Sigma F.S$ = last RL − first RL

2.27 − 8.89 = 93.380 − 100.00

−6.62 = −6.62

Scanned by CamScanner

H.I at station ④ = RL of station ④ + B.S at station ④

= 99.390 + 2.450

= 101.840

RL of station ③ = RL of station ④ + Fall at station ④

= 99.390 + 3.175

= 102.565

RL of station ② = RL of station ③ − Rise at station ③

= 102.565 − 0.825

= 101.740

RL of station ① = RL of station ② − Rise at station ②

= 101.740 − 1.035

= 100.705

H.I at station ① = RL of station ① + B.S at station ①

= 100.705 + 2.485

= 103.190.

check:

$\Sigma B.S - \Sigma F.S$ = Last RL − first RL = Σ Rise − Σ Fall

6.190 − 6.895 = 100 − 100.705 = 3.665 − 4.370

−0.705 = −0.705 = −0.705

INCLUDED ANGLE:

Line	F.B	WCB	B.B
AB	N 50°E	50°	50 + 180 = 230°
BC	N80°E	80°	80+180 = 260
CD	S 40°W	220°	220−180° = 40°
DA	N 70°W	290°	290−180° = 110

B.B = F.B ± 180°

∠A = F.B of AB − B.B of AD

= 50° − 110° = −60 + 360° = 300°

∠B = F.B of BC − B.B of AD

= 80° − 230°

= −150° + 360° = 210°

∠C = F.B of CD − B.B of BC

= 220 − 260

= −40 + 360° = 320°

∠D = F.B of DA − B.B of CD

= 290 − 40 = 250°

∠A + ∠B + ∠C + ∠D = 300° + 210° + 320° + 250° = 1080

check:

$(2n+4) \times 90 = (2 \times 4 + 4) \times 90 = 1080.$

St	B.S	I.S	F.S	Rise	Fall	H.I	R.L	Remarks
1	2.485					103.190	100.705	CP
2		1.450		1.035			101.740	
3		0.625		0.825			102.565	
4	2.450		3.80		3.175	101.840	99.390	CP_1
5		2.155		0.295			99.685	
6		1.945		0.210			99.895	
7	1.255		0.645	1.30		102.450	101.195	CP_2
8			2.450		1.195		100.00	
Sum	6.190		6.895	3.665	4.370			

I.S at station ② = B.S at station ① − Rise at station ②

$= 2.485 - 1.035 = 1.450$

Rise at station ③ = IS at station② − ~~Rise~~ I.S at station③

$= 1.450 - 0.625 = 0.825$

F.S at station ④ = I.S at station ③ + Fall at station ④

$= 0.625 + 3.175 = 3.800$

Rise at station ⑤ = B.S at station ④ − I.S at station ⑤

$= 2.450 - 2.155 = 0.295$

Rise at station ⑥ = I.S at station ⑤ − I.S at station ⑥

$= 2.155 - 1.945 = 0.210$

Rise station 7 = I.s at station 6 − F.s at station 7

= 1.945 − 0.645 = 1.300

Fall station 8 = B.s at station 7 − F.s at station 8

= 1.255 − 2.450

= 1.195

H.I at station 7 = RL of station 8 + F.s at station 8

= 100.00 + 2.450

= 102.450.

RL of station 7 = HI at station 7 − B.s at station 7

= 102.450 − 1.255

= 101.195

RL of station 6 = RL of station 7 − Rise at station 7

= 101.195 − 1.3

= 99.895

RL of station 5 = RL of station 6 − Rise at station 6

= 99.895 − 0.21

= 99.685

RL of station 4 = RL of station 5 − Rise at station 5

= 99.685 − 0.295

= 99.390

CONTOURS

A Contour may be defined as an imaginary line passing through points of equal elevation or R.L / line of intersection of a level line with the ground surface

A map showing only the contour lines of an area is called contour map.

The top surface of still water in a pond is a level surface. The intersection of this level surface with the banks represents a contour line

Uses Of Contours:

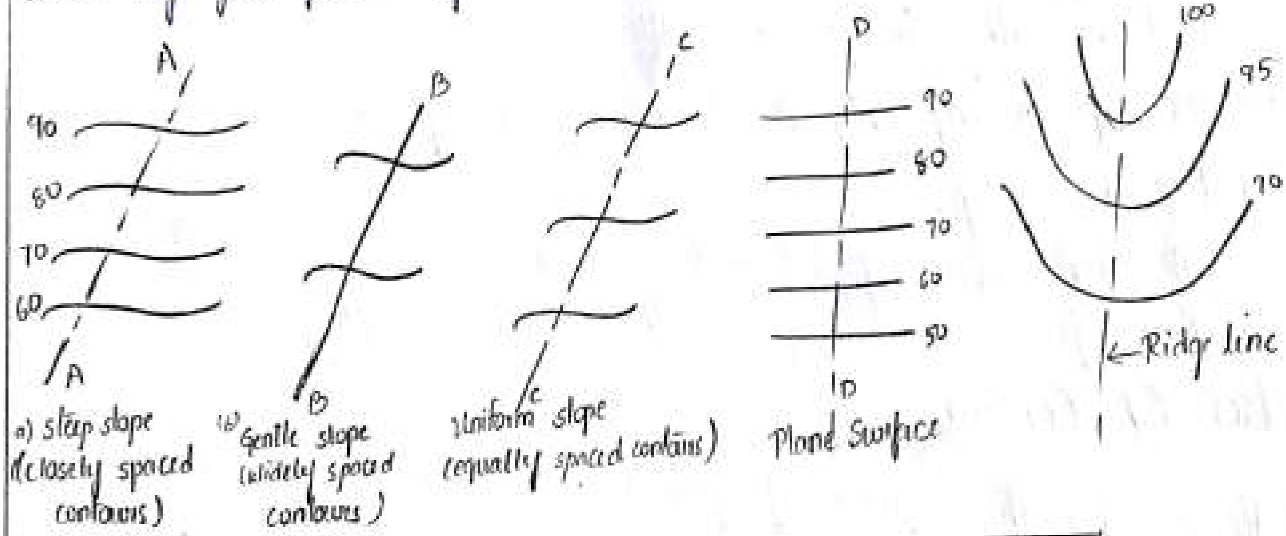
1. To Decide the nature of ground surface, proper location of engg projects such as roads, railways, canals, sewers, transmission lines, industrial plant, dam and reservoir, bridges, buildings etc
2. Contour maps are useful in planning and designing of imp engg. projects.
3. Quantity of cutting, filling and earthwork involved in any project

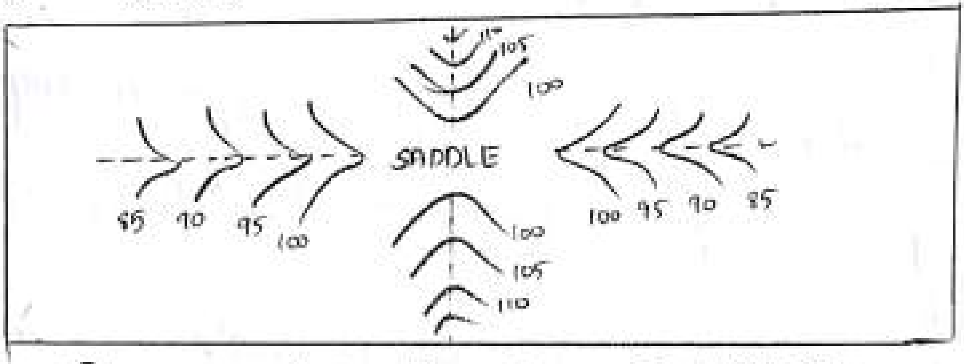
CHARACTERISTICS OF CONTOURS:

1. All the points on the contour line have the same elevations
2. Two contour lines of diff elevations do not cross each other. However Two contour lines of diff elevations can intersect in case of overhanging cliff,
3. Closed contour lines with lower values inside indicate a depression
4. closely spaced contour lines indicate steep slope., When contour lines are spaced far apart, it indicates gentle slope, Uniformly spaced indicate Uniform slope, A series of straight, parallel and equally spaced contour lines represent a plane surface
5. Contour lines crosses a ridge line at right angles. For ridge line, the higher elevation contours are inside the loop.

7. Contour lines crosses a valley line at right angles. For valley line the higher elevation contours are outside the loop

8. A lower region surrounded by hills is called saddle. It is represented by four pairs of contours





CONTOUR INTERVAL AND HORIZONTAL EQUIVALENT

Contour interval : The vertical distance b/w any two consecutive contours is known as contour interval

Contour interval depends upon the following

i) The nature of the ground
ii) The scale of the map
iii) The purpose and the extent of the survey
iv) Time and expense of field and office work

S.No	Purpose of Survey	Scale	Contour interval
1	Building sites	1cm = 10m or less	0.2 to 0.5
2	Town planning	1cm = 50m to 100m	0.5 to 2.0
3	Location Survey, Earthwork	1cm = 50m to 200m	2 to 3

Horizontal equivalent

The horizontal distance b/w any two consecutive contours is known as horizontal equivalent

It varies according to the steepness of the ground. If slope of the ground is steep, its horizontal equivalent will be smaller and vice versa

CONTOUR GRADIENT

The term contour gradient is used to indicate a line lying on the surface of the ground with uniform inclination to the horizontal at all points on it.

METHODS OF LOCATING CONTOURS:

There are two methods of locating contours.

a) Direct method b) indirect method

a) Direct method

In this method the contour to be plotted is actually traced on the ground.

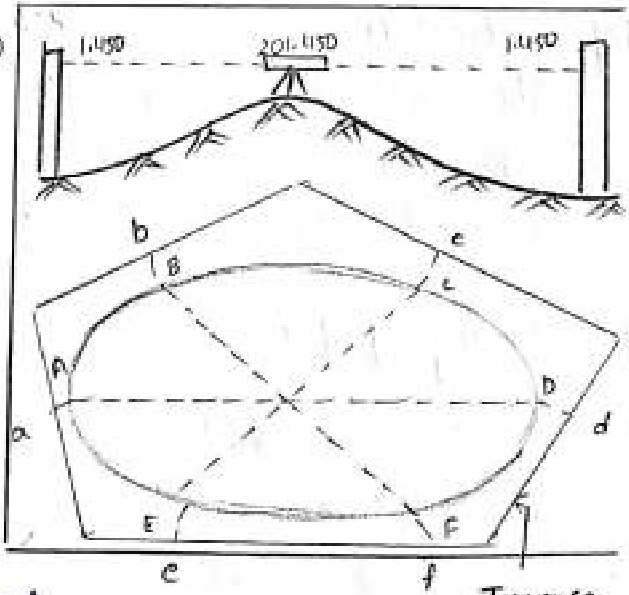
To establish the points of equal elevation on the ground, the instrument is set up at L_1.. The it is levelled staff held on BM and BS is taken

$\therefore$ H.I = R.L of BM + BS

H.I = 201.45 m

We want to draw a contour line of R.L. 200m. The level man direct the staff man to hold staff at 1.45m

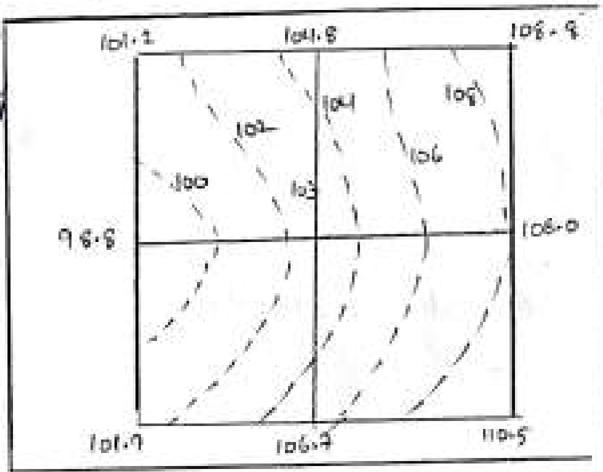
By joining staff readings 1.45m, a contour of RL 200m is traced on the ground



Indirect Method

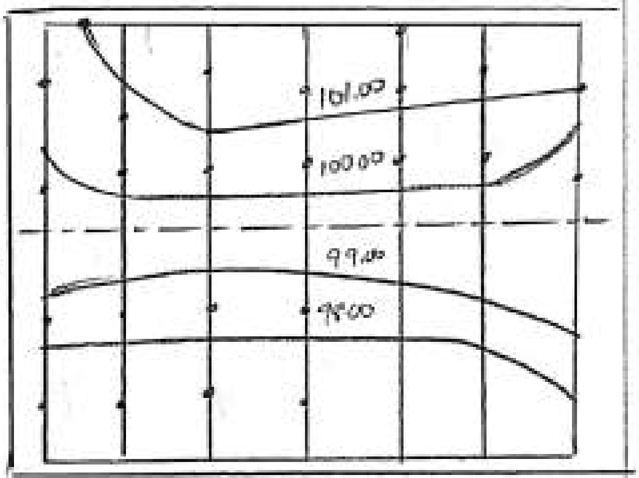
1. Methods of squares 2. Method of Cross-sectioning 3. Method of tacheometry.

1) Methods of squares



This method is employed when ground is small and flat. In this area is divided into grid or series of squares. The grid size may vary from 5m x 5m to 25m x 25m depending on the ground. The grid corners are marked on the ground. Normal method of levelling using a level. The grid is plotted to the scale of the map and the spot levels of the grid corners are ~~marked~~ entered

Method of Cross-Sectioning:



The process of locating the contours proportionately b/w the plotted points is termed as interpolation of contours.

1. By estimation 2) By Arithmetic 3) Tracing

By estimation

The points on the equivalent required contour are located by eye judgement. This method is suitable for small scale maps. It is assumed that slope b/w the ground points is uniform

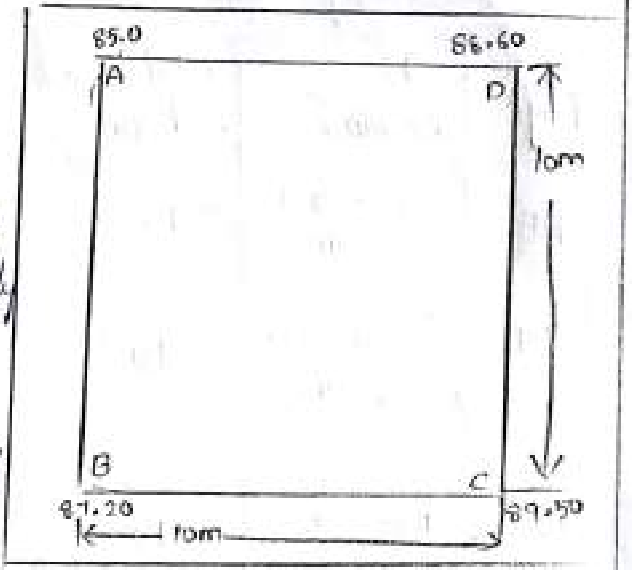
By Tracing paper:

This is a graphical method of interpolation of contours. Tracing paper is used to locate the points on contour.

There are two methods

1. First method - Parallel line method 2. Radial line method

By Arithemetic calculations :



This method is used when high accuracy is required and scale of the map is large

In this method, the distance b/w two points is known elevations are accurately measured. Then with the help of arthematic calculations, the position of the required elevations points are computed

Example: For a square of 10m x 10m R.L of corners are given. Draw contour lines at 1m interval

For edge AB:

B/w 85.0m and 87.20m there will be two contours of 86.0m & 87.0m

Let's calculate horizontal distance of 86.0m contour from A

87.20 - 85.0 = 2.20m

2.20m (VD) = 10m (HD)

1.0m (VD) = ?

$\frac{10 \times 1}{2.20}$ = 4.55m (constant)

Horizontal Distance of 87.0m contour from A

$= \frac{10 \times 2}{2.20} = 9.10m$

For edge AD

B/w 85.0 & 88.60 m there will be 3 contours 86, 87 & 88m

88.60 - 85.0 = 3.60m

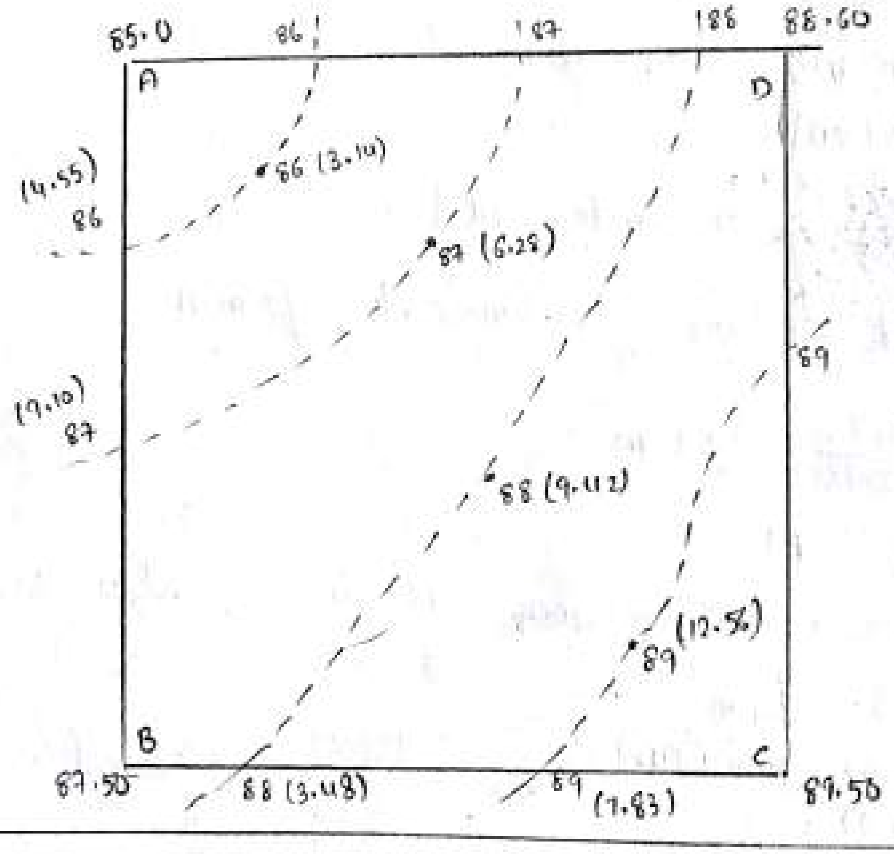
3.60m (V.D) = 10m (HD)

1.0m (V.D) = ?

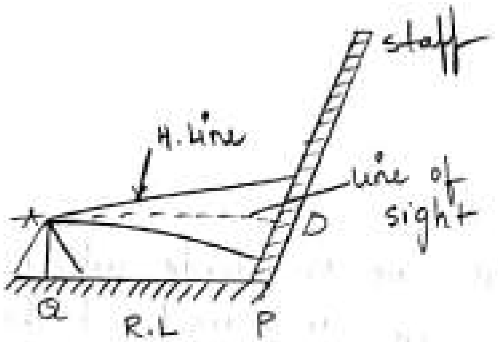
$\Rightarrow \frac{10 \times 1}{3.60}$ = 2.78 m (constant

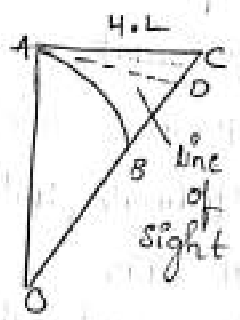
Distance of 87m contour from A = 2 X 2.78 = 5.56 m
Distance of 88m contour from A = 3 X 2.78 = 8.34m

Edge	RL Distance	Total horizo- dist. (m)	constant	Hori. dist. of contour (m)	Contour
AB	87.20 - 85.0 = 2.20m	10	$\frac{10}{2.20} = 4.55$	4.55 X 1 = 4.55 4.55 X 2 = 9.10	86 87
AD	88.60 - 85.0 = 3.60m	10	$\frac{10}{3.60} = 2.78$	2.78 X 1 = 2.78 2.78 X 2 = 5.56 2.78 X 3 = 8.34	86 87 88
BC	89.50 - 87.20 = 2.30m	10	$\frac{10}{2.30} = 4.35$	4.35 X 0.8 = 3.48 4.35 X 1.8 = 7.83	88 89
DC	89.50 - 88.60 = 0.9m	10	$\frac{10}{0.9} = 11.11$	11.11 X 0.4 = 4.44	89
AC	89.50 - 85.0 = 4.50m	$\sqrt{10^2+10^2}$ = 14.14	$\frac{14.14}{4.5} = 3.14$	3.14 X 1 = 3.14 3.14 X 2 = 6.28 3.14 X 3 = 9.42 3.14 X 4 = 12.56	86 87 88 89



Correction for Curvature of Earth and Refraction :-





∴ Curvature correction (-ve)

$\angle COA = 90^\circ$

$\therefore OC^2 = OA^2 + AC^2$

$BC = BC =$ Correction for curvature

$AB = d =$ Horizontal distance b/w A and B

$AO = R =$ Radius of the Earth

$(R + C_c)^2 = R^2 + d^2$

$R^2 + C_c^2 + 2RC_c = R^2 + d^2$

$C_c(2R + C_c) = d^2$

$$\boxed{C_c = \frac{d^2}{2R + C_c}}$$

$C_c = 0.785$

$R = 6370$ km

$C_R = \frac{1}{7} \times \frac{d^2}{2R}$

$C_R = 0.0112 d^2$

Combined correction for C_c and $C_R = C_c - C_R$

$C = 0.0785 d^2 - 0.0112 d^2$

$C = 0.0673 d^2$

⇒ Mid ordinate rule

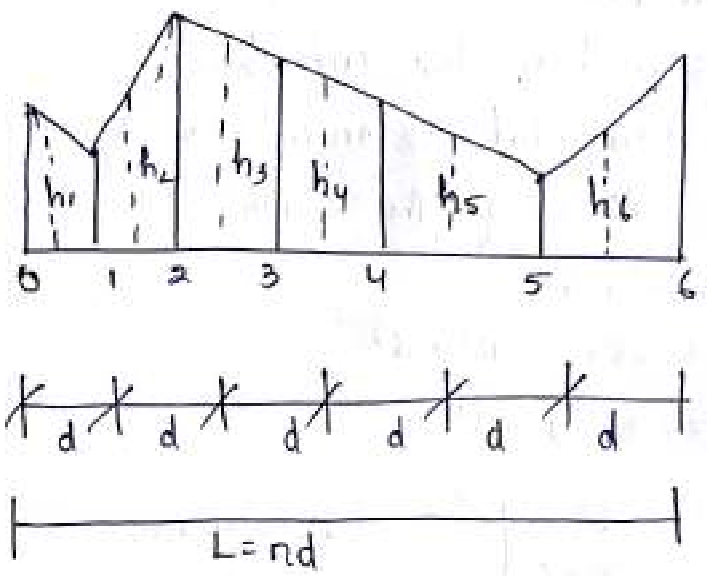
⇒ Average ordinate rule

⇒ Trapezoidal rule

⇒ Simpson's rule

Mid * Ordinate * rule :-

In this method the base line is divide into no. of divisions and the ordinates are measured at the mid point of Each divisions. The boundaries b/w the offsets are considers straight lines



$$\text{Area} = \frac{h_1 + h_2 + h_3 + - - - - - - - + h_n}{n} \times L$$

$$= \frac{h_1 + h_2 + h_3 + - - - - - - - - - + h_n}{n} \times nd$$

$$= (h_1 + h_2 + h_3 + - - - - - - - - + h_n) \times d$$

d = distance of Each division

L = length of base line = nd

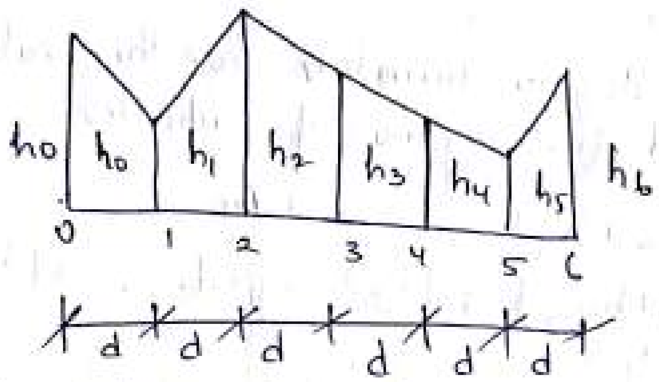
n = no of divisions

2) Average * ordinate * rule :-

This rules are also assumes the boundaries b/w the extremities of the ordinates are straight line

$$A = \frac{h_0 + h_1 + h_2 + h_3 + \text{----} + h_n}{n+1}$$

$$= \frac{h_0 + h_1 + h_2 + h_3 + \text{---} + h_n}{n+1} \times nd$$



d = distance of each division

L = length of base line = nd

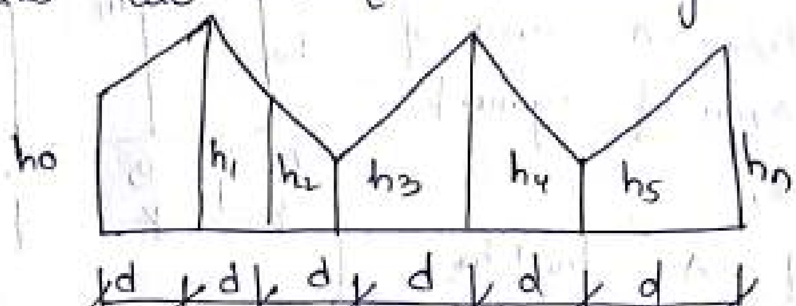
n = no. of divisions.

Where h_0, h_1, h_2, h_3 ---- = ordinates or offsets

n+1 = No of offsets

3) Trapezoidal * rule :-

In this method the entire area is divided into of no of trapezoidals. This rule is more accurate than the previous two rules. The boundaries b/w the ordinates are assumed to straight



let h_1, h_2, h_3 --- h_n be the ordinates at equal interval

d = common distance

1^{st} area $= \frac{h_0+h_1}{2} \times d$

2^{nd} Area $= \frac{h_1+h_2}{2} \times d$

3^{rd} Area $= \frac{h_2+h_3}{2} \times d$

last Area $= \frac{h_{n-1}+h_n}{2} \times d$

Limitation :- There is no limitation for this rule
This rule can be applied for any no. of ordinates

Total area $= A_1 + A_2 + A_3 + ---- + A_n$

$$= \left(\frac{h_0+h_1}{2}\right) \times d + \left(\frac{h_1+h_2}{2}\right) \times d + \left(\frac{h_2+h_3}{2}\right) \times d --- + \left(\frac{h_{n-1}+h_n}{2}\right) d$$

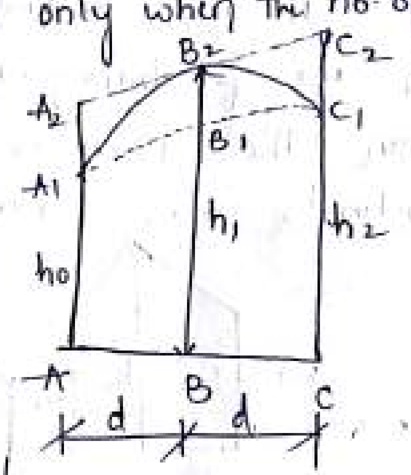
$$= d/2 \left[(h_0+h_n) + 2(h_1+h_2+h_3 + --- h_{n-1}) \right]$$

$A = \frac{\text{Common distance}}{2} \left[(1^{st} \text{ \& last ordinates}) + 2(\text{remaining ordinates}) \right]$

4) Simpson's rule :-

This rule assumes that the short length of boundary b/w the ordinates are parabolic arcs.

Limitation :- This rule is applicable only when the no. of ordinates are odd



let h_0, h_1, h_2 be the ordinates

area of $AA_1B_2C_1CA$ = area of trapezium $AA_1 +$ Area of segment $A_1B_2C_1B_1A_1$

Area of Trapezium, $A_1 = \frac{h_0+h_2}{2} \times 2d$

Area of segment, $A = \frac{2}{3} \times$ Area of parallelogram $A_1A_2C_2C_1$

$$= \frac{2}{3} \times A_1A_2B_1B_2C_2C_1$$

$$= \frac{2}{3} \times B_1B_2 \times 2d$$

$$= \frac{2}{3} \times \left[h_1 - \frac{h_0+h_2}{2}\right] \times 2d$$

$\therefore$ Area b/w the first two divisions

$$A_1 = \frac{(h_0+h_2)}{2} \times 2d + \frac{2}{3}\left[h_1 - \frac{h_0+h_2}{2}\right] \times 2d$$

$$= \frac{d}{3}\left[h_0 + 4h_1 + h_2\right]$$

Similary, the area b/w next 2 divisions

$A_2 = d/3 \left[h_2 + 4h_3 + h_4\right]$ ---- and so on

$\therefore$ total area $= A_1 + A_2 + A_3 + \cdots A_n$

$$= d/3 \left[h_0 + 4h_1 + 2h_2 + 4h_3 + 2h_4 + \cdots\right]$$

$$= d/3 \left[(h_0 + h_n) + 4\left[h_1 + h_3 + \cdots + h_{n-1}\right) + 2\left(h_2 + h_4 + \cdots - h_{n-2}\right)\right]$$

$$\therefore A = d/3 \left[(h_0 + h_n) + 4\left(h_1 + h_3 + \cdots h_{n-1}\right] + 2\left(h_2 + h_4 + \cdots h_{n-2}\right)\right]$$

$A = \frac{\text{common distance}}{3} \times$ [(1st + last ordinates) + 2(sum of odd ordinates] + 4 (sum of Even ordinates)]

3. Theodolite Surveying, ~~an~~ Traversing

1. Transit: This is the operation of the ~~revolo~~ revolving the telescope through 180° in a vertical plane about its horizontal axis. It is also called plunging (or) reversing.
2. Face Right: When the vertical circle of a theodolite is on the right of the observer, the position is called face Right.
3. Face left: When the vertical circle of a theodolite is on the left of the observer, the position is called face left
4. Swinging the telescope: Revolving the telescope in the horizontal plane, about its vertical axis is called swinging.
5. Telescope normal: The telescope is said to be normal (or) direct when its vertical circle is to the left of the observer and the ~~ut~~ bubble is up.
6. Telescope inverted: The telescope is to be inverted when its vertical circle is to the right of the observer and the bubble is down.
7. Horizontal axis: It is the axis about which the telescope can be rotated in a vertical plane.
8. vertical axis: It is the axis about which the telescope can be rotated in a Horizontal plane.
9. Axis of telescope: It is the line Joining the optical centre of the object glass to the centre of the Eyepiece

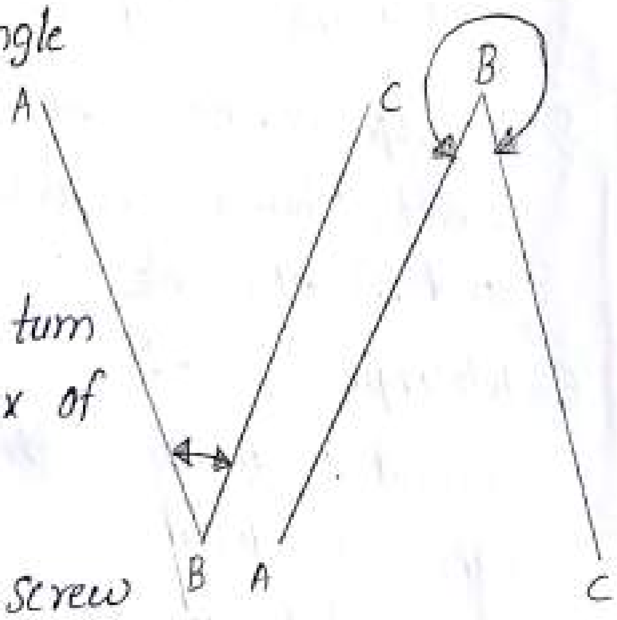
10 Line of Sight: It is an imaginary line Joining the intersection of cross-hairs to the optical Centre of the objective.

11. Axis of Level tube: It is a line tangential to the longitudinal curve of the level tube at its centre

12. Lining in: It is the process of Establishing Intermediate points with a theodolite on a given straight line whose ends are intervisible.

13. Balancing in: It is the process of Establishing Intermediate points with a theodolite on a given straight line whose Ends are not intervisible.

Measurment of Horizontal Angle



1. Set up the instrument over B and level it

2. Loosen the upper clamp and turn the upper plate until the index of the vernier A.

3. Turn the upper Slow motion screw So as to make the two zeros Exactly coincident

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

5. Bisect station A Exactly by using the lower slow motion Screw. The vertical circle clamp and Slow motion screws are used to achieve this

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

7. unclamp, the upper plate, swing the telescope clockwise and bring the station c in the field of view

8. Read both the verniers. The Reading on vernier A directly gives the value of the angle ABC.

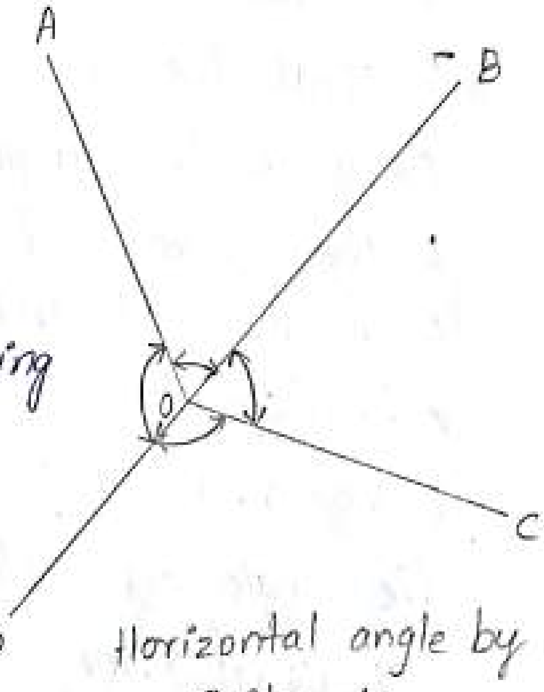
9. change the face of instrument and repeat the procedure. The average of the two values is the required horizontal angle.

Method of Reiteration:

1. To measure angles AOB, BOC, COD and DOA, set up the instrument at O and level it

2. Set the vernier A to read zero using the upper clamp and tangent screw

3. Direct the telescope towards A and bisect it exactly



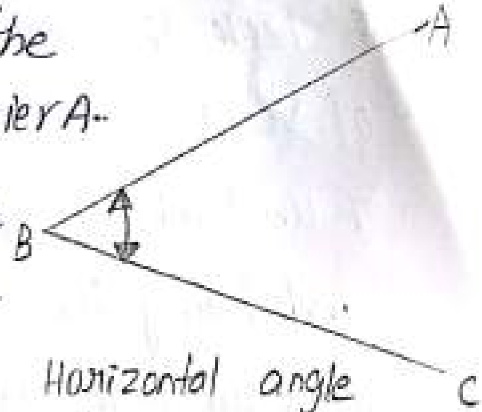
Horizontal angle by Reiteration

4. unclamp the upper plate, swing the telescope clockwise and Bisect B accurately.

5 METHOD OF REPETITION

1. To measure an angle, say ABC, by the method of Repetition set up the instrument at B and level it.

2. Loosen the upper clamp and turn the upper plate until the index of the vernier A.



Horizontal angle by Repetition.

3. unclamp the lower plate and Swing the telescope clockwise and bisect Station A.

4. Read both the verniers. Check the vernier reading

5. Release the upper plate by using the upper clamp and tangent screw.

6- Repeat the process for required number of times, say three times, and find out the value of angle ABC.

7. Repeat the above procedure with the face changed and Calculate the angle ABC.

8. The average of two values of angle ABC. obtained with face left and face right

Advantages:

1. The Errors of graduations are minimised by reading the angle on different parts of the graduated circle.

2. personal Errors of bisection are Eliminated

3. The Errors due to Eccentricity of the centres and that of verniers are Eliminated

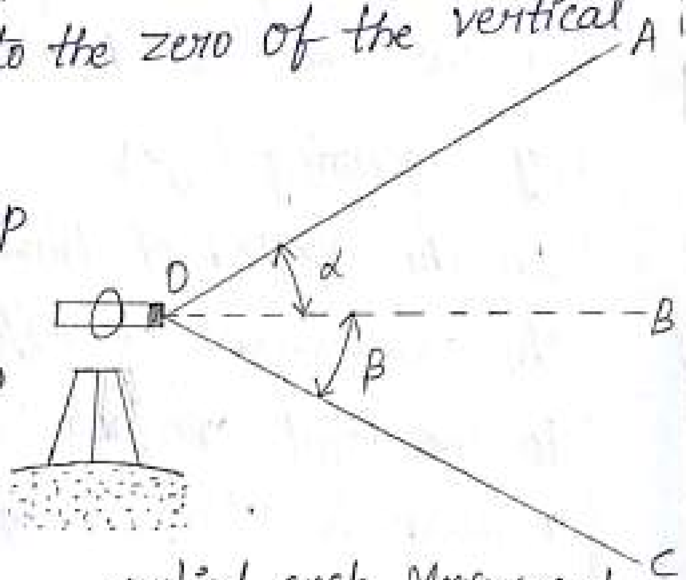
4. Errors due to Eccentricity the line of collimation not being perpendicular to the transverse axis of the telescope

MEASUREMENT OF VERTICAL ANGLE

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

2. Using the upper clamp and upper tangent screw, set the zero of the vertical vernier to the zero of the vertical circle.

3. Loosen the vertical circle clamp and rotate the telescope in a vertical plane and bring station A in the field of view.



vertical angle Measurement

4. change the face and repeat the procedure.

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

METHODS OF TRAVERSING

By chains: The linear as well as angular Measurements are done with the help of chain and tape only. This is a very crude method and cannot be replied upon.

Free or Lose Needle Method:
In this method, the linear Measurements are made with the help of Either chain or tape and the bearings are measured with the help of theodolite whose telescope is inverted for alternate backward and forward Readings.

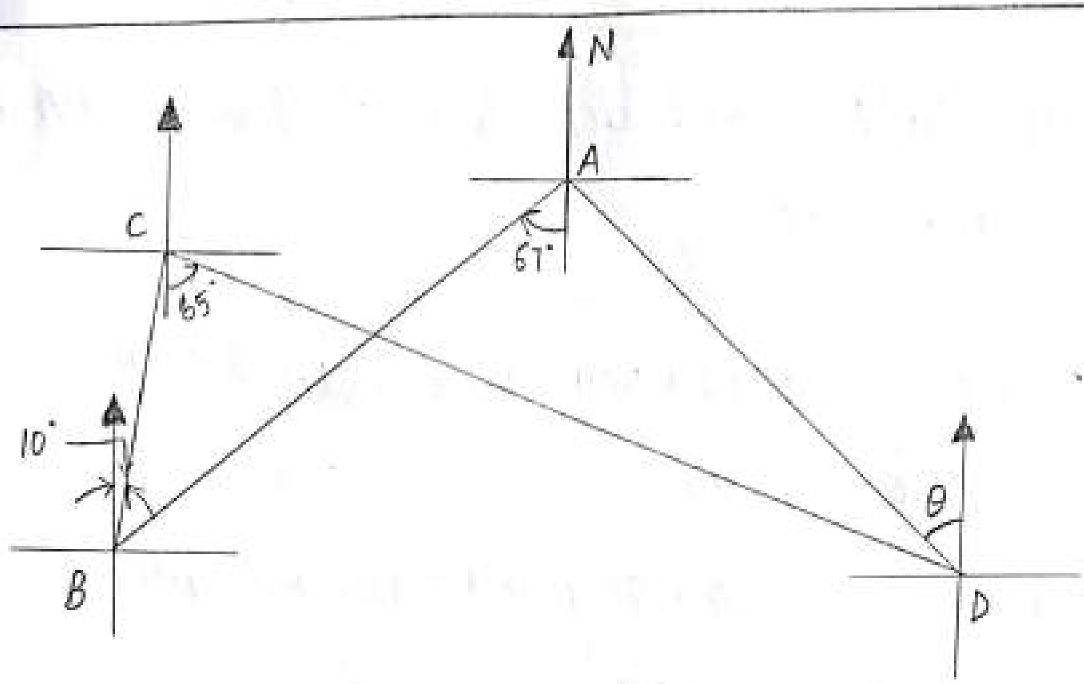
Fast Needle Method:

In this Method, the linear Measurement are made with the help of a chain (or) tape and the bearings are Measured with the help of compass. Both fore and back bearings of lines are observed at Each Station.

By Measuring Angles

In this Method. of traversing, the angular Measurements The Measurement of deflection angles, azimuth, and angles to the right (or) included angles are directly done with a theodolite. This is the most accurate method of traversing.

1. While Making a reconnaissance Survey through the woods, a surveyor with a hand compass, started from a point A and Walked a thousand Steps in the direction S67°W and reached a point B. Then he changed his direction and Walked 512 steps in the direction N10°E and reached at point C. Then he changed his direction and walked 1504 steps in the direction S65°E and reached a point D. Now, the surveyor wants to returns to the Starting point A. In Which direction should be move and how many steps should he take.



Soll

Line	Length	R.B	Quadrant
AB	1000 Steps	67°	SW
BC	512 Steps	10°	NE
CD	1504 Steps	65°	SE
DA	?	?	?

Line AB

Latitude $= -1000 \cos 67° = -390.73$

Departure $= -1000 \sin 67° = -920.50$

Line Bc

Latitude $= 512 \cos 10° = +504.22$

Departure $= 512 \sin 10° = +88.90$

Line CD:

Latitude $= -1504 \cos 65° = -635.61$

Departure $= 1504 \sin 65° = +1363.08$

Scanned by CamScanner

Let the latitude and departure of the line DA be $l\cos\theta$ and $l\sin\theta$

then,

$$\Sigma L = 0 = -390.73 + 504.22 - 635.61 + l\cos\theta$$

$$l\cos\theta = 522.12$$

$$\Sigma D = 0 = -920.50 + 88.90 + 1363.08 + l\sin\theta$$

$$l\cdot\sin\theta = -531.48$$

length of DA, $l = \sqrt{\Sigma L^2 + \Sigma D^2}$

$$= \sqrt{(522.12)^2 + (-531.48)^2}$$

$$= 745.03$$

$$= 745 \text{ steps.}$$

Reducing Bearing, $\theta = \tan^{-1}\left(\frac{531.48}{522.12}\right)$

$$= 45^\circ 30' 32''$$

Hence, the required direction is N $45^\circ 30' 32''$ W.

(NW quadrant, since latitude is +ve and departure is −ve)

Checks on Angular Measurment

Traversing by Included Angles: The sum of interior included angles should be = $(2n+4)$ right angles, the sum of Exterior an Included angles should be = $(2n-4)$ right angles, Where 'n' is the number of sides of traverse.

Traversing by deflection angles: The algebraic sum of the deflections angles should be = 360°

Traversing by direct observation of bearings:
the fore Bearing of the last line is compared with the back Bearing of the line at the initial stations
The two values should have a difference of 180°.

In a quadrilateral ABCD, the co-ordinates of the points are as follows. find the Area.

Point	East	North
A	0	0
B	0	-893.8
C	634.8	728.8
D	1068.4	699.3

Sol: Arrange the co-ordinates in the following manner

$$\begin{matrix} 0 & 0 & 634.8 & 1068.4 & 0 \\ 0 & -893.8 & -728.8 & 699.3 & 0 \end{matrix}$$

$$\text{Area} = \frac{1}{2}\Big[0\times(-893.8) - 0\times 0 + 0\times(-728.8) - 634.8)\times(-893.8)$$
$$+ 634.8\times 699.3 - 1068.4\times(-728.8) + 1068.4\times 0 - 699.3\times 0\Big]$$
$$= \frac{1}{2}\left[634.8\times 893.8 + 634.8\times 699.3 + 1068.4\times 728.8\right]$$
$$= 894974.9 \ m^3$$
$$= 89.4974 \text{ hectares}$$

OMITTED MEASUREMENTS

Often it becomes impossible to measure all the lengths and bearings of a closed traverse. The values of missing quantities can be determined, provided they do not exceed two in number. Since the observed and omitted measurements are part of a closed traverse, the algebraic sum of the all the latitudes and that of all the depratures are each zero, i.e $\Sigma L = 0$ and $\Sigma D = 0$ Thus,

$$\Sigma L = l_1 \cos\theta_1 + l_2 \cos\theta_2 + l_3 \cos\theta_3 + \cdots = 0$$
$$\Sigma D = l_1 \sin\theta_1 + l_2 \sin\theta_2 + l_3 \sin\theta_3 + \cdots = 0$$

Where $l_1, l_2, l_3 \cdots\cdots$ and $\theta_1, \theta_2, \theta_3$ are, respectively, the lengths and bearings of the lines.

From the above two Equations, the two unknowns are obtained.

The following observations were made for a closed traverse round an obstacle. Due to obstructions, length of lines DE and EA could not be measured. find out the missing lengths.

Line	Length(m)	Bearing
AB	500	98°30′
BC	620	30°20′
CD	468	298°30′
DE	?	230°00′
EA	?	150°10′

Sol: The two lines DE and EA are adjacent lines of closed traverse ABCDE. Join D and A by dotted lines so as to obtain a closed traverse ABCD. Now, ΣL and ΣD should be zero for this traverse. let the length and the bearing of line DA be l and θ.

$$\Sigma L = 0 = 500 \cos 98°30' + 620 \cos 30°20' + 468 \cos 298°30' + l \cos\theta$$

$$l \cos\theta = -73.90 - 535.12 - 223.3$$

$$= -684.53 \text{ m}$$

$$\Sigma D = 0 = 500 \sin 98°30' + 620 \sin 30°20' + 468 \sin 298°30' + l \sin\theta$$

$$l \sin\theta = -494.50 - 313.12 + 411.28$$

$$= -396.34 \text{ m}$$

Since latitude and departure both are negative, the line DA lies in the third Quadrant (SW)

$$l = \sqrt{(-684.53)^2 + (-396.34)^2}$$

$$= 790.99 m$$

$$\tan\theta = \frac{\Sigma D}{\Sigma L}$$

$$= \frac{396.34}{684.53}$$

$$\theta = 30^\circ 04' SW$$

Bearing of DA = S $30^\circ 04'$ W = $210^\circ 04'$

Now, in the triangle DAE, the length and the bearing of line DA is known. Also, the Bearings of DE and EA are known.

Bearing of DA = $210^\circ 04'$

Bearing of AD = $210^\circ 04' - 180^\circ = 30^\circ 04'$

Bearing of DE = $230^\circ 00'$

Bearing of EA = $150^\circ 10'$

Since traverse ADE is anticlockwise, the included angles will be the interior angles.

$\angle ADE = 230^\circ 00' - (30^\circ 04' + 180^\circ) = 19^\circ 56'$

$\angle DEA = 150^\circ 10' - (230^\circ - 180^\circ) = 100^\circ 10'$

Line	Length	Included Angle	W.C.B
AB	255m	$\angle A = 93°18'16''$	$140°42'$
BC	656m	$\angle B = 74°16'35''$	
CD	120m	$\angle C = 123°42'00''$	
DA	668m	$\angle D = 68°41'16''$	

Sol: Corrected Included Angles

Sum of the observed included angles of the traverse

$= 93°18'16'' + 74°16'24'' + 123°42'00'' + 68°41'16''$

$= 359°57'56''$

Theoretical Sum of included angles $= (2n-4) \times 90°$

$= (2\times4-4)\times 90° = 360°$

Correction $= 360° - 359°57'56'' = 2'4''$

A correction of $(2'4''/4) = 31''$ should be applied to Each included angle. Hence, corrected Angles are.

$\angle A = 93°18'16'' + 31'' = 93°18'47''$

$\angle B = 74°16'24' + 31'' = 74°16'55''$

$\angle C = 123°42'00'' + 31'' = 123°42'31''$

$\angle D = 68°41'16'' + 31'' = 68°41'47''$

Calculation of Bearings

Bearing of line AB $= 140°42' + 74°16'55''$ (Add $\angle B$)

$= 214°58'55''$

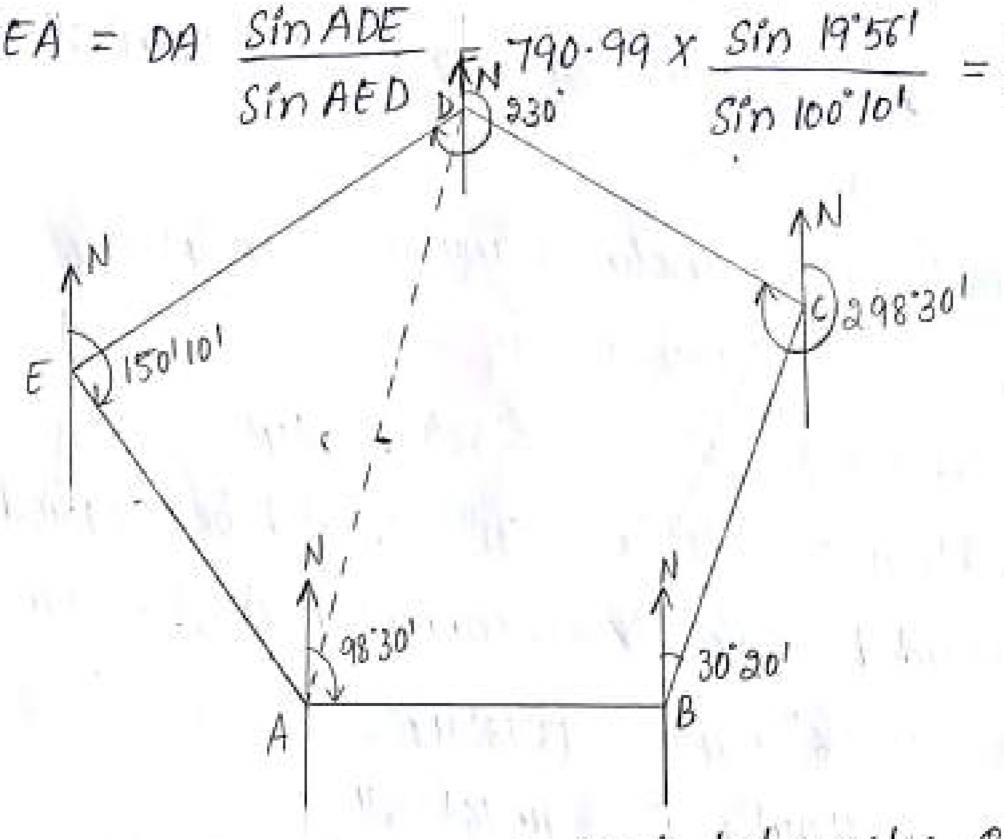
$\angle EAD = 30^\circ 04' - (150^\circ 10' + 180^\circ) = 300^\circ 56' - 360^\circ = 59^\circ 54'$.

The bearings of line DE and EA can be obtained by applying the Sine Rule.

From ΔADE,

$$DE = DA \cdot \frac{\sin EAD}{\sin AED} = 790.99 \times \frac{\sin 59^\circ 54'}{\sin 100^\circ 10'} = 695.24\,m$$

$$EA = DA \frac{\sin ADE}{\sin AED} = 790.99 \times \frac{\sin 19^\circ 56'}{\sin 100^\circ 10'} = 273.97\,m$$



1. The lengths, bearings and included angles of a closed traverse ABCDA, as observed with a transit theodolite are given below. prepare a Gale's traverse table and plot the traverse.

Station B:

$$Latitude = 225 \cos 39°18' = 197.329\,m$$

$$Departure = 225 \sin 39°18' = 161.512\,m$$

Station C:

$$Latitude = 656 \cos 34°58'55'' = 537.482\,m$$

$$Departure = 656 \sin 34°58'55'' = 376.097\,m$$

Station D:

$$Latitude = 120 \cos 21°18'34'' = 111.796\,m$$

$$Departure = 120 \sin 21°18'34'' = 43.608\,m$$

Station A:

$$Latitude = 668 \cos 47°23'13'' = 452.265\,m$$

$$Departure = 668 \sin 47°23'13'' = 491.610\,m$$

Closing Error

$$\Sigma L = +537.482 + 111.796 - 197.329 - 452.265 = -0.316\,m$$

$$\Sigma D = +161.512 + 376.097 - 43.608 - 491.610 = 2.391\,m$$

Hence, there is a closing Error.

$$\text{Closing Error, } e = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$$

$$= \sqrt{(-0.316)^2 + (2.391)^2}$$

$$e = 2.411\,m$$

Bearing of line AB = 214°58'55" − 180° (subtract 180°)
= 34°58'55".

Bearing of line BC = 34°58'55" + 123°42'31"
= 158°41'26" (Add ∠C)
= 158°41'26" + 180° (Add 180°)
= 338°41'26"

Bearing of line CD = 338°41'26" + 68°41'47"
= 407°23'13" (Add ∠D)
= 407°23'13" − 180°
= 227°23'13" (subtract 180°)

Bearing of line DA = 227°23'13" + 93°18'47" (Add ∠A)
= 320°42'00'
= 320°42'00' − 180° (subtract 180°
= 140°42'00"

Line	W.C.B	R.B	Quadrant
AB	140°42'	39°18'	SE
BC	34°58'55"	34°58'55'	NE
CD	338°41'26"	21°18'34"	NW
DA	227°23'13"	47°23'13"	SW

The reducing bearing of closing Error,

$$\theta = \tan^{-1} 2.390/0.316 = 82^\circ 27' 39''.$$

Since ΣL is negative and ΣD is positive, the Quadrant of closing Error is SE

Corrections

Correction to latitude (or) departure of any side

$$= \text{total Error in latitude (or) departure} \times \frac{\text{latitude (or departure) of the side}}{\text{arithmetic sum of latitudes (or departures)}}$$

Line AB:

$$\text{Correction to Southing} = 0.316 \times \frac{197.329}{649.278 + 649.594}$$

$$= 0.048\,m\ (-ve)$$

$$\text{Correction to Easting} = 2.391 \times \frac{161.512}{537.609 + 535.218}$$

$$= 0.360\,m\ (-ve)$$

Line BC:

$$\text{Correction to northing} = 0.316 \times \frac{537.482}{649.278 + 649.594}$$

$$= 0.131\,m\ (+ve)$$

$$\text{correction to Easting} = 0.2391 \times \frac{376.097}{537.609 + 535.218}$$

$$= 0.838\ m\ (-ve)$$

Line CD:

$$\text{correction to northing} = 0.316 \times \frac{111.796}{649.278 + 649.594}$$

$$= 0.027 m\ (+ve)$$

$$\text{correction to Westing} = 2.391 \times \frac{43.608}{537.609 + 535.218}$$

$$= 0.097 m\ (+ve).$$

Line DA:

$$\text{correction to southing} = 0.316 \times \frac{452.265}{649.278 + 649.594}$$

$$= 0.110 m\ (-ve)$$

$$\text{correction to Westing} = 2.391 \times \frac{491.610}{537.609 + 535.218}$$

$$= 1.096 m\ (+ve)$$

Instrumental Station	Angle	correction	Corrected Angle	W.C.B	R.B	Quadrant	length of the line
A	93°18'16"	+31"	93°18'47"	140°42'	39°18'	SE	AB = 225m
B	74°16'24"	+31"	74°16'55"	34°58'55"	34°58'55"	NE	BC = 656m
C	123°42'00"	+31"	123°42'31"	338°41'26"	21°18'34"	NW	CD = 120m
D	68°41'16"	+3"	68°41'47"	227°23'13"	47°23'13"	SW	DA = 668m

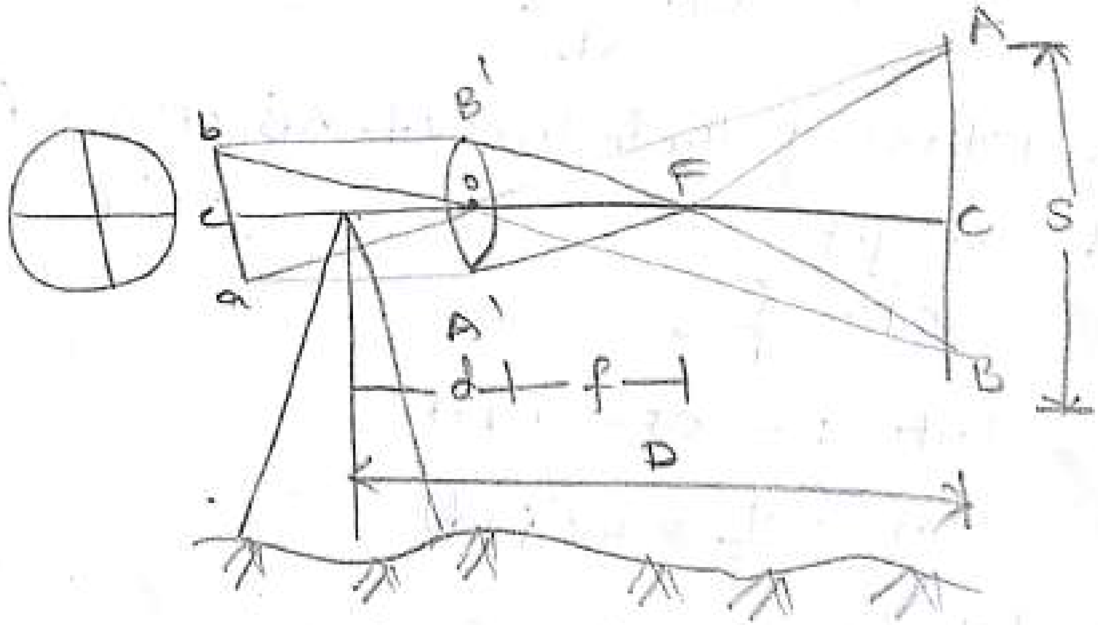
Consecutive co-ordinates (m)				Corrections (m)			
Northing	Southing	Easting	Westing	Northing	Southing	Easting	Westing
	197.329	161.512			−0.048	−0.360	
537.482		376.097		0.131		−0.838	
111.796			43.609	0.027			0.097
	452.265		491.610		−0.110		1.095
ΣN 649.278	ΣS 649.594	ΣE 537.609	ΣW 535.219	ΣN +0.158	ΣS −0.158	ΣE −1.198	ΣW +1.192
−0.316		+2.390		+0.316		−2.390	

Corrected Consecutive Coordinates (m)				Independent co-ordinates (m)	
Northing	Southing	Easting	Westing	Northing	Easting
	197.281	161.152		302.719	661.152
537.613		375.259		840.332	1036.411
111.823			43.706	952.155	992.705
	452.155		492.705	500	500
ΣN 649.436	ΣS 649.436	ΣE 536.411	ΣW = 536.411		

Tacheometric

principle of stadia measurements:-

For the measurement of stadia distances, the Reticle or diaphragm in the telescope of the theodolite (some levels) in equipped with two Additional horizontal hairs in addition to the Normal cross-hairs, one above and other below the main horizontal hair



(b)

principle of transit stadia measurement

Let f = focal length of the object length
i = stadia hair interval ab
s = horizontal distance of the staff from the Instrument axis

d = distance of the optical centre of the object
Instrument axis

From similar Δ^s $A'B'F$ and ABF, we get

$$\frac{CF}{OF} = \frac{AB}{A'B'}$$

$$CF = OF \frac{AB}{A'B'}$$

We have, $A'B' = ab$

$$CF = OF \frac{AB}{ab}$$

substituting the values of AB, OF and ab, we get

$$CF = \frac{f}{i} s$$

But $D = CF + f + d$

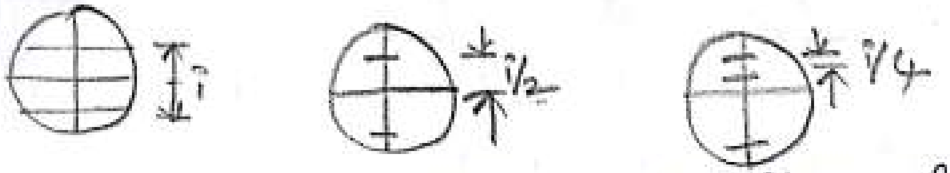
(or) $= \frac{f}{i} s + (f+d)$

substituting K for $\frac{f}{i}$ and c for $(f+d)$ them eqn

becomes, $D = KS + C$

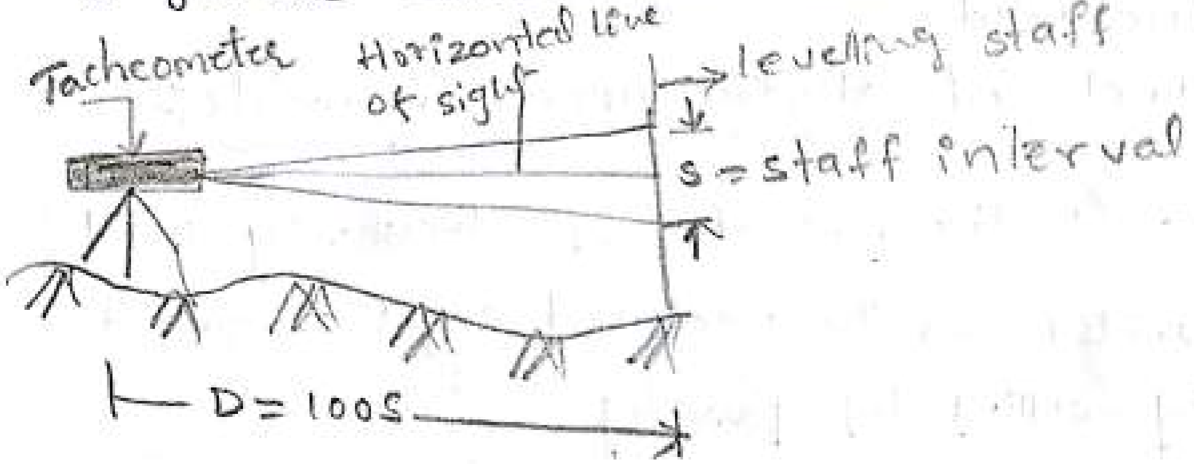
where, K and C are know as the Tachometric constants

Scanned by CamScanner



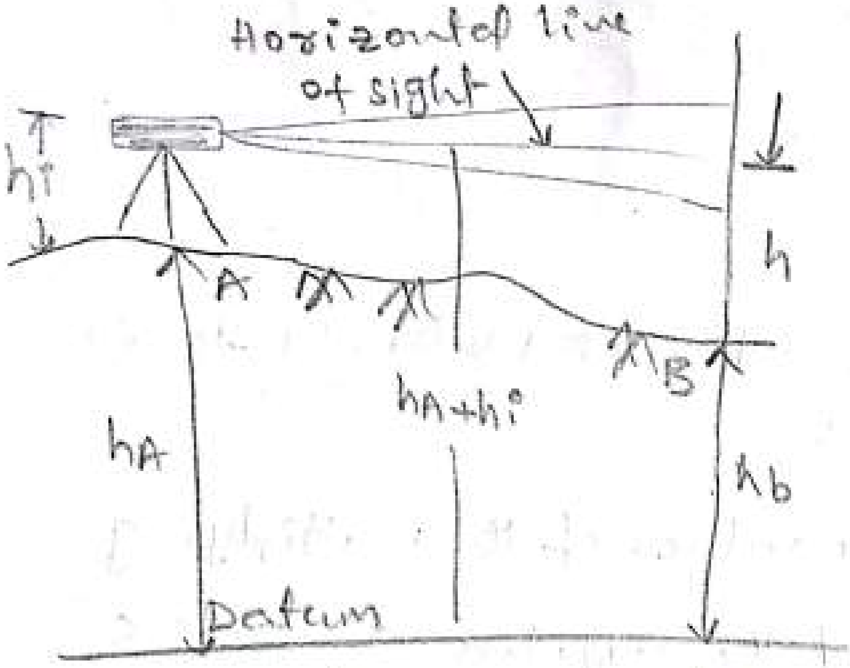
Recticles with different patterns of stadia hairs

* Generally, the values of the multiplying constant K and the additive constant c are kept equal to 100 and zero. the staff interval is then multiplied by 100 to get the horizontal distance D=100S



Horizontal distance by stadia method.

Elevations are obtained by stadia by determining the height of the Instrument HI and the middle hair reading on the staff.



$$h_B = h_A + h_i - h$$

Elevation by stadia method

~~Inclined~~

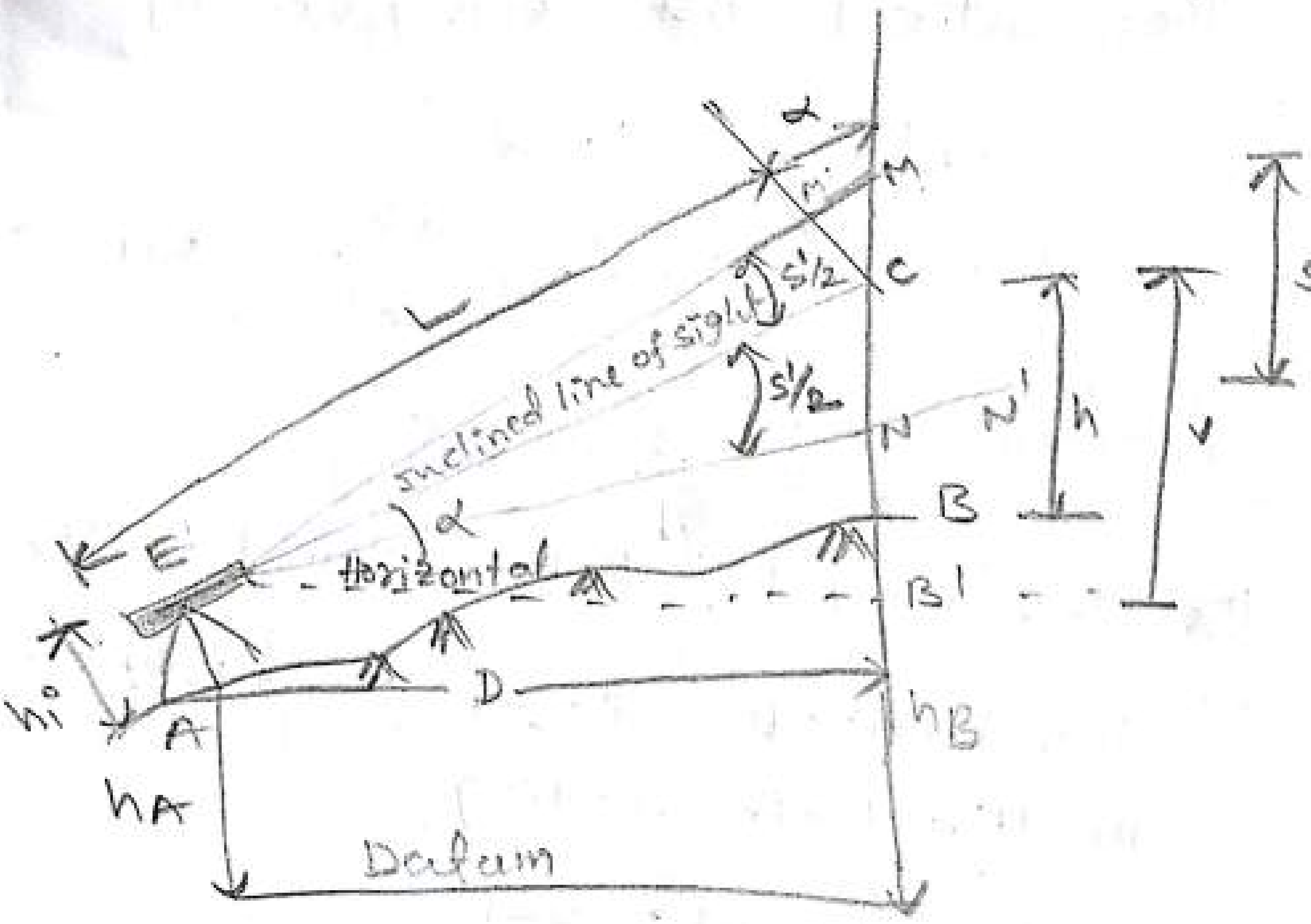
Inclined stadia Measurements:-

The stadia method is particularly well suited for the Inclined sight required By rolling topography

$$CM' = CM \cos\alpha$$

$$\frac{s'}{2} = \frac{S}{2}\cos\alpha \quad (\text{or}) \quad s' = S\cos\alpha$$

where $M'N'$ is the staff internal s'



Inclined stadia measurement (c=0)

The slope distance L from eq taking c=0 is given by

$$L = Ks'$$
$$= K s \cos\alpha$$

and the horizontal distance by

$$D = L\cos\alpha$$

We have, $D = Ks\cos^2\alpha$

The vertical distance is given by

$$CB' = V = D \tan\alpha$$

$$= KS\cos^2\alpha \frac{\sin\alpha}{\cos\alpha} = KS\sin\alpha\cos\alpha$$

$$= \frac{1}{2} KS \sin 2\alpha$$

The elevation h_B of B is equal to the elevation of instrument axis +ve − middle hair reading

$$h_B = h_A + h_i + V - h$$

19) The following reading were taken with a transit fitted with stadia hairs the line of sight was horizontal and the staff was held vertical

Reading on	staff (m)
Top hair	0.875
middle hair	1.340
Bottom hair	1.805

if the tacheometric constant K and C hence the value

As 100 and 0.20m what is the horizontal distance b/w the staff and the Instrument.

Sol: $D = Ks + C$ $K = 100$ m; $C = 0.20$m

$s = 1.805 - 0.875 = 0.930$m

$D = 100 \times 0.930 + 0.20 = 93.200$m

(2) To determine the distance between two points p and q the RL of q the following observations where made

Height of tacheometer at $p = 1.480$m

vertical angle at $P = +5^\circ 20'$

staff readings $= 0.545, 0.905, 1.265,$

RL of $P = 150.000$ m, $k = 100.00$

$c = 0.00$m.

Sol:- we have $D = Ks \cos^2\alpha$

$s = 1.265 - 0.545 = 0.720$m

$\alpha = +5^\circ 20'$

$D = 100 \times 0.720 \times \cos^2 5^\circ 20' = 71.378$m

RL of $q = hp + hi + v - h$

$h_i = 1.480m$

$h = 0.905m$

we have,

$V = \frac{1}{2} KS \sin\alpha$

$= \frac{1}{2} \times 100 \times 0.720 \times \sin(2 \times 5^\circ 20')$

$= 6.663 m$

R.L of Q $= 150.000 + 1.480 + 6.663 - 0.905$

$= \underline{157.238 m}$

Determination of Tacheometric constants

method-1: $D = KS + C$ (or) $K = \frac{D-C}{S}$

method-2: $D_1 = K_1 S_1 + C_1$ $D_3 = K_2 S_2 + C_2$

$D_2 = K_1 S_2 + C_1$, $D_4 = K_2 S_4 + C_2$

the means values of K and C are

$K = \frac{K_1 + K_2}{2}$ and $C = \frac{C_1 + C_2}{2}$

19 To determine the tacheometric constants of a transit filled with stadia hairs

horizontal distance (m)	50	100	150
staff interval (m)	0.495	0.998	1.500

the line of sight was kept horizontal the focal length of the object glass is 20cm and the distance of the objective Glass from the ~~tor~~ trunion axis of the Instrument is 11cm Determine the values of K and C

Sol: we know that,

$$C = f + d$$

$$= 20 + 11 = 0.31m$$

and $D = KS + C$ (or) $K = \frac{D-C}{S}$

(i) For $D = 50m$ and $S = 0.495m$

$$K = \frac{50 - 0.31}{0.495} = 100.383$$

(ii) For $D = 100m$ and $S = 0.998m$

$$K = \frac{100 - 0.31}{0.998} = 99.890$$

(iii) For $D = 150 m$, and $s = 1.500$

$$K = \frac{150 - 0.31}{1.500} = 99.793$$

2Q) The following observations were made on Beeman stadia arc fitted on the vertical circle of a transit
staff reading(m) = 1.772 2, 565, 3.358
V-scale Reading = 56, H-scale reading = 0.46,
RL of the Instrument axis = 300.00m

sol:- we have,

$D = S(100 - H \text{ scale Reading})$

$V = S(V\text{-scale reading} - 50)$

$S = 3.358 - 1.772 = 1.586 m$

$D = 1.586 \times (100 - 0.46) = 157.870 m$

$V = 1.586 \times (56 - 50) = 9.516 m$

As V-scale Reading is more than 50, V is +ve, indicating the angle of elevation.

R.L of the staff station

→ RL the instrument axis + V − central hair reading

$= 300.00 + 9.516 - 2.565$

$= 306.95m$

39 The horizontal angle subtended at the theodolite station by a subtense bar with targets 2m apart is 17'30"

Compute

(a) the horizontal distance b/w the the subtense bar and the theodolite

(b) the error in the horizontal distance if there is an error of 1.5" in the measurement of horizontal angle

(c) the error in the horizontal distance if distance if the bar is 1°

Given that

$s = 2m$

$\theta = 17'30''$

$\delta\theta = +1.5''$

(a) $D = \frac{1}{2} s \cot \theta/2$

$= \frac{1}{2} \times 2 \cot \frac{17'30''}{2}$

$= 392.88\ m$

(b) $\delta D = \frac{D^2}{s} . \delta\theta$

$= \frac{392.88^2}{2} \times \frac{1.5}{206265}$

$= 0.561\ m$

Also from $\delta D = D \frac{\delta\theta}{\theta}$

$= 392.88 \times \frac{1.5}{17 \times 60 + 30}$

$= \frac{392.88 \times 1.5}{1050}$

$= 0.561\ \underline{m}$

(c) Let the angle $A'CA = 1^\circ$

$OC' = D' = A'C' \cot \theta/2$

But $A'C' = A'C \cos 1^\circ$

$D' = A'C \cos 1^\circ \cot \theta/2$

$A'B' = 2m$

$A'C = 1m$

$D' = 1 \times \cos 1^\circ \cot \frac{17'30''}{2}$

$= 392.82\,m$

Error in horizontal distance

$= D - D'$

$392.88 - 392.82 = 0.06m$

Curves

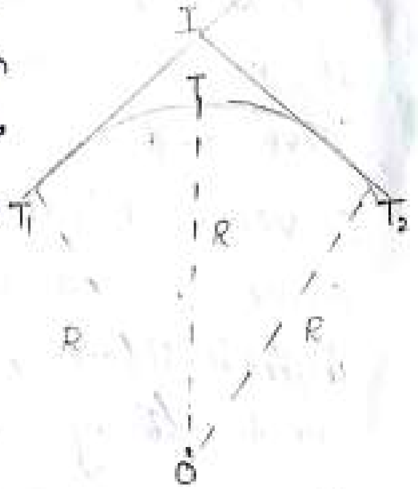
Classification

Curves are basically classified as horizontal or Vertical Curves the former being in the horizontal plane and the latter in the Vertical plane.

The horizontal Curves are further classified as Simple Circular Curve, Compound, reverse, transition, Combined and broken-back Curves, Vertical curve.

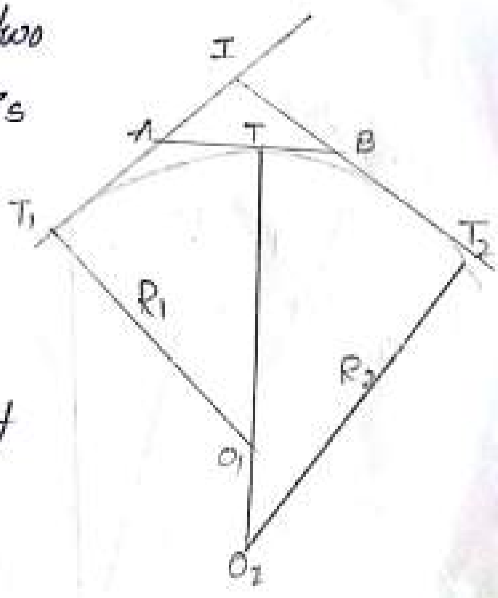
Simple Circular Curve:-

A Curve connecting two intersecting straights having a Constant radius all through is known as simple Circular Curve. It is tangential to the two straights at the joining ends. From fig T_1TT_2 is a simple Circular Curve of radius R, Joining the two straights T_1I and T_2I intersecting at a point I.



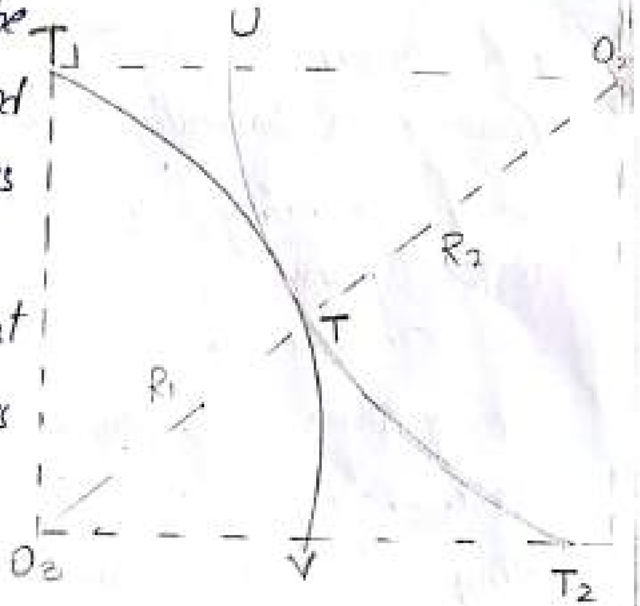
Compound Curve:

When two or more simple Curves, of different radii, turning in the same direction join two intersecting straights, the resultant Curve is called compound Curve. In fig T_1TT_2 is a Compound Curve with two simple Circular Curves T_1T and TT_2 of Radii of R_1 and R_2. ATB is a Common tangent and T is the Common tangent point



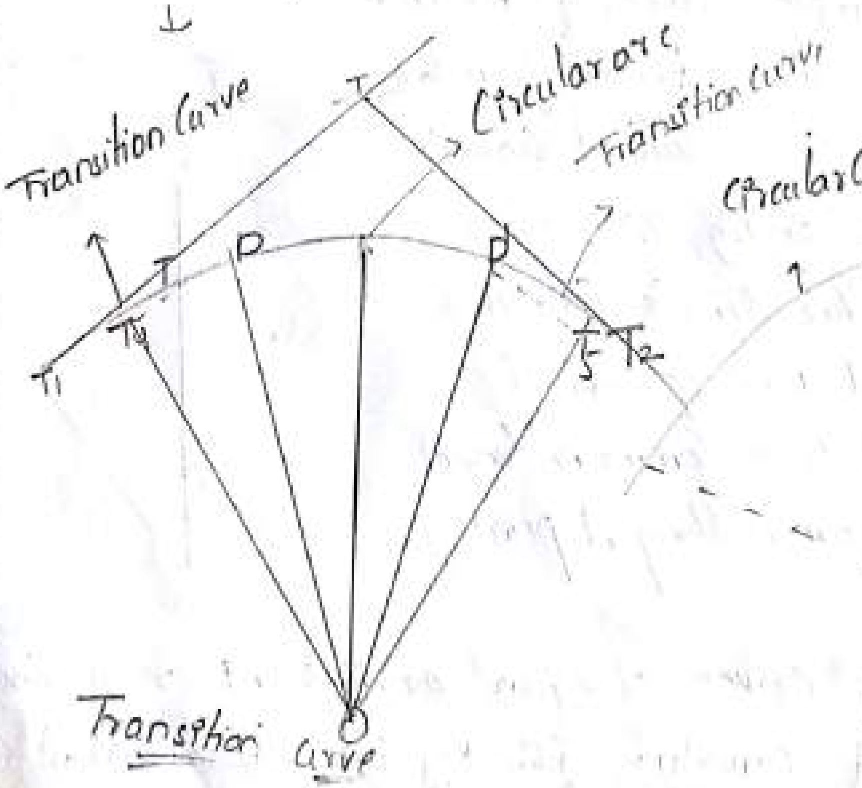
Reverse Curve:

When two simple curves of equal or different radii, having opposite direction of Curvature join together. the resultant Curve is known as reverse curve. In fig T_1TT_2 is a reverse curve formed from the curves T_1TV and T_2TU of radii R_1 and R_2 joining the two straight lines T_1U and T_2V. Reverse curves are quite common in Railway yards but are unsuitable for modern highways. These are serpentine curve or S-curve because of their shape

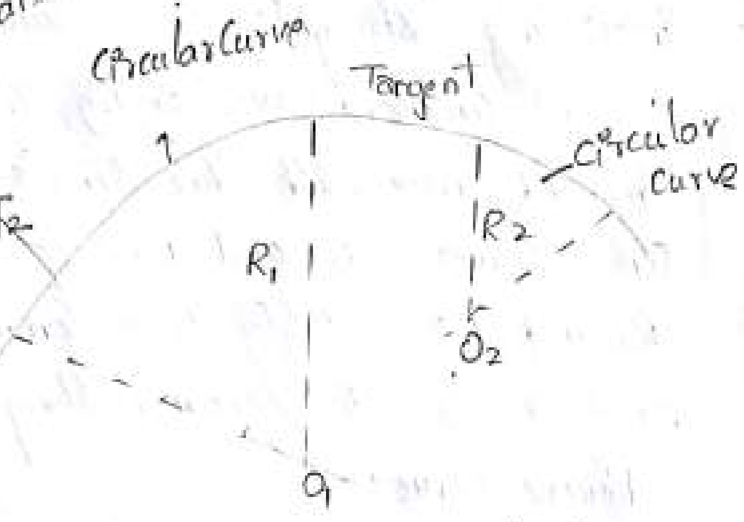


Transition Curve :-

It is a curve usually introduced b/w a simple circular curve and a straight, or between two simple circular curves. It is also known as an easement curve. A transition curve has a radius, gradually changing from a finite to infinite value or vice versa. It is widely used on highways and railways from fig T_1TD is a transition curve introduced b/w a simple curve $T_4DD'T_5$ and the straight T_3I



Transition Curve



Broken back Curve

Broken-back curve:

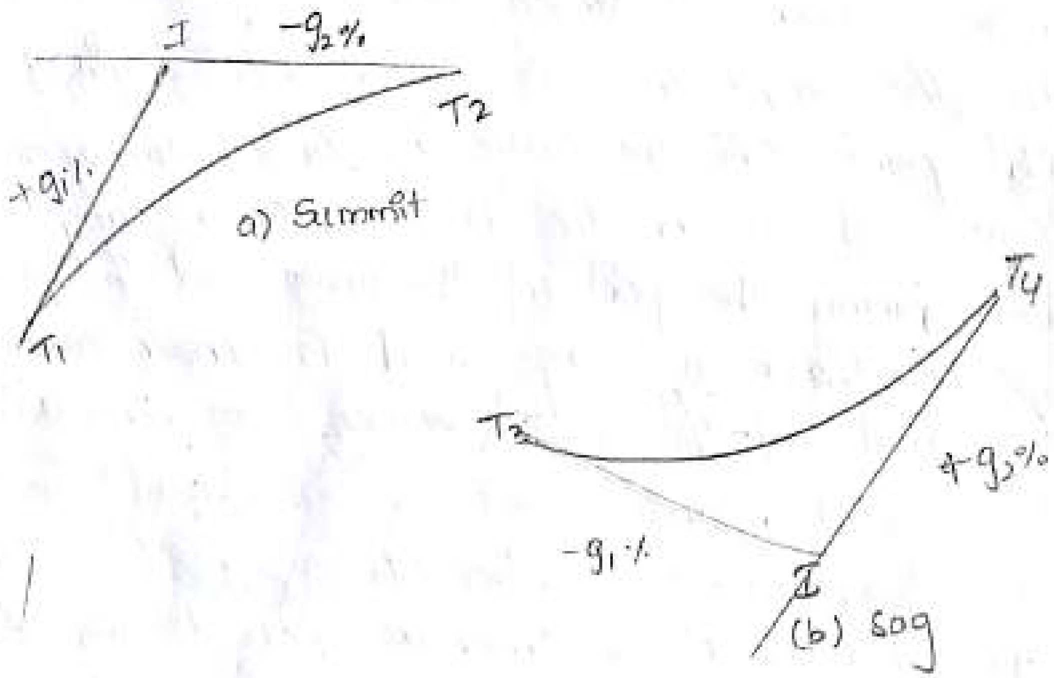
In the past, sometimes, two circular curves having their centres on the same side and connected with a short tangent length were used for railroad traffic since these are not suitable for high speeds, they are not in use nowadays

Combined Curve:

Combined curves are a combination of simple circular curve and transition curves and are preffered in railway and highways

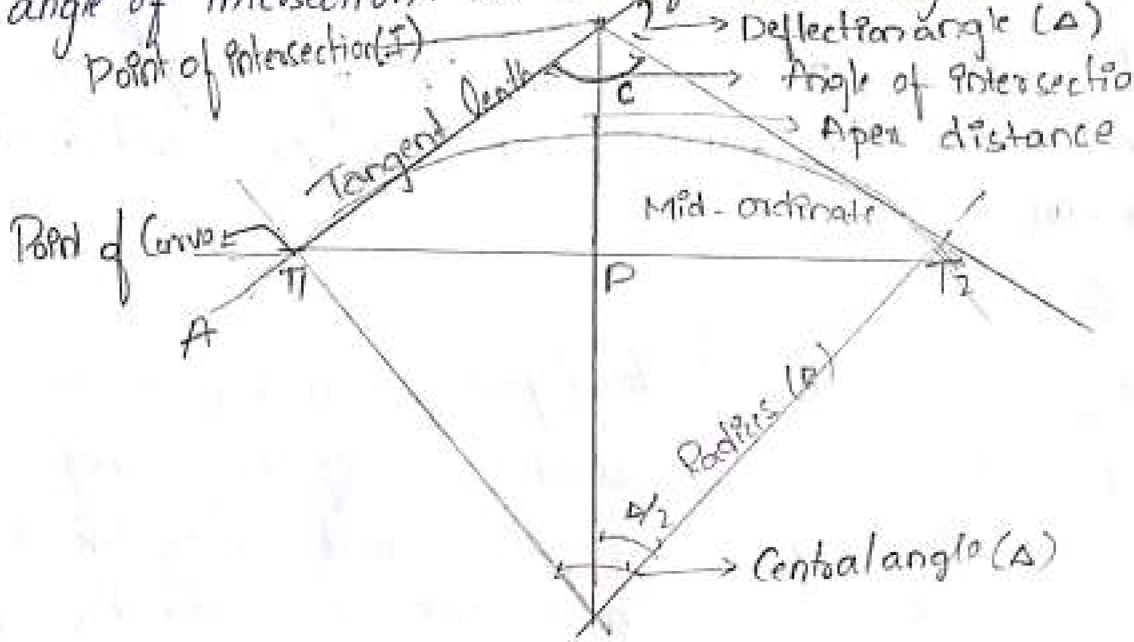
Vertical Curve:

These are curves, in a vertical plane, used to join two intersecting grade lines. The reduced level of these curves change from point to point in a gradual and systematic ~~matter~~ manner. A vertical summit curve is provided when a rising grade (T_1I) join a falling grade (T_2I) joins a rising grade (T_2I). A vertical sag curve is provided when a falling grade (T_3I) joins a rising grade (T_4I) as shown in fig



ELEMENTS OF SIMPLE CURVE

AI and BI are two intersecting straights joined by a simple curve T_1CT_2 of Radius R. The point I is called point of intersection (PI). Δ, the deflection angle is the external angle b/w the two intersecting straights. The internal angle ∠AIB (θ) is called angle of intersection. The sum of these 2 angles Δ and θ is 180°

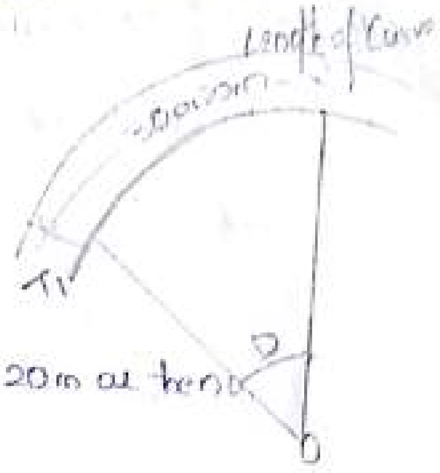


O is the centre of the circular curve. The point T_1 where the circular curve begins is known as point of curve (Pc) The last point of the tangent T_2 is known as point of tangency (PT) IT_1 & IT_2 are known as tangent length and are always equal in length. The length T_1CT_2 is called Total length of the curve. The middle point C of the curve is called the apex or summit of the curve. It lies on the bisection of the angle of intersection the chord joining the point of the curve and the point of tangency is known as long chord is known as mid ordinate (CD). A chord of the curve between two consecutive regular station is called normal chord. Any other chord shorter length is called sub chord. The distance b/w the point of intersection I and apex of curve (C) is known as apex distance. Angle T_1OT_2 subtended at the centre of curve (O) is known as central angle

Scanned by CamScanner

Relation b/w Degree and Radius of curvature

Arc Defination: If R is the radius of a curve and D is the degree for a 30m arc



$R \times D \times \pi/180^\circ = 30$

$$R = \frac{30 \times 180^\circ}{D \times \pi} = \frac{1718.9}{D} = \frac{1719}{D}$$

If D is the degree of a curve for 20m arc hence

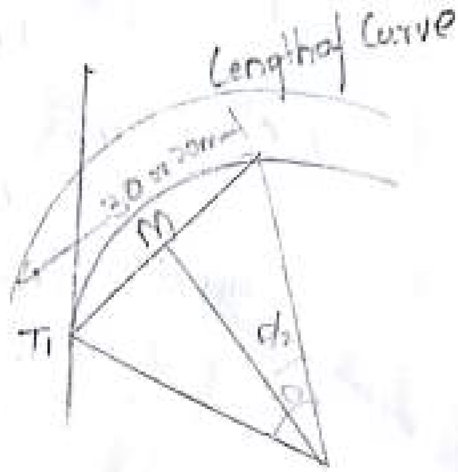
$R \times D \times \pi/180 = 20$

$$R = \frac{20 \times 180^\circ}{D \times \pi} = \frac{1145.9}{D} = \frac{1146}{D}$$

Chord Defination: For a 30m chord from Δ^{le} T_1OM

$$\sin D/2 = \frac{T_1M}{OT_1} = \frac{15}{R}$$

(or) $R = \dfrac{15}{\sin D/2}$



Since D is very small $\sin D/2 = D/2$ hence

$$R = \frac{15}{(D/2) \times (\pi/180^\circ)} = \frac{15 \times 2 \times 180^\circ}{D \times \pi}$$

$$= \frac{1718.9}{D} = \frac{1719}{D}$$

||ly for a 20m chord from Δ^{le} T_1OM

$$\sin D/2 = \frac{T_1M}{OT_1} = \frac{10}{R}$$

or $R = \dfrac{10}{\sin D/2}$

Since D is very small $\sin D/2 = D/2$ hence

$$R = \frac{10}{D/2 \times (\pi/180^\circ)} = \frac{1145.9}{D} = \frac{1146}{D}$$

FORMULAE FOR ELEMENTS OF A SIMPLE CIRCULAR CURVE

Length of the Curve : Let the length of the curve T_1CT_2 be l and let R be the Radius

Hence $l = R\Delta$

$= R\Delta \times \frac{\pi}{180°}$ where Δ is Degrees

If a 30m arc (or) chord defination is used

$$l = \frac{1719}{D} \times \Delta \times \frac{\pi}{180} = \frac{30\Delta}{D}$$

If a 20m arc (or) chord defination used

$$l = \frac{1146}{D} \times \Delta \times \frac{\pi}{180} = \frac{20\Delta}{D}$$

Tangent length:-

$$T = IT_1 = IT_2 = R \times \tan(\Delta/2)$$

Long chord length:-

$$L = T_1DT_2 = 2T_1D$$

from Δ'le OT_1D

$$\sin(\Delta/2) = T_1D/R$$

$$T_1D = R\sin(\Delta/2)$$

$$L = 2R\sin(\Delta/2)$$

Apex distance

$$IC = IO - CO = R\sec\frac{\Delta}{2} - R = R\left[\sec\frac{\Delta}{2} - 1\right]$$

Mid ordinate

$$O_0 = CD = CO - DO = R - R\cos\frac{\Delta}{2} = R\left[1 - \cos\frac{\Delta}{2}\right)$$

$$O_0 = R\,\mathrm{Vers}\,\frac{\Delta}{2}$$

Example :-

A Circular Curve has a 200m radius and 65° deflection angle. What is its degree (i) by arc defination and (ii) by chord defination Also calculate

a) length of Curve (b) tangent length (c) length of long chord (d) apex distance (e) mid ordinate.

Sol (i) Arc defination

Assuming a 30m chord length

$$R \times D \times \frac{\pi}{180^\circ} = 30$$

$$D = \frac{30 \times 180}{R \times \pi} = \frac{30 \times 180}{200 \times \pi} = 8.595^\circ$$

(ii) chord defination

Assuming a 30m chord length

$$R = \frac{15}{\sin(D/2)} \quad \left[\sin D/2 = D/2 \text{ radians} \right]$$

$$= \frac{15}{D/2 \times \frac{\pi}{180^\circ}} = \frac{15 \times 2 \times 180}{\pi \times 200} = 8.595^\circ$$

(a) Length of Curve $L = R \cdot \Delta \cdot \frac{\pi}{180^\circ} = 200 \times 65^\circ \times \frac{\pi}{180^\circ} = 226.89m$

(b) Tangent length $T = R \tan \frac{\Delta}{2} = 20 \tan \frac{65^\circ}{2} = 127.41m$

(c) Length of long Chord $L = 2R \sin \frac{\Delta}{2} = 2 \times 200 \sin \frac{65^\circ}{2} = 214.92m$

(d) Apex distance $= R\left[\sec \frac{\Delta}{2} - 1\right] = 200 \times \left[\sec \frac{65}{2} - 1\right] = 37.13m$

(e) Mid ordinate $= R\left[1 - \cos \frac{\Delta}{2}\right] = 200 \times \left[1 - \cos \frac{65^\circ}{2}\right] = 31.32m$

SETTING OUT A SIMPLE CIRCULAR CURVE:

Setting out a curve means locating various points at equal and convenient distances along the length of the curve. The distance b/w any two successive point is called a peg interval.

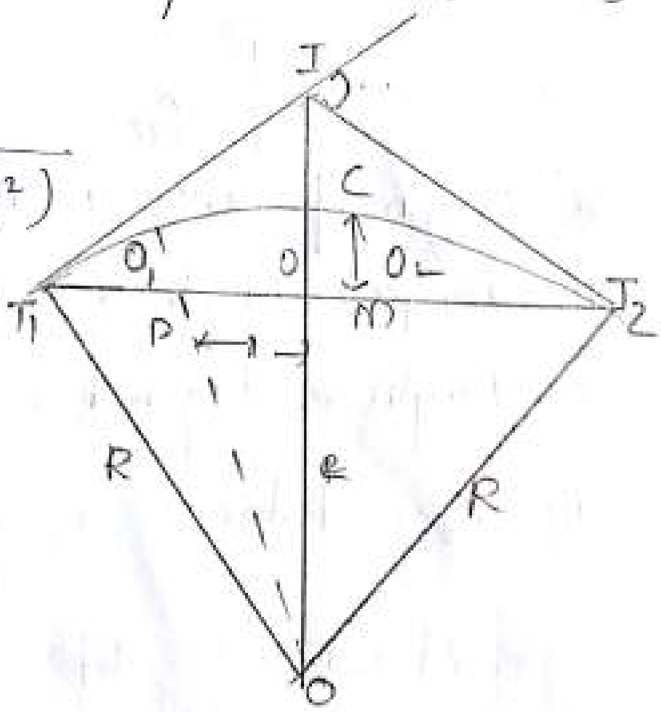
1) Linear Methods: The various linear methods of setting out a simple circular curve are

1) offsets from the long chord
2) perpendicular offsets from the tangent
3) radial offsets from the tangent
4) Successive bisection of arcs
5) offsets from the chord produced

* OFFSETS FROM THE LONG CHORD

Let it be required to lay a curve T_1CT_2 b/w the two intersecting straights T_1I and T_2I. R is the radius of the curve O_0 the mid ordinate, and O_1 the offset at a point P at a distance x from the mid point M of the long chord

From Δ^{le} OMT_1



$$OM = \sqrt{(OT_1^2 - MT_1^2)} = \sqrt{(R^2 - (L/2)^2)}$$

$$CM = OC - OM$$

$$O_0 = R - OM$$

$$O_0 = R - \sqrt{R^2 - (L/2)^2}$$

In Δ^{le} $OP'G$

$$OG = \sqrt{R^2 - x^2} \text{ and } OM = R - O_0$$

The required offset

$$PP' = OG - OM$$

$PP^1 = \sqrt{R^2 - x^2} - (R - O_0)$

$O_x = \sqrt{R^2 - x^2} - (R - O_0)$

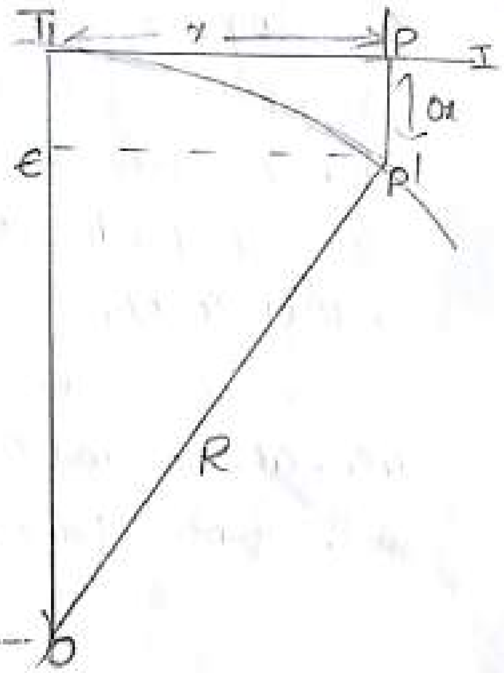
$= R(1 - x^2/R^2)^{1/2} - R + O_0$

$= R(1 - x^2/2R^2) + --) - R + O_0$

$= O_0 - \frac{x^2}{2R}$

PERPENDICULAR OFFSETS FROM TANGENTS:

this method is suitable for small values of Radius, length of the curve and deflection angle. O_x is the offset perpendicular to the tangent at a distance x from the point of curve T_1



In Δ^{le} OEP^1

$P^1O^2 = OE^2 + P^1E^2$

$R^2 = (R - O_x)^2 + x^2$

$(R - O_x)^2 = R^2 - x^2$

$(R - O_x) = \sqrt{R^2 - x^2}$

$O_x = R - \sqrt{R^2 - x^2}$

$= R - (R^2 - x^2)^{1/2}$

$= R - R(1 - x^2/R^2)^{1/2}$

$= R - R((1 - x^2/2R^2) + x^4/8R^4 + ---)$

$= R - R + Rx^2/2R^2$

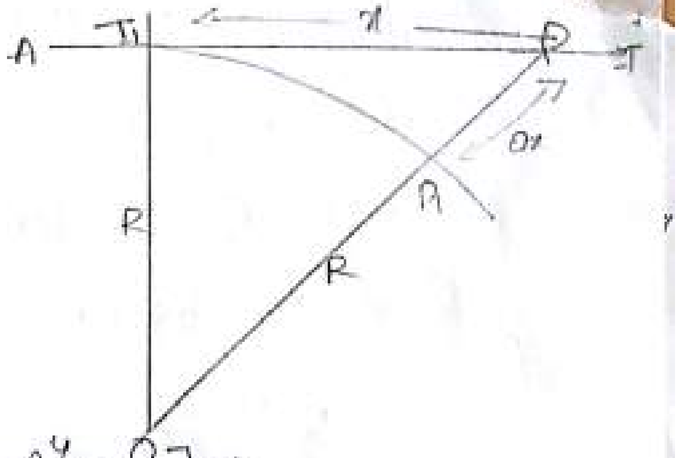
$O_x = x^2/2R$

Radial offsets from the tangent:

O_x is the radial offset PP^1 at any distance x along the tangent from T_1

from Δ^{le} OT_1P

$OP^2 = OT_1^2 + T_1P^2$



$(R + O_x)^2 = R^2 + x^2$

$R + O_x = \sqrt{R^2 + x^2}$

$O_x = \sqrt{R^2 + x^2} - R$

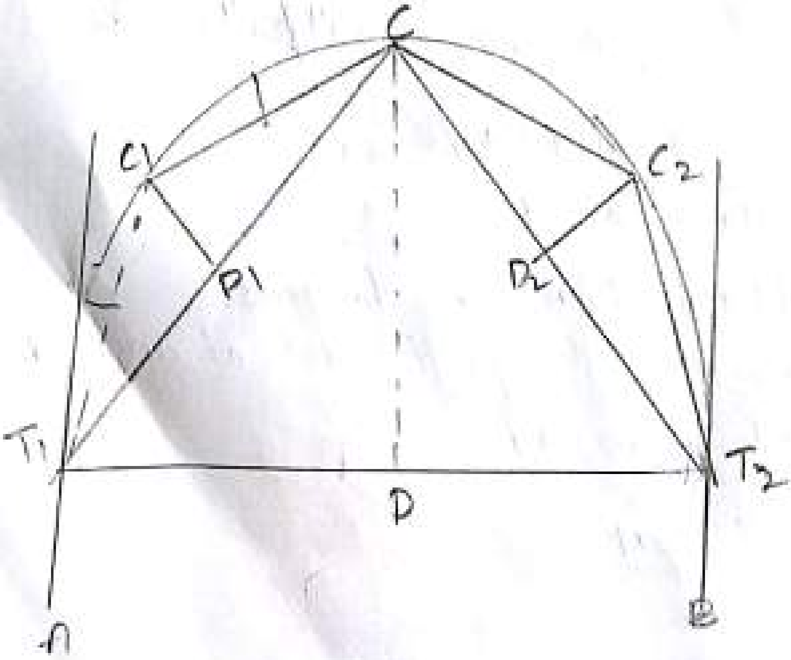
$= R(1 + x^2/R^2)^{1/2} - R$

$= R[1 + x^2/2R^2 + x^2/(8R)^4 + \cdots] - R$

$= R[1 + x^2/(2R)^2] - R$

$O_x = x^2/2R$

4. Successive bisection of arcs or chords. T_1T_2 are tangents. The long chord T_1T_2 is bisected at D. Mid-ordinate is equal to $R(1-\cos\Delta/2)$. The point C is established. T_1C and T_2C are joined. T_1C and T_2C are bisected at D_1 and D_2. Perpendicular offsets D_1C_1 and D_2C_2 each will be equal to $R(1-\cos\Delta/4)$. These offsets are set out giving points C_1 & C_2 on the curve. By the successive bisection of the chords T_1C_1, C_1C, CC_2 and C_2T_2 more points may be obtained which when joined produce the required curve.



5. By offsets from the chords produced

This is the best method for setting out a long curve by linear method and is usually employed for highway curves when a theodolite is not available.

T_1 is PC along the tangent T_1I. T_1a is the first sub chord. C_1 From the PC, a length equal to the first sub chord C_1 (T_1a') is taken. The perpendicular offset (O_1) aa' is set out. The second offset O_2 (bb') set out to get point b. point a and b are joined & produced by distance C_3 the third offset O_3 (cc') is set out to get point c. This procedure is repeated till the curve is completed

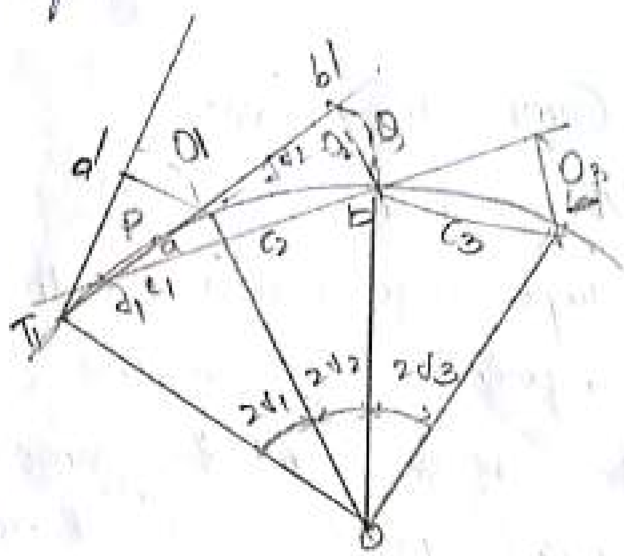
$\angle a'T_1a = \delta_1 =$ deflection angle of 1st chord

$O_1 = a'a = T_1a\delta_1 = C_1\delta_1$

$\angle T_1Oa = 2\angle aT_1a' = 2\delta_1$

$T_1a = R \times 2\delta_1$

$\delta_1 = \frac{T_1a}{2R} = \frac{C_1}{2R}$



putting the value of δ_1

$$O_1 = C_1 \times \frac{C_1}{2R} = \frac{C_1^2}{2R}$$

For computing O_2 a tangent PQ is drawn to the curve a and is produced both ways

$$ab' = ab = C_2$$

$$O_2 = b'b = b'Q + Qb$$

$$b'Q = O_2'$$

$$Qb = O_2''$$

$$O_2'' = C_2 \times \delta_2 = C_2 \times \frac{C_2}{2R} = \frac{C_2^2}{2R}$$

$$O_2' = C_2 \times \delta_1 = C_2 \times \frac{C_1}{2R}$$

$$O_2 = O_2' + O_2''$$

$$= \frac{C_1 C_2}{2R} + \frac{C_2^2}{2R}$$

$$= \frac{C_2}{2R}(C_1 + C_2)$$

$\parallel$ly 3rd offset $\cdot$ $O_3 = \frac{C_3}{2R}(C_2 + C_3) = cd$

Normally $\cdot$ $C_3 = C_4 = \dots = C_{n-1} = C$

the last or the n^{th} offset is given by

$$O_n = \frac{C_n}{2R}(C_{n-1} + C_n)$$

COMPOUND CURVE

A Compound curve is a combination of two or more simple circular curves with different radii. The two centered compound curve has two circular arcs of differend radii that deviate in the same direction and join a common tangent point also known as point of compound curvature.

Elements of the Compound Curve:

AI and BI are two straights intersecting at I. T_1DT_2 is the compound curve consisting of two arcs of Radii R_1 and R_2 and D is the point of compound curvature. MN is the common tangent making deflection angles Δ_1 & Δ_2 at M & N

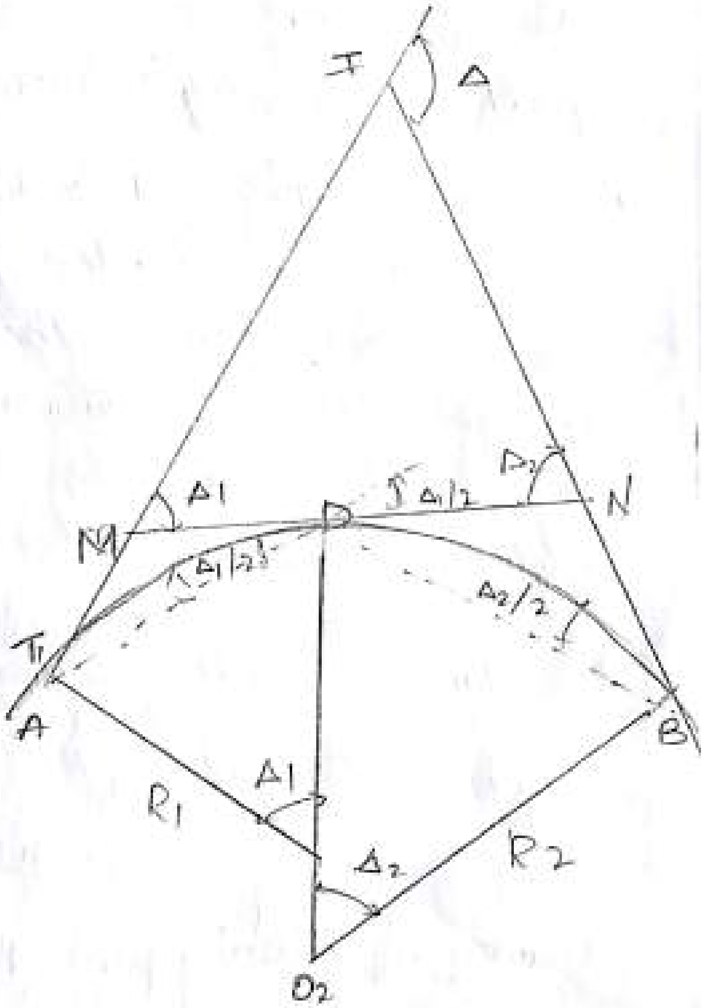
So that $\Delta = \Delta_1 + \Delta_2$

from the Δ^{le} IMN

$$\frac{IM}{\sin\Delta_2} = \frac{IN}{\sin\Delta_1} = \frac{MN}{\sin(180^\circ - (\Delta_1 + \Delta_2))}$$

Hence $IM = \dfrac{MN \sin\Delta_2}{\sin(\Delta_1 + \Delta_2)}$

$$IM = \frac{MN \sin\Delta_2}{\sin\Delta}$$



Also $IN = \dfrac{MN \sin\Delta_1}{\sin(\Delta_1 + \Delta_2)}$

$$IN = \frac{MN \sin\Delta_1}{\sin\Delta}$$

Comparing tangent MN

$$MD = R_1 \tan\frac{\Delta_1}{2}$$

$$DN = R_2 \tan\frac{\Delta_2}{2}$$

$$MN = MD + DN$$

$$= R_1 \tan\frac{\Delta_1}{2} + R_2 \tan\frac{\Delta_2}{2}$$

Length of the Main tangent IT_1 and IT_2

$$IT_1 = T_1M + MI$$

$$= R_1 \tan\frac{\Delta_1}{2} + \frac{MN \sin\Delta_2}{\sin\Delta}$$

$$IT_2 = T_2N + NI$$

$$= R_2 \tan\frac{\Delta_2}{2} + \frac{MN \sin\Delta_1}{\sin\Delta}$$

In total there are seven elements viz. $\Delta_1, \Delta_2, \Delta, R_1, R_2$
IT_2 and IT_1 If any four of these elements are known (including at least one angle and atleast two length) the remaining three elements can be determined by applying the above equation

problems on simple curve:-

1) Two straights AI and BI meet at a chainage of 3450m. A right handed simple circular curve of 250m radius joins them. The deflection angle b/w the two straights is 50°. Tabulate the necessary data to layout the curve by Rankine's method of deflection angles. Take the chord interval as 20m.

Sol) Tangent length $= R\tan\frac{\Delta}{2} = 250 \times \tan 25° = 116.58m$, Set

Length of the curve $= R \times \Delta \times \frac{\pi}{180°} = 250 \times 50° \times \frac{\pi}{180°}$

$= 218.166 = 218.17\ m$

chainage of starting point $T_1 = 3450 - 116.58 = 3333.42m$

chainage of the end point $T_2 = 3333.42 + 218.17 = 3551.59m$

Length of the chords

There will be 12 chords altogether.

First subchord $C_1 = 3340 - 3333.42 = 6.58m$

Last Subchord C_{12} = chainage of $T_2 - 3540$

$= 3551.9 - 3540$

$= 11.59\ m$

Scanned by CamScanner

C_2 to C_{11} = $20 \times 10 = 200\,m$

$d_1 = 1719 \times C_1/R = 1719 \times 6.58/250 = 45'15'' = \Delta_1$

$d_2 = 1719 \times C_2/R = 1719 \times 20/250 = 2°17'31''$

$\Delta_2 = d_2 + \Delta_1 = 2°17'31'' + 45'15'' = 3°2'46''$

$\Delta_3 = d_3 + \Delta_2 = 2°17'31'' + 3°2'46'' = 5°20'17''$

$\Delta_4 = d_4 + \Delta_3 = 2°17'31'' + 5°20'17'' = 7°37'48''$

$\Delta_5 = d_5 + \Delta_4 = 2°17'31'' + 7°37'48'' = 9°55'19''$

$\Delta_6 = d_6 + \Delta_5 = 2°17'31'' + 9°55'19'' = 12°12'50''$

$\Delta_7 = d_7 + \Delta_6 = 2°17'31'' + 12°12'50'' = 14°30'21''$

$\Delta_8 = d_8 + \Delta_7 = 2°17'31'' + 14°30'21'' = 16°47'52''$

$\Delta_9 = d_9 + \Delta_8 = 2°17'31'' + 16°47'52'' = 19°5'23''$

$\Delta_{10} = d_{10} + \Delta_9 = 2°17'31'' + 19°5'23'' = 21°22'54''$

$\Delta_{11} = d_{11} + \Delta_{10} = 2°17'31'' + 21°22'54'' = 21°22'54''$

$\cancel{\Delta_{12} = d_{12} + \Delta_{11} = 2°17'31''} +$

$d_{12} = 1719 \times 11.59/250 = 1°19'42''$

$\Delta_{12} = d_{12} + \Delta_{11} = 1°19'42'' + 23°40'25'' = 25°07'' = 25$

$\Delta_{12} = \frac{\Delta}{2} = \frac{50°}{2} = 25°$

2) Two straights AB and BC intersect at a chainage of 4242.0 The angle of intersection is 140° It is required to set Out a 5° simple Circular Curve to connect the Straights. Calculate all the data necessary to set Out the Curve by the method of offsets from the chord produced with an interval of 30m

The chain used is of 30 m

Radius of curve, $R = \frac{1720}{D} = \frac{1720}{5} = 344$ m

Deflection angle, $\Delta = 180° - 140° = 40°$

Tangent length $BT_1 = R \tan(\Delta/2) = 344 \tan 20° = 125.2$ m

chainage of intersection point $B = 4242.0$ m

chainage of $T_1 = (4242.0 - 125.2) = 4116.8$ m

Length of curve $= R \times \Delta \times \pi/180° = 344 \times 40° \times \pi/180° = 240.16$

chainage of T_2 = chainage of T_1 + length of curve $= 4116.8 + 240.16 = 4356.96$ m

Length of chords

First Subchord $C_1 = 4140 - 4116.8 = 23.2$ m

Last sub chord $C_2 = 4356.96 - 4350.0 = 6.96$ m

there are seven unit chords of 30 m length

Hence, there will be nine chords altogether

$$O_1 = \frac{C_1^2}{2R} = \frac{23.2^2}{2 \times 344} = 0.78 \text{ m}$$

$$O_2 = \frac{C_2(C_1 + C_2)}{2R} = \frac{30 \times (23.2 + 30)}{2 \times 344} = 2.32 \text{ m}$$

$$O_3 = O_4 = O_5 = \cdots O_8 = \frac{C^2}{R} = \frac{30^2}{344} = 2.62 \text{ m}$$

$$O_9 = \frac{C_9(C_8 + C_9)}{2R} = \frac{6.96 \times (30 + 6.96)}{2 \times 344} = 0.37 \text{ m}$$

GPS [Global positining System]

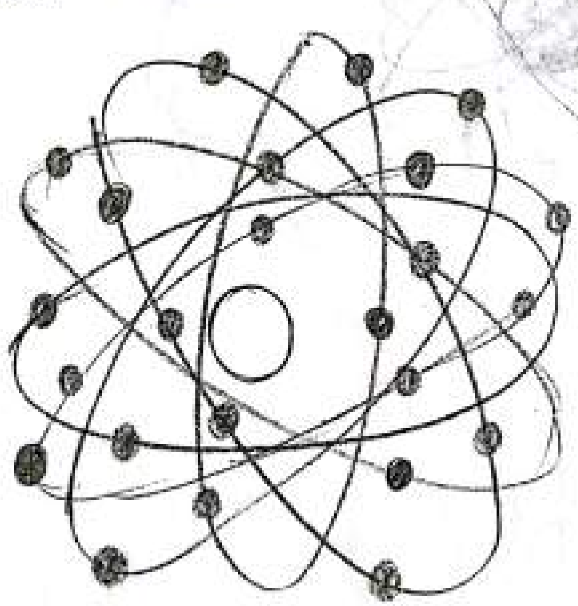
Working principle of Global positioning system :

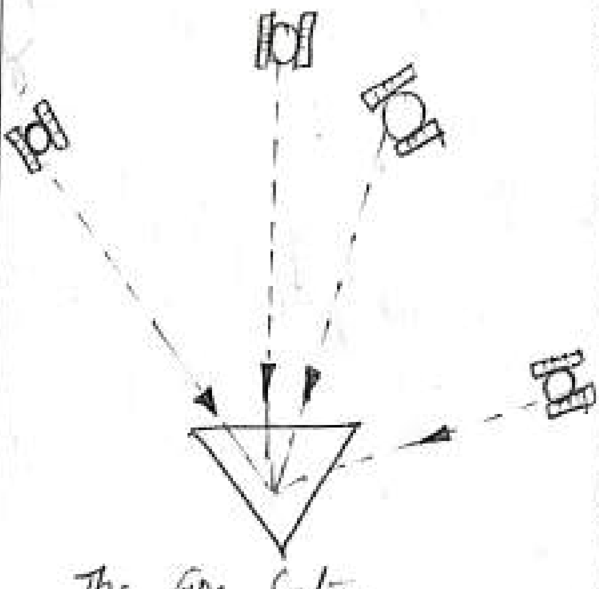
GPS was invented by the United states Department of Defence. Its actual name is NAVASTR (Navigation system with time and ranging). It is a Technique by which the location of any object, its Velocity, direction & time can be known precisely at any time i.e during day or night, whether the object is on the ground, on the sea water surface or in air.

The GPS system consists of 24 satellites placed in near circular orbits arranged in 6 orbital planes at 55° inclination to equator 20,200 km height and 26600 km orbital radius. The period of revolution is 12 hrs, so that atleast 4 satellites are available for observations at anytime. These form space segment of GPS

GPS uses ground stations as well as receivers. GPS is the only system today that can show exact position on the earth anytime

The GPS allows the user to Locate his/her position in three dimensions as well as with respect to time





The GPS System

Functional Segements of GPS:-

The GPS is comprised of three segements

1. Space segement 2. Control segement 3. User segement

Space Segement :

No. of satellites :- 24 satellites
Arrangement :- circular orbits
No. of orbits :- 6
Location :- 55° inclination to equator at 20,200km height
Orbital radius :- 26600 km
Period of revolution :- 12 hrs

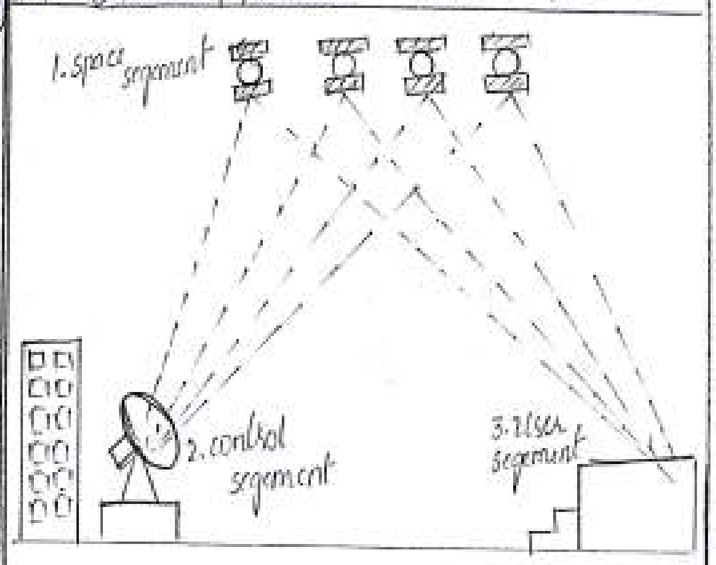
Control Segement

The control segment on the ground tracks and maintains the satellite in the space.

(1) a Master control station
(2) an alternate master control station
(3) four dedicated ground antennas
(4) six dedicated monitor stations

User Segement

In General GPS, receivers are composed of an antenna, tuned to the frequencies transmitted by the satellites, receivers - Producers, and a highly stable clock. They may also include a display for providing location and speed information to the user. A receiver is often described by its number of channels



COMPONENTS OF GIS

1. Computer System: It includes CPU, VDU, keyboard, mouse, digitizer, plotter, printer, CD/DVD drive etc. to store, process and present digital spatial data

2. Software : It includes software like Arc GIS, Map, info, Geomatica, Autodesk map and others to perform GIS operations

3. Data : Geographical data in the form of hardcopy map or digital map, aerial photographs, satellite images, statistical tables and other documents are used as data for GIS operations

4. Procedure : To complete certain tasks, procedures are performed using hardware and software

5. Experts and users : Experts with knowledge are required to apply GIS appropriately. Diff types of users are using the GIS at diff levels

APPLICATIONS OF GIS

- Water resources management and planning
- Environmental & Transportation planning
- Agriculture and Land Use planning
- Town and regional planning
- Forestry and Wildlife management
- Muncipial applications
- Emergency planning and Routing
- Market Analysis
- Land Use planning projects
- Health, Educational or Retail Services

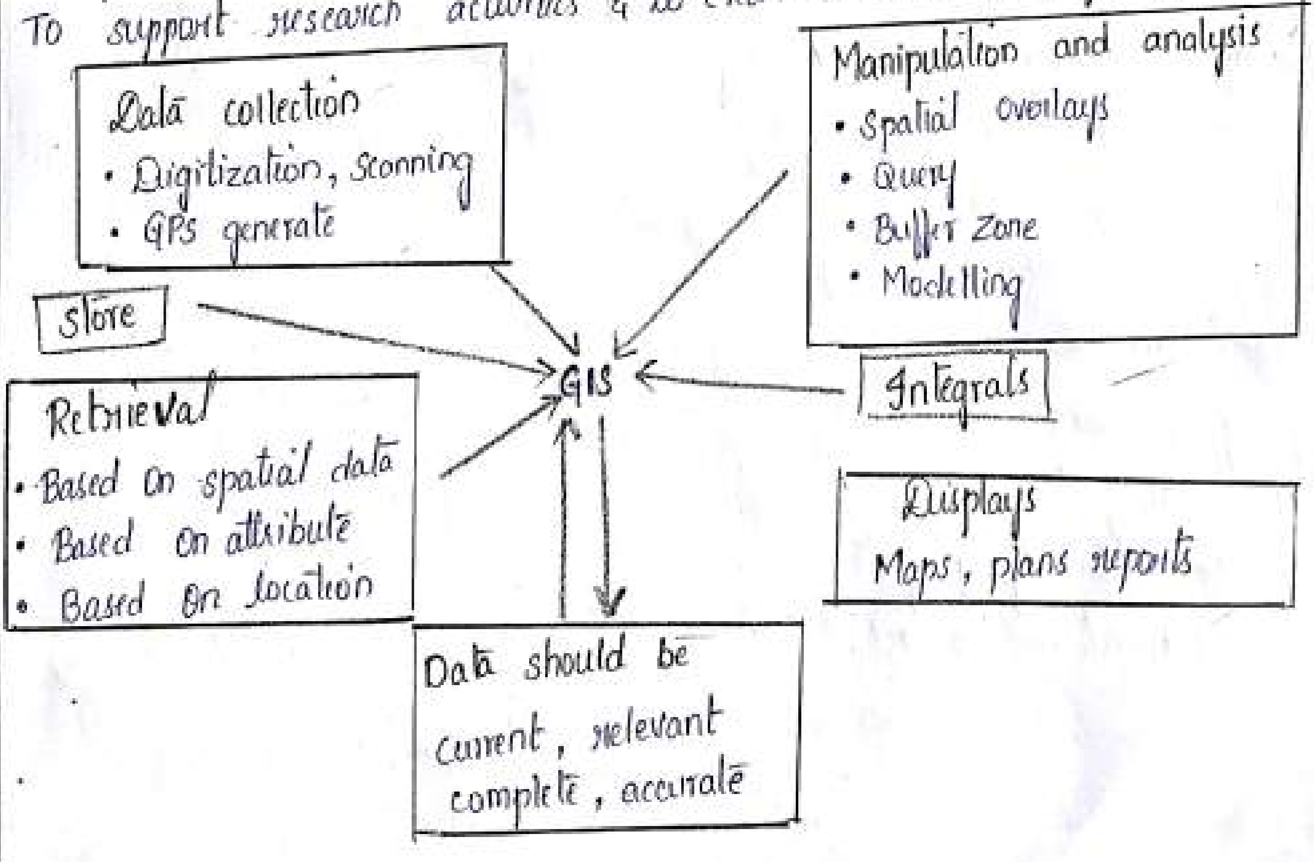
GIS [Geographic Information System]

GIS is an data management system that provides many facilities for surveyors and planners. GIS is a computer based system which collects and stores spatially referenced data with other relevant attributes and enables us to manipulate, analyse and display in suitable formats.

Data is stored in different layers. Once this geographical informat-ion system is developed, the user can access the attribute data of that place. of any place by clicking over the spatial data of that place.

Objectives of GPS

1. To collect, analyse and manipulate spatial data
2. To collect, analyse and manipulate attribute data
3. To produce maps/plans.
4. To support research activities & to create database in useful formats.



COMPONENTS OF GIS

1. Computer System : It includes CPU, VDU, keyboard, mouse, digitizer, plotter, printer, CD/DVD drive etc. to store, process and present digital spatial data

2. Software : It includes software like Arc GIS, Map, info, Geomatica, Autodesk map and other to perform GIS operations

3. Data : Geographical data in the form of hardcopy map or digital map, aerial photographs, satellite images, statistical tables and other documents are used as data for GIS operations

4. Procedure : To complete certain tasks, procedures are performed using hardware and software

5. Experts and users : Experts with knowledge are required to apply GIS appropriately. Diff types of users are using the GIS at diff levels

APPLICATIONS OF GIS

- Water resources management and planning
- Environmental & Transportation planning
- Agriculture and Land use planning
- Town and regional planning
- Forestry and wildlife management
- Muncipial applications
- Emergency planning and Routing
- Market Analysis
- Land use planning projects
- Health, Educational or Retail Services

ADVANTAGES OF GPS:

1. GPS can be used as Vehicle navigator to determine exact location, speed, direction and time
2. Finds location of any object at any time in any weather condition
3. Works 24hrs continuously
4. Manpower and time required is less
5. Accuracy is high
6. GPS has "Panic" Button. If you press panic Button an user can get help at the time of calamity, accident, highjacking etc.

Uses Or Applications Of GPS

Location

GPS is useful for finding location of any object lying on the ground, on the sea surface or in the air at any time

Navigation

GPS helps us to determine exactly where we are, GPS Technology is also useful for transportation management and bearthing of ships.

Tracking:

It is useful for monitoring vehicles and persons

1. Mass Transit
2. ship tracking
3. Vehicle tracking
4. Used by police, ambulance and fire departments widely

Mapping and Survey:

GPS maps regarding Mountains, rivers, forests, towns etc. are prepared. which are useful for

1. Conservation of natural resources

2) Preservation of scare animals
3) Forest mointoring
4. Managing effects
5. Weather forecasting
6. Geodtic surveying

Timings

GPS can also used to determine precise time, time intervals and frequency.. GPS satellites carry highly accurate atomic clocks.

1. Astronomers
2. Computer networks
3. Communication systems
4. Radio and television systems
5. Banks
6. Power companies etc

Electronic Distance Measuring [EDM] instruments are mountable with optic/electronic Theodolites.

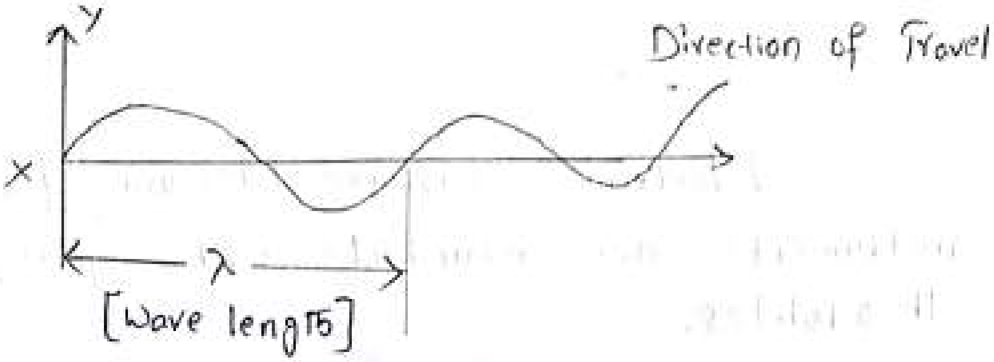
It is used only for distance Measurements.

* Principle of EDM Instrument:-

The principle of measurement device in EDM, is currently used in a total - Station or electronic/optic theodolites, that it calculates the distance by measuring the phase shift during the radiated electromagnetic wave [an infrared light or laser light or microwave] from EDM's main unit which returns by being reflected through reflector, which is positioned at a measurement point.

The wave is travelling along the X axis with a velocity of light.

The frequency of the wave is the time taken for one complete wave length.



$$\lambda = \frac{c}{f}$$

where,

λ = Wave length in metres.

c = Velocity in km/sec.

f = Frequency in hertz [One Cycle per second]

*- Uses of EDM :-

→ Measuring Distances.

→ Measuring Difference in Height.

→ Setting Out Distances.

*- Different Wave length Bands Used by EDM :-

Usually, EDM Uses three different wave length bands and their characteristics are.

* Infrared Systems :- Range upto 3 km, limited to line of Sight and affected by rain, fog, other air borne particles.

* Light Wave Systems ÷ Range upto 5 km. Visible light, lasers. Distance reduced by visibility.
* Microwave Systems ÷ Range upto 150 km. Not limited to line of sight. Unaffected by visibility.

* Features of an EDM instrument ÷

The general features of an EDM instruments are listed below.

* Distance Range : Short range EDM instruments can measure upto 1250 m using a single prism. Long range EDM instruments can measure upto 15 km using 11 prisms.
* Accuracy ÷ For short range EDM instruments ±15 mm + 5 PPM. For Long range EDM instruments : ±3 mm + 1 PPM.
* Measuring Time ÷ The measuring time required is 1.5 sec for short range measurements and upto 4 sec for long range measurements.

* Slope Reduction :- Generally automatic.
The average of repeated measurements is available on some models.

* Battery Capability :- 1500 - 5000 measurements depending on the storage capacity of battery and the temperature.

* Non-prism Measurements :- Non-Prism measurements are available with some models. They can measure upto 100 - 350 m in the case of non-prism measurements

Total station:-

Definition:- A total station is a combination of An electronic theodolite, an electronic distance Measuring device (EDM) and a microprocessor with memory unit.

Introduction:- the electronic digital theodolite was first introduced in the year 1960 by carl zeiss Inc., helped to set the stage for Modern field data collection processing. when the electronic theodolite was used with a built in EDM unit, fully automated surveying started. the earlier name of this instrument was electronic tacheometer, the current name was introduced by Hewlett- packard 30 years Back and which hitted the Market.

with this device one determine angles and Distance from the instrument to the points to Be surveyed. with the aid of trigonometry, The angles and distances used to calculate The actual positions (x, y and z (or) northing, Easting and elivation of surveyed points In absolute terms.

Total stations have an EDM and electronic Angle scanning. the coded scales of the horizontal and vertical circles are scanned electronically, And then the angles and distances are displayed Digitally. the slope distance is measured and Horizontal distance, and height difference And the co-ordinates are calculated with the Help of the built-in programmes and all Measurements and additional information Can be recorded

the EDM Instrument transmits an Infrared beam/ laser beam, which is Reflected back to the EDM unit with the help of a prism, and the EDM uses timing Measurements to calculate the distance Travelled by the beam. with few exceptions, The EDM Instrument require that the Target be highly Reflective, and a reflecting prism is normally used as the target.

Generally, the total stations include Data Recorders. the raw data (Angle and Distances) and coordinates of the points sighted are Recorded along with some Additional information.

Sighting collimator:- It consists of a small △ Mask at top and bottom of telescope. By coinciding the apex of triangle with the prism pole, Rough sighting can be done.

On-Board Battery:- A Rechargeble battery is provided on the side of the instrument. some Modern instrument are having two batteries

Battery Locking Lever:- By pushing the lock Button the battery can be detached from the Instrument for charging purpose.

Telescope Eye piece:- the telescope eye piece is Rotated clockwise or anticlockwise, to view The cross hairs clearly.

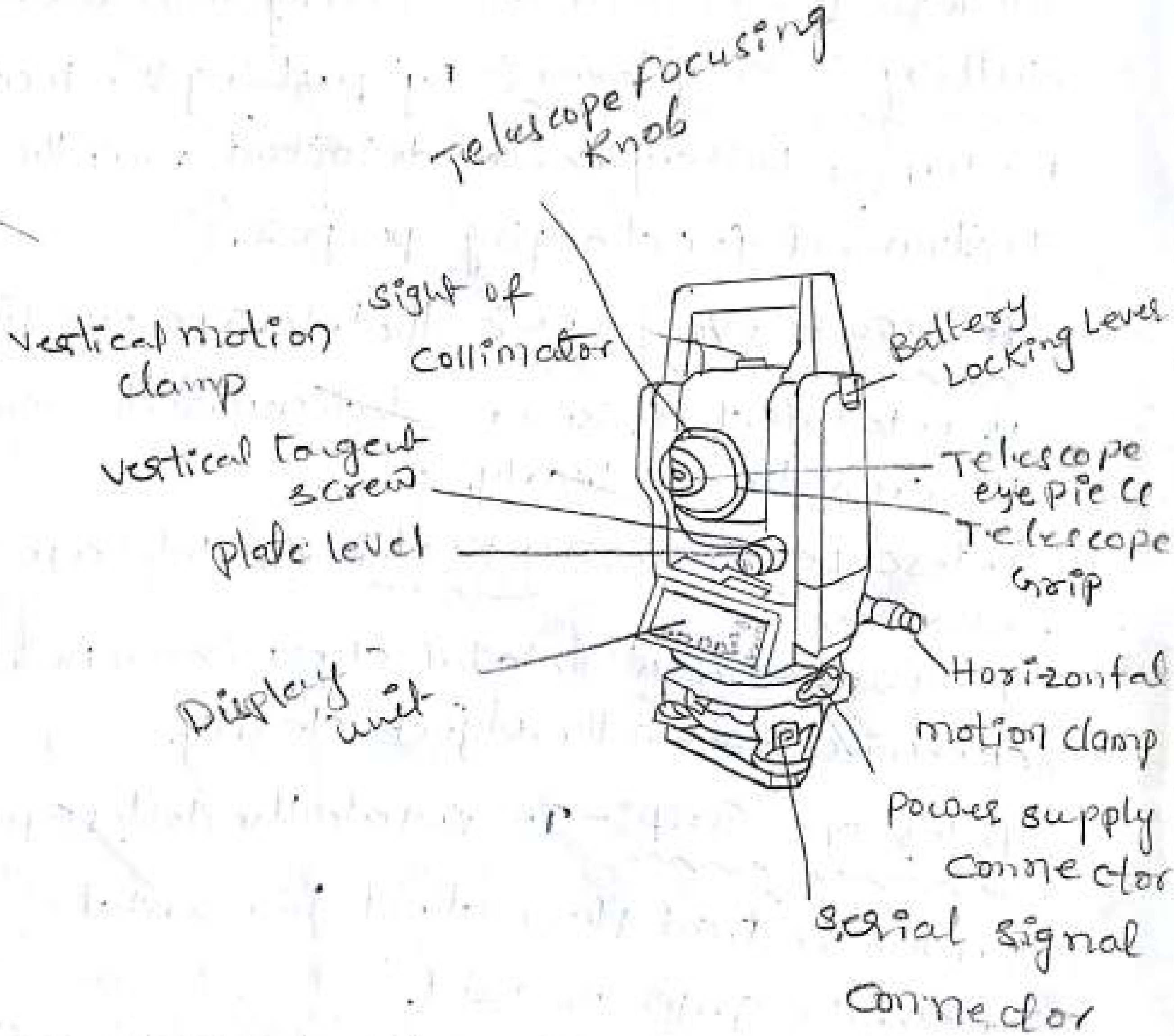
Telescope Focusing Knob:- the telescope focusing knob is Rotated clockwise (or) Anti-clockwise to view the object clearly.

Telescope Grip:- To Rotate the telescope In the vertical plane about horizontal Axis, the grips are used.

Vertical motion clamp:- By clamping this screw The telescope cannot be rotated in vertical plane about the horizontal axis.

Component parts and their functions:-

The following diagram shows the components parts of a total station. the purpose of each components is explained below



Scanned by CamScanner

Vertical Tangent screw:- By Rotating this screw in or out the horizontal cross-hair is bisected with the prism

Horizontal motion clamp:- By clamping this screw, the telescope cannot be rotated in horizontal plane, about vertical axis

Horizontal Tangent screw:- By Rotating this screw in or out the vertical cross hair is bisected with the prism pole, exactly

Instrument Centre mark:- the height Measured from ground to this mark gives The height of instrument, which is to be entered as input data.

plate level:- As like in levelling Instrument A plate level is provided to make the line of collimation horizontal.

Display unit:- most of the Instruments Have two display units, one on each side. The display unit is generally having LCD (or) LED, multi line display, input data like prism height, Instrument height, station name, co-ordinate values of station

Are entered through alpha Numeric Keyboard attached with the display unit The angular and linear measurements After processing by the system are displayed on the displayed on the display unit

Serial signal connector:- through this port a cable can be connected to the computer To Transfer the data from Instrument to the computer and vice versa.

Types of Total stations:-

In the early days, three classes of total stations were available Manual, semiautomatic and automatic.

Manual Total station: it was necessary To Read the horizontal and vertical angles Manually in this type of Instrument. The only value that could be read electronically was the slope distances

Semiautomatic Total stations:-

the user had to manually read the horizontal angles, but the vertical angles were shown digitally. slope distances were measured electronically and the

Instruments could in most cases, Be used to reduce the values to horizontal And vertical component.

Automatic Total stations:- this type is the most common Total station used now-a-days they sense both the horizontal and vertical Angles electronically and measure the slope Distances, compute the horizontal and vertical components of those distances, and Determine the co-ordinates of observed points

Advancement in Total station technology:-

1) Servo-Driven and Robotic Total stations: Refinement to an existing Technology are the classes of servo-driven And Robotic optical total stations. their Added functionality makes them suitable for intense mapping. Because of their capacity to improve the surveying operation significantly, they can be classified into a separate group.

2) Servo- Driven and Robotic Total stations:-

these Instruments are particularly appealing where automatic pointing is Desired. This is done by using motors to Aim and position the Instrument. In the Case of setting out, it makes it feasible TO set control points for surveying with very little sighting through the telescope

when these Instruments are used manu-ally, because they are servo-driven, they Have friction clutches that afford Great speed in pointing, as there are no locks to be adjusted. the servo-driven Instrument has the disadvantages of Data collection and coding occurring At the Instrument it is also mandatory that atleast two people be on the crew

Setting up Total station for Taking observations:-

the following steps are followed to setup of a Total station.

1) select a suitable position for Instrument station, such that an observer can safely operate the Instrument.

2) Remove the plastic cap from the telescopic Tripod, and leave the Instrument in the case until the Tripod is nearly level

③ open the strap of the Tripod legs, release all clips, so that the height of the tripod legs are so adjusted that, after stretching them, the Tripod head is at the level of operators chest, then press the clips

④ the Instrument height is important for an effective and comfortable survey It differs in looking down position and the looking up position. one should not touch or cling to the tripod during the survey.

5) AT a new station without a reference point on the ground, level up the Total station at an arbitrary point, where a stake can easily go in and be steady And put down the stake at the centre Using the plummet.

6) To occupy an existing station above a reference point, first roughly level up The Tripod head right above the point For levelling up, circular bubble attached To the level is usefull, To find out the position, use a plumb-bob or drop a stone pebble through the hole in the Tripod Head.

7) Check the level and adjust the level By changing the leg length.

8) Fix a Tribranch with a plummet, a Tribranch and a prism carrier with A plummet, or a total station with a Built in Plummet on the Tripod head.

9) if levelling is failed, adjusted foot screws of tribranch and repeat the process.

10) put the Total station on tribranch, if it is not there

11) tighten the fixing screw firmly without applying two much pressure Never loosen the screw untill all Measurements are finished.

12) measure the Instrument height, i.e from the mark on the side of the Instrument to the ground and note it.

Advantages and Disadvantages of Total station:-

Advantages:-

1) Quick setting of the Instrument on the Tripod by using laser plummet.

2) Area calculation programme computes And displays the Area of the field by simple observations.

3) on screen, graphical view of plots And land displays the area of the field. By simple observations.

3) on screen, graphical view of plots And land can be made for quick visualization.

4) As soon as the field work is finished the map of the Area with dimensions can be got as output, after data Transfer

5) plotting and Area computation at Any user required scale can be done

6) Integration data base (exporting Map to GIS packages) is possible

7) Using Robotic total station single surveyor can perform surveying work

8) Automation of old maps and full GIS Creations (using map info software) is possible.

Disadvantages:-

1) their use does not provide hard copies of field notes.
Hence field check is not possible
2) For an overall check of the survey, it is necessary to return to the office and prepare the drawings, using appropriate software
3) they should not be used for observations of the sun unless special filters are used if not, the EDM part of the instrument will be damaged.
4) the Instrument is costly and skilled persons are required to operate it.

Total station initial setting:-

the following steps are performed for the initial setting of a total station

* By pressing on button, turn on the Total station display unit
* Release both horizontal and vertical

locks (modern total starting having Auto lock, system hence this operation not required.

* some total stations require, rotating the telescope through 360° along the vertical And horizontal circles to initialize Angles.

* Adjust the eye piece by rotating Inner ring to see the image of the cross-hairs sharp and clear

* Rotate the telescope, by aiming back sight (azimuth) for the approximate Aiming use collimator and rotate the Telescope in horizontal plane, after sighting the prism pole clamp horizontal screw and bisect the pole perfectly by looking through telescope and rotating slow motion (tangent) screw.

* input the azimuth of the back sight Manually in the measurement set-up window (ie, set 0° 00' 00'')

horizontal angles.

* if a station ID and back sight ID Are required, use a 2 or 3 digit serial Number line 101, 102 ---- for each Reference point, use a 4-digit number for unknown points.

* Input station parameters like HI (height of Instrument), Easting, Northing And Altitude (R.L) of the point, where the Instrument is setup, Use 1000, 1000 And 500 for Easting, Northing and (RL) Of the point to avoid negative figures if the exact co-ordinates are known Manually input the data.

* Input the target height (height of Reflector (ie., prism))

* Check the pointing at the prism Again and press measure Key, and Make the back sight measurement

From the LCD display note the horizontal Angle, the vertical angle, slope distance Easting, Northing and RL, and record them in a field book with a Index plan of the Area.

* Create a new Job open an existing Job. A Job is a block of data sets stored in the memory like a file. ~~A Job~~ name is used as an input file Name in total station.

Field book Recording:-

The observer can record all numerical Data and a little text data in the Total station, but descriptive Information and graphic information should be recorded in the field book. The following points can be recorded In the field book.

1) place , data and time
2) Surveyor's name and other members
3) Temperature and atmospheric pressure
4) Station coordinates (Easting, Northing Altitude), UTM by GPS and height of the Instrument
5) Back sight coordinates (slope