Module - 1 Date: 08/08/18

Unit - 1: Introduction

Surveying

Surveying: It is the art of determining the relative positions of point on, above or beneath the surface of the earth by means of direct or indirect measurements of distance, direction & elevation.

Objectives & importance of surveying:

→ Surveying enables us to acquire data on a relative positions, horizontal distances & elevations of points. These data are thus used for operations such as preparation of plans or maps & land records, laying out engineering works & calculating areas & volumes.

→ The objectives of surveying can thus be stated as

i) collect & record data on the relative positions of points on the surface of the earth.

ii) Computations of areas & volumes using surveying data required for various purposes.

iii) To prepare the plans & maps required for various activity.

iv) To laying out various civil engineering works in correct positions.

classification of Surveying:

Primary classification:

→ Primarily surveying can be divided into 2 classes

* Plane surveying
* Geodetic surveying

*. Plane Surveying :- It is the type of surveying in which the mean surface of the earth is considered as a plane & the sph̊eroidal shape is neglected. All triangles formed by survey lines are considered as plane Δ^{les}. The level lines [Horizontal line] is considered as straight & plumb lines are considered as parallel. Accuracy of is quiet less.

*. Geodetic surveying :- It is a type of surveying in which the shape of the earth is taken into account. All lines lying in the surface are curved lines & Δ^{les} are spherical Δ^{les}. Standard of accuracy is quiet high in this type of surveying.

Secondary classification:

a) classification based upon nature of the field survey:

→ There are 3 types, they are

i) Land surveying
ii) Marine or hydrographic survey
iii) Astronomical survey.

i) Land survey: This survey is further classified into 3 types. They are

* Topographical survey:- This consist of horizontal & vertical location of certain points by linear & angular measurements & is made to determine the natural features of a country such as rivers, streams, lakes, hills, woods, etc & artificial features like roads, railways, canals, towns & villages.

*. Cadastral survey:- These are made to fix the property lines, calculation of land area, the transfer of land property from one owner to another owner. They are also made to fix the boundaries of muncipalities & state & federal jurisdictions.

*. City survey:- They are made in connection with the construction of streets, water supply systems, sewers & other works.

ii) Marine or hydrographic survey:- It deals with water bodies for purpose of navigation, water supply, Harbour works or for determination of mean sea level.

iii) Astronomical survey:- It offers the surveyor for determining the absolute location of any point or the absolute location & direction of any line on the surface of the earth. This consists the observations to the heavenly bodies such as the sun or any fixed star.

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b) classification based on the object [Purpose] of the survey

i) Engineering Survey: This is undertaken for the determin-ation of quantities or to afford sufficient data for the designing of engineering works such as roads & reservoirs or those connected with sewage disposal or water supply

ii) Military survey: This is used for determining the points of strategic importance.

iii) Mine survey: This is used for exploring mineral wealth.

iv) Geological Survey: This is used for determining different strata in the earth's crust

v) Archeological survey: this is used for unearthing relics of antiquity [existance of ancestors]

c) Classification based on instruments used

i) chain Surveying:- It is a simplest type of survey in which only linear measurement is taken with the help of chain & tape.

It is generally used when accuracy is not required

ii) Compass survey: In compass survey the horizontal angles are measured with the help of magnetic compass in addition to it linear measurements are taken with the help of chain or tape.

iii) Levelling survey: It is a type of survey in which levelling instrument [Dumpy level] is used for determining relative elevations of various points in a vertical plane.

iv) Plane table surveying: It is a technique where the field work & a part of paper work is done simultaneously in field.

Accuracy in plane table surveying is partly less.

v) Theodolite Survey:- A survey carried out by using theodolite for finding vertical & horizontal angle is known as theodolite survey.

vi) Tacheometry Surveying [Telemetry surveying]:- In this, a special type of theodolite is used to measure vertical & horizontal angles.

Horizontal & vertical distance can also be calculated with the help of stadia readings.

vii) Photogrammetric survey:- It is a science of taking measurement with the help of ~~photograph~~ ph.

viii) EDM survey [Electronic Distance Measurement]:- EDM involves generation, transmission, reception from a reflector at the station of micro wave signal.

Phase difference b/n transmitter & recieved signals enables the distance b/n intruments station & reflective station to be calculated & displayed or stored.

imp *** Principles of Surveying:

→ There are 2 principles in surveying, they are

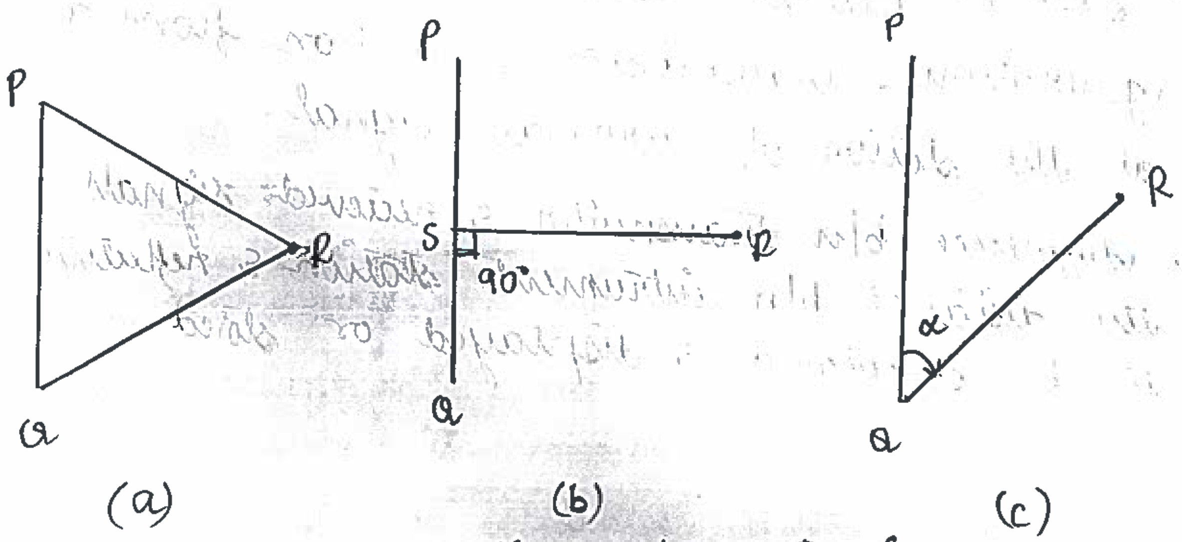
1. Location of a point by measurement from two points of reference

2. working from whole to part.

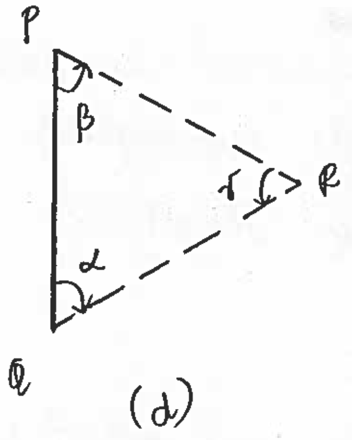
1. Location of point by measurement from 2 points of reference:- The relative positions of the points to be surveyed should be located by measurement from atleast 2 points of reference, the positions of which have already being fixed.

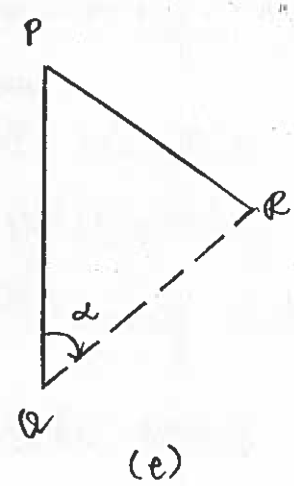
Let P & Q be the reference points on the ground. The distance PQ can be measure accurately & the relative positions of P & Q can be plotted on the sheet to some scale. The points P & Q will thus serve as reference points for fixing the relative positions of other points.

Any other point, such as R can be located by direct methods like :



Location of a Point





11/08/18 ** Location of a point

11/08/18 a). Distances PR & QR can be measured & point can be plotted by swinging the 2 arcs to the same scale to which PQ has been plotted. The principle very much used in plane surveying.

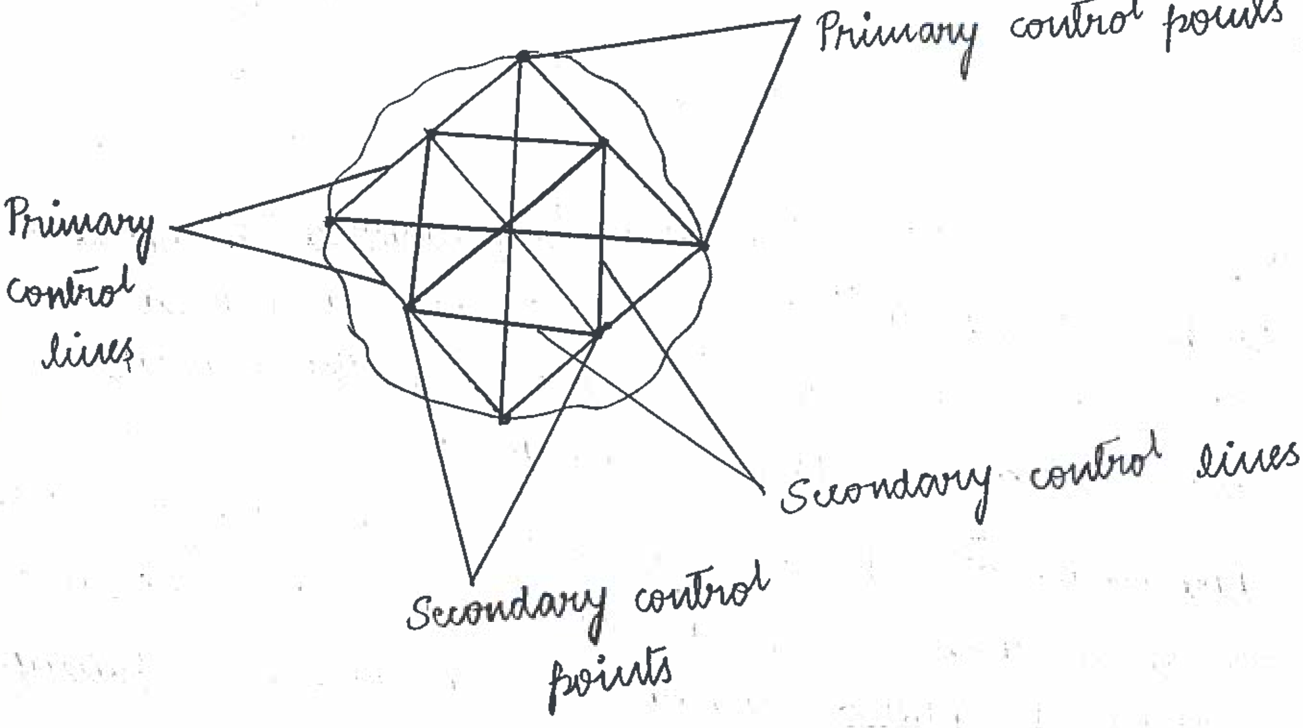
b). A perpendicular SR can be dropped on the reference line PQ & lengths PS & SR are measured. The po R can then be plotted using set square. This prin is used for defining details.

c). The distance QR & angle $P\hat{Q}R$ can be measured & point R is plotted either by means of protracter or trigonometrically [By using trignometric function]. This principle used for traversing.

d). In this method, the distances PR & QR are not measured but angle $R\hat{P}Q$ & angle $R\hat{Q}P$ are measured with an angle measuring instruments. Knowing the distance PQ, point R is plotted either by means of protract & by solution of ΔPQR. This principle is very mu used in triangulation.

e). $\angle RQP$ & distance PR are measured & point R is plotted by protracting an angle & swinging an arc from P. This principle used in travelsing.

2F. Working From Whole to Part



→ Establish first a system of control point & fix them with higher precision.

→ The line joining the control points will be control lines.

→ Minor control points can then be established by less precise methods & details can then be located using these minor control points by running minor travels [Joining of survey lines]

→ The idea of working in this way is to prevent the accumulation of errors & to control & localise minor errors.

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Units of Measurements (or) Surveying Measurements

→ In surveying, there are 4 kinds of measurements

1. Horizontal distance: It is measured in horizontal plane.

2. vertical distance: It is measured along the direction of gravity at the point.

3. Horizontal angle: It is measured b/n 2 lines in a plane ie, horizontal at the point.

4. vertical angle: It is measured b/n 2 lines in a plane ie, vertical at the point.

SI units:

→ International system of units is abbrivated as SI units.

$1m = 100\,cm = 10^2\,cm$

$1m = 1000\,mm = 10^3\,mm$

$1m = 0.001\,km = 10^{-3}\,km$

Linear Measurements:

1 decametre $= 10\,m = 10^1\,m$

1 hectametre $= 100\,m = 10^2\,m$

1 kilometre $= 1000\,m = 10^3\,m$

1 Megametre $= 10^6\,m$

1 Gigametre $= 10^9\,m$

1 millimeter $= 10^{-3}\,m$

1 micrometre = 10^{-6} m

1 nanometre = 10^{-9} m

Angular Measurements: There are 3 systems usually used for angular measurements

1) Sexagesimal System:

→ one circumference = 360° [degrees of arc]

→ one degree = $60'$ [minutes of arc]

→ one minute = $60''$ [seconds of arc]

2) Centesimal System:

→ one circumference = 400^g [Grades]

→ one grade = 100^c [centi Grad]

→ one centigrad = 100^{cc} [centi centi grad]

3) Hours System:

→ one circumference = 24^h [Hours]

→ one hour = 60^m [minutes]

→ one minute = 60^s [seconds of time]

Surveying Errors:

→ The difference b/w measured & true value of the quantity is the "true error" of the measurement."

- **Sources of Error:**

→ Basically there are 3 sources of error, they are,

1) Instrumental Error

2) Personal Error

3) Natural Error

1) Instrumental Errors: These are errors which occur due to malfunctioning of the instruments. In-correct graduations on a tape is an example

2) Personal Errors: These errors are due to limits on human perception, reading of a vernier scale by judging the co-incidence of graduations can be a source of error. similarly, sighting & bisecting an object accurately can be a source of error.

3) Natural Errors: This errors occured due to temperature variations, magnetic substances, refraction, etc.

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Types of Errors: Errors which occur in survey mainly classified into 3 types, they are

i) By mistake

ii) Systematic Error

iii) Accidental Error

i) By mistake: Mistakes are errors which arrives from inattention, inexperience, carelessness & from poor judgement.

If the mistake is undetected, it produces severe of errors upon the final results. These errors are very large & can be easily detected.

These errors can be avoided by,

→ carefully targeting the object & taking readings
→ Taking additional readings for checking.
→ The person who takes the reading should loudly announce the reading so that recorder hear the reading properly.

ii) Systematic Error: Errors which follow well defined pattern are classified as systematic errors. Since, systematic error always follows the definite pattern, correction can be determined by mathematical expressions.

This type of errors are called cumulative errors since each measurements adds to the errors in the same sense.

iii) Accidental Error [compensating Error]: These are errors ~~are error~~ which remain after mistakes & systematic errors & this, is beyond the ability of the observer to control.

Their magnitude may vary from reading to reading.

They are propabalistic since it can not be estimated using standard functional relation.

Eg: Human eye has a limitation of distinguishing b/w 2 close readings

****imp****

Precision and Accuracy:

→ Precision is a degree of perfection used in by instruments, the methods & observations.

→ Accuracy is a degree of perfection obtained

Accuracy depends on,

i) Precise instruments

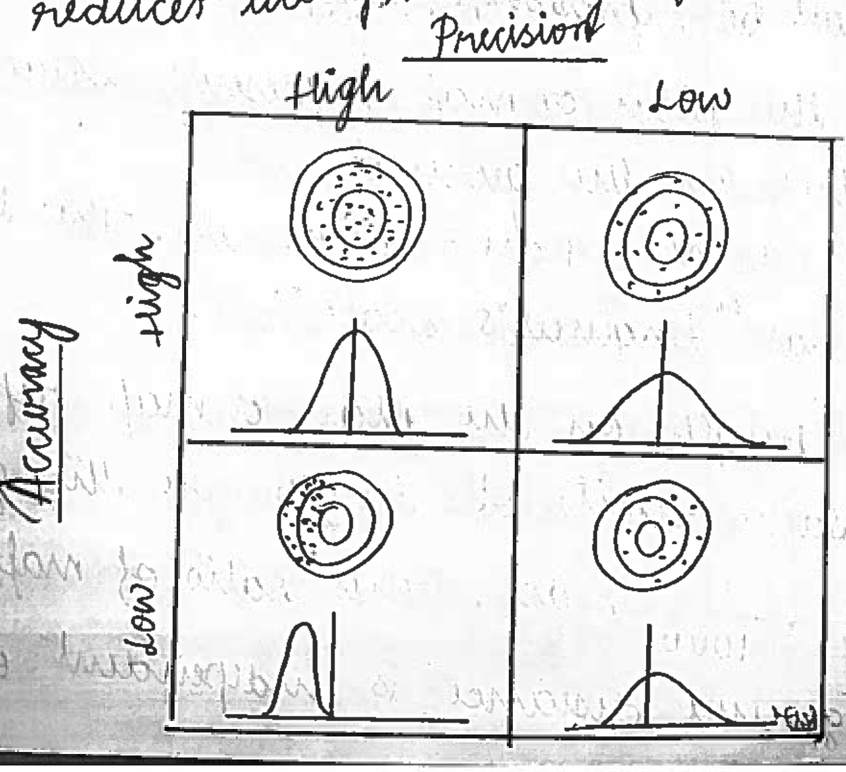
ii) Precise methods

iii) Good planning

The use of precise instruments simplify the work, save time & provide economy.

The use of precise methods eliminate or try to reduce the effect of all types of errors.

Good planning, which includes proper choice & arrangements of survey control & the proper choice of instruments & methods for each operation. saves time & reduces the ~~probability~~ possibility of errors.



** imp Plan and Map:

→ A map is a graphical representation of larger area using smaller scale.

→ A plan is a graphical representation of smaller area using larger scale.

Classification of maps:

1. Larger scale map : 1 cm = 10 m or less than that
2. Medium scale map : 1 cm = 1100 m upto 1 cm = 100 m
3. Smaller scale map : Any value which is more than 100 m

Map Scale: The area that is surveyed is vast & therefore plans or maps are made to some scale.

→ Scale is a fixed ratio that every distance on the plan or map ~~bo~~ bears indicate with corresponding distance on the ground.

Scale ~~will~~ can be represented by 2 methods:

i) 1 cm on the plan or map represents some whole number of metres on the ground. Such as 1 cm = 10 m, 1 cm = 100 m, etc. This type of scale is called as "Engineer's Scale"

ii) 1 unit of length on the plan or map represents some no of same units of length on the ground. Such as $\frac{1}{500}$, $\frac{1}{1000}$, etc. This ratio of map distance to the corresponding ground distance is independent of units

of measurement & is called "representative fraction" (RF
The RF can be very easily found for a given engine
-r's scale.

Eg: If the scale is 1 cm = 50 m,

then RF = $\frac{1}{5000}$ ie, 50 × 100 = 5000

The engineer's scale & RF are also known as Numerical scale.

Convensional Symbols

1. Triangulation Station.	2. Traverse Station	3. Tie Station.	4. Chain Line.
5. Wood fencing.	6. Pipe Railing.	7. Wire Fencing.	8. Demarcated property Boundary.
9. Undemarcated Property Boundary.	10. Compound Wall.	11. Stream.	12. River.
13. Cart track.	14. Canal.	15. Railway line.	16. Railway Double Line.
17. Unmetalled Road.	18. Metalled Road.	19. Pucca Building.	20. Katcha Building.
21. Hedge	22. Trees.	23. Woods.	24. Orchard.
25. Cultivated Land.	26. Swamps.	27. Culvert.	28. Bridge
29. Embankment.	30. Cutting.	31. Railway Bridge.	32. Temple.
33. Mosque.	34. Church.	35. Pond or lake.	36. North line.
37. Gates.	38. Well.	39. Bench Mark. BM 15,000	40. Pucca Drain.
41. Katcha Drain.	42. Electric Line.	43. Shed.	44. Gate & wall.
45. pasture.	46. Cemetry.	47. Foot path.	48. Lawn.

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Topographic Maps:

→ A topographic map is a detailed & accurate 2 dimensional representation of natural & human made features on the earth's surface. These maps a used for a number of applications from camping, hunting, fishing & hiking to to urban planning, resource management & surveying.

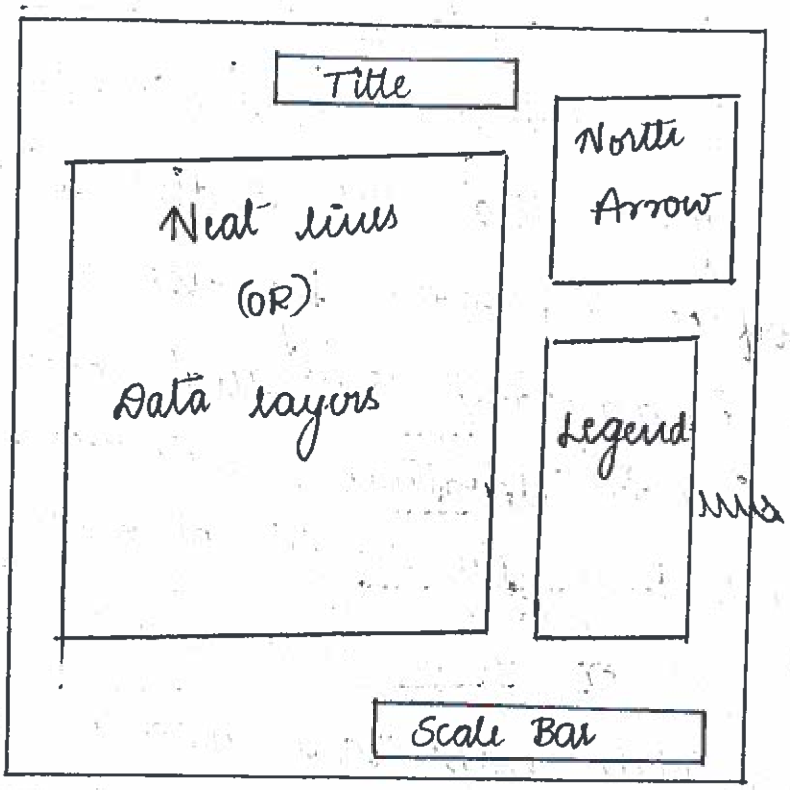
The most distinctive characteristics of a topographic map is that the 3 dimensional shape of the earth surface is moulded by use of contour lines. Contours are imaginary lines that connect locations of ~~similar~~ similar elevations. Contours make it possible to represent the height of mountains & steepness of slopes on a 2 dimensional map surface.

Topographic maps also used in a variety of symbols to describe both natural & human made features such as roads, buildings, quarries, lakes, st & vegetations.

Map Layout:

→ The process of map composition starts with preparing a layout for the map. Apart from the data a map has certain other things that make map a package of effective & clear communication. These provide critical informations to uses & are known a map elements.

A layout specifies the space & positions for different map elements such as neat lines [Data layers], title, north arrow [direction], scale bar, legend, etc. Preparing an effective layout often requires experimentatio with the available space.



Map Layout

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Survey of India Map Numbering System:

→ The system followed by the survey of India for map numbering is as follows

1. Each sheet is bound by 4° lattitude & 4° langitude
2. The sheet numbering starts from north west corner 40° north lattitude & 44° East as first sheet & increas from north to south.
3. Sheets falling in the sea are not numbered.
4. Sheets covering India are numbered from 39 to 58.

5. 4° lattitude & 4° longitude sheets are called $\frac{1}{M}$ sheets. [million sheets]

6. The $\frac{1}{M}$ sheets are further divided into 16 equal parts of $1^\circ \times 1^\circ$.

7. The sheets are in 1 : 25000 scale.

8. These sheets are numbered from 'A' to 'P' & each grid is called by the sheet no followed by the alphabet

i.e, for $\frac{1}{M}$ sheet of 55, 16 components are 55A to 55P.

70° 71° 72° 73° 74°

24°				
	A	E	I	M
23°				
	B	F	J	N
22°				
	C	G	55 K	O
21°				
	D	H	L	P
20°				

The $1^\circ \times 1^\circ$ map (degree sheet) is subdivided into 16 equal parts. Each of 15' minutes of lattitude & 15' of longitude (15' × 15').

These sheets are numbered from 1 to 16. & each grid is called by the degree sheet number followed by the number from 1-16.

i.e., for the 55A degree sheet, the 16 components are 55A1 to 55A16 & scale is 1 : 50,000

70° 70°15' 70°30' 70°45' 71°

24°				
	1	5	9	13
23°45'				
	2	6	10	14
23°30'				
	3	7 55A	11	15
23°15'				
	4	8	12	16

Module - 01

Unit - 02

Measurement Of Horizontal Distance

Chain surveying:

→ Measurement of horizontal distance by using chain (or) tape is known as chain surveying.

Measuring tapes & types

→ Tapes are used for measuring lines & off sets & are classified according to material of which they are made. such as

1. cloth or linen tape.
2. Metallic tape
3. steel tape
4. Invar tape.

1. Cloth or Linen tape:

→ There are available in lengths of 10, 20, 25 & 30 metres

→ These tapes are light & flexible & hence easy to handle.

→ Width of tape will be 12 to 15 mm.

→ The entire tape will be varnished to resist moisture

→ A cloth tape is rarely used for making accurate measurements, because of the following reasons.

i). It is easily affected by moisture or dampness

& thus shrinks.

ii). Its length get alter by stretching

iii). They are not strong & they are likely to get twisted.

2. Metallic tape :

→ These tapes are available in lengths of 12, 20 30 & 50 metre

→ Metallic tapes are light & flexible & are not easily broken.

→ These are made up of varnished strip of water proof linen interwoven with a small brass, a or Bronze wires & does not stretch as easily as cloth tape.

→ The width of tape will be 12-15 mm.

3. Steel tape :

→ It consists of 6-10 mm wide strip of steel with metal rings at free end.

→ These are available in lengths of 1m, 2m, 1 20m, 30m & 50m.

→ Steel tapes are superior to metallic tapes as per as accuracy is considered.

4. Invar tape :

→ It is made up of an alloy of Ni & steel, which has very low co-efficient of thermal expansion

→ The width of tape is 6mm & it is available in lengths of 30m, 50m & 100m.

→ It is most accurate tape but it is expensive.

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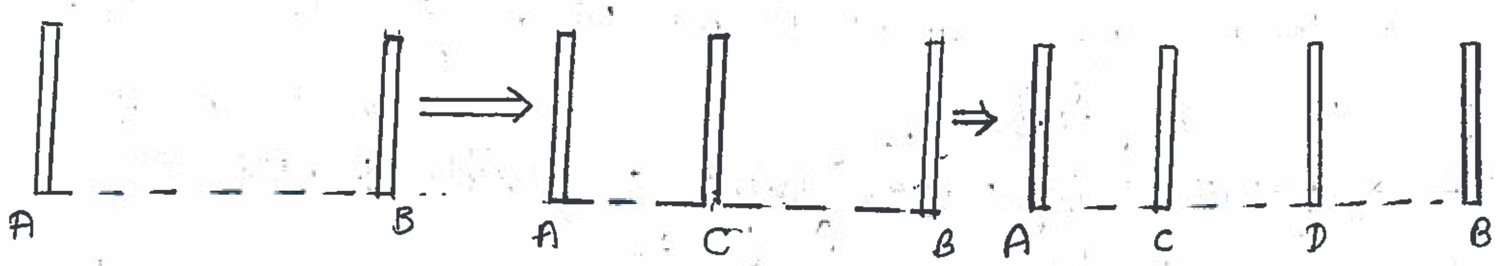
Taping on level Ground:

Direct Ranging: It is possible only when first & last points on survey line are inter visible.



→ Let A & B be the 2 points at the end of survey line.

→ One ranging rod is erected at B while surveyor starts at another ranging rod at A.

→ An assistant then goes with another ranging rod & establishes the another ranging rod at point approximately inline with AB at a distance not greater than 1 chain length at A

→ Surveyor at A signals the assistant to move towards the chain line till he is inline with A & B

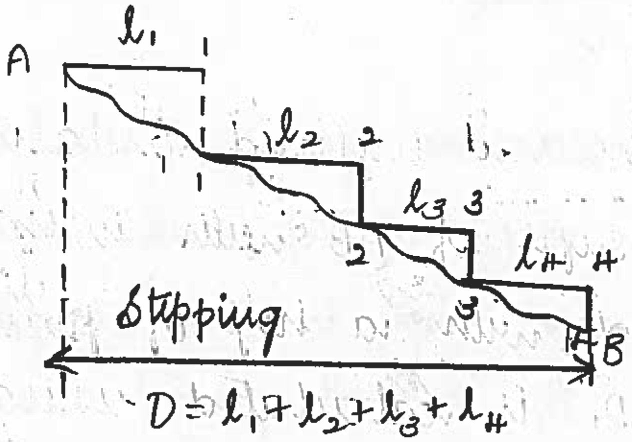
→ III^ly other intermediate points can be established b/w the points A & B & then measure the distance

Taping on Sloping Ground:

→ There are 2 methods of measurement of distance over sloping ground; they are

1) Direct method
2) Indirect method

1) Direct method: It is also known as method of stepping.



→ Let AB be the length of line to be measure on a sloping ground.

→ The distance l_1, l_2, l_3 & l_4 are taken to our convenient distance depending upon the slope [Ground Profile] using tape

→ The points 1, 2, 3 & 4 are transfer to the ground using plumba.

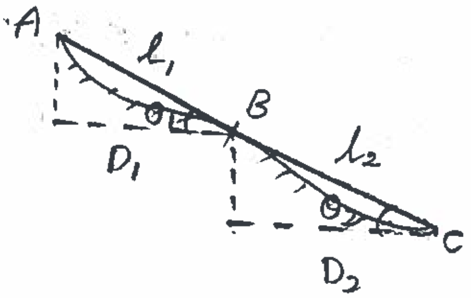
→ The total distance b/n A & B is given by,

$D = l_1 + l_2 + l_3 + l_4$

→ This method is suitable only when there is a irregular slope

2) Indirect method:

case i) Angle measured: In this case, total length to be measured is divided into several segments. Each segments will be having particular slope.



→ Let l_1 be measured inclined distance b/n AB. & θ_1 is the slope of AB with horizontal.

→ θ_1 is measured with a help of ~~plan~~ clinometer

→ The distance D_1 is calculated using the formula

$$\cos\theta_1 = \frac{D_1}{l_1}$$

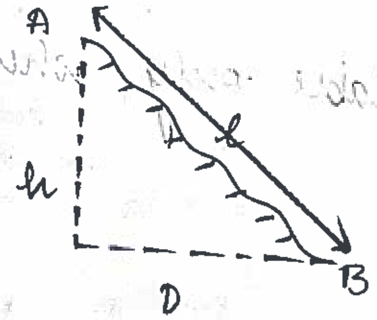
$$\therefore D_1 = l_1 \times \cos\theta_1$$

$$\text{III}^{ly} \quad D_2 = l_2 \times \cos\theta_2$$

In general, $D_n = l_n \times \cos\theta_n$

→ The horizontal distance $D = D_1 + D_2 + \ldots + D_n$

case ii) Difference in level measured [Elevation measured]:



→ In this method, using levelling instrument [Dumpi level] the difference in elevation b/n points A & B [ie, h] is measured.

→ After measuring sloping ground l, the equivale
horizontal length D can be calculated as,

$l^2 = h^2 + D^2$

$\therefore D^2 = l^2 - h^2$

$D = \sqrt{l^2 - h^2}$

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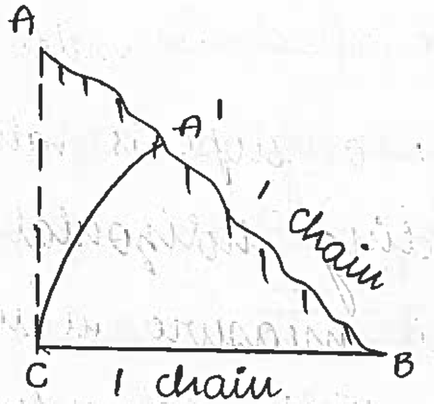
case iii) Hypotenusal Allowance:

B.



→ In this method, a correction is applied in field at every chain length & at every point whe the slope changes.

→ When the chain or tape is stretched on the slope, the arrow is not pegged at the end of chain but it is placed in advance of the end by an amount which allows the slope correctio

In the figure, BA' is one chain length [20m or 30m] on slope, the arrow is not pegge at A' but it is pegged at the point A. The distance AA' is known as hypotenusal allowance

BA = 100 sec θ links

BA' = 100 links

Hence, Hypotenusal Allowance (HA) = AA'

$AA' = BA - BA'$

$= 100 \sec\theta - 100$

$\boxed{HA = AA' = 100(\sec\theta - 1) \text{ links for 20 m}}$

$HA = AA' = 150(\sec\theta - 1)$ links for 30 m

(or) $HA = l(\sec\theta - 1)$ m

Where l = length of chain in metre.

Note: The length along a slope is always more than the corresponding horizontal distance. Thus it is possible to measure a longer distance along a slope such that equivalent horizontal distance is known.

PROBLEMS:

1). The distance b/w 2 points measured along slope is 428 m. Find horizontal distance b/w them.

a). Angle of slope is 8°

b). Difference in level is 62 m

c). Slope is 1:4

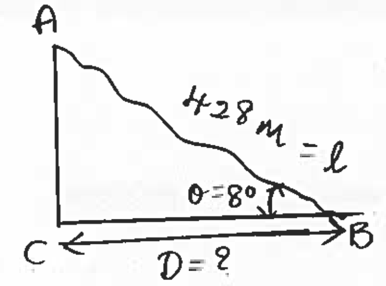
→ $l = 428$ m

a) $\theta = 8°$

b) $h = 62$ m

c) Slope of 1:4 opp : Adj

a) Angle measured.

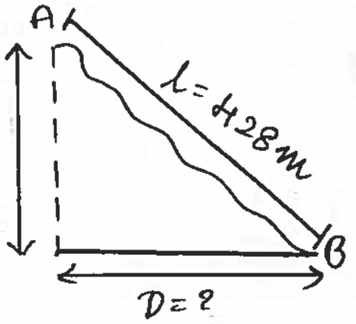


$D = l\cos\theta$

$= 428\cos(8)$

$D = 423.83$ m

b) Difference in level measured

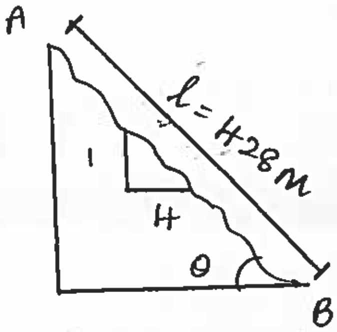


$D = \sqrt{l^2 - h^2}$

$= \sqrt{(428)^2 - (62)^2}$

$D = 423.485$ m.

c)



$\tan\theta = \frac{1}{4}$

$\theta = \tan^{-1}(1/4)$

$\theta = 14.036^\circ$

$= 14^\circ\, 2'\, 11''$

$D = l\cos\theta$

$= 428\cos(14.036)$

$D = 415.220$ m

27. The distance b/n the points measured along slope is 129 m. Find the horizontal distance them, if

i) Angle of slope is $6^\circ 30'$

ii) Difference in level is 30 m

iii) slope is 1:4.

$\rightarrow \quad l = 129\ m$

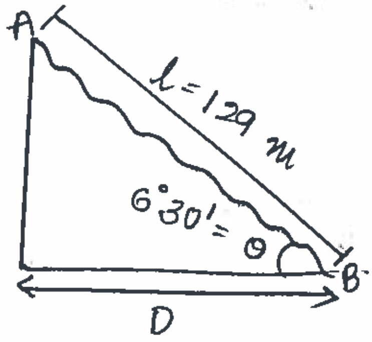
i) $\theta = 6^\circ 30'$

ii) $h = 30\ m$

iii) slope of 1:4.

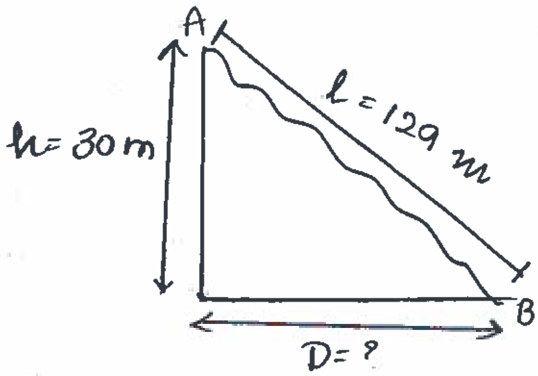
i)

$$D = l\cos\theta$$
$$= 129\cos(6.5)$$
$$= 128.17\ m$$



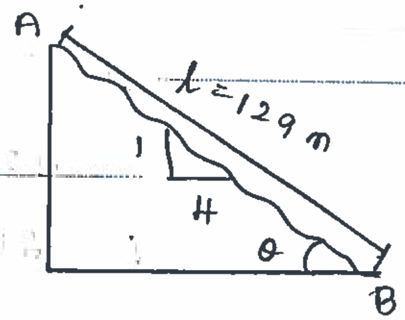
ii)

$$D = \sqrt{l^2 - h^2}$$
$$= \sqrt{(129)^2 - (30)^2}$$
$$D = 125.463\ m$$



iii)

$$\tan\theta = \frac{1}{4}$$
$$\theta = \tan^{-1}(1/4)$$
$$= 14.036^\circ$$
$$= 14^\circ 2' 11''$$



$$D = l\cos\theta$$
$$= 129\cos(14.036)$$
$$D = 125.14\ m$$

35. Find hypotenusal allowance per chain of 20 m length. If i) Angle of slope is 10°

ii) Ground raises by 4m in 1

chain length

→ Length of chain = 20 m = 100 links

i) $\theta = 10°$

ii) slope H : 20 ie 1 : 5

i) Hypotenuseal Allowance :

$$HA = 100(\sec\theta - 1) \text{ links}$$

$$= 100(\sec 10 - 1)$$

$$= 100\ (1.0154 - 1)$$

$$= 100\ (0.01542)$$

$$= 1.5426 \text{ links}$$

$$= 1.5426 \times 0.2$$

$$HA = 0.3085 \text{ m}$$

(or)

$$HA = l(\sec\theta - 1)$$

$$= 20(\sec 10 - 1)$$

$$= 20(1.0154 - 1)$$

$$HA = 0.308 \text{ m}$$

ii) slope of H : 20 = 1 : 5

$$\tan\theta = \frac{1}{5}$$

$$\theta = \tan^{-1}(1/5)$$

$$\theta = 11.309$$

$$\theta = 11.31°$$

$$\theta = 11° 18' 35.76''$$

$HA = 100(\sec 11.31^\circ - 1)$

$= 100(1.0198 - 1)$

$HA = 1.9804$ links

$= 1.9804 \times 0.2$

$HA = 0.3960$ m

(or)

$HA = l(\sec\theta - 1)$

$= 20(\sec 11.31^\circ - 1)$

$HA = 0.396$ m

44. Find the hypotenusal Allowance for the chain of 20m length. ~~the~~ angle If

i) Angle of slope is 12°

ii) Slope is 1:5

→ Length of chain = 20m = 100 links

~~i) $\theta = 12^\circ$~~

ii) slope of 1:5

i) $HA = 100(\sec\theta - 1)$

$= 100(\sec 12^\circ - 1)$

$HA = 2.2340$ links

$= 2.2340 \times 0.2$

$HA = 0.4468$ m

(or)

$HA = 20(\sec 12^\circ - 1)$

$HA = 0.4468$ m

ii) Slope is 1:5

$1\ kg = 9.81\ N$

$\tan\theta = \frac{1}{5}$

$\frac{1000N}{9.81} = 101.8 = 1$

$\theta = \tan^{-1}(1/5)$

$\theta = 11.30°$

$\theta = 11°18'35.76''$

$HA = 100(\sec 11.31° - 1)$

$HA = 1.9804$ links

$= 1.9804 \times 0.2$

$HA = 0.3960\ m$

(or)

$HA = l(\sec\theta - 1)$

$= 20(\sec 11.31° - 1)$

$HA = 0.396\ m$

29/08/18

Errors & Corrections in Tape Measurement:

→ ~~Errors & mistakes may arise from~~

i) Erroneous [Incorrect] length of tape
ii) Careless holding & marking
iii) Bad ranging
iv) Bad straightening
v) Non-horizontality
vi) Sag in tape
vii) Variation in Temperature
viii) Variation in pull

ix) Personal mistakes like,

→ displacement of arrows

→ misreading

→ Erroneous Booking

The tape corrections are,

1) correction ~~of~~ for Absolute length

2) correction for temperature

3) correction for pull or Tension

4) correction for sag

5) correction for slope.

6) correction for alignment

7) Reduction to sea level

8) correction to measurement in vertical plane.

1) correction for absolute length:

→ If the absolute [Actual] length of the tape is not equal to its nominal or designated length, a correction will have to be applied to the measured length of the line.

→ If the absolute length of the tape is greater than the disignated length, the measured distance will be too short & the correction will be additive (+ve).

→ If the absolute length of the tape is lesser

than the designated length., the measured distan
will be too great & the correction will be
subtractive (-ve).

Thus, $\boxed{C_a = \frac{L \times c}{l}}$

where, C_a = correction for absolute length

L = measured length of the line in metre

c = $l_a - l$ = correction per tape length

l_a = Absolute length of the tape in metre

l = Designated length of the tape in metre.

$\therefore$ The correct length $L' = L \pm C_a$

2. Correction for temperature:

→ If the temperature of the field is more than the temperature at which the tape was standar the length of the tape increases, measured dist become less & the correction is additive

→ If the temperature of the field is less than the temperature at which the tape was standardised, the length of the tape decreases, measured dist becomes more & the correction is negative.

→ The temperature correction is given by,

$$\boxed{C_t = \alpha (T_m - T_0) L}$$

where, α = Co-efficient of thermal expansion in $/^\circ C$

T_m = Mean Temperature in the field during measurement in $^\circ C$

T_0 = Temperature during standardisation of the tape in $^\circ C$

L = Measured length in metre.

$\therefore$ corrected length $L' = L \pm C_t$

3f. Correction For Pull or Tension :

→ If the pull applied during measurement is more than the pull at which the tape was standardised, the length of tape ↑es, measured distance becomes less & correction is +ve.

→ If the pull applied during measurement is less than the pull at which the tape was standardised, the length of tape ↓es, measured distance becomes more & correction is -ve.

The correction for pull is given by,

$$\boxed{C_p = \frac{(P - P_0) \times L}{AE}}$$

where, P = Pull applied during measurement in

P_0 = standard pull in N

L = Measured length in metre

A = Cross sectional area of the tape in cm^2

E = Modulus of elasticity in N/cm^2.

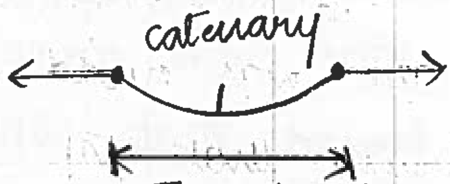
$\therefore$ Corrected length $L' = L \pm C_p$

4. Correction for sag:

→ While taking reading, if the tape is suspended b/n 2 supports, the tape sag under its own weight. The shape of the tape is catenary. Hence measured length is more than the actual length.

→ The correction for sag is given by,

$$\boxed{C_{sag} = \frac{1}{24}\left(\frac{W}{P}\right)^2 \times L}$$



where, W = weight of the tape in N

P = pull applied during the measurement in N

L = Measured length in metre

$\therefore$ corrected length $L' = L - C_{sag}$

This correction is always subtractive (-ve)

5. Correction for slope:

→ If the length measured is L & difference in level of 1st & last point is h, Then the correction of a slope is given by,

$$\boxed{C_{sl} = \frac{h^2}{2L}}$$

→ If the measured length is L & ~~at~~ its slope θ is given, the correction for slope is given by,

$$C_{sl} = L(1 - \cos\theta)$$

∴ corrected length $L' = L - C_{sl}$

08/18 This correction is always negative.

PROBLEMS:

1. The distance b/w 2 points measured by 20 m tape was record as 720 m. It was afterwards found that tape used was 4 cm too long. what was the true distance b/w 2 points.

→ $l = 20\ m$

$l_a = 20.04\ m$

$L = 720\ m$

$L' = ?$

$c = l_a - l$

(+ve) ~~20.04 - 20~~

$c = 0.04\ m$

$$C_a = \frac{L \times c}{l} = \frac{720 \times 0.04}{20} = 1.44 m\ (+ve)$$

The designated length is less than absolute length. Hence +ve

True distance, $L' = L \pm C_a$

$L' = L + C_a$

$= 720 + 1.44$

$L' = 721.44$ m.

Q2. Length of line measured with 30m chain was found to be 4920 m. if chain was 0.2 link too short, find the true length of the line.

→ $L = 4920$ m.

l_a 0.2 link = 20 cm.

$l = 30$ m

$l_a = 30 - (0.2 \times 0.2)$

$= 29.96$ m

$e = l_a - l$

$= 29.96 - 30$

$e = 0.04$ (-ve)

$$C_a = \frac{L \times e}{l} = \frac{4920 \times (-0.04)}{30} = 6.56 \text{ m } (-ve)$$

The absolute length is less than designated length hence, correction is -ve.

True length, $L' = L \pm C_a$

$= 4920 - 6.56$

$L' = 4913.44$ m

Q3. Length of line measured with 20m chain was 1341 m. The same line when measured with 30m chain which was 20 cm too short was found to be

1350 m. Find the error in 20 m chain.

→ For 20 m:

$l = 20$ m

$L = 1341$ m

For 30 m:

$l = 30$ m

$L = 1350$ m

$l_a = 30 - 0.2$

$= 29.8$ m

For 30 m chain:

$$C_a = \frac{L \times (l_a - l)}{l}$$

$$= \frac{1350 \times (29.8 - 30)}{30}$$

$C_a = 9$ m (-ve)

Since the absolute length is less than designated length, hence correction is -ve.

$L' = L - C_a$

$= 1350 - 9$

$L' = 1341$ m

There is no error in 20 m chain

The error will be zero in 20 m chain because true distance of 30 m chain [1341 m] is equal to measured distance of 20 m chain.

4f. The distance b/w 2 stations was measured with 20 m chain & found to be 1500 m. The same was measured with a 30 m chain & found to be 1476 m. If 20 m chain was 5 cm too short, what was the error in 30 m chain.

→ For 20 m:

$L = 1500$ m

$l = 20$ m

$l_a = 20 - 0.05$

$= 19.95$ m

For 30 m:

$L = 1476$ m

$l = 30$ m.

For 20 m chain:

$$C_a = \frac{L \times (l_a - l)}{l}$$

$$= \frac{1500 \times (19.95 - 20)}{20}$$

$= 3.75$ m ($-ve$)

Absolute length is less than designated length hence correction is $-ve$.

True distance, $L' = L - C_a$

$= 1500 - 3.75$

$L' = 1496.25$ m

For 30 m chain, correction for absolute:

C_a = True distance − Measured length

$= 1496.25 - 1476$

$= 20.25$ m ($+ve$)

WKT,

$$C_a = \frac{L \times (l_a - l)}{l}$$

$$20.25 = \frac{1476\,(l_a - 30)}{30}$$

$$607.5 = 1476\,l_a - 44280$$

$$l_a = 30.41\ m$$

The 30 m chain was 41 cm (0.41m) too long.

5f. ~~The length of survey line~~ The distance of 2000 m was measured with 30 m chain. After measurement the chain was found to be 10 cm ~~too~~ long. It was found to be 15 cm longer after another 500 m measured distance. If the length of chain was correct before measurement, determine exact length after whole measurement.

→ For 2000 m measured distance:

Average correction per tape length $C_{avg} = \frac{0+10}{2}$

$C_{avg} = 5$ cm

$= 0.05$ m.

$$\therefore\ C_a = \frac{L \times C_{avg}}{l} = \frac{2000 \times 0.05}{30} = 3.33\ (+ve)$$

The absolute length is more than designated length hence +ve.

$$L' = L + C_a$$

$$= 2000 + 3.33$$

$$= 2003.33\ m$$

For 500 m measured distance:

Average correction per tape length, $C_{avg} = \frac{10+15}{2}$

$= 12.5$ cm

$= 0.125$

$\therefore C_a = \frac{L \times C_{avg}}{l} = \frac{500 \times 0.125}{30} = 2.083$ (+ve)

The absolute length is more than designated length hence +ve.

$L' = L + C_a$

$= 500 + 2.083$

$L' = 502.08$ m.

Before exact length of line of whole measurement,

$2003.33 + 502.08 = 2505.413$ m.

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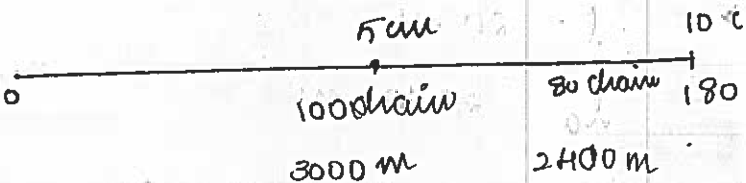
6) A 30 m chain was tested before commencement of days work & found to be correct. After chaining 100 chains, chain was found to be 5 cm too lon at the end of days work. After chaining total distance of 180 chains, chains was found to be 10 cm too long. Find the true distance chained.

→ ~~For 30 m chains~~

For first 3000 m



$C_{avg} = \frac{0+5}{2}$

$= 2.5$ cm

$= 0.025$ m

$$C_a = \frac{L_1 \times C_{avg}}{l}$$

$$= \frac{3000 \times 0.025}{30}$$

$= 2.5$ m (+ve)

$L_1' = L_1 + C_a$

$= 3000 + 2.5$

$= 3002.5$ m.

For the next 2400 m:

$$C_{avg} = \frac{5+10}{2}$$

$= 7.5$ cm

$= 0.075$ m

$$C_a = \frac{L_2 \times C_{avg}}{l}$$

$$= \frac{2400 \times 0.075}{30}$$

$= 6$ m (+ve)

$L_2' = L_2 + C_a$

$= 2400 + 6$

$= 2406$ m.

$\therefore$ Total true distance chained $= L_1' + L_2'$

$= 3002.5 + 2406$

$= 5408.5$ m.

77. A chain was tested before starting survey & was found to be exactly 20 m, at the end of survey it was tested again & was found to be 20.12 m. Area of plan of field drawn to a scale of 1 cm = 6 m was 50.4 cm². Find the true area of the field in m².

→ $l = 20$ m

$l_a = 20.128$ m

~~Area~~ scale 1 cm = 6 m

Area of field on plan = 50.4 cm²

Since scale of the plan is 1 cm = 6 m

Measured area of the field = 50.4 × 6-

$A = 1814.4\ m^2$

Actual area or true area of field = $A' = A \times \left(\frac{l_a}{l}\right)^2$

$= 1814.4 \times \left(\frac{20.12}{20}\right)^2$

$= 1836.23\ m^2$

8). A 30 m chain used to find volume of cylindri water tank, was found to be 2 cm too long. Th cylindrical water tank measured by 30 m chain was found to have a diameter of 8.45 m. Th height of tank is 20 m. Find the true cubic content [volume] of water tank.

→ $l = 30\ m$

$l_a = 30.02\ m$

diameter of cylindrical water tank = $d = 8.45\ m$

Height of tank = $H = 20\ m$

Measured volume of the cylindrical tank

ie, $V = \frac{\pi d^2}{4} \times H$

$= \frac{\pi (8.45)^2}{4} \times 20$

$V = 1121.58\ m^3$

∴ Actual or true cubic content of cylindrical wate tank,

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

$$= 1121.58 \times \left(\frac{30.02}{30}\right)^2$$

$$V' = 1123.07 \ m^3$$

Q6. The length of chain measured on slope of 14° is 272 m. It was found that the tape is 22.2 m long instead of 20 m. Calculate the correct horizontal length

→ $\theta = 14^\circ$

$L = 272 m$

$l = 20 m$

$la = 22.2 m$

Slope correction $C_{SL} = L(1 - \cos\theta)$

$$= 272(1 - \cos 14^\circ)$$

$$= 8.079 \ m \ (-ve)$$

Correction for absolute length.

$$C_a = \frac{L \times (la - l)}{l}$$

$$= \frac{272(22.2 - 20)}{20}$$

$$C_a = 29.92 \ m \ (+ve)$$

∴ True horizontal length $L' = L - C_{SL} + C_a$

$$= 272 - 8.079 + 29.92$$

$$L' = 293.841 \ m$$

10f. A tape 20 m long of standard length at 84°F was used to measure a line, the mean temperature during measurement being 65°F & measured distance was 882.10 m. $\alpha = 65\times10^{-7}/°F$

Following are the varying slopes.

i) 2° 10' for 100 m

ii) 4° 12' for 150 m

iii) 1° 6' for 50 m

iv) 7° 48' for 200 m

v) 3° 0' for 300 m

vi) 5° 10' for 82.1 m

Find true length of line.

$\rightarrow$ $L = 882.10\ m$, $T_0 = 84°F$ $\alpha = 65\times10^{-7}/°F$

$l = 20\ m$. $T_m = 65°F$

~~correction for temperature~~, $C_t = \alpha (T_m - T_0) \times L$

$$= 65\times10^{-7}(65-84)\times 882.10$$

$$= 0.1089 \quad (-ve)$$

correction for slope,

$$C_{sL} = L_1(1-\cos\theta_1) + L_2(1-\cos\theta_2) + L_3(1-\cos\theta_3) + L_4(1-\cos\theta_4) + L_5(1-\cos\theta_5) + L_6(1-\cos\theta_6)$$

$$= 100(\cancel{2°10'}1-\cos(2°10')) + 150(1-\cos(4°12')) + \cancel{50(1°6')}\ 50(1-\cos(1°6')) + 200(1-\cos(7°48')) + 300(1-\cos(3°0')) + 82.1(1-\cos(5°10'))$$

$C_{sl} = 3.078$ (-ve)

True distance, $l' = l - C_t - C_{sl}$

$= 882.1 - 0.1089 - 3.078$

$l' = \underline{878.91\ m}$

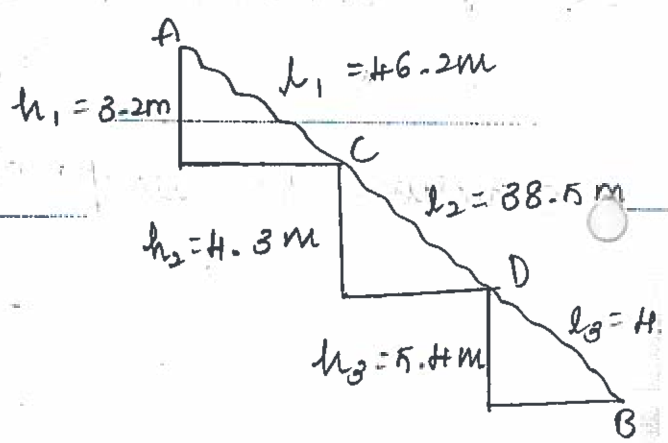
03/09/18

11. The following are the slope distance when measured with 15 m tape.

Slope distance (m)	Difference in elevation of end points (m)
46.2	3.2
38.5	4.3
42.6	5.4

Find the total horizontal distance by using exact formula & approximate formula.

→ By exact formula:



we have, $D = \sqrt{l^2 - h^2}$

$D_1 = \sqrt{l_1^2 - h_1^2}$

$= \sqrt{(46.2)^2 - (3.2)^2}$

$D_1 = 46.089m$

$D_2 = \sqrt{l_2^2 - h_2^2}$

$= \sqrt{(38.5)^2 - (4.3)^2}$

$= 38.259m$

$D_3 = \sqrt{l_3^2 - h_3^2} = \sqrt{(42.6)^2 - (5.4)^2}$

$= 42.256\ m$

$\therefore$ True horizontal distance ...

$= 46.089 + 38.259 + 42.0...$

$= 126.604\ m$

<u>By Approximate Formula:</u>

The correction for slope is given by,

$$C_{sl} = \frac{h^2}{2L}$$

$$C_{sl_1} = \frac{h_1^2}{2L_1} = \frac{(3.2)^2}{2(46.2)} = 0.1108\ m$$

$$L_1' = 46.2 - 0.1108 = 46.089\ m$$

$$C_{sl_2} = \frac{h_2^2}{2L_2} = \frac{(4.3)^2}{2(38.5)} = 0.2401\ m$$

$$L_2' = 38.5 - 0.2401 = 38.259\ m$$

$$C_{sl_3} = \frac{h_3^2}{2L_3} = \frac{(5.4)^2}{2(42.6)} = 0.3422\ m$$

$$L_3' = 42.6 - 0.3422 = 42.257\ m$$

$\therefore$ True horizontal distance $= L_1' + L_2' + L_3'$

$= 46.089 + 38.259 + 42.2...$

$= 126.605\ m.$

12. To measure a base line, a steel tape 30 m le... standardised at 15°C with a pull of 100 N was used. Find correction per tape length, if the temp at the time of measurement of 20°C ... pull exerted was 160 N. If the length of 250 m is measured on slope of 1:4, find the

horizontal length. If $E = 2.1 \times 10^{5}$ N/mm², $\alpha = 11.2 \times 10^{-6}$/°C

& cross sectional area of tape is 0.08 cm².

→ Given, $T_0 = 15°C$

$P_0 = 100N$

$T_m = 20°C$

$P = 160N$

Sloping distance, $L = 250$ m

$\tan\theta = \frac{1}{4} \Rightarrow \theta = 14.036°$

$E = 2.1 \times 10^{5}$ N/mm²

$\alpha = 11.2 \times 10^{-6}$ /°C

$A = 0.08$ cm² $= 0.08 \times 10^{2}$ mm² $= 8$ mm²

1 cm = 10 mm

1 cm² = 10^2 mm²

Solⁿ:

correction for temp is given by,

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

$$= 11.2 \times 10^{-6} (20 - 15) \times 250$$

$$C_t = 0.014 \text{ m. } (+ve)$$

correction for pull is given by,

$$C_p = \frac{(P - P_0) \times L}{AE}$$

$$= \frac{(160 - 100) \times 250}{8 \times 2.1 \times 10^{5}}$$

$$C_p = 8.928 \times 10^{-3} \ (+ve)$$

correction for slope is given by,

$$C_{sL} = L(1 - \cos\theta)$$

$= 250(1 - \cos 14.036)$

$= 7.464$ (-ve)

$\therefore$ True horizontal distance, $L' = L + C_p + C_t - C_{sl}$

$= 250 + 0.014 + 8.928$

$- 7.464$

$L' = 242.558$ m

13) Calculate sag correction for 30 m steel tape under a pull of 100N in 3 equal spans of 10 m each. Unit weight of steel tape is 78.6 KN/m³. Area of cross section of tape is 8 mm²

→ $L = 30$ m

$1 KN = 102$ kg.

$P = 100N$

$A = 8 \text{ mm}^2 = (8 \times 10^{-3})^2 \text{ m}^2$

Unit weight of steel tape $= 78.6$ KN/m³

volume $= A \times L$

$= 8 \times 10^{-6} \times 30$

$= \cancel{0.24 \text{ m}^3}$.

$= 2.4 \times 10^{-4} \text{ m}^3$

1 m³ volume of tape weighs $= 78.6$ KN $= 78.6 \times 10^3$ N

$\therefore$ volume of 30 m tape $= A \times L$

$= 8 \times 10^{-6} \times 30$

$= 2.4 \times 10^{-4} \text{ m}^3$

$$C_{sag} = \frac{1}{24}\left(\frac{W}{P}\right)^2 \times L$$

$1 m^3 = 78.6 \times 10^3$

$2.4 \times 10^{-4} = ?$

$= 78.6 \times 10^3 \times 2.4 \times 10^{-4}$

$w = 18.864 \ N$

$\therefore$ 2.4×10^{-4} m^3 of tape weighs = unit weight × volume

$= 78.6 \times 10^3 \times 2.4 \times 10^{-4}$

$= 18.864 \ N$.

$\therefore$ $C_{sag} = \frac{1}{24}\left(\frac{W}{P}\right)^2 \times L$

$= \frac{1}{24}\left(\frac{18.86}{100}\right)^2 \times 30$

$C_{sag} =$ ~~0.2357~~

$C_{sag} = 0.0444 \ m$ (-ve)

04/09/18

14\. A 30 m long tape was standardised that 20°C & under a pull of 100 N. The tape was used to measure a distance AB where temp was 45°C & pull was 150 N. The tape was supportive at ends only. Find correction per tape length if cross sectional area of tape was 4 mm², unit weight of tape material is 78.6 KN/m³. & $\alpha = 11.5 \times 10^{-6} /°C$ & $E = 20,00,000 \ KN/m^2$

→ $T_0 = 20°C$

$P_0 = 100 N$.

$T_m = 45°C$

$P = 150 N$.

$L = 30\ m$

$A = 4 \times 10^{-6}\ m^2$.

Unit weight $= 78.6\ kN/m^3 = 78.6 \times 10^3\ N/m^3$

$\alpha = 11.5 \times 10^{-6} /^{\circ}C$

$E = 2000000 \times 10^3\ N/m^2$.

Correction for temp

$$C_t = \alpha (T_m - T_0) \times L$$
$$= 11.5 \times 10^{-6} (45 - 20) \times 30$$
$$= 8.625 \times 10^{-3}\ m \quad (+ve)$$

$$C_p = \frac{(P - P_0) \times L}{AE}$$
$$= \frac{(150 - 100) \times 30}{4 \times 10^{-6} \times 2000000 \times 10^3}$$
$$= 0.1875\ m \quad (+ve)$$

$$C_{sag} = \frac{1}{24} \left(\frac{W}{P}\right)^2 \times L.$$

Volume of 30 m $= A \times L$
$$= 4 \times 10^{-6} \times 30$$
$$= 1.2 \times 10^{-4}\ m^3.$$

$$= 78.6 \times 10^3 \times 1.2 \times 10^{-4}$$

$$W = 9.432\ N$$

$$C_{sag} = \frac{1}{24} \left(\frac{W}{P}\right)^2 \times L = \frac{1}{24} \left(\frac{9.432}{150}\right)^2 \times 30$$

$C_{sag} = 0.\cancel{7413}\ m\ (-ve)$

$$C_{sag} = 4.942 \times 10^{-3}\ m \quad (-ve)$$

$$L' = L + C_t + C_p - C_{sag}$$

$$= 30 + 8.625 \times 10^{-3} + 0.1875 - 4.942 \times 10^{-3}$$

$$L' = \underline{30.191\ m}$$

05/09/18

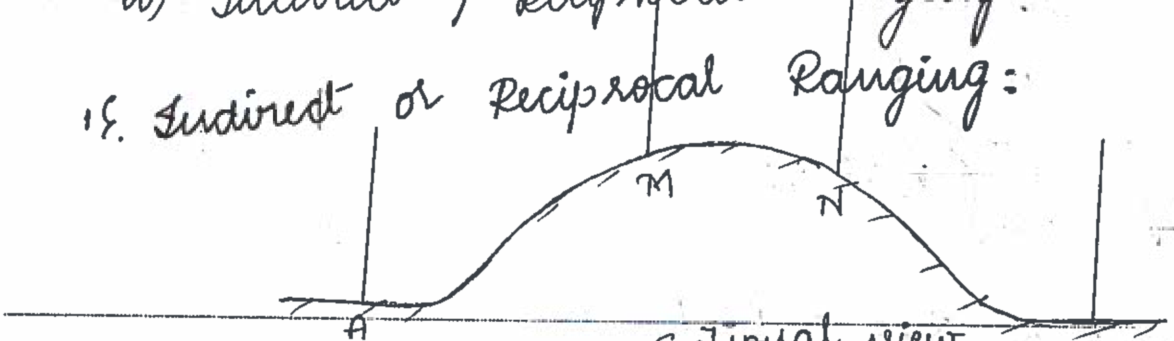
Ranging of Lines:

→ When a survey line is longer than a chain length, it is necessary to establish intermediate points on survey lines. The process of locating intermediate points on survey line is known as ranging.

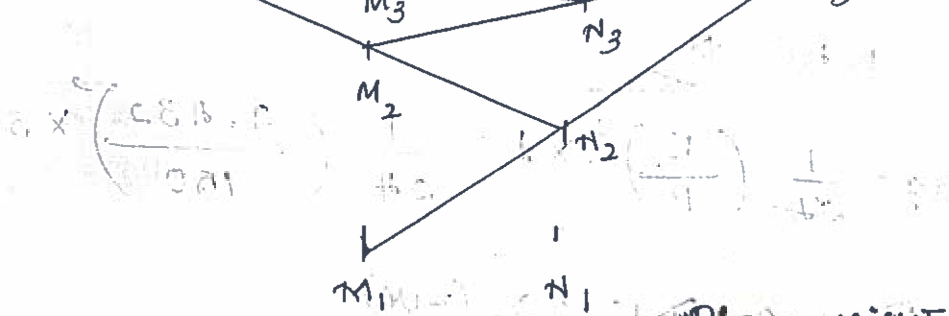
The methods of ranging are classified as

i) Direct ranging.
ii) Indirect / reciprocal ranging.

1. Indirect or Reciprocal Ranging:



Sectional view



Plan view

→ If the 2 end points of the line to be measured are not intervisible, the surveyor has to go for indirect ranging.

→ The intervisibility of points may be due to unevenness of ground or due to long distance.

→ M & N are 2 points to be fixed on AB such that M & N points are ||le to AB & for the person near M, the point N & point B is visible & for the person N, the points M & A should be visible.

→ 2 surveyors themselves at M_1 & N_1 with ranging rods.

→ The person at M_1 then directs the person N_1 to move to a new position N_2 ~~and~~ inline with M_1B.

→ The person at N_2 then directs the person at M_1 to move to a new position M_2 inline with N_2A.

Thus, the 2 persons are now at M_2 & N_2 which are nearer to the chain line than the position M_1 & N_1.

→ The process is repeated till the points M & N are located in such a way that the person at M finds the person at N inline with MB & the person at N finds the person at M inline with NA. Thus, M & N are established.

Electronic Distance Measurement [EDM]:

(depend).

→ In EDM, distances are measured that relay on propagation, reflection & subsequent reception of electro magnetic waves.

→ EDM enables the accuracies upto 1×10^{-5} & ranges upto 100 km.

→ Depending upon the carrier wave employed, EDM instruments can be classified under 3 heads

i) Microwave instrument : Eg: Tellurometer

ii) visible light instrument : Eg: Geodemeter

iii) Infrared wave instrument : Eg: Distometer.

Basic principle of EDMI

→ Basic principle of EDM instrument is the determination of time required for electromagnetic waves to travel b/w 2 stations. Here, the velocity of electromagnetic wave is the basis for computations of the distance.

Field Book:

→ Field book is a book for recording all field observations. It is a oblong book of size 120 mm × 200 (b × l) which can be carried in the pocket.

There are 2 forms of the book

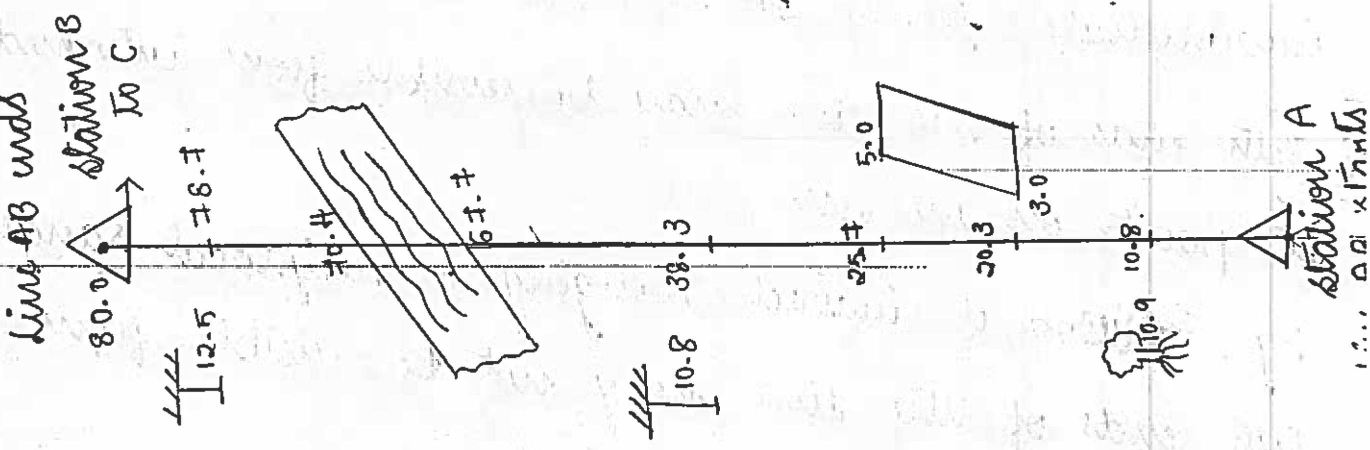
i) single line book

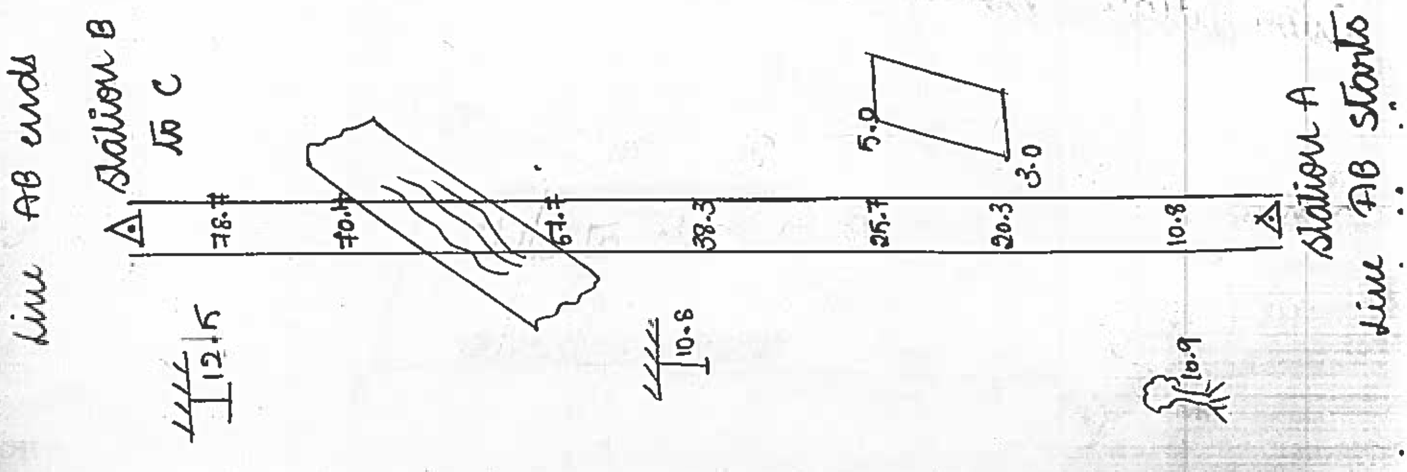
ii) Double line book.

→ The pages of single line books have a red li along the length of the paper in the middle of wid It indicates chain line. All chainages are recorded across it. The space on either side of the line is used for sketching the objects & noting offset distan

→ In double line book, there are 2 blue lines with a space of 150 to 20 mm in the middle in each page of the book. The space b/n the two lin is utilized for noting the chainages.

→ For ordinary works, double line books are used whereas single line books are used for large scale & much detailed work.





06/10.

Obstacles in tape survey:

→ obstacles to chaining prevent chain man from measuring directly b/n 2 points & give rise to set of problems in which distances are found by indirect measurements.

There are 3 types of obstacles in chaining, they are

i) obstacles to ranging but not chaining.

ii) obstacles to chaining but not ranging.

iii) obstacles to both chaining & ranging.

i) obstacles to ranging but not chaining.

→ There will be 2 cases of these obstacle

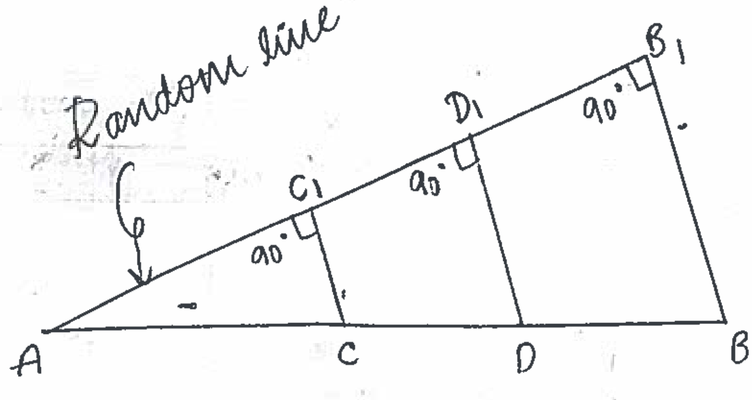
a) Both ends of the line may be visible from intermediate points on the line.

b). Both ends of the line may not be visible from intermediate points on the line.

a). Both ends of the line may be visible from intermediate points on the line:

Eg: Method of indirect ranging or reciprocal ranging

b) Both ends of the line may not be visible from intermediate points on the line.



*. Let AB be the line in which ... visible from intermediate points on it.

*. Through A, draw a random line AB_1 in any convenient direction but as nearly towards B as possible.

*. The point B_1 should be show & choosen that,

→ B_1 is visible from B.

→ BB_1 is perpendicular to random line AB_1

*. Measure BB_1 & AB_1 & Select point C_1 & D_1 on the random line & erect perpendicular C_1C & D_1D on it

$$CC_1 = \frac{BB_1}{AB_1} \times AC_1$$

$$DD_1 = \frac{BB_1}{AB_1} \times AD_1$$

*. Join C & D & prolong.

ii) Obstacles to chaining but not ranging.

→ There may be 2 cases of this obstacle, they are

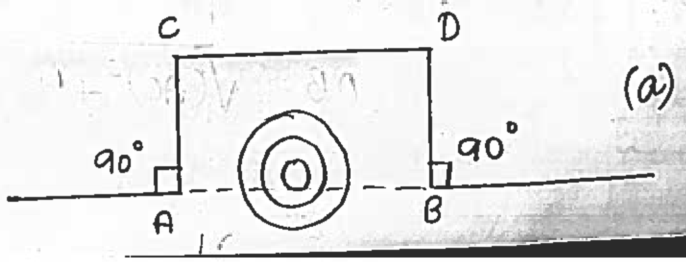
1) When it is possible to chain around the obstacle

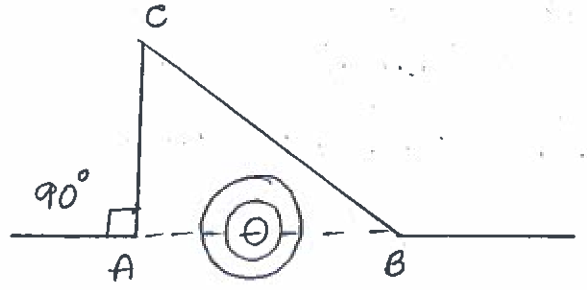
Eg: Pond, hedge [Group of bushes], etc

2) When it is not possible to chain around the obstacle

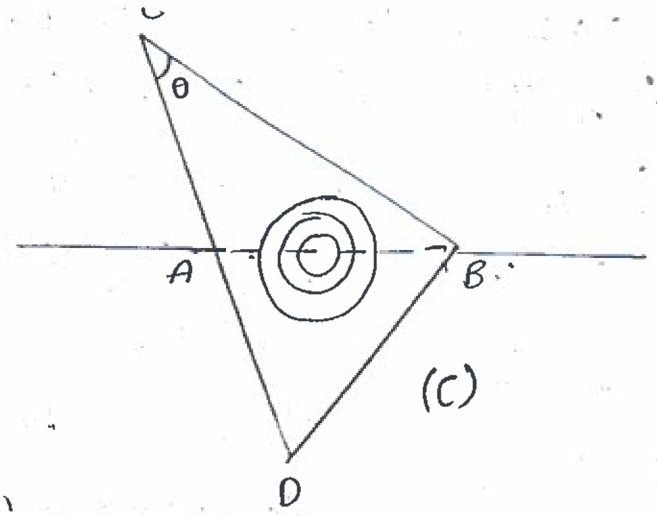
Eg: River.

1) When it is possible to chain around the obstacle.





(b)



(C)

Method (a):

→ Set out equal perpendicular AC & BD

→ Measure CD, then CD = AB.

Method (b):

→ Set out perpendicular AC to chain line

→ Measure AC & BC

→ The length AB is calculated from the relation,

$$AB = \sqrt{(BC)^2 - (AC)^2}$$

07/09/18

Method (C):

→ Select 2 points C & D to both sides of A & in the same line.

→ Measure AC, AD, BC & BD.

→ Let angle $B\hat{C}D = \theta$

→ From ΔBCD, $(BD)^2 = (BC)^2 + (CD)^2 - 2 \times BC \times CD \times \cos\theta$

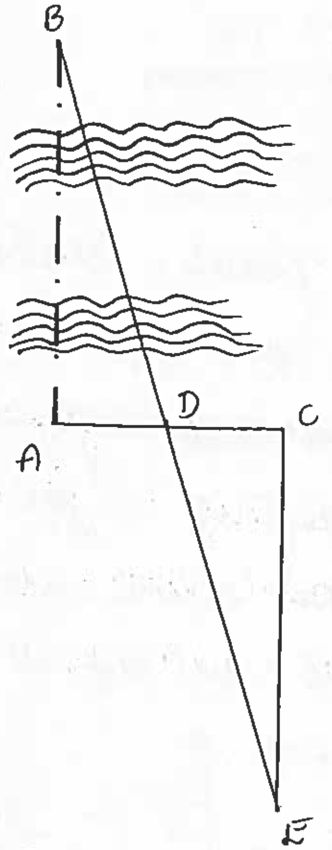
$$\cos\theta = \frac{(BC)^2 + (CD)^2 - (BD)^2}{2 \times BC \times CD}.$$

lly from ΔBCA, $(AB)^2 = (AC)^2 + (BC)^2 - 2(AC)(BC)\cos\theta$

$$AB = \sqrt{(AC)^2 + (BC)^2 - 2 \times AC \times BC \times \cos\theta}$$

2) when it is not possible to chain around the

Obstacle :



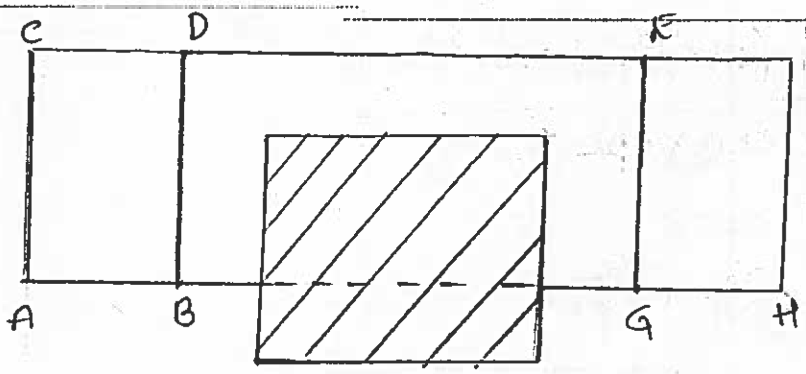
→ Erect perpendicular AC & bisect it at D.

→ Erect perpendicular at C & range E in line with BD

→ Measure CE, then AB = CE

iii) Obstacles to both chaining & Ranging:

→ A building is the typical example of this type of obstacle.



→ Choose 2 points A & B on one side & erect Perpendiculars AC & BD of equal length

→ Join CD & prolong to past the obstacle

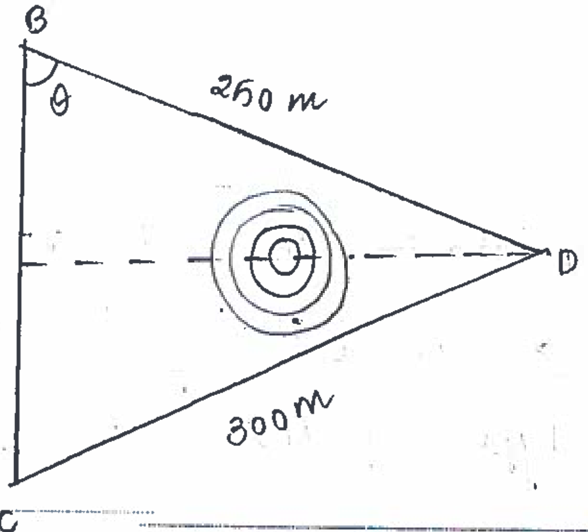
→ Choose 2 points E & F on CD & erect perpen ulars EG & FH equal to that of AC or BD.

→ Join GH & prolong it.

→ Measure DE, then BG = DE

imp ***
PROBLEMS on Obstacles :

18. In chaining, past a pond, stations A & D on main line but taken on opposite line of side. Two lines BD & CD measuring 250 m & 300 m were laid down to left & right of AD. The point A, B & C are in same line. AB & AC are measured & found to be equal to 130 m & 120 m. Find length of line AD.



→ Let $\angle CBD = \theta$

from cosine rule,

$$\cos\theta = \frac{(BD)^2 + (BC)^2 - (CD)^2}{2 \times BC \times BD}$$

$$= \frac{(250)^2 + (250)^2 - (300)^2}{2 \times 250 \times 250}$$

$\cos\theta = 0.28$

$\theta = \cos^{-1}(0.28)$

$\theta = 73.73^\circ$

For $\triangle ABD$, from cosine rule,

$$AD = \sqrt{(AB)^2 + (BD)^2 - 2 \times AB \times BD \times \cos\theta}$$

$$= \sqrt{(130)^2 + (250)^2 - 2 \times 130 \times 250 \times 0.28}$$

$AD = 247.88$ m.

$CD^2 = AC^2 + AD^2$

$AD^2 = CD^2 - AC^2$

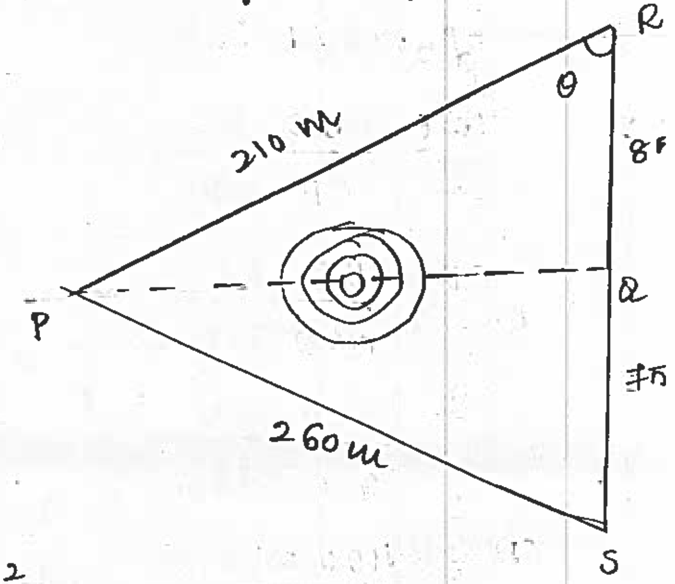
$AD = \sqrt{(CD)^2 - (AC)^2}$

$= \sqrt{(300)^2 - (120)^2}$

$= 274.95$

∴ The length of AD = 24.08 m

28. 2 stations P & Q on main survey line, were taken on opposite sides of a pond. On right of PQ a line PR, 210 m long was layed down & another line PS, 260 m long was layed down on left of PQ. The points R, Q & S are all same straight line. The measured lengths of RQ & QS are 85 m & 75 m respectively. What is the length of PQ.



→ Let $\angle PRQ = \theta$

From cosine rule,

$$\cos\theta = \frac{(PR)^2 + (RS)^2 - (PS)^2}{2 \times PR \times RS}$$

$$= \frac{(210)^2 + (160)^2 - (260)^2}{2 \times 210 \times 160}$$

$$\cos\theta = 0.0312$$

$$\theta = \cos^{-1}(0.0312)$$

$$= 88.212^\circ$$

For ΔPQR, ΔPRQ, from cosine rule

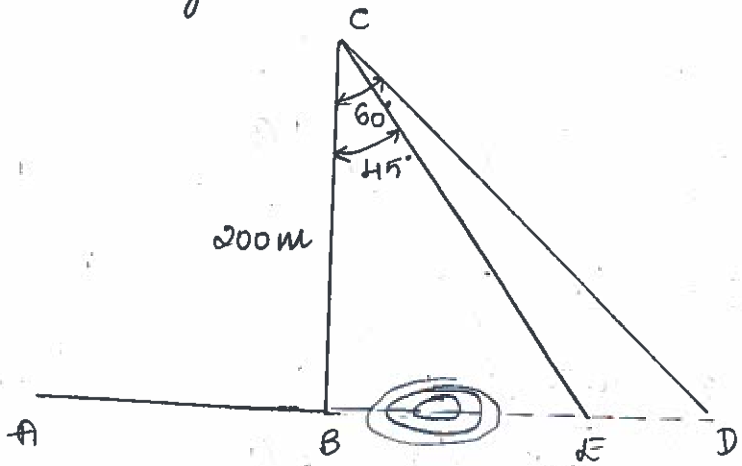
$$PQ = \sqrt{(RQ)^2 + (PR)^2 - 2 \times RQ \times PR \times \cos\theta}$$

$$= \sqrt{(85)^2 + (210)^2 - 2 \times 85 \times 210 \times 0.0312}$$

$$PQ = 224.07\ m$$

∴ The length of PQ = 224.07 m //

Q4. To continue a survey line AB past an obstacle, a line BC 200 m long was set out perpendicular to AB & from C angles $B\hat{C}D$ & $B\hat{C}E$ were set out at 60° & 45° respectively. Determine the lengths which must be chained of along CD & CE in order that ED may be in AB produced. Also, determine the obstacted length BE.



→ from ΔBCD,

$$\cos\theta = \frac{BC}{CD}$$

$$CD = \frac{BC}{\cos\theta}$$

$$= \frac{200}{\cos 60}$$

$$CD = \underline{400\ m}$$

from ΔBCE,

$$\cos\theta = \frac{BC}{CE}$$

$$CE = \frac{BC}{\cos\theta}$$

$$= \frac{200}{\cos 45}$$

$$CE = \underline{282.84\ m}$$

from ΔBCE

$$CE^2 = BC^2 + BE^2$$

$$\tan\theta = \frac{BE}{200}$$

$BE = \tan 45^\circ \times 200$

$BE = 200\,m.$

(or)

From $\triangle BCE$

$CE^2 = BE^2 + CB^2$

$BE = \sqrt{CE^2 - CB^2}$

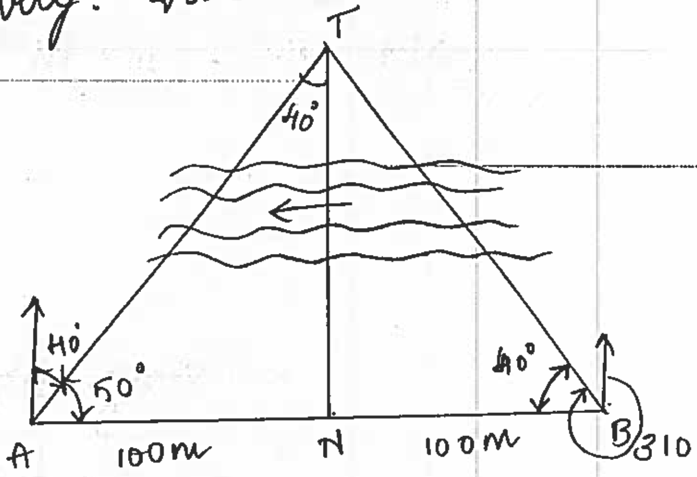
$= \sqrt{(282.84)^2 - (200)^2}$

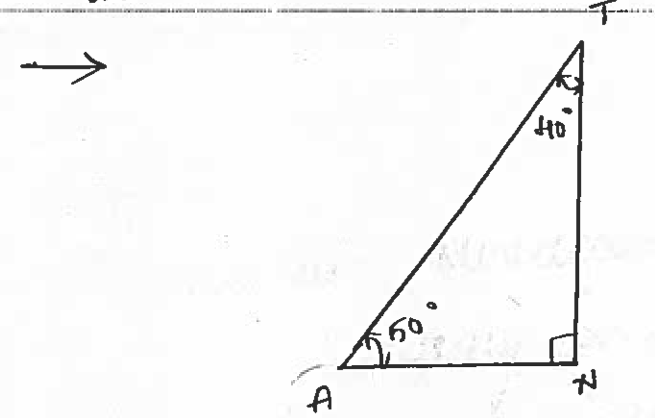
$= 199.99$

$\underline{BE = 200\,m}$

The obstacle length $BE = 200\,m$

45. A & B are 2 points 200m apart along bank of river flowing from east to west. The bearing of tower on other side of bank has observed from A & B are 40° & 310° respectively. Find the width of the river.





From $\triangle TAN$,

$\tan\theta = \dfrac{TN}{AN}$

$TN = \tan\theta \times AN$

$= \tan 50^\circ \times 100$

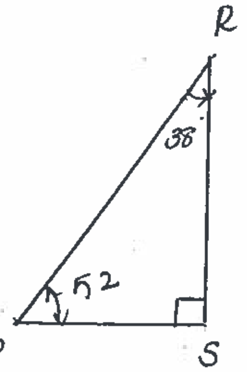
$= \underline{119.17\,m}$

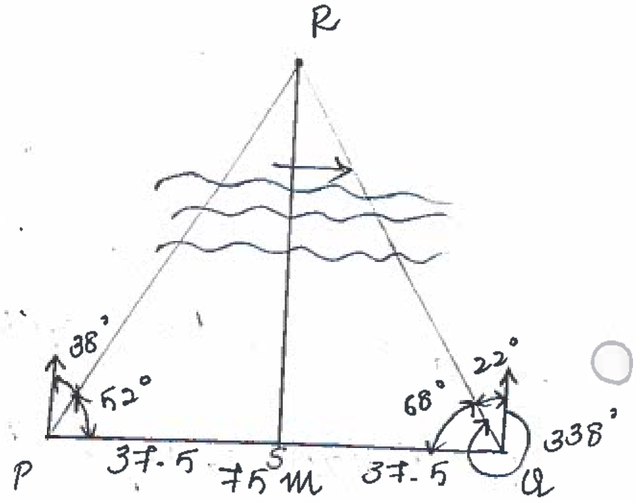
or by using sine rule

TN should be more than 100 m

57. 2 stations P & Q were taken on southern side & river flowing west to east point. P is westwards of point Q at 75 m apart. Bearing of tree R on northern side of bank is observed to be equal to 38° & 338° respectively from P & Q. Calculate the width of the river.

→





From ΔRPS,

$$\tan\theta = \frac{RS}{PS}$$

$$RS = PS \times \tan\theta$$

$$= 37.5 \times \tan 52$$

$$RS = 48 \text{ m}.$$

Module - 2

Measurement of Directions And Angles

Unit - 1 : Compass Survey

The Branch of surveying in which direction of survey lines are determined by a compass & their length by chaining on the surface of the earth is known as "compass surveying".

Definitions of Important Terms:

i) Meridian : It is the fixed direction on the surface of the earth, with reference to which bearings of survey lines are expressed.

a) True Meridian: The true or geographical meridian passing thro a place is the line of intersection of the earth surface by a plane containing north & south poles & also the given place.

b) Magnetic Meridian: The geometrical longitudinal axis of a freely suspended & properly with balanced magnetic needles, unaffected by local attractive forces, defines the magnetic meridian.

ii) Bearing: The horizontal angle b/w the meridian & the survey line, measured in clock wise

Module-02

Measurement of Directions And Angles

Compass Survey:-

Branch of Surveying in which direction (angle) of survey lines are determined by a compass and their length by chaining on the surface of the Earth is known as Compass Surveying

Definitions of Important terms:-

1. **Meridian:-**

It is the fixed direction on the surface of the Earth with reference to which bearing of survey line are Expressed

a) **True Meridian:-**

The true @ geographical meridian passing through the plane, is the line of intersection of the Earth surface by a plane containing North and South poles and also the given place

b) **Magnetic Meridian:-**

The Geometrical longitudinal axis of a freely suspended and properly balanced Magnetic Needle unaffected by local attractive forces, defines the Magnetic Meridian

2. **Bearing:-**

The horizontal angle b/w the Meridian and the Survey line, Measured in clockwise direction is called bearing

a) **True bearing:-**

The horizontal angle b/w the true meridian and the survey line.

b) **Magnetic bearing:-**

The horizontal angle b/w the Magnetic Meridian and the Survey line.

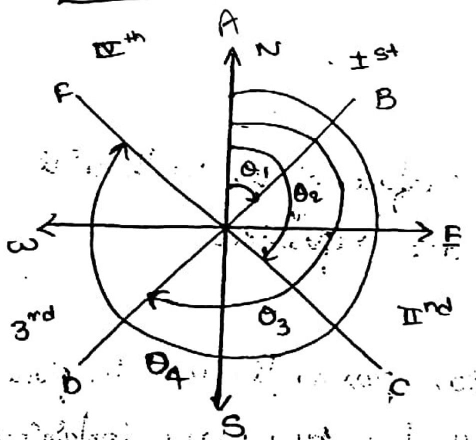
Designation of bearing:-

Bearings are designated in two System.

i.e, whole circle bearing (WCB) & ~~Readucing~~ Reduced Bearing [RB @ QB]

1. Whole Circle Bearing:-

In this system the bearing of survey line measured with Magnetic North in clockwise direction, utilizing whole circle of graduation. The whole circle bearing varies from 0 to 360. Prismatic Compass is graduated on this system.

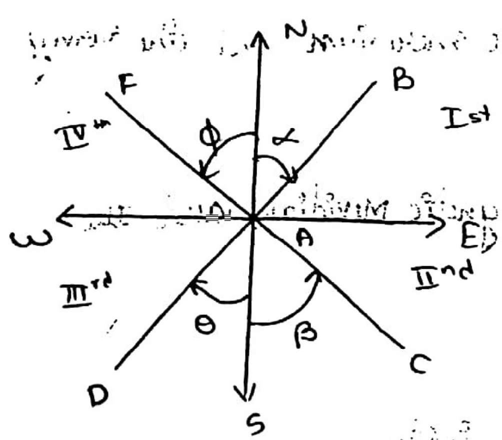


* The WCB of line AB is θ_1
* The WCB of line AC is θ_2
* The WCB of line AD is θ_3
* The WCB of line AF is θ_4

2. Reduce bearing System @ Quadrantal bearing System:-

In this System the bearing of a line is Measured East ward (@) West ward

From North (@) South which ever is nearer. Thus, Both North and South are used as Reference Meridian and direction can be Either clockwise (@) anticlockwise depending upon the position of the line. In this System the Quadrant in which the line lies will have to be mentioned. These bearings are observed from Survey Compass.



* The RB of line AB is NαE
* The RB of line AC is SβE
* The RB of line AD is SθW
* The RB of line AF is NϕW

Scanned with CamScanner

Conversions of Bearings from one System to other.

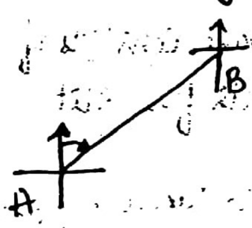
1. Conversion from whole circle bearing to Reduce bearing.

line	WCB	rule of 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

2. Conversions from RB to WCB

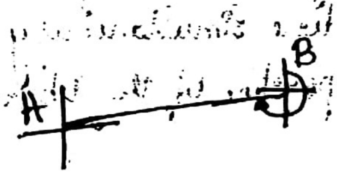
line	RB	Rule for WCB	WCB
AB	N α E	WCB = RB	0 to 90°
AC	S β E	WCB = 180 - RB	90 to 180
AD	S θ W	WCB = 180 + RB	180° to 270°
AF	N φ W	WCB = 360 - RB	270° to 360

Fore Bearing (FB):- difference b/w FB & BB



The bearing of line AB measured from A towards B known as Fore Bearing or Forward Bearing.

Back Bearing (BB):-



The bearing of line AB measured from B towards A is known as Back bearing (BB) or Backward bearing.

The general relation b/w back bearing and

$\boxed{\text{Fore bearing is } BB = FB \pm 180°}$

* Use + Sign when FB is less than 180°
* Use - Sign when FB is More than 180°

Comparision b/w Prismatic Compass And Surveyor's Compass.

Item	Prismatic Compass	Surveyor's Compass
1. Magnetic Needle	The Needle is of broad Needle type. The Needle doesn't act as index	The Needle is of Edge bar type. The Needle acts as a index.
2. Graduated ring	* Graduated ring is attached with the Needle. The ring doesn't rotate along with the line of sight * The Graduations are in WCB System, having 0° at S, 90° at W, 180° at N and 270° at E.	* The graduated ring is attached to the box and Not to the Needle. The ring rotate along with the line of sight * The graduations are in QB System, having 0° at N & S and 90° at E & W
3. Sighting Vanes	* The object vane consists of Metal vane with Vertical hair * The Eye vane consists of a Small metal vane with slit	* The object vane consists of metal vane with verticle hair * The eye of vane consists of metal vane with fine slit
4. Reading	* The reading is taken with the help of prism provided at the Eye-slit * Sighting & Reading taking can be done simultaneously ~~can be used even by~~ from one Position of the observer.	* The Reading is taken by directly Seeing through the top of the glass * Sighting and reading taken can't be done simultaneously from one position of the observer.
Tripod.	Tripod may/may Not be used. The instrument can be used Even by holding suitable in hand	* The instrument can't be used without the tripod.

least count

least Count | * The least Count is 30' | * The least Count is 1°

Temporary Adjustments of Prismatic Compass

Temporary adjustment are those adjustment which have to be made at every set up of the instrument.

6m There are 3 steps in temporary adjustment of Compass they are

* Centring
* Leveling
* Focusing the prism.

Centring :-

It is the process of keeping the instruments exactly over the station the centring is in variably done by adjusting & Manipulating the legs of the tripod. A plumbob may be used to judge the Centring & if it is not available, it may be judge by droping a piece of stone from the center of bottom of instrument.

leveling :-

If the instrument is handly instrument it must be held in hand in such a way that graduated disc is swinging freely and appears to be leveled as judged from the top of the glass. Generally a tripod is provided with the ball & socket arrangment with the help of which the top of the box can be leveled.

3. Focusing the prism :-

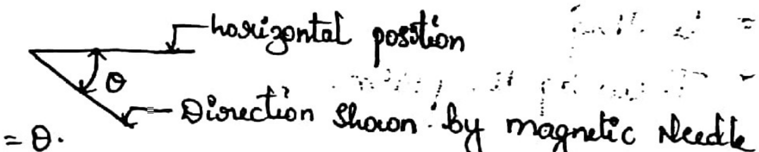
The prism attachment is slided up/down for focusing till the readings are seen to be sharp & clear

Dip And declination :-

Dip :-

A perfectly balanced, freely suspended magnetic needle dips towards its Northern end in Northern hemisphere and dips towards Southern end in Southern hemisphere. If it is at North pole the Needle takes verticle position.

The vertical angle b/w the horizontal and the direction shown by perfectly balanced and freely suspended magnetic Needle is known as the magnetic dip at that place.



Dip = θ.

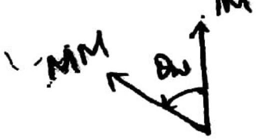
Magnetic Declination :-

Magnetic declination at a place is the horizontal angle b/w the true meridian and the magnetic meridian shown by the magnetic needle at the time of observation.

If the Magnetic Meridian is to the right (Eastern side) of the true Meridian the declination is said to be Eastern ⊕ Positive

Declination East

If the Magnetic meridian is to the left (Western) side of the true Meridian the declination is said to be Western / negative



* The declination at any particular location can be obtained by establishing true meridian from astronomical observation and Magnetic Meridian from Compass.

* Isogonic line is the line drawn through the points of same declination.

* Agonic line is the line drawn through the points of zero declination.

Relation b/w true bearing And Magnetic bearing

True bearing (TB) = Magnetic Bearing (MB) ± Declination

Use +ve sign is the declination is to the east And -ve sign if it is to the west.

Local Attraction :-

A freely suspended, properly balanced magnetic needle is normally expected to show true magnetic meridian. However, some local objects like steel structures and Electric wire influence that the needle and attracts towards them. Thus needle is forced to show slightly different direction. This disturbence is called local attraction.

List of object which caused local attraction

i) Magnetic rock (or) Iron ore

ii) Steel structure, rails, Electric poles and cables.

iii) Bunch of keys, Knife, Iron buttons

iv) chains, arrows, etc.

Detection of local attraction:-

i) Take fore bearing and back bearing of all survey line of closed traverse.

ii) If stations are affected by local attraction then difference of Fore bearing and Back bearing will not be equal 180°

iii) If difference of fore bearing and back bearing is equal to 180° then stations are free from local attraction

iv) If the difference is not equal 180 go back to previous station and check whether the difference is due to personal error.

v) If still differences are exist there is local attraction at one @ Both the stations

PLANE TABLE SURVEYING {or Traverse Table}

Plane Table & Accessories.

A plane tabling is a graphical method of survey in which the field observations & plotting proceed simultaneously.

3rd Civil
B Section

Plane Table consists of the following
1. Drawing board mounted on a tripod.
2. Straight edge called an alidade.
3. Accessories to the plane table.

1. The drawing board: Drawing board is made of well-seasoned wood & varies in size from (40 cm X 30 cm) to (75 cm x 60 cm).

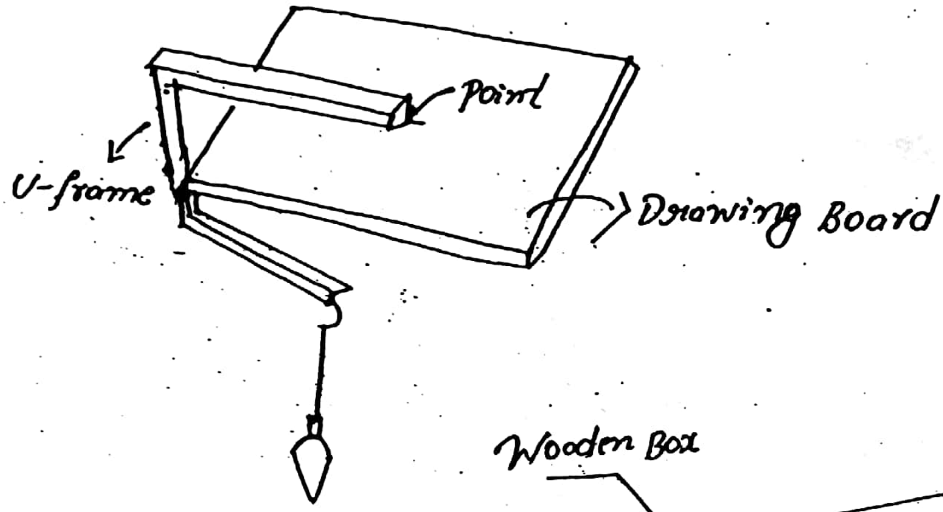
2. Alidade

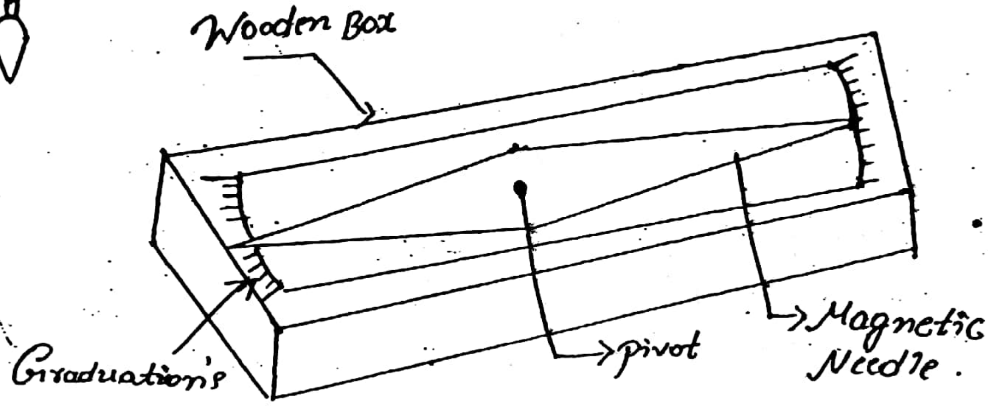
Alidade is a straight-edge ruler. One of the edge is graduated & used for drawing line of sights. Depending upon the type of line of sight provided there are two types of alidade.

i) Plane ~~Table~~ Alidade ii) Telescope Alidade.

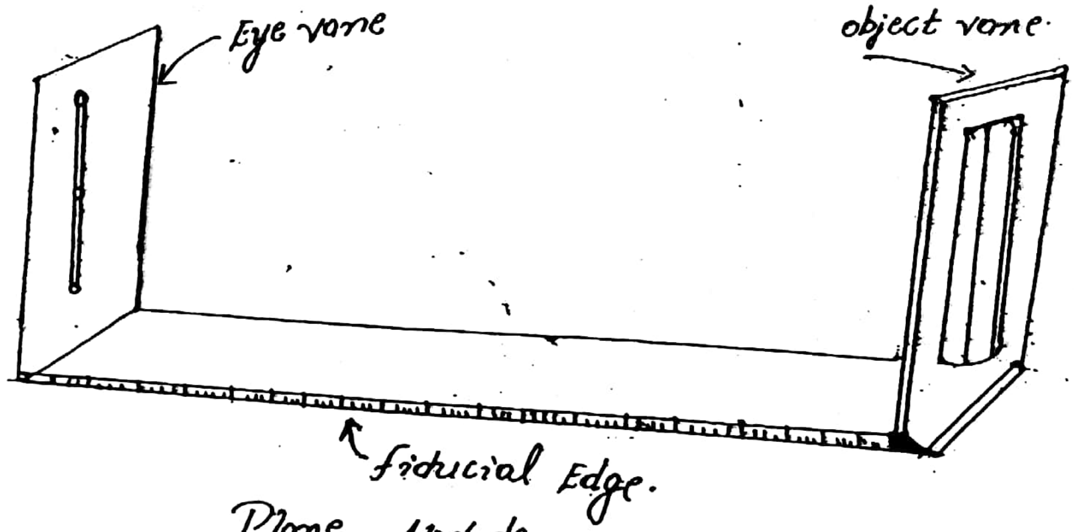
3. Accessories to the plane Table.

a. Trough Compass → The compass is used to mark the direction of meridian on the paper.
b. U-frame (Plumbing fork) → used for centering the table.
c. Water proof cover → To protect the sheet from rain.
d. Spirit level or level tube → To level the plane table.
e. Drawing sheet, pencil, Eraser.
→ Drawing paper used should be of superior quality. For work of high precision, fibre glass sheets or paper backed with aluminium sheet are used.





Trough Compass.



Plane Alidade.

37

Advantages and Disadvantages (Limitations) of Plane Tabling

Advantages:

1. There is no possibility of omitting measurements.
2. The Surveyor can compare the plotted work in the field itself.
3. Local attractions will not influence the plotting.
4. No great skill is required to produce satisfactory map
5. Possibilities of booking errors are eliminated.
6. Irregular objects are represented more accurately since they are seen while plotting.
7. Simple and cheaper than theodolite
8. Suitable for small scale maps & method is quite fast

Limitations (or Disadvantages)

1. Reproduction of map to different scale is difficult.
2. Plane table is heavy, inconvenient to transport.
3. Plane table is not intended for ~~heavy~~ Accurate work.
4. It needs several accessories.
5. Survey cannot be conducted in wet weather & rainy day.

Setting up the Plane Table.

Setting up the plane table includes the following three operations.

1. Centering the plane table.
2. Levelling the plane table.
3. Orientation of plane table.

centering the plane Table.

This is the process of adjusting the position of point on plane table exactly over it's position on ground station. This is acheived using plumbing fork & by moving legs of tripod.

Leveling the plane Table.

The process of levelling is carried out with the help of level tube. The bubble of level tube is brought to center in two directions, which are right angles to each other. This is achieved by moving legs.

orientation: (Imp)

orientation is the process of setting plane table at a station such that all the lines plotted are parallel to corresponding lines on the ground. Accuracy of plane table survey mainly depends upon how accurately at each station perfect orientation is acheived. It can be acheived by one of the following methods.

i) using trough compass.
ii) By back sighting.

i) Using trough compass:- for orientation, the compass is placed on the plane table such that the needle floats centrally, A line is drawn to show North direction. This orientation is to be maintained at all subsequent stations.

At any other station, where the table is to be oriented, the compass is placed against this line & table is oriented by turning it until the needle floats centrally.

* When speed is more important than accuracy this method is used.

This method is less accurate because local attraction to compass affects proper orientation.

ii) Orientation by Back Sighting.

It is commonly employed method.

* Before shifting the table, from station A to station B line ab is drawn from plotted position of station A towards next station B.
* Distance AB is measured & plotted position b of station B is located.
* Then plane table is shifted to station B, & centred such that the point 'b' is exactly over station B.
* keeping alidade along ba station A is sighted & clamped. This gives the required orientation.

Methods of Plane Table Surveying:

The following four methods are available for carrying out plane table survey:

i) Radiation
ii) Intersection
iii) Traversing
iv) Resection

Note (Imp)

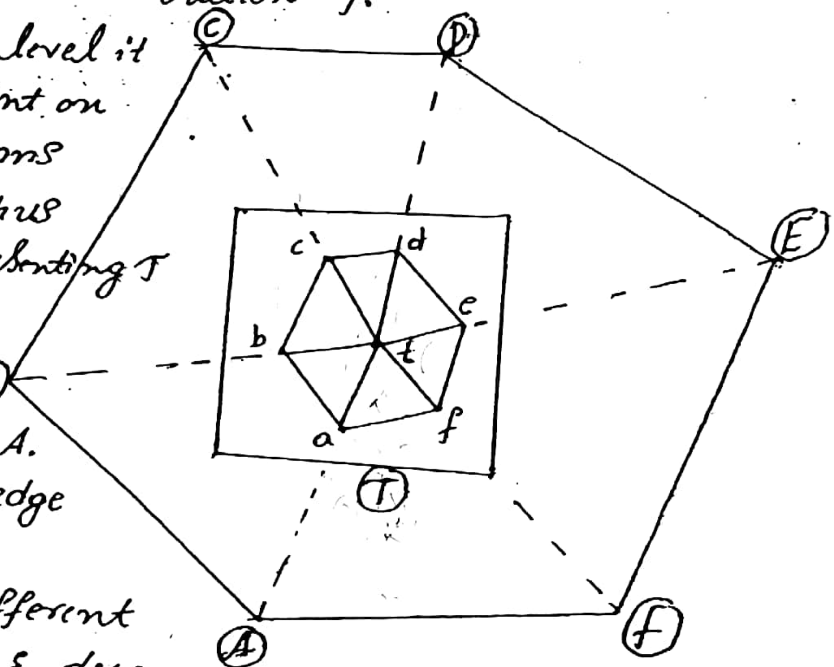
* Radiation & Intersection methods are generally employed for locating the field details.
* Traversing & Resection are used for locating & testing the positions of plane table stations on drawing sheet.

Radiation.

* In this method, a ray is drawn from the instrument station towards the point, the distance is measured between the instrument station & that point. Point is then located by plotting ~~to some save~~ the measured distance to some scale.

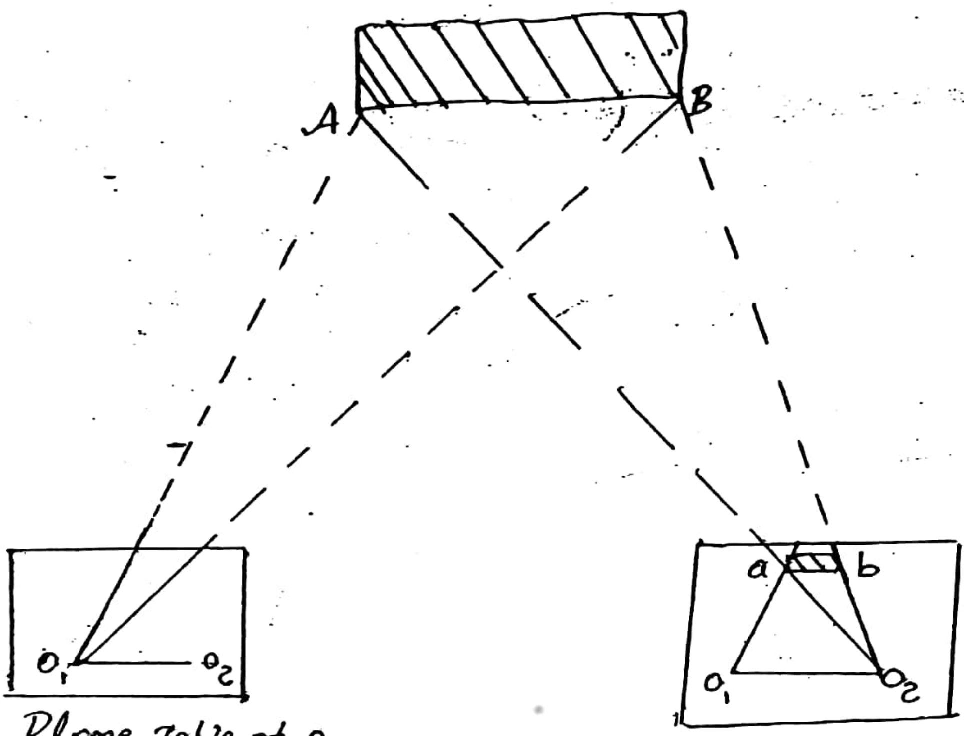
* This method is more suitable when the distances are small

following steps are necessary to locate the points from an instrument station T.



1. Set the table at T, level it & transfer the point on to the sheet by means of plumbing fork, thus getting point t representing T. Clamp the table.
2. keep the alidade touching t & sight to A. Draw the ray the edge of the alidade.
3. Similarly sight different points B, C, D, E, F & draw corresponding rays.
4. Measure TA, TB, TC, TD, TE etc in the field & plot their distances to some scale along the corresponding rays, thus getting a, b, c, d, e etc.

Intersection

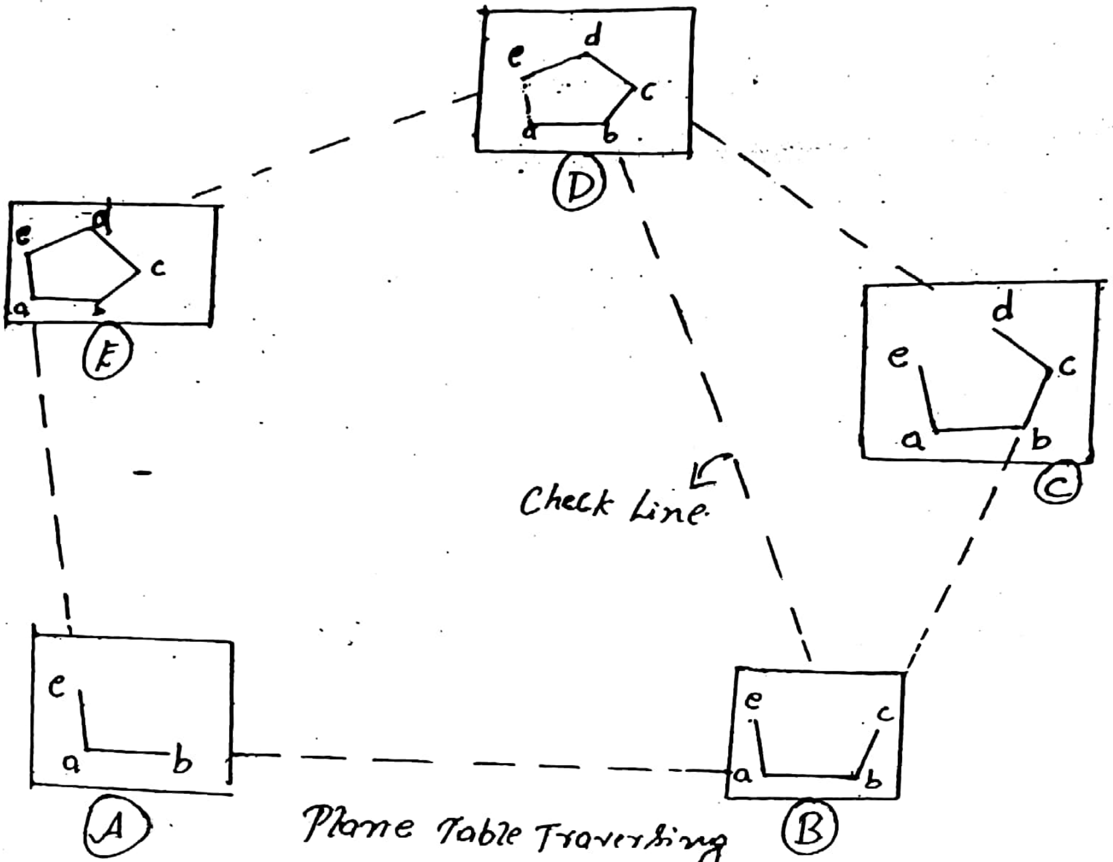


Plane Table at o_1

Plane Table at o_2

* In this method rays are drawn to an object from plotted positions of two stations & the intersection is the plotted position of the object.
* It needs linear measurements b/w the two station points & there is no need to measure distances upto objects.
* O_1 & O_2 are the plotted position's of stations. After setting the plane table at station O_1, the rays O_1A, O_1B, etc are drawn.
* Then plane table is shifted to O_2 & orientation is done by backsighting. then ray O_2A & O_2B are drawn
* The intersection of lines O_1A, O_1B with O_2A & O_2B gives points a & b. Join thes points to get distance b/w two inaccessible points.

Traversing



Plane Table Traversing

(*) This method is used for locating instrument stations in closed traverse or in an open traverse.

(*) From the first station (A) before shifting to next station (B) a ray is ~~dr~~ drawn towards next station. In closed traverse a ray is drawn to the last station (E) also.

(*) ~~Before going to next station~~ The ~~the~~ distance b/w the stations is measured & plotted to scale.

(*) Shift the table to station B. ~~B~~ After centring & levelling by backsighting table is oriented.

(*) A ray is drawn towards next station (c), then details are completed & proceeded to new station. Subsequently traverse is completed.

Resection:

Resection is the method used to locate the plotted position of the survey station by drawing ~~by drawing~~ resectors from plotted position of the objects.

Resectors: The rays drawn from the unplotted position of a station to the points of known location are called resectors.

Resection can be done after orientation of table by any one of the following methods.

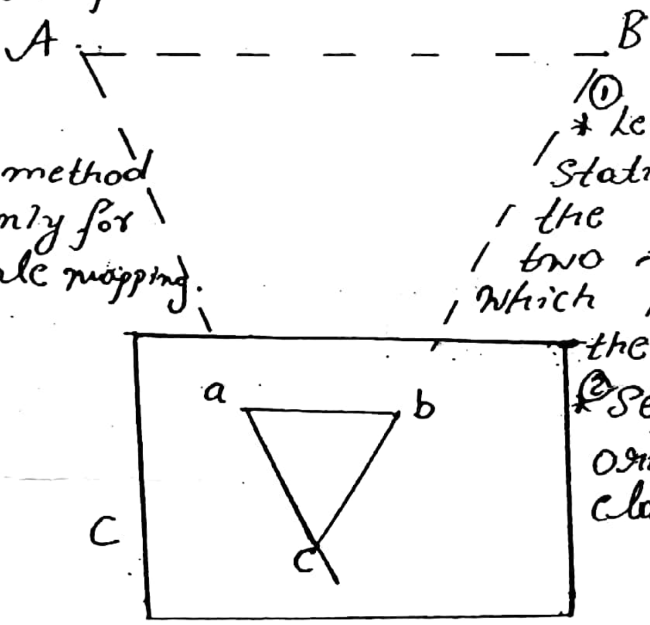
i) By Compass

ii) By Backsighting

iii) By Solving two point problems.

iv) By Solving 3 point problems.

i) By Compass.



Note: This method is used only for small scale mapping.

① * Let C be the instrument station to be located on the plan. Let A & B be two visible points on field which have been plotted on the sheet as a & b.

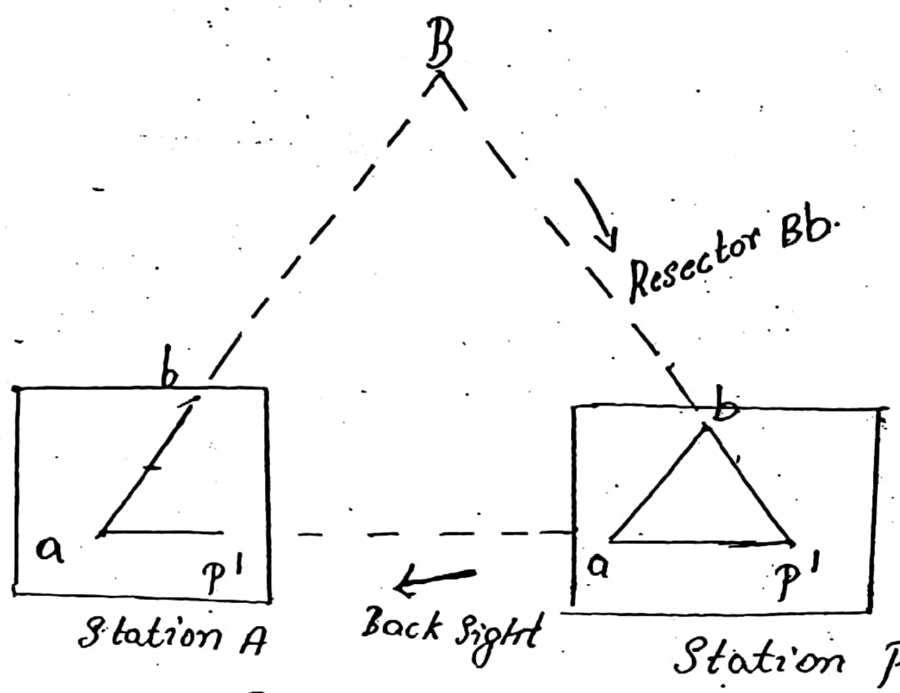
② * Set the table at C & orient it with compass. Clamp the table.

Resection after orientation by compass.

*③ Pivot the alidade about a, draw a resector towards A; Similarly sight B from b & draw a resector. The intersection of 2 resectors will give c.

10)

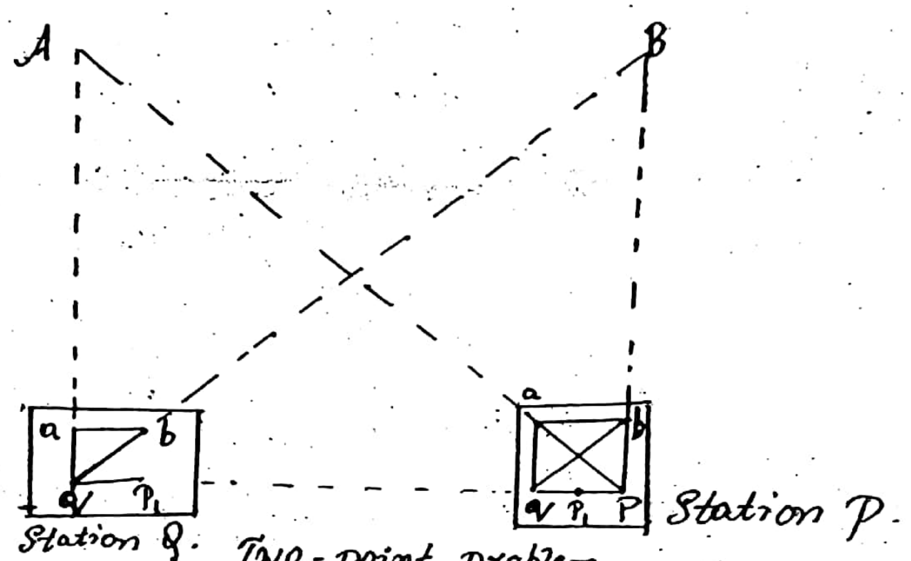
ii) Resection after orientation by Back Sighting



Resection after Back Sighting.

* From Station A, B has been already plotted as b.
* Before shifting table to Station P from Station A a ray is drawn towards P say ap'.
* Plane table is approximately centred over Station P & along p'a alidade is kept & the table is oriented by back sighting Station A & table is clamped.
* Pivote the alidade about b. resector Bb is drawn
* Intersection of this resector with ap' gives the position of P.
* Thus 'P' is located without measuring the distance b/w A&P.

iii) Two point Problem & its Solution-



Two-point problem.

The problem of finding position of the station point occupied by the plane table with the help of two well-defined points, the plotted positions of which are known, is called two-point problem.

- Select a suitable auxiliary point Q near P such that angles PAQ & PBQ are not too acute.
- Roughly orient the table at Q & draw resectors Aa & Bb.
- Draw a ray qP_1 sighting Station P.
- Shift the plane table to P, orient the table by backsighting to Station Q.
- Draw resector Aa & Bb to intesect at point P.
- Thus, making use of positions of two well-defined points the plane table position P is obtained on the drawing sheet.

(12)

iv) Three point problem (Bessel's Graphical Solution)

fixing the plotted position of the station occupied by the plane table by means of observation to three well-defined points whose plotted positions are known. is called three-point problem.

Procedure: (As per manual)

- Diagram (As field map)

Errors in Plane Table Surveying:

The possible errors in plane table survey may be grouped into

i) Instrumental errors. &

ii). Personal Errors.

i) Instrumental Errors: This type of errors are listed below.

(a) Surface of plane Table. May not be perfectly plane
(b) Edge of the alidade may be straight
(c) Sight vanes of alidade ~~no~~ may not be $\perp^r$ to the base
(d) Plane table clamp being loose.
(e) Sluggish magnetic Compass
(f) Defective bubble tube

Personal Errors.

1. Centring Errors
2. Levelling Errors
3. Orientation Errors
4. Errors due to instability of Tripod
5. Sighting Errors & Plotting errors

To avoid personal errors.

i) Set the tripod on firm ground
ii) Do not apply undue pressure on table
iii) Use sharp edged pencil
iv) Take all the care to draw rays correctly.

Three-Point Problem

Procedure:-

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

1. After having set the table at station P, keep the alidade on ba, and rotate the table, so that A is bisected. Clamp the table,
2. Pivoting the alidade about b, sight to C and draw the ray xy along the edge of the alidade fig 7(a)
3. Keep the alidade along ab and rotate the table till B is bisected. Clamp the table
4. Pivoting the alidade about a, sight to C, Draw the ray along the Edge of the alidade to intersect the ray xy in c fig 7(b) Join cc'
5. Keep the alidade along cc' and rotate the table till C is bisected. Clamp the table. The table is correctly oriented Fig (7c)

Pivoting the alidade about b, sight to B. Draw the ray to intersect cc' in p. Similarly, if alidade is pivoted about 'a' And A is sighted, the ray will pass through p if the work is accurate

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

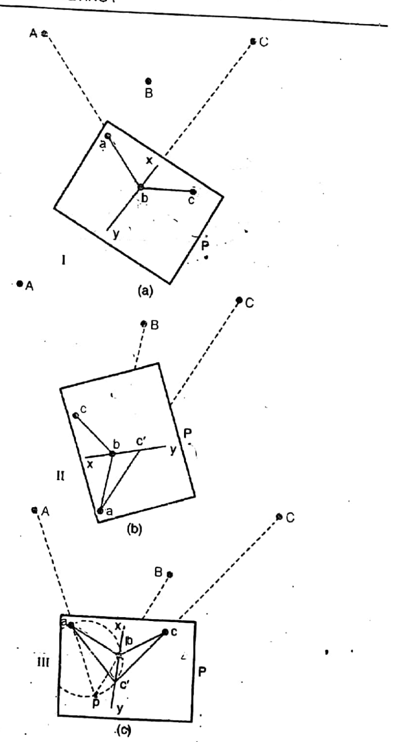


Fig. 11.15 Three-Point Problem: Bessel's Method.

4. To orient the table, keep the alidade alo...

UNIT - III

Surya

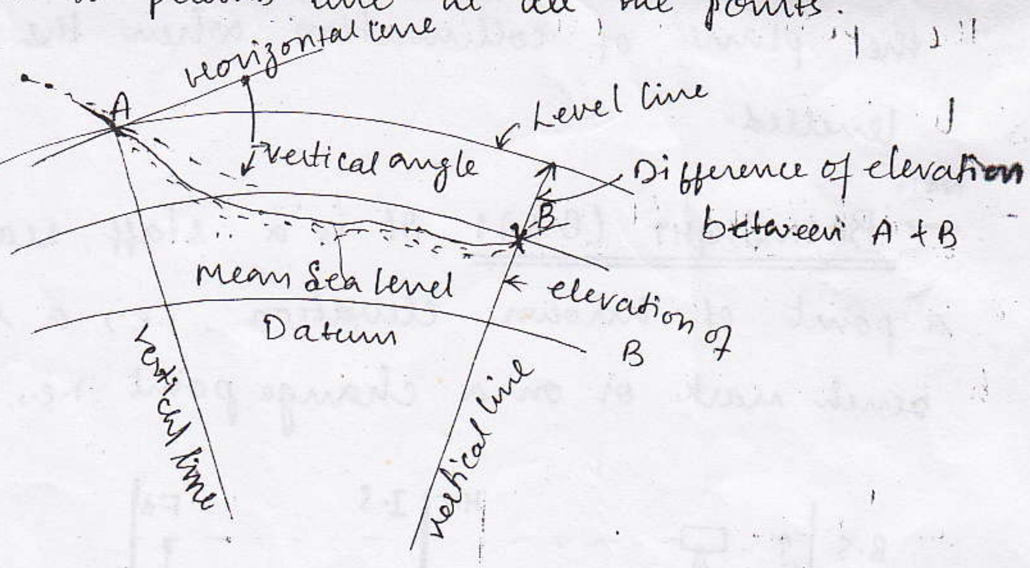
LEVELLING AND CONTOURING

→ **LEVELING:** The operation of determing the difference of elevation of points with respect to each other on the surface of the earth is called levelling.

→ **LEVEL SURFACE:** A surface parallel to the mean spheroidal surface of the earth is called level surface.

→ **VERTICAL LINE:** It is a line from any point on the earth's surface to the centre of the earth and is commonly considered to be the line defined by a plumb line.

→ **LEVEL LINE:** It is a line lying on a level surface and normal to plumb line at all the points.



→ **HORIZONTAL PLANE:** It is a plane tangential to the level surface at the point under consideration and perpendicular to plumb line.

→ **HORIZONTAL LINE:** It is a line lying in the horizontal plane. It is a straight line tangential to level line

→ ELEVATION: It is a vertical distance above or below the datum. It is also known as reduced level (R.L.)

→ AXIS OF TELESCOPE: It is a line joining the optical Centre of the objective to the Centre of the eyepiece

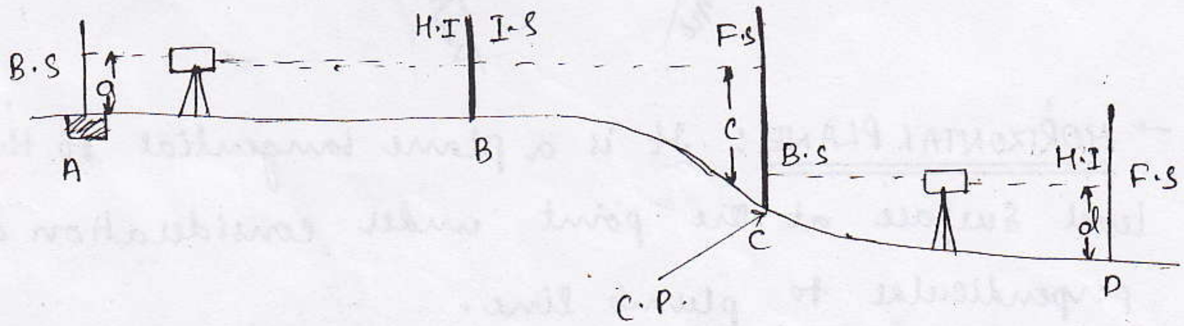
→ LINE OF SIGHT (or) LINE OF COLLIMATION: It is a line joining the intersection of the cross-hairs to the Centre of the objective and its Continuation.

→ AXIS OF LEVEL TUBE or BUBBLE TUBE: It is an imaginary line tangential to the longitudinal curve of the tube at its mid-point.

→ HEIGHT OF INSTRUMENT (H.I.): It is the elevation of the plane of collimation when the instrument is levelled.

No:

→ BACK SIGHT (B.S.): It is a staff reading taken on a point of known elevation i.e., a sight on a bench mark or on a change point i.e., station 'C' in (fig.1)



→ FORE SIGHT (F.S.): It is a staff reading taken on a point whose elevation is to be determined, eg. a sight on a change point, i.e., station C & D in (fig.(1))

→ **INTERMEDIATE SIGHT (I.S.)**: It is a staff reading taken on a point of unknown elevation between backsight and foresight, e.g. a sight on Station B in fig (1).

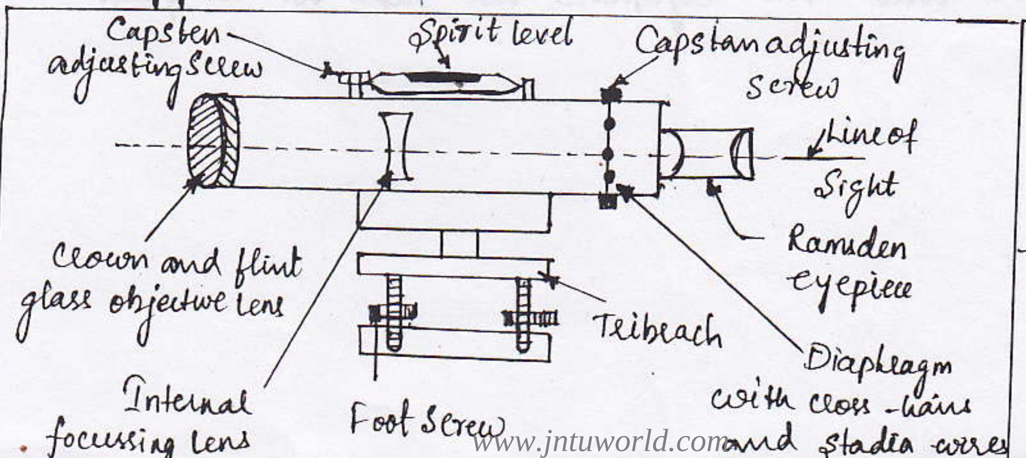
→ **CHANGE POINT (C.P.) or TURNING POINT (T.P.)**: It is a point denoting the shifting of the level. Both F.S. and B.S. are taken on this point. eg. Station C in fig. 1.

→ **STATION**: A point whose elevation is to be determined is called station. The shaft is kept at this point, e.g. A, B and C in fig. 1

→ **PARALLAX**: It is the apparent movement of the image relative to the cross-hairs when the image formed by the objective does not fall in the plane of the diaphragm.

→ **BENCH MARK (B.M.)**: It is a fixed reference point of known elevation.

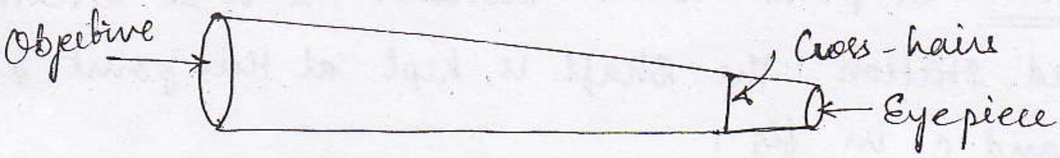
→ **LEVEL**: The instrument which is used to determine the vertical distance of points is known as level. A level consists of a telescope to provide the line of sight, a level tube to make the line of sight horizontal, a levelling head to bring the bubble of level tube at the centre, and a tripod to support the level.



→ Sketch of Dumpy level

TELESCOPE:-

- A telescope consists of a diaphragm ring and two convex lenses. The lens near the eye is called eyepiece and that towards object is called objective.
- The diaphragm ring consists of cross-hairs near the eye piece.



- Focussing screw is provided to give lateral movement of telescope.
- Adjusting screw is provided to give small movement of telescope about vertical axis.

External Focussing Telescope:-

- Focussing is achieved by the external movement of objective tube.
- The movement is done with the help of a focussing screw.

Internal Focussing Telescope:-

- Focussing is done internally with a negative lens by moving the negative lens with a focussing screw.
- The objective and the eyepiece are kept at a fixed distance.

Advantages of internal focussing telescope:-

- Short in length and compact
- Difficult for dust or moisture to enter the telescope barrel.
- The error, tending to cause the movement of internal lens to become non-axial, will be much less than that resulting from a change in the position the line of sight of an external focussing telescope when the objective or eyepiece slide becomes loose.

Disadvantages of internal focussing telescope:-

- Reduction in brilliancy of the image resulting from the passage of light rays through the additional internal lens.
- Tends to be more expensive.

LEVELLING HEAD:-

- It is generally a conical socket attached with a triangular base called tribrach, over which level tube is provided. It is having three or four levelling screws.
- The bubble in the level tube is brought to its centre by adjusting the level screws.

LEVEL TUBE:

- It is a glass tube filled with a sensitive liquid such as alcohol or ether leaving enough space to form a bubble.

- The tube is provided with uniform graduations (generally of 2mm length) to show the exact positions of bubble.
- The bubble is made to come at the centre of level by levelling screws in all directions so that the instrument is horizontal.

SENSITIVENESS OF LEVEL TUBE:-

Measurement of Sensitivity:

(i) Fix two points at a known distance apart, say 100 m.

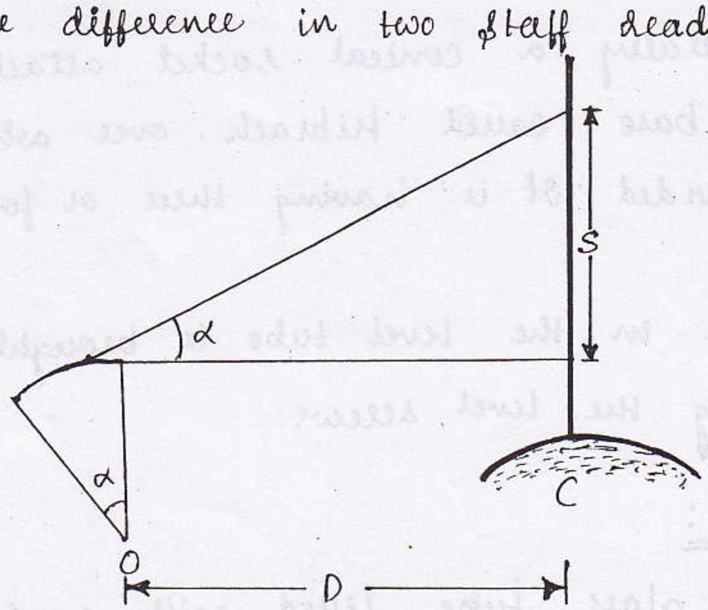
(ii) Set up and level the instrument at 'O'.

(iii) Take the reading on staff held vertical at C.

(iv) By turning the foot screw, move the bubble to n divisions.

(v) Read the staff again.

(vi) Find the difference in two staff readings.



$$\alpha = S/D = n \times (l/R) \quad \rightarrow (1)$$

The term l/R is called sensitivity of bubble tube and equal to α'

From $\alpha' = l/R = S/nD$ (in radians)

$\therefore \alpha' = l/R = S/nD \times 206265$ (in seconds)

where α = angle between the line of sights in radians

D = distance of the instrument from staff.

n = number of divisions through which the bubble is moved.

R = Radius of curvature of tube.

S = Staff intercept

l = length of one division of bubble tube (usually 2mm)

TYPES OF LEVELS:-

DUMPY LEVEL:

- Most widely used level and its telescope is rigidly fixed to its support. The telescope can neither be rotated about its longitudinal axis nor it can be removed from its support.

WYE - LEVEL:

- Similar to dumpy level except that the telescope is supported by two Y - sloped uprights, so that the telescope can be lifted clear of the Y - supports.

COOKE'S REVERSIBLE LEVEL

- It combines good features of both dumpy and wye - levels. It may be rotated about its line of sight giving a bubble left and bubble right reading. Thus, the collimation error is eliminated.

CUSHING'S LEVEL:

- The telescope can neither be revolved about its longitudinal axis nor can it be removed from its socket. However, the Object glass and the eye piece along with the diaphragm are reversible and can be interchanged.

AUTOMATIC (or) AUTO (or) ADJUSTING LEVEL

- It is similar to dumpy level with its telescope fixed to the tribrach.
- For approximate levelling, a circular spirit bubble is attached to the side of telescope.
- For accurate levelling, a stabilizer or compensator is fitted inside the telescope, which automatically levels the instrument.

TILTING LEVEL:

- The telescope can be rotated about horizontal axis by adjusting tilting screws.
- It enables surveyor to quickly centre the bubble and thus bring line of sight into the horizontal plane.

DIGITAL LEVEL:

- Has the advantage of being able to measure and record electronically.

HAND LEVEL:

- It is held in the hand while in use and is adjusted by hand alone.
- This is recommended for short sights and high accuracy is not required.

ABNEY HAND LEVEL:

- Also known as clinometer. It is similar to hand level with the difference that the bubble tube is movable with respect to the sighting tube.
- Vertical angles can also be measured.

LEVELLING STAFF:-

- A levelling staff is a straight, rectangular, wooden rod graduated into metres and smaller divisions.
- The reading given by the line of sight on a levelling staff is the height of the line of collimation from the point on which the staff is held vertically.

SELF READING LEVELLING STAFF:

- These may be 3-5 metre in length. A solid staff is usually 3m long, and a folding staff is generally 4m in length. The folding staff is made of two pieces each of 2m length.
- The minimum graduation on the staff is 5mm

INVAR PRECISION LEVELLING STAFF:

- It is used for precise levelling work and is 3m long
- An Invar band graduated at 5 or 10 mm intervals is fitted to a wooden staff.

SOPWITH TELESCOPIC STAFF:

- It is 14 ft. long and made into three pieces, the top and middle ones being 4 ft 5" and the bottom one is 5 ft. long. Each length slides into the lower length when folded.

TARGET STAFF:

- It is 13 ft. long and consists of two lengths held together by means of a brass clamping screw. One of the lengths can be slided over the other. Both the faces are graduated but in opposite directions. It is facilitated with movable target having a vernier that can facilitate reading.

TEMPORARY ADJUSTMENTS:

- The temporary adjustments consists of setting up, levelling and elimination of parallax.

SETTING UP:

- While locating the level, the ground point should be so chosen that (a) the instrument is not too low or to high to facilitate reading on a bench mark (b) the length of back sight should preferably be not more than 98.0 m and (c) the back sight distance and fore sight distance should be equal, and the foresight should be so located that it advances the line of levels.

- Setting up includes fixing the instrument and approximate levelling by leg adjustment.

FIXING THE INSTRUMENT OVER TRIPOD:

- The clamp screw of the instrument is released. The level is held in the right hand. It is fixed on the tripod by turning round the lower part with the left hand and is firmly screwed over the tripod.

LEG ADJUSTMENT:

- The instrument is placed at a convenient height with the tripod legs spread well apart and so adjusted that the tripod head is nearly horizontal as can be judged by eye. Fix any two legs of the tripod firmly into the ground and move the third leg right or left in a circumferential direction untill the main bubble is approximately at the centre. The third leg is then pushed in to the ground.

LEVELLING UP:

Levelling with a three-screw head:

(1) The clamp is loosened and the upper plate is turned until the longitudinal axis of the plate is parallel to

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

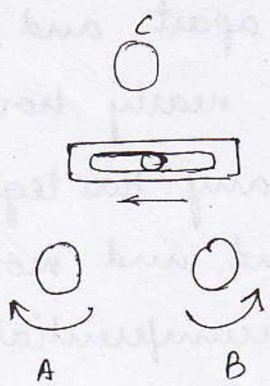
(2) The foot screws are turned uniformly towards eachother or away from each other until the plate bubble is central.

(3) The telescope is rotated through 90° so that it lies over the third foot screw.

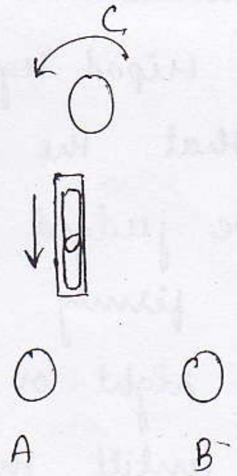
(4) The third screw is turned until the plate bubble is central.

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

(6) The telescope is now rotated through 180°. The bubble should remain central if the instrument is in proper adjustment.



(a)



(b)

ELIMINATION OF PARALLAX:-

- It consists of focussing the eye piece and objective of the level.

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Focussing the eye-piece:

- The operation is done to make the cross-hairs appear distinct and clearly visible. The following steps are involved.

(i) The telescope is directed skywards or a sheet of white paper is held infront of the objective.

(ii) The eyepiece is moved in or out till the cross-hairs appear distinct.

Focussing the objective:

- This operation is done to bring the image of the object in the plane of cross-hairs. The following steps are involved.

(i) The telescope is directed towards the staff.

(ii) The focussing screw is turned until the image appears clear and sharp.

PERMANENT ADJUSTMENTS:-

- These are the adjustments that are done to set the essential parts of the instrument in their true positions relative to each other.

- For a level, if care is taken to equalize backsight and foresight distances, any error due to imperfect permanent adjustment is eliminated.

ective

PRINCIPAL OF REVERSAL (or) PRINCIPAL OF REVERSION:-

- The testing of the level is based on this principle which states that if there exits any error in a certain part, it gets doubled by reversing, i.e. revolving the position of that part through 180°.

FUNDEMENTAL LINES OF A LEVEL:-

- These are the axis of the bubble tube, the vertical axis, the axis of telescope, and the line of collimation.

Relationship between fundemental lines:

- There exist fixed relationships between these fundemental lines. These relationships generally get disturbed because of mishandling of the level during its usage in the field and need frequent adjustment.

- The desired relationship of the fundemental lines are as follows:

(1) The vertical axis of the level should be perpendicular to the axis of the plate bubble tube.

(2) The line of collimation should be perpendicular to the vertical axis.

(3) The axis of telescope and the line of collimation should coincide.

- The following two permanent adjustments are required for a dumpy level:

(i) Desired Relation :

To move the vertical axis of the level perpendicular to the axis of the plate bubble tube.

Test :

(a) The instrument is levelled as described under temporary adjustments.

(b) Rotate the telescope through 180°. If the bubble runs out of the bubble tube centre, the adjustment is not in order.

(c) If it is so, count the number of graduations on the bubble tube by which the bubble has run out of its central position.

Adjustment:

(a) Bring the bubble halfway back to a central position by using the two footscrews. This makes the vertical axis truly vertical.

(b) Bring the bubble to the centre of its run by means of the capstan screw provided at one of the ends of the bubble tube. This makes the axis of the bubble tube truly horizontal.

(ii) Desired Relation:

To make the line of sight perpendicular to the vertical axis (or parallel to the axis of bubble) when the instrument is truly levelled.

Test:

(a) This test is known as the two-peg test.

(b) Choose two suitable points A and B about 60m apart and place the level mid way at C as shown below.

(c) Level the instrument and read the staff at A and B. Calculate the difference in elevation between A and B. The difference will be correct even if the line of sight is not parallel to the axis of tube as the error is directly proportional to the length of sight.

(d) Choose another point D in line with A and B about 15 m ahead of B.

(e) Level the instrument at D and again take the observations at A and B.

(f) Calculate the difference in elevations. If it is same as calculated before (step c), the adjustment is correct.

(g) If not, the reading at A will have a bigger error than that at B, since the error is proportional to sight distance.

<u>Adjustment:</u>

(a) The error is removed with the capstan screws securing the diaphragm to the telescope.

(b) Make the diaphragm screws in steps and keep on reading at A and B, till the correct difference is obtained. This is, therefore, a trial and error procedure.

<u>Alternative Adjustment:</u>

(a) Workout the reading that should be obtained at A from D, to make the line of sight horizontal. Let reading at A from C − reading at B from C = h_1

and reading at A from D − reading at B from D = h_2

Required increase in the reading at A = $(h_1 - h_2) \times \frac{DA}{BA}$

(b) Correct the staff reading at A.

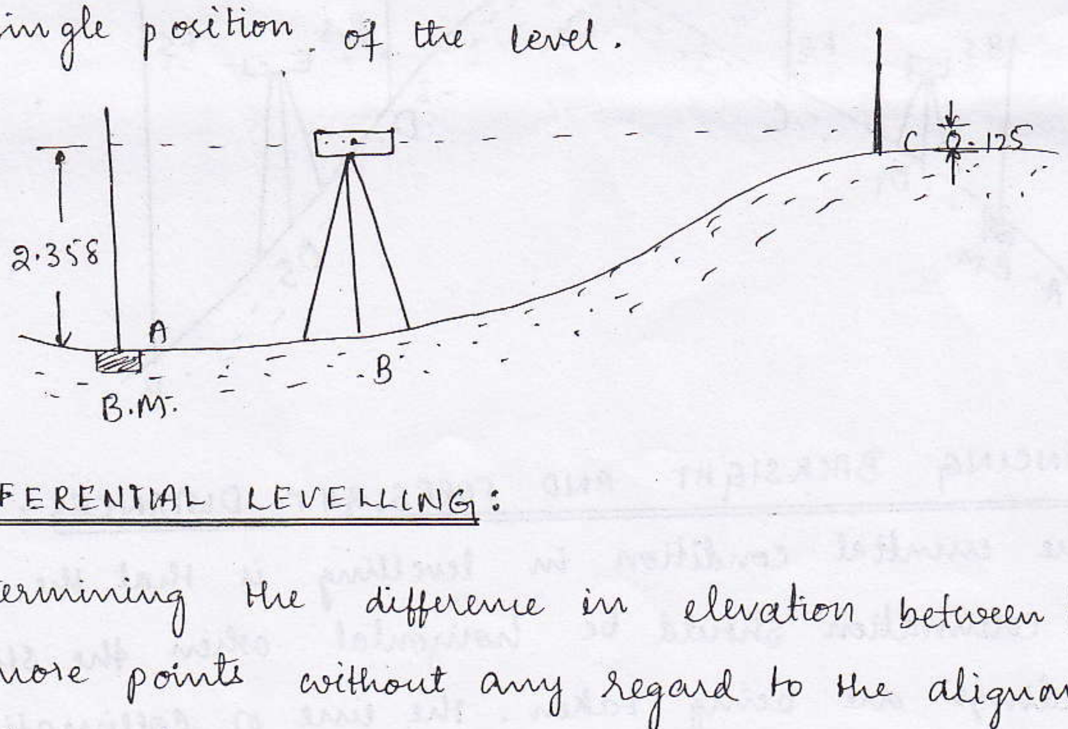
(c) Keep the staff at A and take the observation from D

(d) Diaphragm capstan screws are moved to get the same staff reading as calculated above. (Step a)

PRINCIPLES OF LEVELLING :-

SIMPLE LEVELLING :

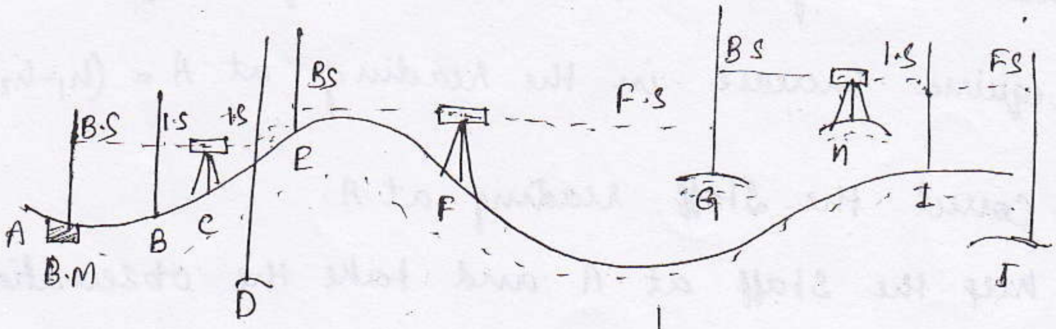
- It is the simplest operation in levelling, when it is required to find the difference in elevation between two points, both of which are visible from a single position of the level.

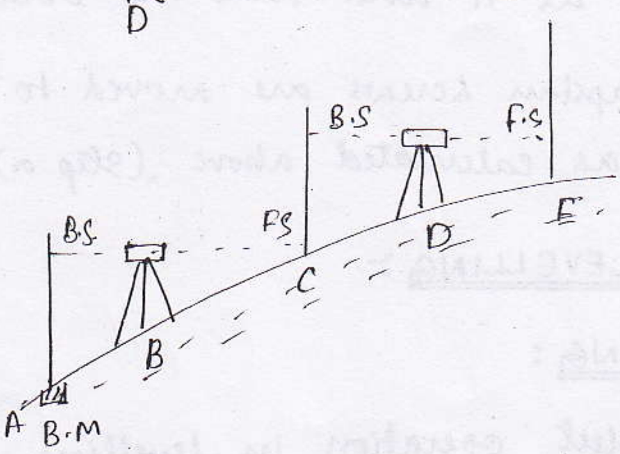


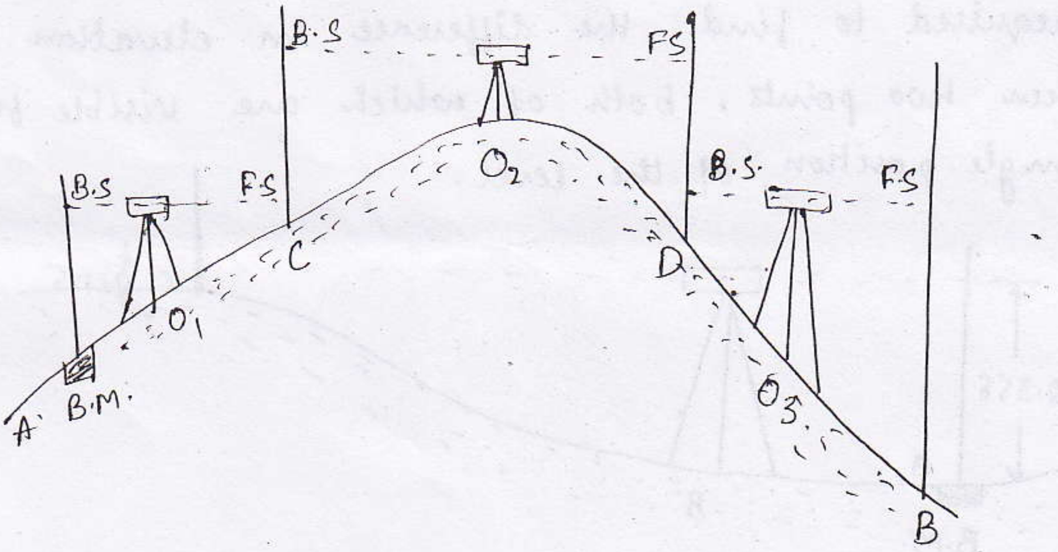
DIFFERENTIAL LEVELLING :

- Determining the difference in elevation between two or more points without any regard to the alignment of the points is called differential levelling.

It is used when (a) two points at a large distance apart (b) the difference in elevation between the two points is large and (c) some obstacle intervenes between the points.

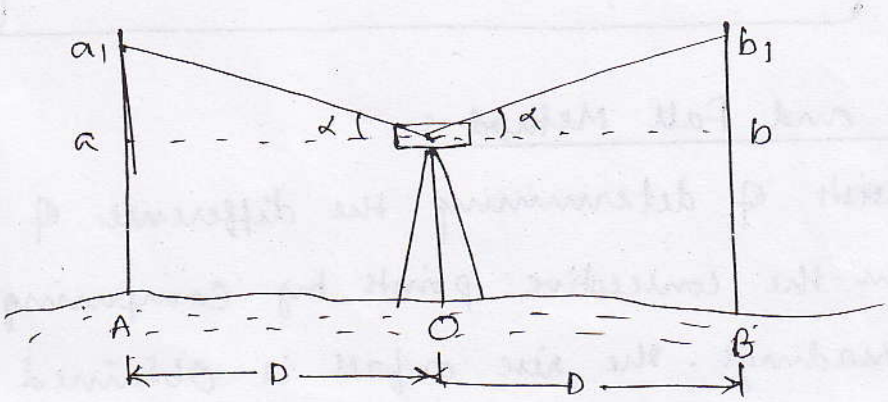






BALANCING BACKSIGHT AND FORESIGHT DISTANCES :

- The essential condition in levelling is that the line of collimation should be horizontal when the staff readings are being taken. The line of collimation is horizontal only when the bubble is at its centre. But this can seldom be ensured with absolute exactness and usually, when the bubble appears to be central, the line of collimation will make a small angle with the horizontal. Since the error is proportional to the length of sight, the error due to non-parallelism can be eliminated by keeping the lengths of backsight and foresight nearly equal. Therefore, to find the true difference of levels between two points, the level must be kept exactly midway between them, but not necessarily on the line joining them.



BOOKING AND READING THE LEVEL:-

- The observations are recorded in a level book. There are two methods of booking and reading the levels of the points from the observed staff readings.

(1) Collimation Method or Height of Instrument Method:-

- The elevation of plane of collimation or height of instrument for the first setup of the level is determined by adding backsight to the reduced level of a B.M.
- The reduced levels of intermediate points and the first change point are obtained by subtracting the staff readings. The rise or fall is obtained by

subtracting the staff readings taken on these points and a new plane of collimation is set by taking a B.S. on the change point. The height of instrument is obtained by adding this B.S. to its R.L. which was already calculated and the process continues.

Check: $\boxed{\Sigma B.S. - \Sigma F.S. = \text{Last } R.L. - \text{First } R.L}$

(ii) Rise and Fall Method :

- It consists of determining the difference of levels between the consecutive points by comparing their staff readings. The rise or fall is obtained by calculating the difference between the consecutive staff readings.
- A rise is indicated if the backsight is more than the foresight, and a fall if the backsight is less than the foresight.
- The reduced level of each point is obtained by adding the rise to, or by subtracting the fall from the reduced level of the preceding point.

Check:

$$\boxed{\Sigma B.S. - \Sigma F.S. = \Sigma \text{Rise} - \Sigma \text{fall} = \text{Last } RL - \text{first } R.L}$$

METHODS OF LEVELLING:-

(1) RECIPROCAL LEVELLING:

- It is the operation of levelling in which the difference in elevation between two points by two sets of observations.
- This method is very useful when the instrument cannot be setup between the two points due to an obstruction such as a valley, river, etc and if the sights are much longer than those which are ordinarily permissible.
- For such long sights the errors of reading the staff the curvature of earth, and the imperfect adjustments of the instrument become prominent. Special methods like reciprocal levelling should be used to minimise these errors.
- In this method the instrument is setup near one point say A on one side on the valley, and a reading is taken on the staff held at A near the instrument and on the staff at B on the other side of valley. Let these reading b would have an error due to curvature, refraction & collimation.
- Let these readings be c and d. The near reading c is without error, where as reading d would contain an error e due to the reasons discussed above. Let h be the true difference of elevation between A and B.

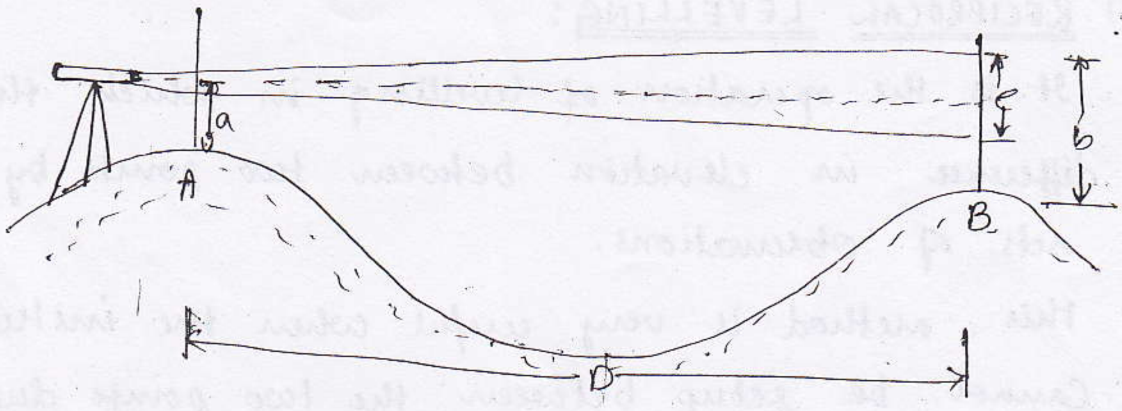


Fig (a)

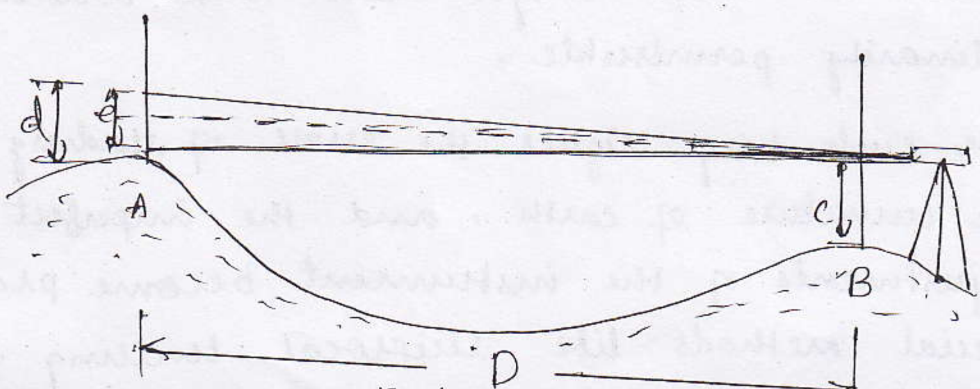


Fig (b)

In the first case (Fig (a))

$$h = (b-e) - a \rightarrow (1)$$

In second case (Fig (b))

$$h = c - (d-e) \rightarrow (2)$$

Adding (1) & (2) we get

$$2h = b - e - a + c - d + e$$

$$2h = b - a - c - d$$

$$2h = (b-a) + (c\ d)$$

$$h = \frac{1}{2}\left[(b-a) + (c-d)\right]$$

As 'e' is eliminated like this, therefore reciprocal levelling eliminates the effect of atmospheric refraction, and earth curvature, as well as the effect of not adjusting the line of collimation.

(2) PRECISE LEVELLING:

- This is the operation of levelling in which precise instruments are used.
- In ordinary levelling, the distances between check points are relatively short.
- In precise levelling, the level loop may be of substantial length and efforts are made to control all the sources of errors.
- The most important error control in precise levelling is the balancing of foresight and backsight distances. This eliminates the collimation error and errors due to curvature, and minimizes errors due to refraction.
- Temperatures are read at intervals to correct the graduations along the length of the staff.
- Precise levelling is used for establishing benchmarks with high precision.

(3) FLY LEVELLING:

- It is an operation of levelling in which a line of levels is run to determine the approximate elevations
- It is carried out for reconnaissance of the area.

(4) CHECK LEVELLING:

- It is the operation of running levels to check the accuracy of the benchmarks previously fixed. At the end of each day's work, a line of levels is run, returning to the B.M. with a view check the work done on that day.

(5) TRIGONOMETRIC LEVELLING:

- This is an indirect method of levelling in which the difference in elevation of the points is determined from the observed vertical angles and measured distances.
- The vertical angles are measured with a transit and the distances are measured directly (plane survey) or computed trigonometrically (geodetic survey).
- This is commonly used in topographical work to find out the elevation of the top of buildings, chimneys, church spires, and so on.

(6) BAROMETRIC LEVELLING:

- The principle used in barometric levelling is that the elevation of a point is inversely proportional to the weight of the air column above the observer.
- The instrument used for measuring pressure is called Barometer. The modified form of a barometer used to find relative elevations of points on the surface of earth is called Altimeter.

- The method used to measure elevations with an altimeter is known as Single Base method. Two altimeters are required. One altimeter is placed at a point of known elevation and the other altimeter is placed at the desired point whose elevation is desired and the readings of these two barometers are noted.
- The difference in elevation between the two points may be obtained by the following formula.

$$\boxed{H = 18336.6\left(\log_{10} h_1 - \log_{10} h_2\right)\left[1 + \frac{T_1 + T_2}{500}\right]}$$

where H = the difference in elevation between two points

h_1, h_2 = the barometric readings (in cm) at the lower and higher points respectively.

T_1, T_2 = temperatures of air (in $0^\circ C$) at the lower and higher points respectively.

(7) HYPSOMETRY:

- The altitude of various points may be obtained by using an instrument known as hypsometer.
- It works on the principle that a liquid boils when its vapour pressure is equal to the atmospheric pressure.
- It may be noted that the boiling point of water is lowered as the pressure decreases i.e., as a higher altitude is attained.

- This method consists in determining the boiling point temperatures at various station. The corresponding atmospheric pressures may be obtained from the tables. In the absence of tables, the following approximate formula may be used.

$$\boxed{h = 76.00 \pm 2.679\, t}$$

where h = pressure in cm

t = difference of boiling point from 100°C

- The difference in elevation between two points is obtained by

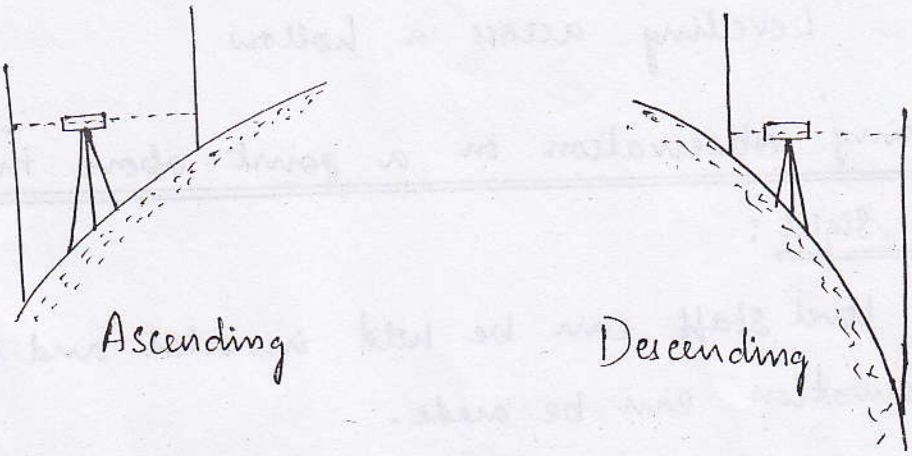
$$\boxed{H = 18336.6 \left(\log_{10} h_1 - \log_{10} h_2\right)\left[1 + \frac{T_1 + T_2}{500}\right]}$$

LEVELLING DIFFICULTIES:-

(1) Ascending or Descending of a hill:

- A plumb bob must be used to check the verticality of the staff, as it is difficult to ascertain the verticality of the staff on slope.
- While ascending a hill it is difficult to take foresight of the staff, as the staff may be held at a higher level and it is possible to take foresight only near the foot of the staff. Similarly in case of descending a hill, the backsight may be taken at the foot of the staff which is difficult.

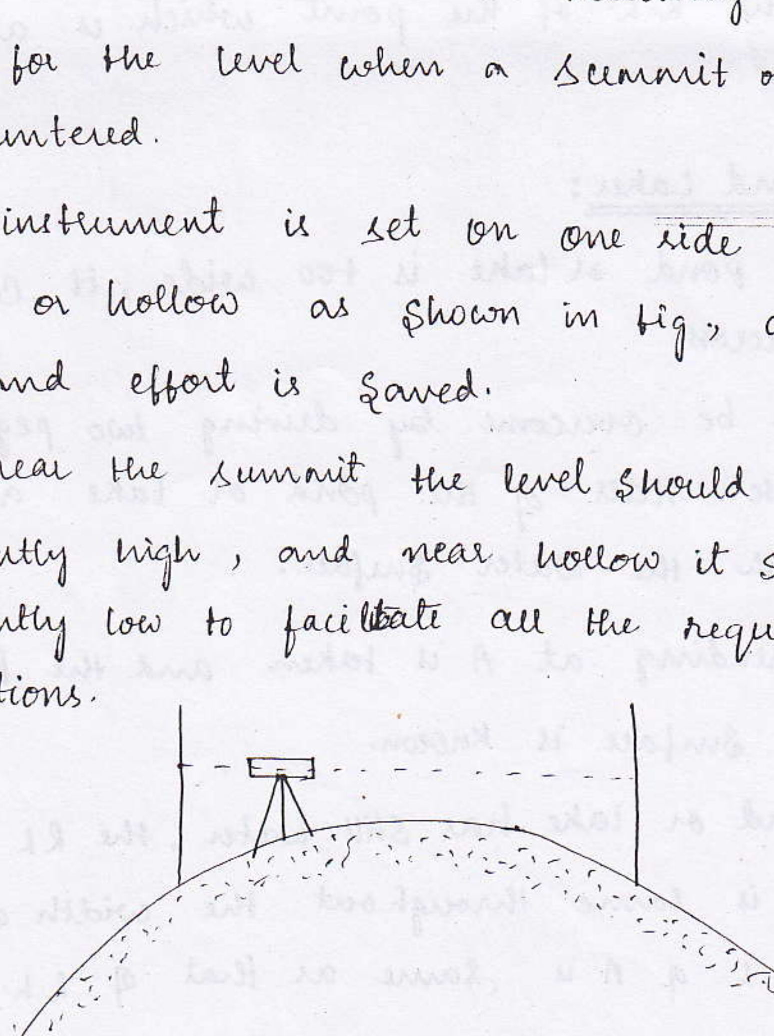
To overcome this difficulty, the staff is placed near the instrument during a foresight while ascending a hill, and during a backsight while descending the hill. Another way can be to place the instrument away from the line of levels and take zig-zag sights.



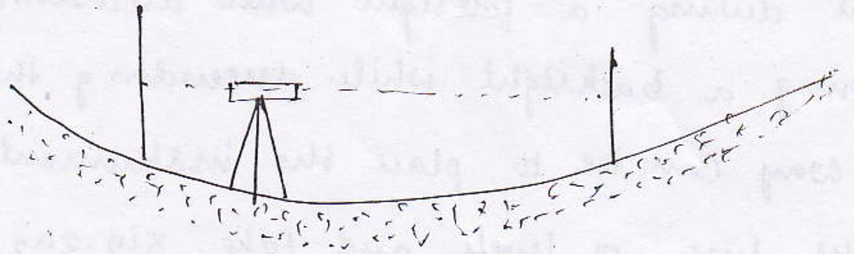
Ascending   Descending

(2) <u>Levelling across a summit or a hollow:</u>

- Care should be exercised in selecting a suitable position for the level when a summit or a hollow is encountered.
- If the instrument is set on one side of the summit or hollow as shown in fig, a lot of time and effort is saved.
- Also, near the summit the level should be set sufficiently high, and near hollow it should be sufficiently low to facilitate all the required observations.



Levelling across a Summit



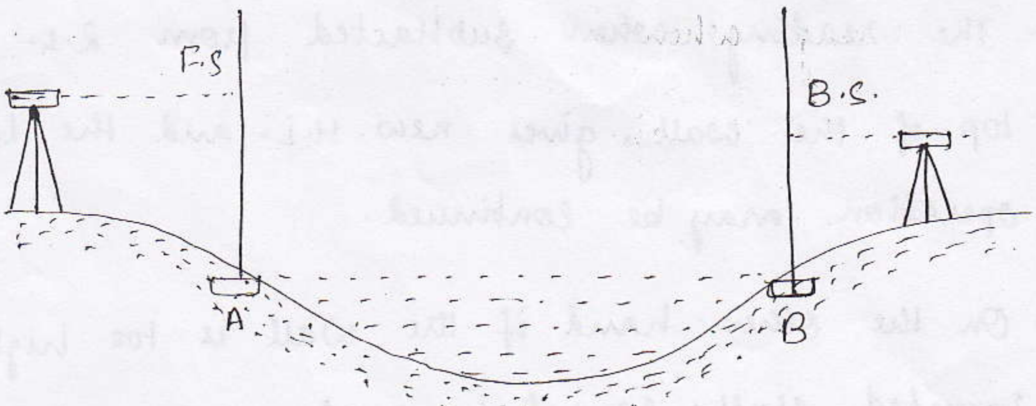
Levelling across a hollow

(3) Taking Observation on a point above the line of Sight:

- The level staff can be held inverted and the observation can be made.
- The staff reading thus obtained is added to the height of instrument instead of subtracting to find the R.L of the point which is above line of sight.

(4) Ponds and lakes:

- When a pond or lake is too wide, it cannot be sighted across.
- This can be overcome by driving two pegs say A and B on opposite sides of the pond or lake and flush with the water surface.
- The staff reading at A is taken and the RL of A or the water surface is known.
- As the pond or lake has still water, the RL of surface of water is same throughout the width of water. Therefore R.L. of A is same as that of R.L of B.



- Now the instrument is shifted to the other side of pond and the staff reading at B is added to the R.L. of B (or A), and H.I. of instrument is known and levelling can be continued.

(5) Levelling Across a River:

- To level across a river, the method of reciprocal levelling is used.

(6) Levelling Past a Wall:

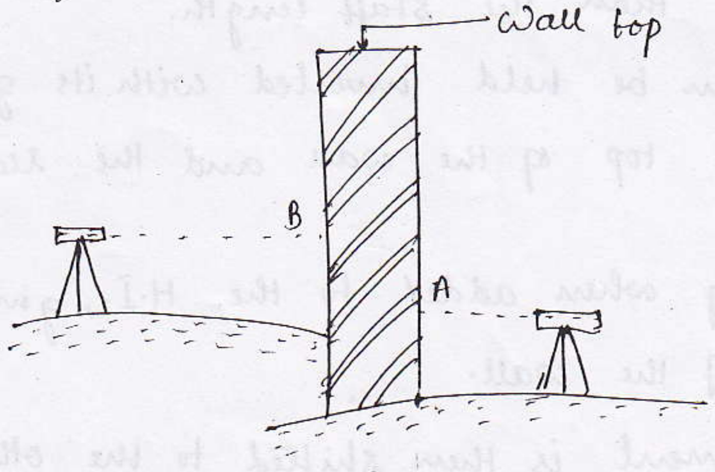
- During levelling, if a wall falls on the course, the R.L. of the top of the wall is found by inverted staff reading when the heigt of the wall above the line of sight is less than the staff length.
- The staff can be held inverted with its zero end touching the top of the wall and the reading is observed.
- This reading when added to the H.I., gives the R.L. of the top of the wall.
- The instrument is then shifted to the other side of the wall. The staff is kept inverted with its zero end touching the top of the wall and facing the instrument.

Surya

- The reading when subtracted from R.L. of the top of the wall, gives new H.I. and the levelling operation may be continued.

- On the other hand if the wall is too high and an inverted staff cannot be used, a tape is used. With the instrument to one side of the wall, a mark is made on the wall when the line of sight strikes it, say a point A. the height from A to the top of the wall is measured with a tape suspended from the top of the wall. When this height is added to the H.I., the R.L. of the top of the wall is determined.

- The instrument is shifted to the otherside of the wall and same procedure is repeated for measuring the height of wall with same tape as it is done when the instrument was at the previous station



Height when subtracted from the R.L. of the wall gives the new H.I the levelling operation is then continued

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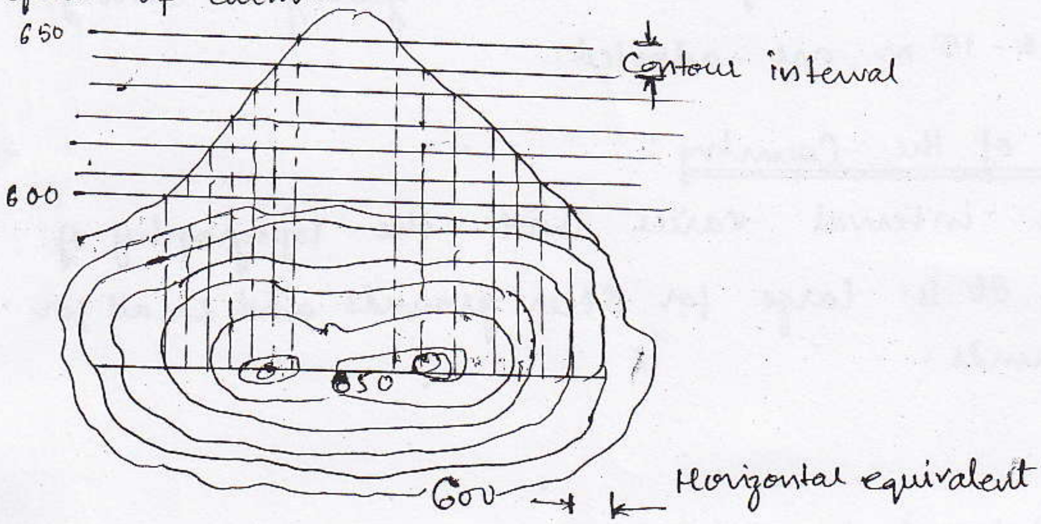
CONTOURING:-

CONTOUR: A contour may be defined as an imaginary passing through points of equal elevation.

(or)

It is defined as the intersection of a level with the surface of earth.



CONTOUR INTERVAL:-

- The vertical distance between consecutive contours is termed as contour interval
- It is desirable to have a constant interval throughout the map.

The contour interval depends on the following factors

(1) **Scale of the Map:**

Contour interval is inversely proportional to the scale of the map. For a topographic map, the interval may range as shown below

Ground Surface	Large Scale (1cm = 1-10m)	Intermediate Scale (1cm = 10-100m)	Small Scale (1cm = 100m onwards)
Flat	0.2 - 0.5m	0.5 - 1.0m	1 - 3m
Rolling	0.5 - 1m	0.5 - 1.5 m	2 - 5m
Hilly	[illegible]	[illegible]	[illegible]

(2) Purpose of the map:

- Contour interval is kept large upto 2.0 m for projects such as highways and railways, whereas it is kept as small as 0.5 m for measurement of earth works, building sites, dams etc. For a city survey, a contour interval of 0.5 m may be adopted, and for more extended surveys such as a geological survey, usually 6 - 15 m are adopted.

(3) Nature of the Country:

- Contour interval varies with the topography of the area. It is large for steep grounds and small for flat grounds.

(4) Time:

- Contour interval is kept large when time is less.

(5) Funds:

- Contour interval is kept large when funds are short and limited.

HORIZONTAL EQUIVALENT:-

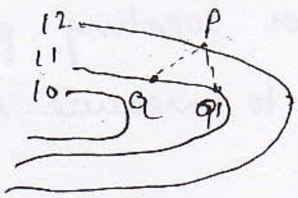
- The horizontal distance between consecutive contours is termed as horizontal equivalent.
- Steeper the ground, lesser the horizontal equivalent.

CONTOUR GRADIENT:

- A line lying on the ground surface throughout and maintaining a constant inclination is called contour gradient.

GRADE CONTOURS:

- The lines having equal gradient along a slope are called grade contours.
- The difference in elevation of two points of grade contours divided by the distance between them is always a constant gradient.



PQ = PQ'

The gradient of $PQ = \frac{(12-11)}{PQ}$

The gradient of $PQ' = \frac{(12-11)}{PQ}$

As PQ = PQ'

$$\therefore \boxed{\frac{12-11}{PQ} = \frac{12-11}{PQ'}}$$

Therefore PQ and PQ' are Grade contours as their gradients are equal.

USES OF CONTOURS:-

(i) With the help of contour map proper and precise location of engineering works such as roads, canals etc can be decided.

(ii) In location of water supply, water distribution and to solve the problems of stream pollution, etc.

(iii) Planning and designing of dams, reservoirs, aqueducts, transmission lines.

(iv) To select sites for new industrial plants.

(v) To ascertain the intervisibility of station.

(vi) To ascertain the profile of the country along any direction.

(vii) To estimate quantity of cutting, filling and the capacity of reservoirs.

GHAT TRACER:-

- This is an instrument used for locating points on a given contour gradient (or) to measure slope or gradient between two points.

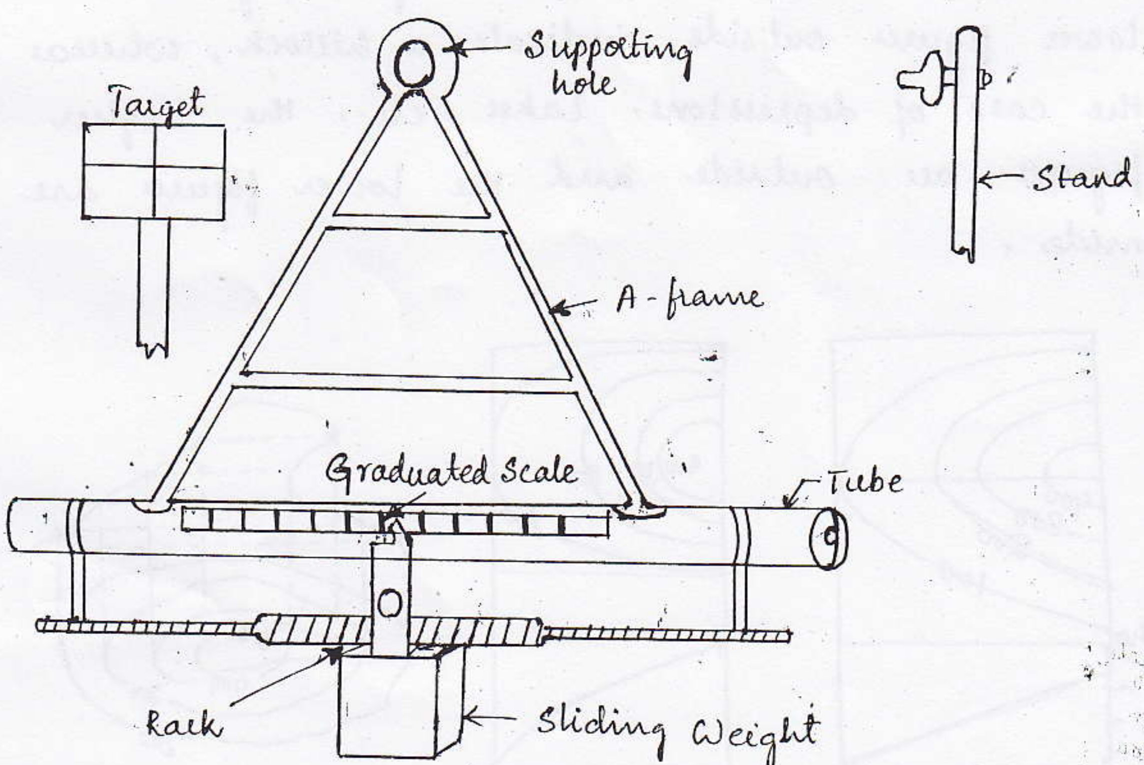
CONSTRUCTION:

- It consists of a hollow tube with an eye-hole at one end and cross wire at the other end to provide a line of sight. It is attached to a sliding A-shaped bracket with a hole for suspension. A weight is attached to the tube.

When the weight is at zero, the line of sight is horizontal. In case the weight is towards the observer, the line of sight is elevated and when towards the cross-wire, it is depressed. The scale attached to the tube gives the gradient.

WORKING:

- The instrument is placed on a point A, with its centre above it on the line, say AB whose gradient is to be measured. The target is placed at the same height as that of the centre of tube and kept on the other point B. The observer sees through the eye-hole and moves the weight till the target is bisected. The corresponding reading on the tube is noted. Incase the points are to be established along a given gradient say 1 in 30, from a point say A, the instrument is kept at A with the reading on the tube as 30. The target, set at the same height as that of the centre of the tube is directed to move along the line of sight till it is bisected. This fixes the point, say B, on the desired gradient. The instrument is moved from A to B and the procedure is repeated for any number of desired points.



GHAT TRACER

CHARACTERISTICS OF CONTOUR LINES:-

(i) All the points on a contour line have the same elevation. The elevations of the contour are indicated either by inserting the figure in a break in the respective contour or printed close to the contour.

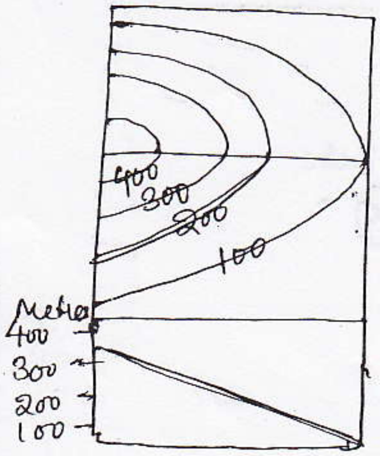
When no value is present, it indicates a flat terrain. A zero meter contour line represents the coast line.

(ii) Two Contour lines do not intersect each other except in the cases of an overhanging cliff or a cave penetrating a hill side.

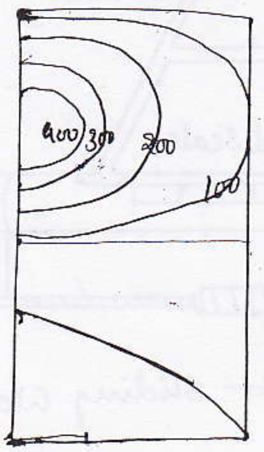
(iii) A Contour line must close onto itself, not necessarily within the limits of a map.

(iv) Equally Spaced contour represent a uniform slope and contours that are well apart indicate a gentle slope.

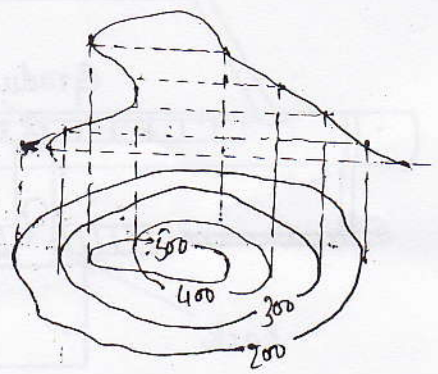
(v) A set of close contours with higher figures inside and lower figures outside indicate a hillock, whereas in the case of depressions, lakes, etc. the higher figures are outside and the lower figures are inside.



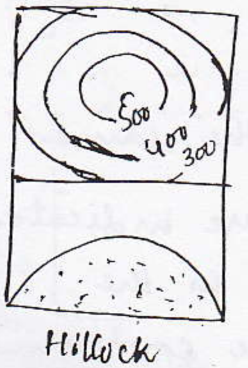
Gentle Slope



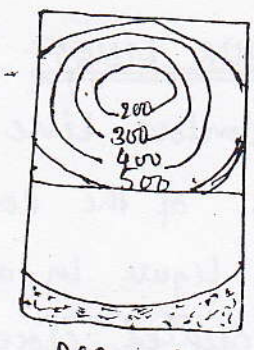
Uniform Slope



Overhanging cliff.

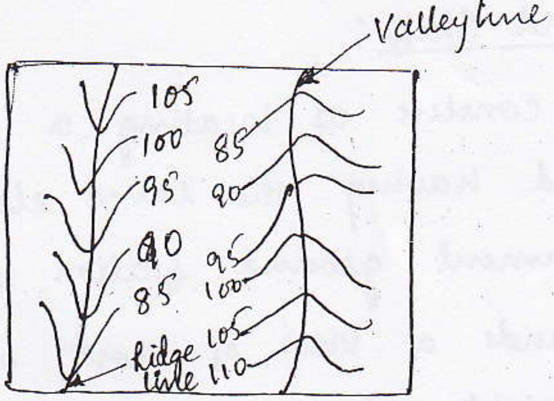


Hillock

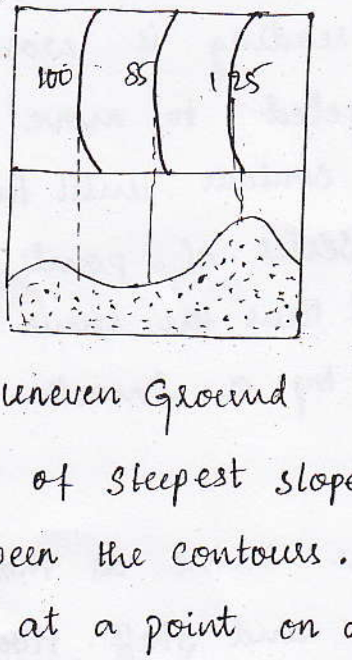


Depression

(vi) A watershed or ridge line (line joining the highest points of a series of hills) and the thalweg or valley line (line joining the lowest points of a valley) cross the contours at right angles.

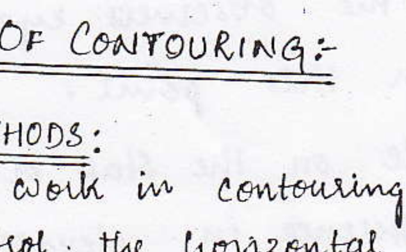


(vii) Irregular contours represent uneven ground.



Uneven Ground

(viii) The direction of steepest slope is along the shortest distance between the contours. The direction of the steepest slope at a point on a contour is, therefore, at right angles to the contour.



Among PQ, PQ_1 and PQ_2, PQ is the steepest slope in the figure.

METHODS OF CONTOURING:-

DIRECT METHODS:

The field work in contouring consists of horizontal and vertical control. The horizontal control for a small area can be exercised by a chain or tape and by compass, theodilite or plane table for a large area. For vertical control either a level and staff or a hand level may be used.

(1) <u>By level and Staff</u>:

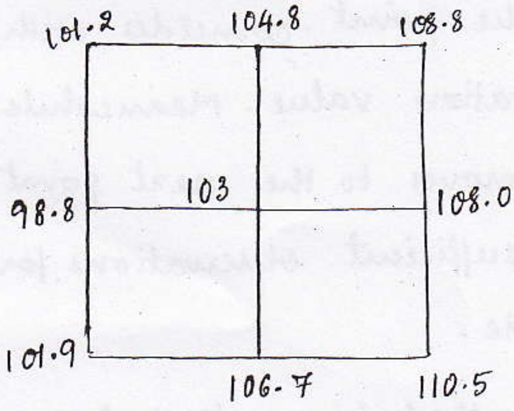
- The method consists of locating a series of points on the ground having the same elevation. To do this an instrument ground station is selected so that it commands a view of most of the area to be surveyed. The height of the instrument is fixed from the nearest benchmark. For a particular contour value, the staff reading is worked out. Staff man is then directed to move right or left along the excepted contour until the required reading is observed. A series of points having the same staff readings, and thus the same elevations, are plotted and joined by a smooth curve.

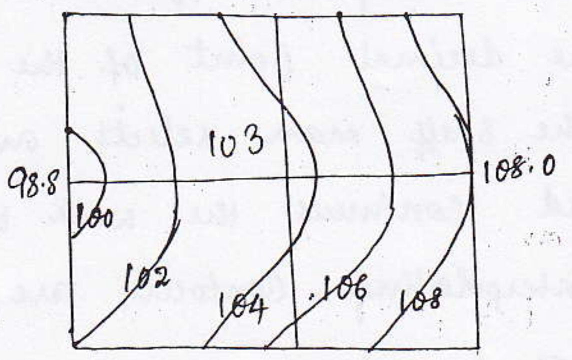
(2) <u>By hand level</u>:

- The principle used is the same as that used in the method using a level and staff. However, this method is very rapid and is preferred for certain works. The instruments used are a hand level, giving an indication of the horizontal line from the eye of the observer, and level staff or a pole having a zero mark at the height of the observer's eye and graduated up and down from this point. When an observation is made on the staff or pole, the reading on it is the difference in elevation between the foot of the observer and that of the pole. In this method the instrument man stands over the bench mark and the staff man is moved near to a point on the contour which has to be plotted. As soon as the instrument man observes the required staff reading for a particular contour, he instructs the staff man to stop and locate the position of the point.

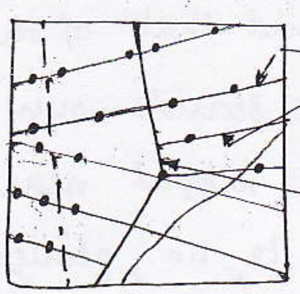
INDIRECT METHODS:-

(1) Method of Squares: This is also called coordinate method of locating contours. The entire area is divided into squares or rectangles forming a grid. The elevations of the corners are then determined by spirit levelling. The levels are then interpolated. This method is very suitable for a small open area where contours are required at a close vertical interval.





(2) Method of Cross-sections: In this method a transit traverse is run. Then suitably spaced sections are projected from traverse lines. The observations are made in the usual manner with a level, clinometer, or theodolite at points on these transverse lines. The contours are then interpolated. This method is suitable for road, railway and canal survey.



(3) Plane table Method : A plane table is placed on the traverse station and an alidade is sighted on a rod with two targets at a fixed distance apart (1-2m). The direction of the line is drawn along the ruling edge of the alidade. With a tangent clinometer the vertical angles are read corresponding to the two targets. The distance and elevation of the staff point is reduced by trigonometric relations. The contours are then interpolated. The observer scales the computed distance along the plotted line to locate the point and writes the computed elevation in such a way that the plotted position of the point coincides with the decimal point of the elevation value. Meanwhile the staff man selects and moves to the next point and continues the work till sufficient observations for interpolating contours are made.

(4) Tacheometric Method : This method is particularly suitable for hilly areas and at places where plane tabling is impractical. First of all, reconnaissance of the area is done and a network of traverses is arranged in such a way that the entire area can be covered. The traverse stations are so chosen that large vertical angles, particularly for long sights are avoided. From these traverse stations a number of

radial lines are drawn at some angular interval depending upon the nature of the country. A tacheometer, fitted with an anallactic lens, is placed on the traverse stations. The observations corresponding to cross-wire, stadia wires and vertical angles are carried out on all the control stations and on the points of detail. The elevations and distances are then calculated and from the observed data, contours are interpolated.

INTERPOLATION OF CONTROLS:-

(1) By Estimation: This is a very crude method and is usually adopted where the ground forms are quite regular, the scale of the map is small, and high accuracy is not required. The positions of the contour points between the ground points are estimated and contours are drawn through them. It is assumed that the slope between the ground points is uniform.

(2) By Arithmetic Calculations: This method is used when high accuracy is required and the scale of the map is of intermediate or large. In this method the distance between two points of known elevations are accurately measured. Then with the help of arithmetic calculations, the positions of the required elevation points are computed.

Let A and B be two points with R.L. 52.60 m and 55.80 m respectively. and at a distance of 16.00 m apart. Let the contour interval be 1.00 m and let it be required to locate a contour between the two points with values 53.00 m. The contour can be located as follows.

Difference of level between A and B = 55.80 − 52.60

= 3.20 m

Difference of level between point A and 53.00 m Contour

= 53.00 − 52.60 = 0.40 m

Distance of 53.00 m contour from point A

$$= \frac{0.40}{3.20} \times 16 = 2.00 \text{ m}$$

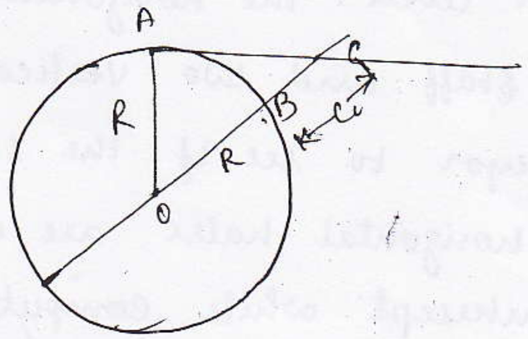
(3) <u>By Graphical Method</u>: When high accuracy is required and many interpolations are to be made, this method of plotting contours proves to be the most rapid and convenient.

On tracing paper parallel lines are drawn at some fixed interval, say 0.5 m. Every tenth line is made thicker. Let A and B be two points of elevation at 50.50 m and 64.50 m, respectively. The tracing paper is placed with point A on the line 50.50 m and is turned till the point B is on line 64.50 m. The intersections of the line AB and the lines of the required elevations point will give the position of the point on the respective contour.

CURVATURE AND REFRACTION :-

Curvature Correction (C_c) : The effect of curvature is to cause the objects sighted, to appear lower than they really are.

The vertical distance between a horizontal line and the level line represents the effect of curvature of the earth



Here BC = Curvature correction

$\therefore$ BC = C_c is given by the formula $\boxed{\dfrac{D^2}{2R}}$

where D = Horizontal distance between level and Staff = AC

and R = Radius of earth = 6370 km.

$$\therefore C_c = \frac{D^2}{2R} = \frac{D^2}{2 \times 6370} = 7.849 \times 10^{-5} D^2 \text{ km}$$

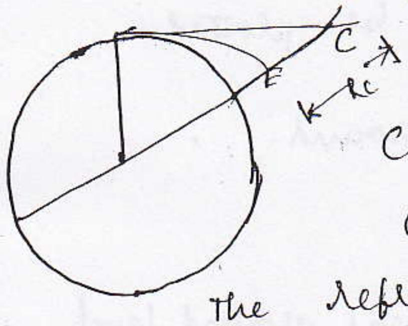
(or)

$$\boxed{C_c = 0.0785 D^2 m}$$

- Since the curvature increases the staff reading, the correction is therefore subtractive

$\therefore$ $\boxed{\text{True staff reading = Observed staff reading} - 0.0785D^2}$

Refraction Correction (R_c): The effect of refraction is to make the objects appear higher than they really are



$C_E = R_C$

CE is the amount of refraction correction.

The refraction correction can be taken as 1/7 th of curvature correction

$$\boxed{R_C = \frac{1}{7} \times C_C = 0.0112\ D^2\ m \quad (\text{or})\ 1.12 \times 10^{-5}\ D^2\ km}$$

The correction due to refraction is additive.

Combined Correction: Since, the effect of curvature and refraction when combined is to make the objects sighted appear low, the overall correction is subtractive.

$\therefore$ Combined Correction $= -0.0185D^2 + 0.0112\ D^2$

($-$ve sign is used for subtractive nature and $+$ ve for additive nature)

$= -0.0673\ D^2$ (subtractive) in metres

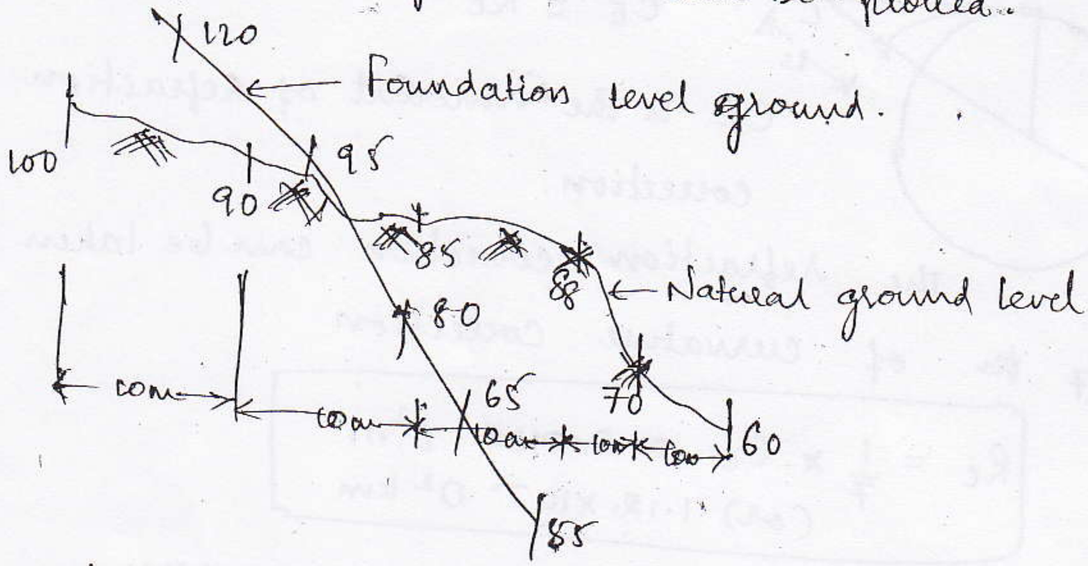
or $6.73 \times 10^{-5}\ D^2$ km

$\therefore$ $\boxed{\text{True Staff Reading} = \text{Observed Staff Reading} - 0.0673\ D^2}$

PROFILE LEVELLING (or) LONGITUDINAL SECTIONING (L.S):-

- It is the operation to determine the elevations of points spaced apart at known distances along a given line in order to obtain the accurate outline of the surface of the ground profile.

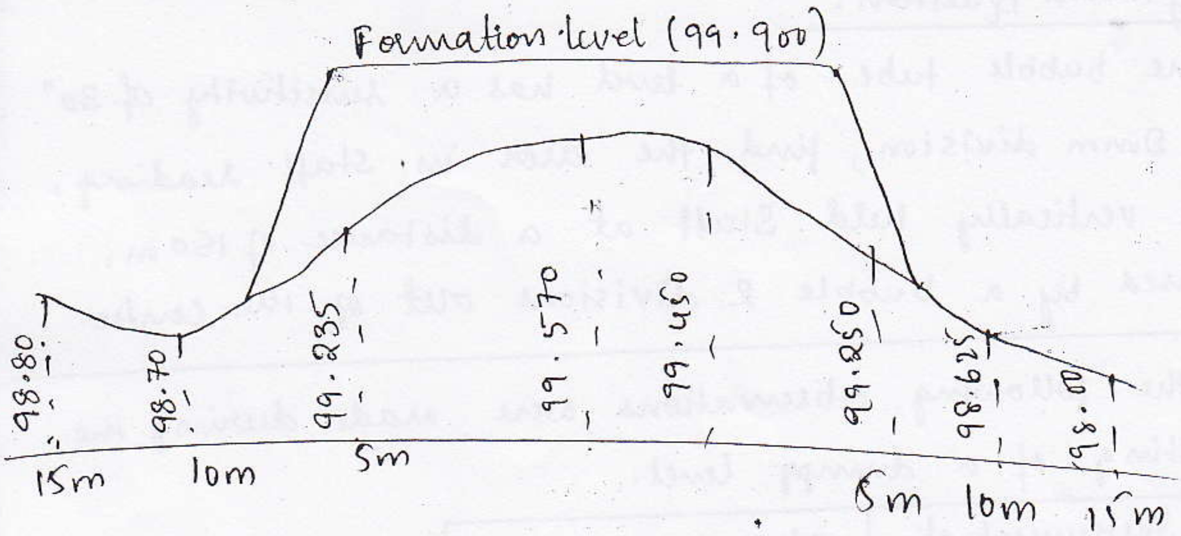
- The purpose of profile levelling is to provide data from which a vertical section of the ground surface along a surveyed line can be plotted.



The natural ground levels (R.L's). of the ground along the proposed road are calculated. The profiles (or) R.L's of the natural ground along with the formation levels of the ground are plotted as a graph where the vertical ordinates represent elevations and horizontal ordinates represent horizontal distance between the points.

CROSS - SECTIONING :-

It is the operation of levelling to determine elevation of the points at right angles (across) on either side of the centre of the proposed route. The detailed information regarding the levels of the ground on either side of the longitudinal section helps in computing the quantity of earth work. The cross - sections are plotted in the same manner as longitudinal - sections.



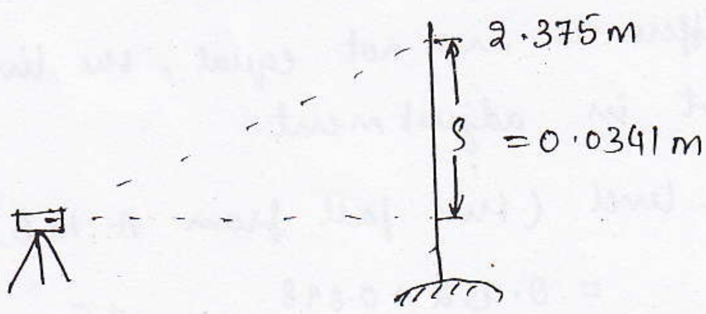
NUMERICALS

1) A level sight on a staff held vertical at distance of 88 m from the instrument reads 2.375 and the bubble is found to be two divisions off the centre of the towards the staff. If the level tube is in adjustment and has a sensitivity of 40°, what is the true reading on the staff? Take $\sin 1'' = \frac{1}{206,265}$ radians

Bubble Sensitivity, $\alpha' = l/R = \frac{S}{nD}$

$$\frac{40''}{206,265} = \frac{S}{2 \times 88}$$

Staff intercept, $S = 0.0341\text{ m}$



True Reading on Staff

$= 2.375 - S$

$= 2.375 - 0.0341$

$= 2.3409 \simeq 2.340\text{m}$ (approximately)

Assignment Question:

If the bubble tube of a level has a sensitivity of 30" per 2mm division, find the error in staff reading, on a vertically held staff at a distance of 150 m, caused by a bubble 2 divisions out of the centre.

→ ② The following observations were made during the testing of a dumpy level.

Instrument at	Staff readings on	
	A	B
A	1.792	2.244
B	2.146	3.044

Distance AB = 150 m. Once the instrument in adjustment? To what reading should the line of collimation be adjusted when the instrument was at B? If the R.L. of A = 432.052. What should be the R.L. of B?

Sol: Instrument at A:

Apparent difference of level $= 2.244 - 1.792$

$= 0.452$ m (A being higher)

Instrument at B:

Apparent difference of level $= 3.044 - 2.146$

$= 0.898$ m (A being higher)

Since the two differences are not equal, the line of collimation is not in adjustment.

True difference of level (true fall from A to B)

$$= \frac{0.452 + 0.898}{2} = 0.675 \text{ m}$$

R.L. of A = 432.052 m

R.L. of B = R.L. of A − true difference of level
(true fall from A to B)

Note : (−)ve Sign is used as there is fall from A to B

R.L. of B = 432.052 − 0.675
= 431.375 m

Collimation error when the instrument is at B.

correct reading on B = 3.044 m

correct reading on A will be

= 3.044 − 0.675
= 2.369 m

The observed reading on A (2.146 m) being less than the correct one (2.369 m), the line of collimation is inclined downwards

The amount of inclination (or) collimation error = 2.369 − 2.146 = 0.223 m

---

Assignment Question :

The following observations were taken during the testing of a dumpy level.

Instrument at	Staff reading at	
	A	B
A	1.275	2.005
B	1.040	1.660

Once the instrument in adjustment? To what readings should the line of collimation be adjusted when the instrument is at B?

→ (3) The following consecutive readings were taken with a dumpy level: 6.21, 4.92, 6.12, 8.42, 9.1, 6.63, 7.91, 8.26, 9.71, 10.21. The level was shifted after 4th, 6th and 9th readings. The R.L of first point was 125.00. Rule out a page of level field work and fill all the columns. Calculate the reduced levels and apply usual checks by both height of instrument method and Rise and fall method.

Sol

Height of Instrument Method:

Station	B.S.	F.S.	I.S.	H.I. (or) Collimation	R.L.	Remarks
1	6.21			131.210	125.00	B.M.
2			4.92		126.290	
3			6.12		125.090	
4	9.1	8.42		131.890	122.790	C.P.
5	7.91	6.63		133.170	125.260	C.P
6			8.26		124.910	
7	10.21	9.71		133.670	123.460	C.P.

$\Sigma B.S. = 23.22$; $\Sigma F.S. = 24.76$

Check : $\Sigma B.S. - \Sigma F.S. =$ Last R.L. − First R.L. $= -1.54\,m$

Rise and fall Method

Station	B.S.	I.S.	F.S.	Rise	Fall	R.L.	Remark
1	6.21					125.00	B.M.
2		4.92		1.290		126.290	–
3		6.12			1.200	125.090	–
4	9.1		8.42		2.300	122.790	C.P
5	7.91		6.63	2.470		125.260	C.P
6		8.26				124.910	
7	10.21		9.71			123.460	C.P.
	ΣB.S		ΣF.S.	ΣRise	ΣFall		

Check: Σ B.S. − Σ F.S. = Σ Rise − Σ fall = Last R.L. − First R.L = −1.54

(4) Complete the following level-book and find the R.L's of all stations including BM-A. Also apply the arithmetical check.

B.S.	I.S.	F.S.	Ht. of Collimation	R.L.	Remarks
3.145			514.835	511.690	BM-A
	2.725			512.110	
0.975		1.855	513.955	512.980	C.P.
1.365		2.450	512.870	511.505	C.P.
	0.475			512.395	
2.805		2.405	513.270	510.465	C.P.
3.065		1.685	514.650	511.585	C.P.
1.500		1.400	514.750	513.250	C.P.
		2.750		512.00	B.M-B.
ΣBS=12.855		ΣFS= 12.545			

Check :-

ΣB.S. − ΣF.S. = 12.855 − 12.545 = 0.310m

Last R.L. − First R.L. = 512.00 − 511.690 = 0.310m

$\therefore$ Σ B.S. − Σ F.S = Last R.L. − First R.L = 0.310m

Assignment Question:

he following Consecutive readings were taken with a level and 4.0m staff on a continuously sloping ground at a common interval of 30m: 0.780, 1.535, 1.955, 2.430, 2.985, 3.480, 1.155, 1.960, 2.365, 3.640, 0.935, 1.045, 1.630 and 2.545.

The reduced level of the first point A was 180.750 m. Rule out a page of a level field book and enter the bove readings. Calculate reduced levels of points by llimation system, and rise and fall system. Apply usual ecks.

(5) Fill up the missing entries. Also apply usual checks

Station	B.S.	I.S.	F.S.	Rise	Fall	R.L.	Remarks
1	3.125					124.18	BM
2	2.265		1.800	1.325		125.505	CP
3		2.320			0.055	125.450	
4		1.920		0.400		125.850	
5	1.040		2.655		0.735	125.115	CP
6	1.620		3.205		2.165	122.950	C.P
7		3.625			8.005	120.945	
8			1.480	2.145		123.090	

Σ B.S. = 8.05 ; Σ F.S = 9.14 ; Σ Rise = 3.87 ; Σ fall = 4.96

Check :

$\Sigma B.S. - \Sigma F.S. = 8.05 - 9.14 = -1.09$

$\Sigma Rise - \Sigma Fall = 3.87 - 4.96 = -1.09$

Last R.L − First R.L = 123.090 − 124.18 = −1.09

$\therefore$ $\Sigma B.S - \Sigma F.S = \Sigma Rise - \Sigma Fall$ = Last RL − First RL = −1.09

Assignment Question:-

Fill in the missing entries and apply usual checks

Station	B.S.	I.S.	F.S.	Rise	Fall	R.L.	Remarks
1	5.250					?	B.M
2	1.755		?		0.750	?	CP
3		1.950				?	
4	?		1.920			?	
5		2.340		1.500		?	
6		?		1.000		?	
7	1.850		2.185			250.00	CP
8		1.575				?	
9		?				?	
10	?		1.895		1.650	?	CP
11			1.350	0.750		?	Last Point

(6) A dumpy level was setup at L, exactly midway between A and B, 50 m apart. The readings on the staff when held on A and B were, respectively, 1.40 m & 2.40 m. The instrument was then shifted and setup at point L_2 on the line BA produced and 10 m from A. The readings on the staff held at A and B were, respectively 1.50 and 2.60. Determine the correct readings and the R.L. of B if that of A is 200.00

Sol True difference of elevation when the level is kept midway. = Difference in staff readings.

$= \underset{(B)}{2.40} - \underset{(A)}{1.40} = 1.00$ m (A being at higher elevation)

Apparent difference of elevation, when the level is kept at a distance of 10 m from A

= Difference in staff readings

$= \underset{(B)}{2.60} - \underset{(A)}{1.50} = 1.10$ m (A being at higher elevation)

Collimation error for a distance of 50 m between A & B

= Difference of true difference of elevation and apparent difference of elevation

$= 1.10 - 1.00 = 0.100$ m

Finding out whether the line of collimation is inclined upwards or downwards:

Assuming staff reading at A is correct, when the level is kept at 10 m from A, as the sight distance is small.

∴ Correct reading at A = 1.50 m

Correct reading at B = Correct reading at A + true difference of elevation

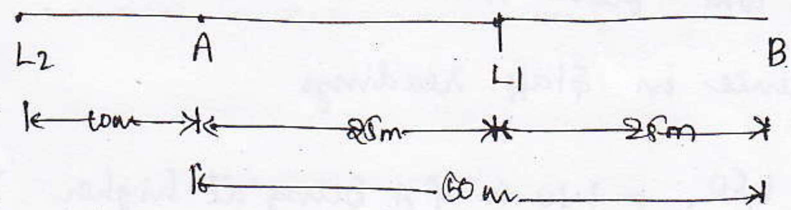
$$= 1.500 + 1.00$$

$$= 2.500 \text{ m}$$

(+) ve sign is used as A is at higher elevation and staff reading at B would be higher.

But observed reading at B (2.600 m) is more than the correct staff reading at B (2.500 m), therefore the line of collimation is inclined upwards

Finding the collimation errors at A and B, when instrument is at L_2:



At A from L_2:

collimation error for 50m = 0.100 m

collimation error for 10 m at A = ?

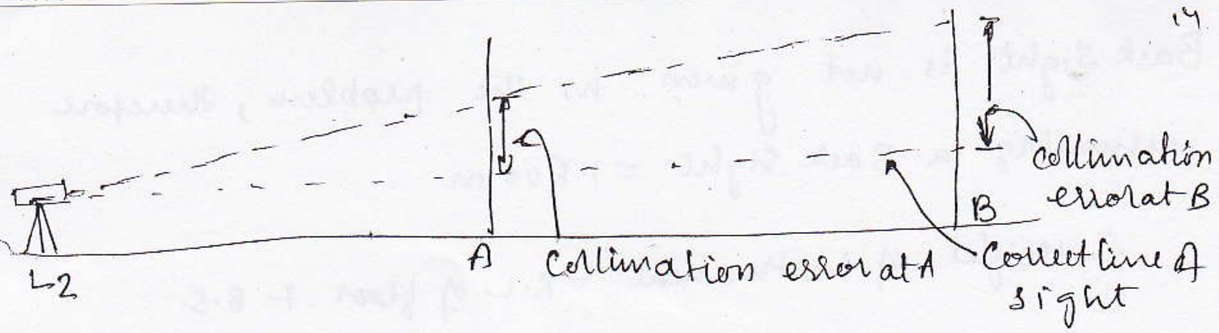
$$\frac{10 \times 0.100}{50} = 0.02 \text{ m}$$

At B from L_2:

collimation error for 50 m = 0.100 m

collimation error for 60m at B = ?

$$\frac{60}{50} \times 0.100 = 0.120 \text{ m}$$



Correct staff readings at A and B:

at A = Observed staff reading (at A from L_2) − Collimation error at A

= 1.500 − 0.020 = 1.480 m

at B = observed staff reading at B from L_2 − Collimation error at B

= 2.600 − 0.120 = 2.480 m

Note: Error will be deducted from observed staff readings, if line of collimation is inclined downwards and added to the observed staff reading when line of collimation is inclined downwards.

Check : 2.480 − 1.480 = 1.00 m = True difference of elevation.

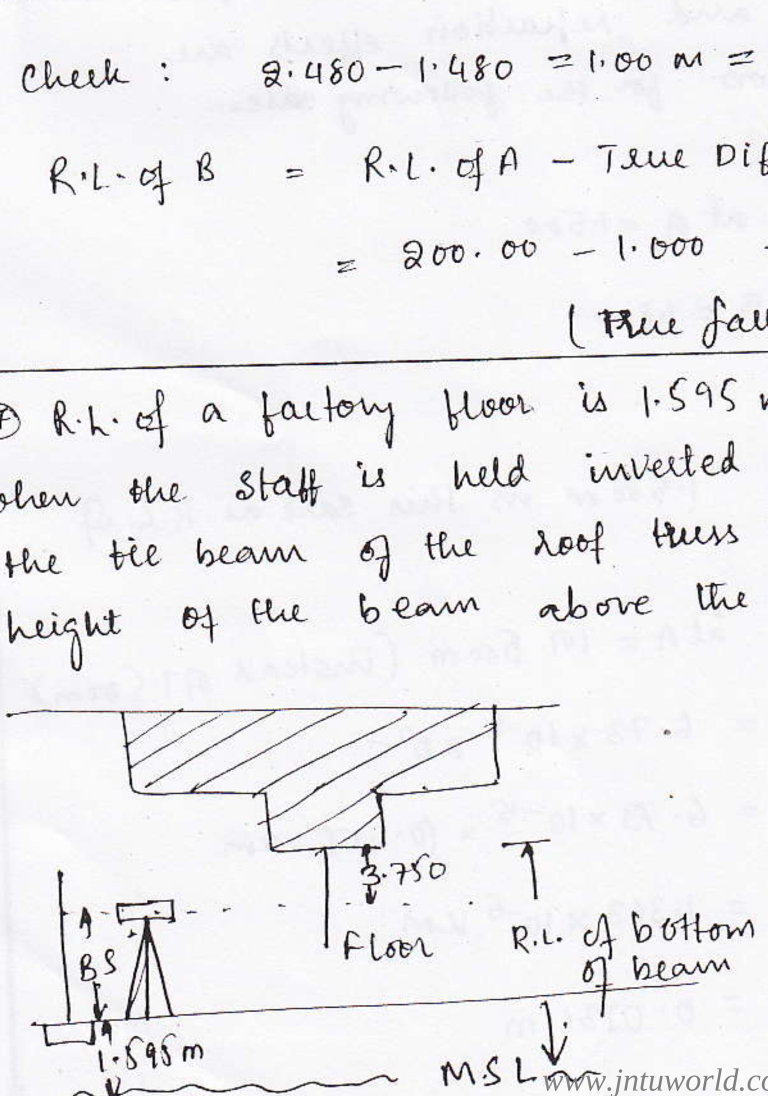
R.L. of B = R.L. of A − True Difference of elevation

= 200.00 − 1.000 = 199.00 m

(True fall from A to B)

⑦ R.L. of a factory floor is 1.595 m and staff reading when the staff is held inverted with bottom touching the tie beam of the roof truss is 3.750 m. Find the height of the beam above the floor.

[R.L. of any point cannot be usually 1.595 m, except where the point is very nearer to the sea]



Back Sight is not given in the problem, therefore assuming a Back Sight = 1.800 m

∴ Height of Instrument = R.L. of floor + B.S.

= 1.595 + 1.800 = 3.095 m

R.L. of bottom of beam (As per inverted staff concept)

= H.I. + Reading on inverted staff

= 3.095 + 3.750 = 6.845 m

Height of the bottom of the beam from the floor

= R.L. of bottom of beam − RL of floor

= 6.845 − 1.595 = 5.250 m

---

(8) Find out the difference in levels between points A and B if curvature and refraction effects are taken into consideration for the following case.

R.L. of A = 100.00

Height of Instrument at A = 1.500

Reading of staff at B = 1.800

Distance AB = 450 m

Sol H.I. at A cannot be 1.500 m in this case as R.L. of A is already 100.00 m.

Therefore assuming H.I. at A = 101.500 m (instead of 1.500 m)

∴ Combined correction $= 6.73 \times 10^{-5} \times D^2$

$= 6.73 \times 10^{-5} \times (0.45)^2$ km

$= 1.363 \times 10^{-5}$ KM

$= 0.0136$ m

Correct staff readings:

When the instrument at A, there is no error

$\therefore$ the reading at A would be correct staff reading.

$\therefore$ Correct staff reading at A = H.I. − R.L.

$= 101.500 - 100.00$

$= 1.500\ m$

Correct staff reading at B = Observed staff reading at B − Combined Correction

[$\because$ Combined correction is subtractive]

$\therefore = 1.800 - 0.0136 = 1.786\ m$

$\therefore$ True difference of levels between A and B

$= \underset{(B)}{1.786} - \underset{(A)}{1.500} = 0.286\ m$ (True fall from A to B)

---

(9) Reciprocal levelling between two points A and B, 630.5m apart, on opposite sides of a river gave the following results:

Instrument at	Height of Instrument	Staff at	Staff reading
A	1.360 m	B	1.585 m
B	1.335 m	A	0.890 m

Determine the difference in levels between A and B and amount of collimation error.

Note: If H.I. is given, R.L should also be given. Therefore in the problem, RL is not given, the H.I. column should be considered as staff reading at instrument itself.

Surya

Difference of elevation between A and B is given by $h = \frac{1}{2}\left[(b-a)+(c-d)\right]$

$$= \frac{1}{2}\left[(1.585-1.360)+(1.335-0.890)\right]$$

$= 0.335\,m$ (True fall from A to B)

Finding out total error :-

instrument at A :

Correct reading at A = 1.360 m

Correct reading at B = Observed reading at A + True difference of elevation

(+ve sign is used as A is at higher elevation and B is very high staff Reading)

$= 1.360 + 0.335 = 1.695\,m$

Total error :

Total error = Difference of obs. reading & correct reading at B

$= 1.695 - 1.585 = 0.110\,m$

Combined error due to curvature and Refractions

$= 6.73 \times 10^{-5} \times D^2$ km $= 6.73 \times 10^{-5} \times (0.6305)^2$

$= 2.6754 \times 10^{-5}$ km

$= 0.0267\,m$

Total error = combined error [curvature correction + Refractive correction] + Collimation error

$\therefore$ Collimation error = Total error − Combined error

Module-05

Unit:1 Areas And Volumes

Objective of land surveying:-

[Is to determine the area of track surveying and to determine the Quantities of Earth work]

General Methods of Determining areas:-

→ 1. By Computation based directly on field Measurement

a) By dividing area into a no of triangle, rectangles and trapizoides

b) By offset to base line

c) By lattitude and departures

d) By Co-ordinates.

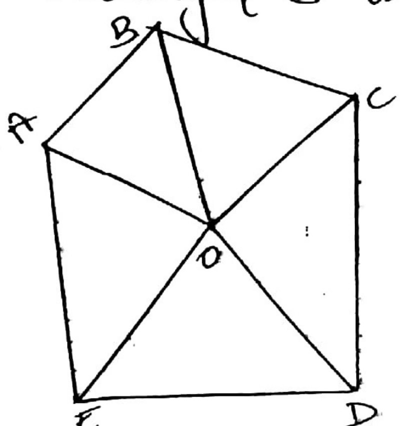
2. By Computation based on Measurement taken from the map

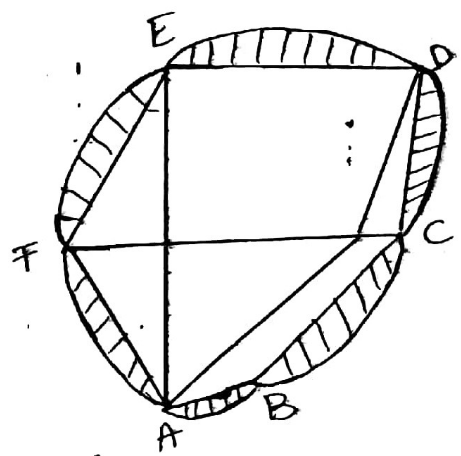
3. By Mechanical method

a) usually By means of planimeter

Computations based on Field Measurement

(I) By dividing the area into a no of triangles or rectangles or trapezoids





Triangle:-

If the base width b, and height is 'h' then

$$\text{area } A = \frac{1}{2} \times b \times h$$

when the lengths of 3 sides of Δ^{le} are measures its area is computed by the Equation

$$\text{Area } A = \sqrt{s(s-a)(s-b)(s-c)}$$

where, s = Half perimeter

$$= \frac{a+bc}{2}$$

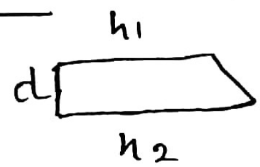
Rectangle:-

Area of rectangle $A = b \times d$

where, b & d are sides of rectangle

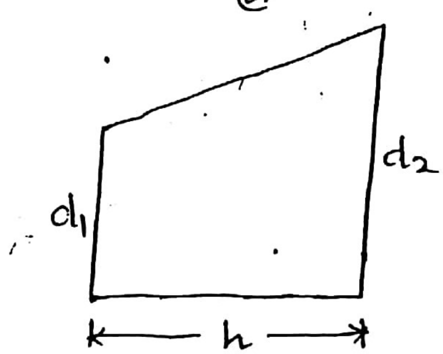
Trapezium

i)



$$\text{Area of Trapezium } A = \left(\frac{h_1+h_2}{2}\right)d$$

(or)



$$A = \left(\frac{d_1+d_2}{2}\right)h.$$

ii) where h_1 & h_2 are length of parallel sides

where 'd' is distance b/w h_1 & h_2

(II) Areas from offsets to a base line:-

Offsets at regular intervals :-

* This method is suitable for long narrow strip of land
* The offsets are measured from the boundary to the base line @ survey line at regular intervals

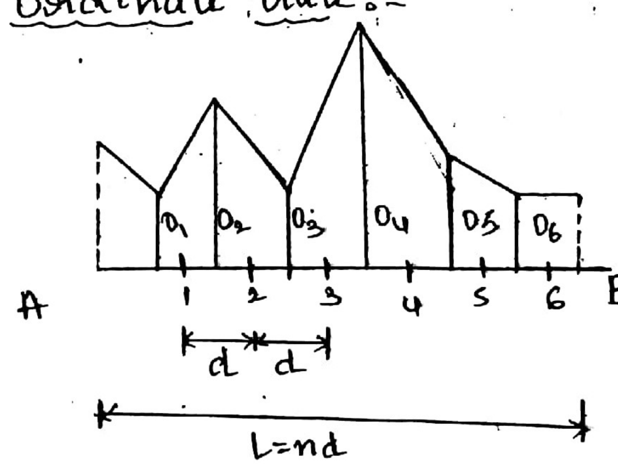
In this method major part of the area is divided into regular shapes and found @ calculated

Remaining area Enclosed b/w survey line Irregular boundry is calculated from drawing offset at regular intervals

The area may calculated by the following rules:-

1. Mid ordinates rule
2. Average ordinates rule
3. Trapezodial rule
4. Simphson's $\frac{1}{3}^{rd}$ rule @ prismoidal rule

1. Mid ordinate rule:-



This Method is used with the assumption that the boundaries b/w the Extremities of the offsets are straight lines

The base line is divided into no of divisions and ordinates are measured at the mid point of each division

The area is calculated by the formula

$$A = \text{Average ordinate} \times \text{length of base}$$

$$\boxed{A = \frac{O_1 + O_2 + \cdots + O_n}{n} \times L}$$

$$A \Rightarrow d\Sigma O$$

where; $O_1, O_2 \cdots$ = Ordinate of midpoint of each division

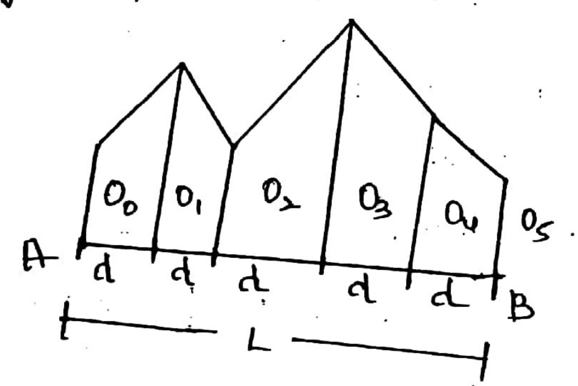
ΣO = Sum of mid ordinates

n = no of divisions

L = length of base line $= nd$

d = distance of each division $= \frac{L}{n}$

2. Average ordinate rule



Area, A = Average ordinate × length of the base

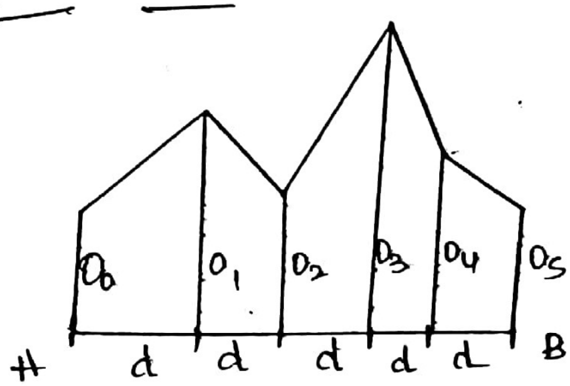
$$\Rightarrow \left[\frac{O_0 + O_1 + O_2 + \cdots O_n}{n+1}\right] \times L$$

$$\boxed{A = \frac{L}{n+1} \Sigma O}$$

where, O_0 = Ordinate at the one end of the base

O_n = Ordinate at other end of the base

3. Trapezoidal rule



This rule is based on assumption that the fig are trapezoides. The area of the first trapezoid is given by

$$A_1 = \frac{O_0 + O_1}{2} \times d$$

lly The area of the 2nd Trapezoid is given by

$$A_2 = \frac{O_1 + O_2}{2} d$$

Area of the n^{th} Trapezoid is given by

$$A_n = \frac{O_{n-1} + O_n}{2} \times d$$

Hence the total area of the fig is given by

$$A = A_1 + A_2 + \cdots + A_n$$

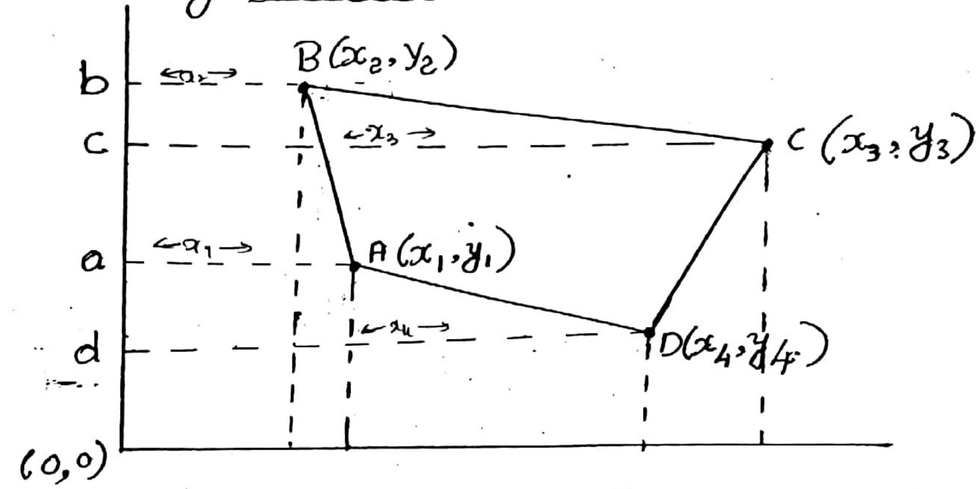
$$= \frac{O_0 + O_1}{2} d + \frac{O_1 + O_2}{2} d + \cdots + \frac{O_{n-1} + O_n}{2} \times d$$

$$\boxed{A = \left[\frac{O_0 + O_n}{2} + O_1 + O_2 + \cdots + O_{n-1}\right] d} \rightarrow (1)$$

Eqn (1) gives trapezoidal rule, which may be expressed as "Add the average of the end offsets to the sum of intermediate offsets multiply the total sum by common distance b/w the ordinates to get the required area

Imp Problem @ Derivation on this

Derivation of Area by Co-ordinates method



$(x_1, y_1), (x_2, y_2), (x_3, y_3)$ & $(x_4, y_4) \rightarrow$ Co-ordinates of Points A, B, C & D

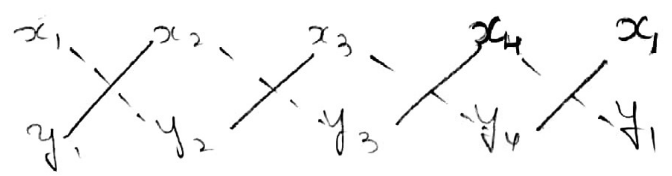
Area of (bBCc) + Area of (cCDd) − Area of (bBAa) − Area of (aADd)

$$A = \left\{ \left(\frac{x_2+x_3}{2}\right) \times (y_2 - y_3) + \left(\frac{x_3+x_4}{2}\right) \times (y_3 - y_4) - \left(\frac{x_2+x_1}{2}\right)(y_2 - y_1) - \left(\frac{x_1+x_4}{2}\right) \times (y_1 - y_4) \right\}$$

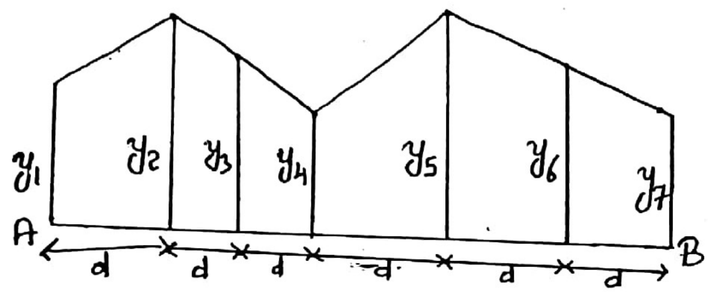
For Problem Note:-

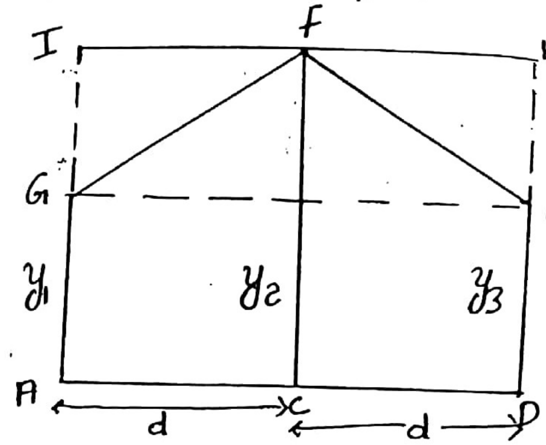
$A = \frac{1}{2}$ {(Σ Pair of Co-ordinates joined by continuous line) − (Σ Pair of co-ordinates joined by dotted lines)}

$$A = \frac{1}{2}\left\{ \left[(y_1 x_2) + (y_2 x_3) + (y_3 x_4) + (y_4 x_1)\right] - \left[(y_2 x_1) + (y_3 x_2) + (y_4 x_3) + (y_1 x_4)\right] \right\}$$



Imp // Simpson 1/3rd Rule





Area enclosed between ADEFG = [Area of the Trapezium ADEG
+
2/3 [Area of the Trapezium IGEH

$$A_1 = \frac{(y_1+y_3)}{\cancel{2}} \times \cancel{2}d + \frac{2}{3}\left[\frac{(y_2-y_1)+(y_2-y_3)}{\cancel{2}}\right] \times 2d$$

$$A_1 = \frac{[3y_1+3y_3+2y_2-2y_1+2y_2-2y_3] \times d}{3}$$

$$A_1 = \left[\frac{y_1+4y_2+y_3}{3}\right] d$$

|||ly
$$A_2 = \left[\frac{y_3+4y_4+y_5}{3}\right] d$$

$$A_3 = \left[\frac{y_5+4y_6+y_7}{3}\right] d$$

$$\frac{[3y_1+3y_3+2y_2-2y_1+2y_2-2y_3]d}{\cancel{3}}$$

$$\left[\frac{y_1+y_3+4y_2}{3}\right] d$$

$$\therefore A = \frac{d}{3}\left\{(y_1+y_n) + 4(\overbrace{y_2+y_4+\cdots+y_{n-1}}^{\text{Even no}}) + 2(\overbrace{y_3+y_5+\cdots+y_{n-2}}^{\text{odd no}})\right\}$$

$$A = \frac{\text{Length of each division}}{3}\left\{(1^{st}\text{ offset} + \text{last offset}) + 4(\text{Even offset}) + 2(\text{odd offset})\right\}$$

Note:- Area of FGE = 2/3 [Area of Trapezium IGEH]

Calculation of volumes

-> Trapezoidal formula for volume calculation

$$V = \left[\frac{(A_1 + A_n)}{2} + A_2 + A_3 + A_4 + \cdots + A_{n-1}\right] \times d$$

-> Prismoidal formula (Simpson's $1/3^{rd}$ Rule) for volume calculation.

$$V = \frac{d}{3}\left[(A_1 + A_n) + 4(A_2 + A_4 + \cdots + A_{n-1}) + 2(A_3 + A_5 + \cdots A_{n-2})\right]$$

An embankment is 30m wide at the top with the slope of 2:1. The ground levels at 100m intervals along a line AB are as under.

A→ 170.30, 169.10, 168.50, 168.10, 166.50 – B

The formation level at A is 178.70m with uniform falling ground of 1 in 50 from A to B. Determine the volume of earth work by prismoidal formula. Assume the ground to be cross-section.

Ans

formation width(b) = 30 (Top width)

Side Slope (n) = $H/V = 2/1 = 2 \Rightarrow n$

NOTE

$$A = \frac{n(b/2)^2 + r^2((b \times y) + ny^2)}{r^2 - n^2}$$

Distance	RL	formation Level	Height	Area
0 (A)	170.30	178.70	8.4	393.39
100	169.10	176.70	7.6	344.25
200	168.50	174.7	6.2	263.48
300	168.10	172.7	4.6	180.78
400(B)	166.50	170.7	4.2	161.71

formation level at zero chainage = 178.70

uniform falling gradient → $1/50$

for 50m (Horizontal Distance) ⟶ 1m vertical fall

for 100m horizontal Distance ⟶ ? $\frac{100 \times 1}{50} = 2m$

∴ formation level at 100m chainage ⇒ formation level at 0 chainage – 2m = 178.70 – 2 = 176.70m

formation level at 200m chainage = 176.7 – 2 = 174.7m.

$$A_1 = \frac{n(b/2)^2 + r^2[(b \times y_1) + ny_1^2]}{r^2 - n^2} = \frac{2(30/2)^2 + (50)^2[(30 \times 8.4) + 2(8.4)^2]}{(50)^2 - (2)^2}$$

$A_2 = 344.25m^2$ $A_3 = 263.48m^2$ $A_4 = 180.78m^2$ $A_5 = 161.71m^2$

$$V = d/3\{(A_1 + A_5) + 4(A_2 + A_4) + 2A_3\} = 21130.41m^3$$

Problems on contours

* Area enclosed by Contours in a water pond are as follows

Contour (m)	270	275	280	285	290
Area (m^2)	2050	8400	16300	24600	31500

Calculate volume of contours b/w water level 270m & 290m by Trapezoidal & Prismoidal rule

Ans Bottom most part of water pond = 270m
Top most part of water pond = 290m

Trapezoidal rule

$$V = \left[\left(\frac{A_1 + A_n}{2}\right) + A_2 + A_3 + A_4\right] d$$

d = Contour interval = 5m

$$V = \left[\left(\frac{2050 + 31500}{2}\right) + 8400 + 16300 + 24600\right] = 330.375 \times 10^3 m^3$$

Simpson's 1/3rd rule

$$V = \frac{d}{3}\left[(A_1 + A_5) + 4(A_2 + A_4) + 2A_3)\right]$$

$$V = \frac{5}{3}\left[(2050 + 31500) + 4(8400 + 24600) + 2(16300)\right]$$

$$\boxed{V = 330.25 \times 10^3 m^3}$$

* Find the capacity of reservoir from the Contour data given in the table. Plan is drawn to a scale of 1:4000.

Contour (m)	260	258	256	254	252	250	248	246	244	242	240
Area (m^2)	400	367.5	327.5	310	277.5	243.75	205	177.5	147.5	115	0

Scale :

For 4000 cm in the field → 1cm in drawing sheet

$4000 cm^2 \rightarrow 1 cm^2$

Countours (m)	260	258	256	254	252	250	248	246	244	242	240
Area $(m^2) \times 10^3$	640	588	524	496	444	390	328	284	236	184	0

~~400~~

$1 cm^2 \rightarrow (4000)^2 cm^2$

$400 cm^2 \rightarrow ?$

$\frac{400 \times (4000)^2}{1} = 6.4 \times 10^9 cm^2$

$= 6.4 \times 10^5 m^2$

$= 640 \times 10^3 m^2$

lly all calulation areas are put up in table

Simpson's 1/3rd Rule →

$$V = \frac{d}{3}\left[(A_1 + A_n) + 4(A_2 + A_4 + A_6 + A_8 + A_{10}) + 2(A_3 + A_5 + A_7 + A_9)\right]$$

$\cancel{= \frac{640+0}{2}}$

$$= \frac{2}{3} \times 10^3 \left[(640+0) + 4(588 + 496 + 390 + 284 + 184) + 2(524 + 444 + 328 + 236)\right]$$

$V = \underline{7648000\, m^3}$

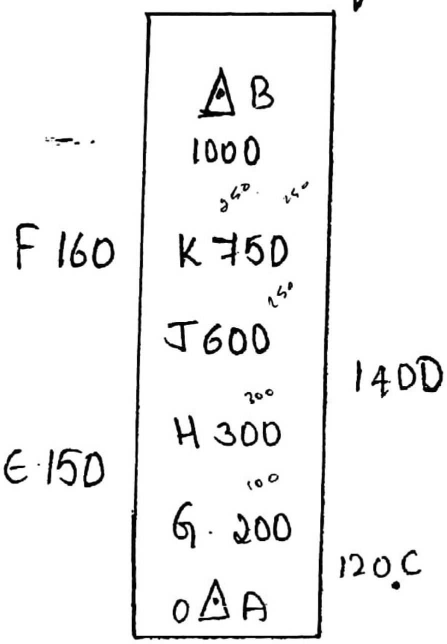
Trapizoidal Rule :-

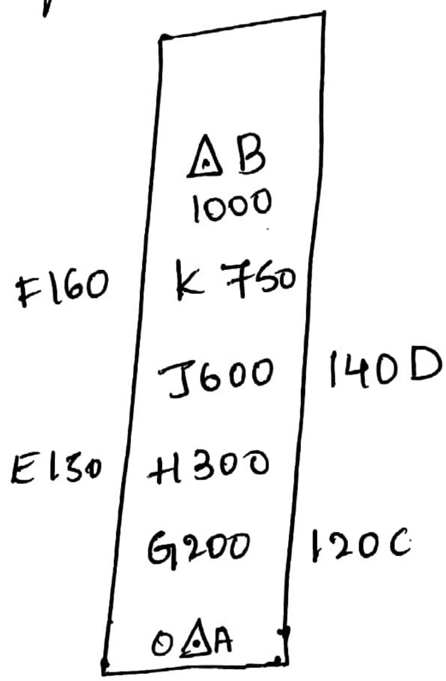
$$V = \left[\left(\frac{A_1 + A_n}{2}\right) + A_2 + A_3 + A_4 + A_5 + A_6 + A_7 + A_8 + A_9 + A_{10}\right] d$$

$$= \left[\left(\frac{640+0}{2}\right) + 588 + 524 + 496 + 444 + 390 + 328 + 284 + 236 + 184\right] \times 2 \times 10^3$$

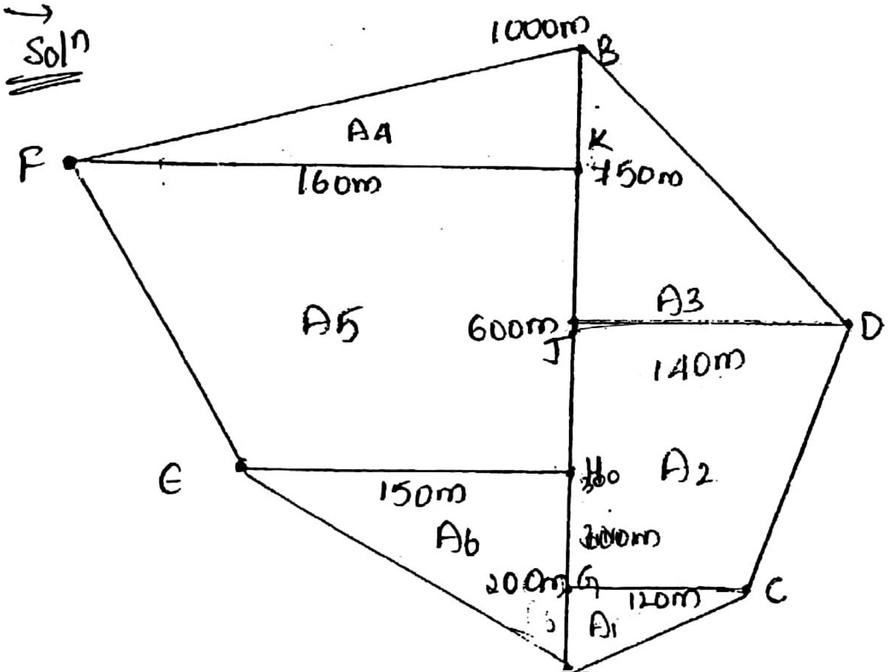
$V = \underline{7588000\, m^3}$

1. Plot the cross shaft survey of a field ABCDFE form the field book measurements given in the fig below and determine the area of the field.





Sol^n



Area of the field, $A = A_1 + A_2 + A_3 + A_4 + A_5 + A_6$

$A_1 = \frac{1}{2} \times \text{base} \times \text{height}$

$= \frac{1}{2} \times GO \times GC$

$= \frac{1}{2} \times 200 \times 120$

$= 12000 \text{ m}^2$

$$A_2 = \left(\frac{JD + GC}{2}\right) \times GJ$$

$$= \left(\frac{140 + 120}{2}\right) \times 400 = \underline{\underline{52000\ m^2}}$$ (13)

$$A_3 = \frac{1}{2} \times JD \times JB = \frac{1}{2} \times 140 \times 400 = \underline{\underline{28000\ m^2}}$$

$600 - 750 = 150$
$750 - 100 = 250$
$400 \rightarrow A_2 \rightarrow JB$

$$A_4 = \frac{1}{2} \times KB \times KF = \frac{1}{2} \times 250 \times 160 = \underline{\underline{20,000\ m^2}}$$

$$A_5 = \left[\frac{EH + FK}{2}\right] \times KH = \left(\frac{150 + 160}{2}\right) \times 450 = \underline{\underline{69750\ m^2}}$$

$750 - 600 = 150$
$600 - 300 = 300$
$450 \rightarrow KH \rightarrow A_5$

$$A_6 = \frac{1}{2} \times EH \times HO = \frac{1}{2} \times 150 \times 300 = \underline{\underline{22,500\ m^2}}$$

$$\therefore A = \underline{\underline{20,4250\ m^2}}$$

Contouring

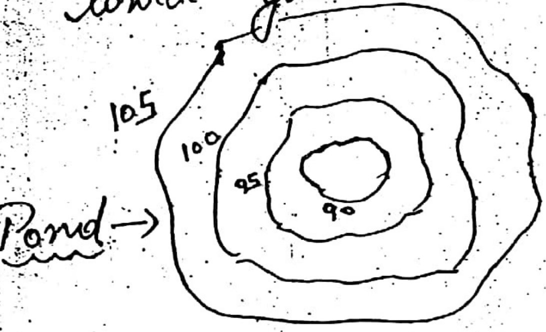
Contour: Contour line is a imaginary line which connects points of equal elevation.

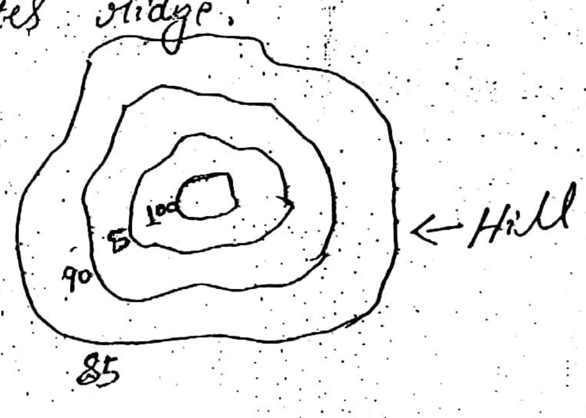
Contour Intervals: The vertical distance b/w the consecutive contour lines is called contour intervals. It depends on
i) Scale of Map ii) Purpose of Map iii) Nature of Country iv) Time & funds.

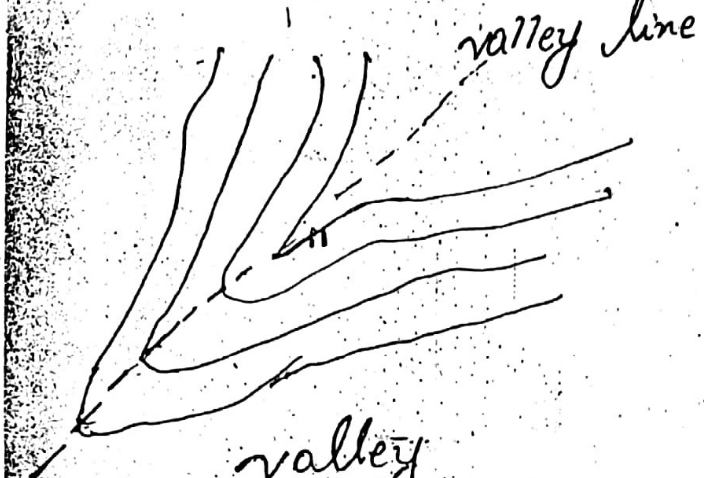
Horizontal Equivalent: The horizontal distance b/w 2 points or b/w any 2 consecutive contours is called horizontal equivalent.

Characteristics of Contour: (Imp)

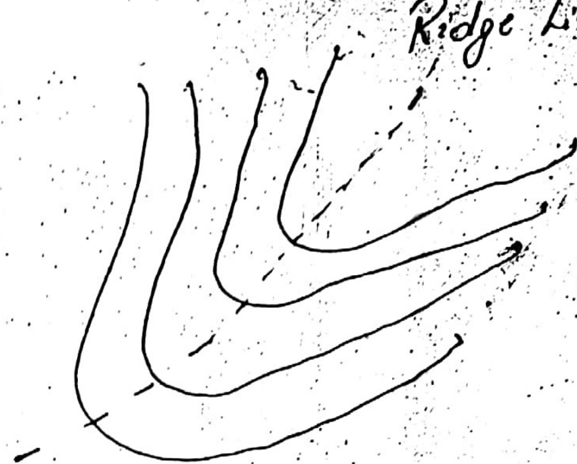
1. Contour lines must close, not necessarily in limits of plan.
2. Widely spaced contours indicates flat surface
3. Closely spaced contours indicates steep ground.
4. Equally spaced contours indicates uniform slope.
5. Irregular contour indicates uneven surface.
6. Concentric closed contours with decreasing values towards the center indicates pond.
7. Concentric closed contours with increasing value towards center indicats hill.
8. Contour lines donot meet or intersect each other.
9. Contour lines with V-shape with convexity towards higher ground indicates valley.
10. Contour lines with V-shape with convexity towards lower ground indicates ridge.







valley



Ridge

Uses of Contour (Imp)

i) A Contour map is useful to Study the possible location of a dam & the volume of water to be Contained.

ii) The most economical & Suitable Site for Structures such as building, bridges, dams etc Can be found from large Scale Contour maps.

iii) The extent of a drainage area (catchment area) Can be be estimated on a Contour map.

iv) A Contour map is useful to Study the profile along any line which is normally required for earthwork Calculation

v) A Contour map is used to determine the intervisibility of the triangulation Station.

vi) Contour map is used to locate Routes.

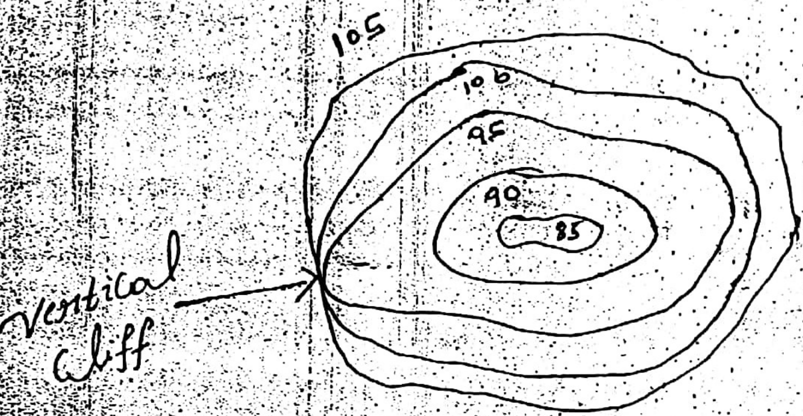
Grade Contour: The lines having equal gradient along a slope are Called grade Contours.
The difference in elevation of two points of grade Contours divided by the distance b/w them is always a Constant.

Contour gradient:- Contour gradient is a line laying throughout on the Surface of the ground & preserving a Constant inclination to the horizontal.

3) (16)

Vertical Cliff

These are the steep rock faces along the sea coast & may be vertical where the contour lines coincide with each other



Interpolation of Contours

Interpolation of contour is the process of spacing the contours proportionately between the plotted ground points established by indirect methods. The methods of interpolation are based on the assumption that the slope of ground b/w the two points is uniform.

The ~~three~~ chief method of interpolation are

i) Estimation ii) Arithmetic Calculation

iii) Graphical method

Methods of Contouring

Contouring consists of finding elevations of various points in the area surveyed.

This is done by

1) Direct method

2) Indirect method
- i) Method of Squares
- ii) Cross-Section method
- iii) Radial line method

34
40 } 50%

49

(1478)

A-144 2nd Lft 55

NOTE :

The Combined Correction due to curvature and refraction is

$$-\frac{6}{7}\frac{d^2}{2R}$$

d is in km
R = 6370 km

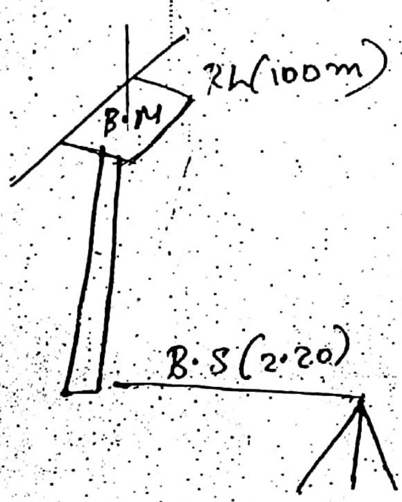
Levelling (Remaining notes)

Note :

If Chejja is taken as B.M, then inverted Staff reading is taken.

To calculate the H.I = R.L of B.M (chejja) − B.S

eg



H.I = 100 − 2.20 ⇒ 97.80m

* Find the correction for curvature & for refraction for a distance of

(a) 1200 metres

(b) 2.48 km

(a) $C.C = 0.07849\, d^2$ (m)
(km)

= 0.113m

(b) $C.R = \frac{1}{7} C_c = 0.016$ m

Module 5

$$\Delta_1 = \frac{O_0 + O_1}{2} d$$

Similarly, the area of the second trapezoid is given by $\Delta_2 = \frac{O_1 + O_2}{2} d$

Area of the last trapezoid (nth) is given by

$$\Delta_n = \frac{O_{n-1} + O_n}{2} d$$

Hence the total area of the figure is given by

$$\Delta = \Delta_1 + \Delta_2 + \ldots\ldots \Delta_n = \frac{O_0 + O_1}{2} d + \frac{O_1 + O_2}{2} d + \ldots\ldots + \frac{O_{n-1} + O_n}{2} d$$

or

$$\Delta = \left(\frac{O_0 + O_n}{2} + O_1 + O_2 + \ldots\ldots + O_{n-1}\right) d \qquad \ldots(12.5)$$

Equation (12.5) gives the trapezoidal rule which may be expressed as below:

Add the average of the end offsets to the sum of the intermediate offsets. Multiply the total sum thus obtained by the common distance between the ordinates to get the required area.

4. Simpson's One-Third Rule

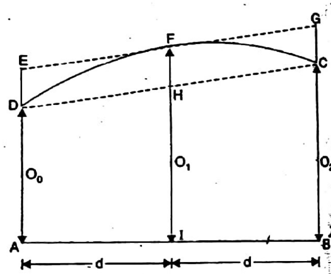


Fig. 12.4

This rule assumes that the short lengths of boundary between the ordinates are parabolic arcs. This method is more useful when the boundary line departs considerably from the straight line.

Thus, in Fig. 12.4, the area between the line *AB* and the curve *DFC* may be considered to be equal to the area of the trapezoid *ABCD* plus the area of the segment between the parabolic arc *DFC* and the corresponding chord *DC*.

Let O_0, O_1, O_2 = any three consecutive ordinates taken at regular interval of *d*.

Through *F*, draw a line *EG* parallel to the chord *DG* to cut the ordinates in *E* and *G*.

$$\text{Area of trapezoid} \quad ABCD = \frac{O_0 + O_2}{2} \cdot 2d \qquad \ldots(1)$$

To calculate the area of the segment of the curve, we will utilize the property of the parabola that area of a segment (such as *DFC*) is equal to two-third the area of the enclosing parallelogram (such as *CDEG*).

$$\text{Thus, area of segment} \quad DFC = \frac{2}{3} (FH \times AB) = \frac{2}{3}\left\{\left(O_1 - \frac{O_0 + O_2}{2}\right) 2d\right\} \qquad \ldots(2)$$

Adding (1) and (2), we get the required area ($\Delta_{1,2}$) of first two intervals. Thus,

$$\Delta_{1,2} = \frac{O_0 + O_2}{2} . 2d + \frac{2}{3}\left\{\left(O_1 - \frac{O_0 + O_2}{2}\right) 2d\right\} = \frac{d}{3}(O_0 + 4O_1 + O_2) \ldots(3)$$

Prakruthi. N

Similarly, the area of next two intervals ($\Delta_{3,4}$) is given by

$$\Delta_{3,4} = \frac{d}{3}(O_2 + 4O_3 + O_4) \qquad \ldots(4)$$

Area of the last two intervals (Δ_{n-1}, Δ_n) is given by

$$O_{n-1,n} = \frac{d}{3}(O_{n-2} + 4O_{n-1} + O_n) \qquad \ldots(5)$$

Adding all these to get the total area (Δ), we get

$$\Delta = \frac{d}{3}[O_0 + 4O_1 + 2O_2 + 4O_3 \ldots\ldots + 2O_{n-2} + 4O_{n-1} + O_n]$$

or

$$\Delta = \frac{d}{3}[(O_0 + O_n) + 4(O_1 + O_3 + \ldots\ldots + O_{n-1}) + 2(O_2 + O_4 + \ldots\ldots O_{n-2})] \quad \ldots(12.6)$$

It is clear that the rule is applicable only when the number of divisions of the area is *even i.e.,* the total number of ordinates is *odd*. If there is an odd number of divisions (resulting in even number of ordinates), the area of the last division must be calculated separately, and added to equation 12.6.

Simpson's one-third rule may be stated as follows: ***The area is equal to the sum of the two end ordinates plus four times the sum of the even intermediate ordinates + twice the sum of the odd intermediate ordinates, the whole multiplied by one-third the common interval between them.***

Comparison of Rules: The results obtained by the use of Simpson's rule are in all cases the more accurate. The results obtained by using Simpson's rule are greater or smaller than those obtained by using the trapezoidal rule according as the curve of the boundary is concave or convex towards the base line. In dealing with irregularly shaped figures, the degree of precision of either method can be increased by increasing the number of ordinates.

Example 12.1: *The following perpendicular offsets were taken at 10 metres intervals from a survey line to an irregular boundary line:*

1 2 3 4 5 6 7 8 9 $\quad n-1 = 9-1 = 8$

3.25, 5.60, 4.20, 6.65, 8.75, 6.20, 3.25, 4.20, 5.65

O_0 O_1 O_2 O_3 O_4 O_5 O_6 O_7 O_8

Calculate the area enclosed between the survey line, the irregular boundary line, and the first and last offsets, by the application of (a) average ordinate rule, (b) trapezoidal rule, and (c) Simpson's rule.

Solution:

$O_0 + O_1$

(a) *By average ordinate rule*

From equation 12.4 (a), we have $\Delta = \frac{L}{n+1}\Sigma O$

Here n = number of divisions = 8; $n + 1$ = number of ordinates = 8 + 1 = 9

L = Length of base = 10 × 8 = 80 m

ΣO = 3.25 + 5.60 + 4.20 + 6.65 + 8.75 + 6.20 + 3.25 + 4.20 + 5.65 = 47.75 m

$\therefore \quad \Delta = \frac{80}{9} \times 47.75$ = 424.44 sq. metres = **4.2444 ares.**

$L = n \times d$

(b) *By trapezoidal rule*

From Eq. 12.5, $\Delta = \left(\frac{O_0 + O_n}{2} + O_1 + O_2 + ... + O_{n-1}\right)d$

Here $d = 10$ m; $\frac{O_0 + O_n}{2} = \frac{3.25 + 5.65}{2} = 4.45$ m

$O_1 + O_2 ...... O_{n-1} = 5.60 + 4.20 + 6.65 + 8.75 + 6.20 + 3.25 + 4.20 = 38.85$ m

$\Delta = (4.45 + 38.85)\ 10 = \mathbf{433}$ sq. metres = **4.33 ares**

(c) *By Simpson's rule*

From Eq. 12.6, $\Delta = \frac{d}{3}\ [(O_0 + O_n) + 4(O_1 + O_3 + ... + O_{n-1}) + 2(O_2 + O_4 ...... + O_{n-2})]$

Here $d = 10$ m; $O_0 + O_n = 3.25 + 5.65 = 8.9$ m

$4\ (O_1 + O_3 + ... O_{n-1}) = 4\ (5.60 + 6.65 + 6.20 + 4.20) = 90.60$

$2\ (O_2 + O_4 + ... O_{n-2}) = 2\ (4.20 + 8.75 + 3.25) = 32.40$

$\therefore$ $\Delta = \frac{10}{3}\ (8.9 + 90.60 + 32.40) = \mathbf{439.67}$ sq. metres = **4.3967** ares.

Example 12.2: *A series of offsets were taken from a chain line to a curved boundary line at intervals of 15 metres in the following order.*

0, 2.65, 3.80, 3.75, 4.65, 3.60, 4.95, 5.85 m.

Compute the area between the chain line, the curved boundary and the end offsets by (a) average ordinate rule, (b) trapezoidal rule, and (c) Simpson's rule.

Solution:

(a) *By average ordinate rule*

From Eq. 12.4 (a), we have $\Delta = \frac{L}{n+1}\Sigma O$

Hence $n = 7$; $n + 1 = 8$

$L = nd = 7 \times 15 = 105$ m

$\Sigma O = 0 + 2.65 + 3.80 + 3.75 + 4.65 + 3.60 + 4.95 + 5.85 = 29.25$ m

$\therefore$ $\Delta = \frac{105}{8} \times 29.25 = \mathbf{383.91}$ sq. m = 3.8391 ares.

(b) *By trapezoidal rule*

From equation 12.5 $\Delta = \left(\frac{O_0 + O_n}{2} + O_1 + O_2 + ... O_{n-1}\right)d$

Here $d = 15$ m; $\frac{O_0 + O_n}{2} = \frac{0 + 5.85}{2} = 2.925$ m

$O_1 + O_2 + ... O_{n-1} = 2.65 + 3.80 + 3.75 + 4.65 + 3.60 + 4.95 = 23.40$

$\therefore$ $\Delta = (2.925 + 23.40)\ 15 = \mathbf{394.87}$ sq. m = 3.9487 ares.

(c) *By Simpson's rule*

From equation 12.6, $\Delta = \frac{d}{3}\ [(O_0 + O_n) + 4\ (O_1 + O_3 + ... O_{n-1}) + 2(O_2 + O_4 + ... O_{n-2})]$

Here, $\frac{d}{3} = \frac{15}{3} = 5$ m.

It will be seen that the Simpson's rule is not directly applicable here since the number of ordinates (n) is even. However, the area between the first and seventh offsets may be calculated by Simpson's rule, and the area enclosed between the seventh and last offsets may be found by the trapezoidal rule.

Thus, $(O_0 + O_n) = 0 + 4.95 = 4.95$

$4(O_1 + O_3 + ...O_{n-1}) = 4(2.65 + 3.75 + 3.60) = 40$

$2(O_2 + O_4 + ...O_{n-2}) = 2(3.80 + 4.65) = 16.90$

$\therefore$ $\Delta' = 5(4.95 + 40 + 16.90) = 309.25$ sq. m.

Area of the last trapezoid $= (4.95 + 5.85)\ \frac{15}{2} = 81.0$ sq. m.

$\therefore$ Total area $= 309.25 + 81.0 = \mathbf{390.25}$ sq. m = 3.9025 ares.

12.5 OFFSETS AT IRREGULAR INTERVALS

(a) *First Method* (Fig. 12.5): In this method, the area of each trapezoid is calculated separately and then added together to calculate the total area. Thus, from Fig. 12.5,

$$\Delta = \frac{d_1}{2}\ (O_1 + O_2) + \frac{d_2}{2}\ (O_2 + O_3) + \frac{d_3}{2}\ (O_3 + O_4) \quad ...(12.7)$$

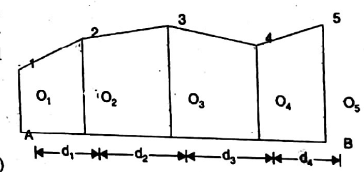


Fig. 12.5

(b) *Second Method:* By method of co-ordinates: See § 12.7

Example 12.3: *The following perpendicular offsets were taken from a chain line to an irregular boundary:*

Chainage	0	10	25	42	60	75 m
Offset	15,5	26.2	31.8	25.6	29.0	31.5

Calculate the area between the chain line, the boundary and the end offsets.

Solution:

Area of first trapezoid $= \Delta_1 = \frac{10 - 0}{2}\ (15.5 + 26.2) = 208.5$ m²

Area of second trapezoid $= \Delta_2 = \frac{25 - 10}{2}\ (26.2 + 31.8) = 435$ m²

Area of third trapezoid $= \Delta_3 = \frac{42 - 25}{2}\ (31.8 + 25.6) = 487.9$ m²

Area of fourth trapezoid $= \Delta_4 = \frac{60 - 42}{2}\ (25.6 + 29.0) = 491.4$ m²

Area of fifth trapezoid $= \Delta_5 = \frac{75 - 60}{2}\ (29.0 + 31.5) = 453.7$ m²

$\therefore$ Total area $= \Delta = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 + \Delta_5$

$= 208.5 + 435 + 487.9 + 491.4 + 453.7$

$= \mathbf{2076.5\ m^2 = 20.765\ ares.}$

Example 12.4: *The following perpendicular offsets were taken from a chain line to a hedge:*

Chainage (m)	0	15	30	45	60	70	80	100	120	140
Offsets (m)	7.60	8.5	10.7	12.8	10.6	9.5	8.3	7.9	6.4	4.4

Calculate the area between the survey line, the hedge and the end offsets by (a) Trapezoidal rule (b) Simpson's rule.

Solution:

(a) **By Trapezoidal rule:** The interval is constant from first offset to 5th offset. There is another interval between the 5th and 7th offset and a third interval between 7th offset and 10th offset. The total area Δ can, therefore, be divided into three sections.

$\therefore \qquad \Delta = \Delta_1 + \Delta_2 + \Delta_3$

where Δ_1 = area of first section; Δ_2 = area of second section

Δ_3 = area of third section; d_1 = interval for first section = 15 m

d_2 = interval for second section = 10 m; d_3 = interval for third section = 20 m

Now 0

n+1

$$\Delta_1 = \left(\frac{7.60+10.6}{2}+8.5+10.7+12.8\right)15 = 616.3\ \text{m}^2$$

$$\Delta_2 = \left(\frac{10.6+8.3}{2}+9.5\right)10 = 189.5\ \text{m}^2$$

$$\Delta_3 = \left(\frac{8.3+4.4}{2}+7.9+6.4\right)20 = 413\ \text{m}^2$$

$$\Delta = 616.5 + 189.5 + 413 = 1219\ \text{m}^2 = 12.19\ \text{ares.}$$

(b) **By Simpson's Rule:** The first section and the second section have odd number of ordinates, and therefore, Simpson's rule is directly applicable. The third section has 4 ordinates (even number); the rule is applicable for the first three ordinates only:

$$\Delta_1 = \frac{15}{3}[(7.60 + 10.6) + 4\ (8.5 + 12.8) + 2\ (10.7)] = 624\ \text{m}^2$$

$$\Delta_2 = \frac{10}{3}[(10.6 + 8.3) + 4\ (9.5)] = 189.7\ \text{m}^2$$

$$\Delta_3 = \frac{20}{3}[(8.3 + 6.4) + 4\ (7.9)] + \frac{20}{2}(6.4 + 4.4)$$

$$= 308.6 + 108 = 416.6\ \text{m}^2$$

$$\therefore \qquad \Delta = 624 + 189.7 + 416.6 = \mathbf{1230.3\ m^2} = 12.303\ \text{ares.}$$

12.6 AREA BY DOUBLE MERIDIAN DISTANCES

This method is the one most often used for computing the area of a closed traverse. This method is known as D.M.D. method. To calculate the area by this method, the latitudes and departures of each line of the traverse are calculated. The traverse is then balanced. A reference meridian is then assumed to pass through the *most westerly station* of the traverse and the double meridian distances of the lines are computed.

Meridian Distances

The meridian distance of any point in a traverse is the distance of that point to the reference meridian, measured at right angles to the meridian. The meridian distance of a survey line is defined as the meridian distance of its mid-point. The meridian distance (abbreviated as M.D.) is also sometimes called as the *longitude*.

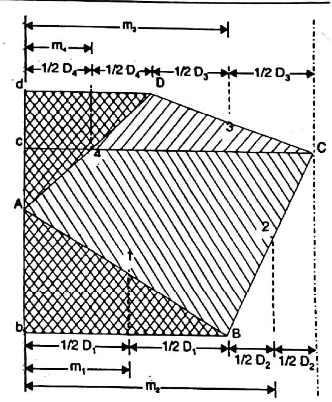


Fig. 12.6

Thus, in Fig. 12.6, if the reference meridian is chosen through the most westerly station *A*, the meridian distance (represented by symbol *m*) of the line *AB* will be equal to half its departure. The meridian distance of the second line *BC* will be given by

$$m_2 = m_1 + \frac{D_1}{2} + \frac{D_2}{2}$$

Similarly, the meridian distance of the third line *CD* is given by

$$m_3 = m_2 + \frac{D_2}{2} + \left(-\frac{D_3}{2}\right) = m_2 + \frac{D_2}{2} - \frac{D_3}{2}$$

And, the meridian distance of the fourth (last) line *DA* is given by

$$m_4 = m_3 + \left(-\frac{D_3}{2}\right) + \left(-\frac{D_4}{2}\right) = m_3 - \frac{D_3}{2} - \frac{D_4}{2} = \frac{D_4}{2}$$

Hence, the rule for the meridian distance may be stated as follows: ***The meridian distance of any line is equal to the meridian distance of the preceding line plus half the departure of the preceding line plus half the departure of the line itself.***

According to the above, the meridian distance of the first line will be equal to half its departure. In applying the rule, proper attention should be paid to the signs of the departures i.e., positive sign for eastern departure and negative sign for western departure.

Area by Latitudes and Meridian Distances

In Fig. 12.6, east-west lines are drawn from each station to the reference meridian, thus getting triangles and trapeziums. One side of each triangle or trapezium (so formed) will be one of the lines, the *base* of the triangle or trapezium will be the *latitude* of the line, and the *height* of the triangle or trapezium will be the *meridian distance* of that line. Thus,

area of each triangle or trapezium = latitude of the line × meridian distance of the line.

or $\qquad A_1 = L_1 \times m_1$

The latitude (*L*) will be taken positive if it is a northing, and negative if it is a southing.

In Fig. 12.6, the area of the traverse *ABCD* is equal to the algebraic sum of the areas of *dDCc*, *CcbB*, *dDA* and *ABb*. Thus,

$$A = \text{Area of } dDCc + \text{area of } cCBb - \text{area of } dDA - \text{area of } ABb$$

or $$A = L_3 m_3 + L_2 m_2 - L_4 m_4 - L_1 m_1 = \Sigma Lm.$$

(It is to be noted that the quantities $L_4 m_4$ and $L_1 m_1$ bear negative sign since L_4 and L_1 of *DA* and *AB* are negative).

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For exam

Example 12.5: The following table gives the corrected latitudes and departures (in metres) of the sides of a closed traverse ABCD:

	N	S	E	W
AB	108		4	
BC	15		249	
CD		123	4	
DA	0			257

Compute its area by (i) M.D. method, (ii) D.M.D. method, (iii) Departures and total latitudes, (iv) Co-ordinate method.

Solution:

1. *By meridian distances and latitudes*

$$\text{Area} = \Sigma(mL)$$

Calculate the meridian distance of each line. The calculations are arranged in the tabular form below. By the inspection of the latitudes and departures, point *A* is the most westerly station. *AB* is taken as the first line and *DA* as the last line.

AB	+ 108	+ 4	+ 2	2	+ 216
BC	+ 15	+ 249	+ 124.5	128.5	+ 1928
CD	– 123	+ 4	+ 2	255	– 31365
DA	0	– 257	– 128.5	128.5	0
				Sum	– 29221

∴ **Total area = $\Delta = \Sigma mL = -29221$ m^2**

Since the negative sign does not have any significance.

∴ **The actual area = 29221 m^2 = 2.9221 hectares.**

2. *By D.M.D. method:* Area = $\frac{1}{2}\Sigma mL$

AB	+ 108	+ 4	4	+ 432
BC	+ 15	+ 249	257	+ 3855
CD	– 123	+ 4	510	– 62,730
DA	0	– 257	257	0
			Sum	– 58,443

∴ **Area = $\frac{1}{2}\Sigma mL$ = 29221 m^2 = 2.9221 hectares.**

3. ***By Departure and total latitudes:*** **Let us first calculate the total latitudes of the point, starting with *A* as the reference point,**

Thus, total latitude of *B* = + 108
total latitude of *C* = + 108 + 15 = + 123
total latitude of *D* = + 123 – 123 = 0
total latitude of *A* = 0 + 0 = 0

The area = $\frac{1}{2}\Sigma$ (Total latitude × algebraic sum of adjoining departures)

AB	+ 108	+ 4				
			B	+ 108	+ 253	+ 27,324
BC	+ 15	+ 249				
			C	+ 123	+ 253	+ 31,119
CD	– 123	+ 4				
			D	0	– 253	0
DA	0	– 257				
			A	0	– 253	0
					Sum	58,443

∴ **Area = $\frac{1}{2}$(58,443) m^2 = 29221 m^2 = 2.9221 hectares.**

4. ***By Co-ordinates:*** **For calculation of area by co-ordinates, it is customary to calculate the independent co-ordinates of all the points. This can be done by taking the co-ordinates of *A* as (+ 100, + 100). The results are tabulated below:**

line (latitude)	N&S (Departure)	E&W	Station	Independent Co-ordinates North (y)	East (x)
			A	100 = y1	100 = x1
AB	+ 108	+ 4			
			B	208 = y2	104 = x2
BC	+ 15	+ 249			
			C	223 = y3	353 = x3
CD	– 123	+ 4			
			D	100 = y4	357 = x4
DA	0	– 257			
			A	100 = y1	100 = x1

Substituting the values of *x* and *y* in equation 12.7, we get

$$A = \tfrac{1}{2}[(y_1(x_2 - x_4) + y_2(x_3 - x_1) + y_3(x_4 - x_2) + y_4(x_1 - x_3)]$$
$$= \tfrac{1}{2}[100(208 - 100) + 104\,(223 - 100) + 353\,(100 - 208) + 357\,(100 - 223)]$$
$$= \tfrac{1}{2}(10800 + 12792 - 38124 - 43911) = -29221 \text{ m}^2$$

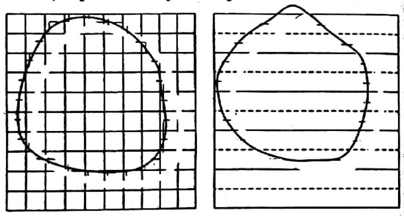
Since the negative sign does not have significance, the area = **2.9221 hectares.**

12.8 AREA COMPUTED FROM MAP MEASUREMENTS

(*a*) **By sub-division of the area into geometric figures:** The area of the plan is sub-divided into common geometric figures, such as triangles, rectangles, squares, trapezoids etc. The length and latitude of each such figure is scaled off from the map and the area is calculated by using the usual formulae.

(*b*) **By sub-division into squares:** Fig. 12.10 (*a*): The method consists in drawing squares on a tracing paper each square representing some definite number of square metres. The tracing paper is placed on the drawing and the number of squares enclosed in the figure are calculated. The positions of the fractional squares at the curved boundary are estimated. The total area of the figure will then be equal to the total number of squares multiplied by the factor (*i.e.*, sq. metres) represented by each square.

(*c*) **By division into trapezoids:** Fig. 12.10 (*b*): In this method, a number of parallel lines, at constant distance apart, are drawn on a tracing paper. The constant between the consecutive parallel lines represents some distance in metres or links. Midway between each pair of lines there is drawn another pair of lines in a different colour or dotted. The tracing is then placed on the drawing in such a way that the area is exactly enclosed between two of the parallel lines. The figure is thus divided into a number of strips. Assuming that the strips are either trapezoids or triangles, the area of each is equal to the length of the mid-ordinate multiplied by the constant breadth. The mid-ordinates of the strips are represented by the length of the dotted lines intercepted within the maps. The total sum of these intercepted dotted lines is measured and multiplied by the constant breadth to get the required area. More accuracy will be obtained if the strips are placed nearer.



(a) (b)

Fig. 12.10

12.9 AREA BY PLANIMETER

A planimeter is an instrument which measures the area of plan of any shape very accurately. There are two types of planimeters: (1) Amsler Polar Planimeter, and (2) Roller Planimeter. The polar planimeter is most commonly used and is, therefore discussed here.

Fig. 12.11 shows the essential parts of a polar planimeter. It consists of two arms hinged at a point known as the pivot point. One of the two arms carries an anchor at its end, and is known as the anchor arm. The length of anchor arm is generally fixed, but in some of the planimeters a variable length of anchor arm is also provided. The other arm carries a tracing point at its end, and is known as the tracing arm. The length of the tracing arm can be varied by means of a fixed screw and its corresponding slow motion screw. The tracing point is moved along the boundary of the plan the area of which is to be determined. The normal displacement of the tracing arm is measured by means of a wheel whose axis is kept parallel to the tracing arm. The wheel may either be placed between the hinge and the tracing point or is placed beyond the pivot point away from the tracing point. The wheel carries a concentric drum which is divided into 100 divisions. A small vernier attached near the drum reads one-tenth of the drum division.

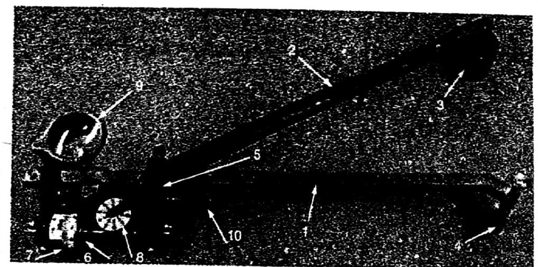


Fig. 12.11 Amsler polar planimeter.

1. Tracing Arm
2. Anchor Arm
3. Anchor
4. Tracing Point
5. Hinge
6. Wheel
7. Graduated Drum
8. Disc
9. Magnifier
10. Adjusting Screw for 1

The complete revolutions of the wheel are read on a disc actuated by a suitable gearing to the wheel. Thus, each reading is of four digits—the units being read on the disc, the tenths and hundredths on the drum, and the thousandths on the vernier. In addition to this, a fixed index near the disc can be utilised to know the number of the times the zero of the disc has crossed the index.

It is clear from Fig. 12.11 that the planimeter rests on three points—the wheel, the anchor point and the tracing point. Out of these three, the anchor point remains fixed in position while the wheel partly rolls and partly slides as the tracing point is moved along the boundary. Since the plane of the wheel is perpendicular to the plane of the centre line of the tracing arm, the wheel measures only normal displacement—when it actually rolls.

To find the area of the plan, the anchor point is either placed outside the area (if the area is small) or it is placed inside the area (if the area is large). A point is then marked on the boundary of area and the tracing point kept exactly over it. The initial reading of the wheel is then taken. The tracing point is now moved *clockwise* along the boundary till it comes to the starting point. The final reading of the drum is taken. The area of the figure is then calculated from the following formula:

Area $(\Delta) = M(F - I \pm 10N + C)$ ...(12.7)

Example 13.1: A railway embankment is 10 m wide with side slopes $1\frac{1}{2}$ to 1. Assuming the ground to be level in a direction transverse to the centre line, calculate the volume contained in a length of 120 metres, the centre heights at 20 m intervals being in metres 2.2, 3.7, 3.8, 4.0, 3.8, 2.8, 2.5.

Solution: For a level section, the area is given by $A = (b + nh)h$

Slope is $1\frac{1}{2} : 1$. Hence $n = 1.5$

The areas at different sections will be as under:

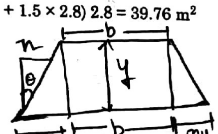
$A_1 = (10 + 1.5 \times 2.2)\ 2.2 = 29.26\ \text{m}^2$; $A_2 = (10 + 1.5 \times 3.7)\ 3.7 = 57.54\ \text{m}^2$

$A_3 = (10 + 1.5 \times 3.8)\ 3.8 = 59.66\ \text{m}^2$; $A_4 = (10 + 1.5 \times 4.0)\ 4.0 = 64.00\ \text{m}^2$

$A_5 = (10 + 1.5 \times 3.8)\ 3.8 = 59.66\ \text{m}^2$; $A_6 = (10 + 1.5 \times 2.8)\ 2.8 = 39.76\ \text{m}^2$

and $A_7 = (10 + 1.5 \times 2.5)\ 2.5 = 34.37\ \text{m}^2$

Volume by trapezoidal rule is given by



$$V = d\left[\frac{A_1 + A_n}{2} + A_2 + A_3 + \ldots A_{n-1}\right]$$

$$= 20\left[\frac{29.26 + 34.37}{2} + 57.54 + 59.66 + 64.00 + 59.66 + 39.76\right] = 6258.9\ \text{m}^3$$

Volume by prismoidal rule is given by

$$A = \frac{b + b + 2ny}{2}\,y = b + ny\,y$$

$$V = \frac{d}{3}[A_1 + 4(A_2 + A_4 + A_6 + \ldots) + 2(A_3 + A_5 + \ldots + A_{n-2}) + A_n]$$

$$= \frac{20}{3}[29.26 + 4\,(57.54 + 64.00 + 39.76) + 2\,(59.66 + 59.66) + 34.37] = 6316.5\ \text{m}^3.$$

Example 13.2: A railway embankment 400 m long is 12 m wide at the formation level and has the side slope 2 to 1. The ground levels at every 100 m along the centre line are as under:

Distance	*0*	*100*	*200*	*300*	*400*
R.L.	*204.8*	*206.2*	*207.5*	*207.2*	*208.3*

The formation level at zero chainage is 207.00 and the embankment has a rising gradient of 1 in 100. The ground is level across the centre line. Calculate the volume of earthwork.

Solution: Since the embankment level is to have a rising gradient of 1 in 100 the formation level at every section can be easily calculated as tabulated below:

0	204.8	207.0	2.2
100	206.2	208.0	1.8
200	207.5	209.0	1.5
300	207.2	210.0	2.8
400	208.3	211.0	2.7

The area of section is given by $A = (b + nh)\,h = (12 + 2h)\,h$

$A_1 = (12 + 2 \times 2.2)\ 2.2 = 36.08\ \text{m}^2$; $A_2 = (12 + 2 \times 1.8)\ 1.8 = 28.06\ \text{m}^2$

$A_3 = (12 + 2 \times 1.5)\ 1.5 = 22.50\ \text{m}^2$; $A_4 = (12 + 2 \times 2.8)\ 2.8 = 49.28\ \text{m}^2$

and $A_5 = (12 + 2 \times 2.7)\ 2.7 = 46.98\ \text{m}^2$

Volume by trapezoidal rule is given by

$$V = d\left[\frac{A_1 + A_n}{2} + A_2 + A_3 + \ldots A_{n-1}\right]$$

$$= 100\left[\frac{36.08 + 46.98}{2} + 28.06 + 22.50 + 49.28\right] = 14{,}137\ \text{m}^3$$

Volume by prismoidal rule is given by

$$V = \frac{d}{3}\,[(A_1 + A_5) + 4(A_2 + A_4) + 2(A_3)]$$

$$V = \frac{100}{3}\,[(36.08 + 49.98) + 4(28.06 + 49.28) + (2 \times 22.50)] = 14{,}581\ \text{m}^3.$$

Example 13.3: Find out the volume of earth work in a road cutting 120 metres long from the following data:

The formation width 10 metres; side slopes 1 to 1; average depth of cutting along the centre of line 5 m; slopes of ground in cross-section 10 to 1.

Solution: The cross-sectional area, in terms of m and n, is given by equation 13.8

Thus,
$$A = \frac{n\left(\frac{b}{2}\right)^2 + m^2(bh + nh^2)}{m^2 - n^2}$$

Here $n = 1$; $m = 10$; $h = 5$; and $b = 10$.

$$\therefore \quad A = \frac{1\left(\frac{10}{2}\right)^2 + 10^2(10 \times 5 + 1 \times 5^2)}{10^2 - 1^2} = 76\ \text{m}^2$$

$$\therefore \quad V = A \times L = 76 \times 120 = 9120\ \text{cubic metres.}$$

Example 13.4: A road embankment 10 m wide at the formation level, with side slopes of 2 to 1 and with an average height of 5 m is constructed with an average gradient 1 in 40 from contour 220 metres to 280 metres. Find the volume of earth work.

Solution: Difference in level between both the ends of the road

= contour 280 – contour 220 = 60 m

∴ Length of the road = 60 × 40 = 2400 metres.

Area of the cross-section = $(b + nh)\,h$

Here, $b = 10$ m; $n = 2$ m; $h = 5$ m.

∴ $A = (10 + 2 \times 5)\ 5 = 100$ sq. m.

∴ Volume of embankment = Length × Area = 2400 × 100 = 2,40,000 cubic metres.

Module-1: Theodolite Survey and Instrument Adjustment

Theodolite

The system of surveying in which the angles are measured with the help of a theodolite is called Theodolite surveying.

A theodolite is a precision optical instrument for measuring angles between designated visible points in the horizontal and vertical planes. The traditional use has been for land surveying, but they are also used extensively for building and infrastructure construction, and some specialized applications such as meteorology and rocket launching.

It consists of a moveable telescope mounted so it can rotate around horizontal and vertical axes and provide angular readouts. These indicate the orientation of the telescope, and are used to relate the first point sighted through the telescope to subsequent sightings of other points from the same theodolite position. These angles can be measured with great accuracy, typically to mili-radian or seconds of arc. From these readings a plan can be drawn, or objects can be positioned in accordance with an existing plan. The modern theodolite has evolved into what is known as a total station where angles and distances are measured electronically, and are read directly to computer memory.

Types of Theodolite

Theodolites are broadly classified as,

a) Transit
b) Non-transit

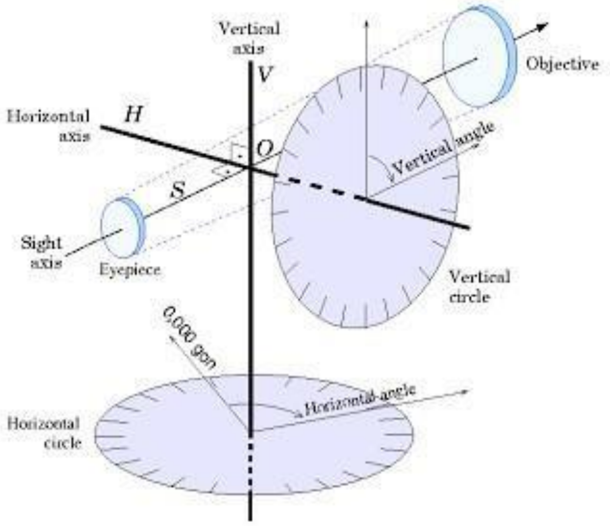
- **Transit theodolite:** A theodolite in which if the telescope can be revolved through a complete resolute about its horizontal axis in the vertical plane is called as a transit theodolite.
- **Non transit theodolite:** These theodolites are plain or 'Y' theodolites, in which the telescope cannot be transited or vertical rotation of the telescope is restricted to a limited arc.

Based on the system used to observe the reading, Theodolites are also classified as,

a) Vernier theodolites
b) Micrometer theodolites

- **Vernier theodolite:** verniers are used to measure accurately the horizontal and vertical angles. A 20" vernier theodolite is usually used.
- **Micrometer theodolite:** An optical system or a micrometer is used to read the angles in this case. The precision can be as high as 1".

Fundamental Axes of transit theodolite



The fundamental Axes are:

1) Vertical axis
2) Horizontal axis
3) Line of collimation (line of sight)
4) Axis of plate level
5) Axis of altitude level
6) Axis of striding level, if provided

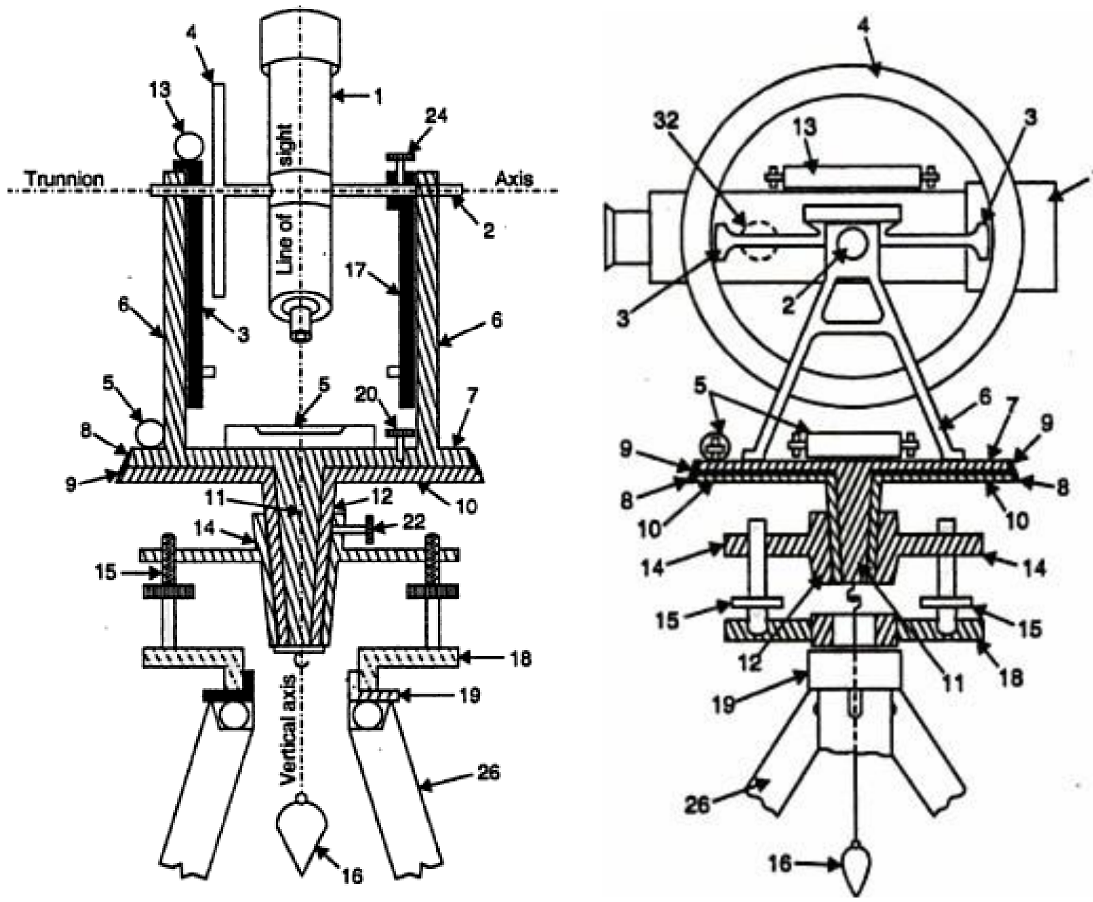
1. The vertical axis: It is the axis about which the instrument can be rotated in a horizontal plane.
2. The horizontal axis: It is the axis about which the telescope can be rotated in the vertical plane. It is also called the trunnion axis.
3. Line of sight or line of collimation: It is the imaginary line passing through the intersection of the cross hairs (vertical and horizontal) and the optical center of the object glass and its continuation.
4. Axis of plate level: It is also called as bubble line; it is the straight tangential line to the longitudinal curve of the level tube at its centre.
5. Axis of the altitude level: It is the axis of the level tube in altitude spirit level.
6. Striding level: It is used for testing the horizontality of the transit axis or trunnion axis.

Parts of transit theodolite

1. Telescope: The telescope of the theodolite is mounted on a spindle known as 'Trunnion axis'. In most of the transit theodolite an internal focusing telescope is used. It consists of object glass, a diaphragm and an eye-piece. The main function of the telescope is to provide line of sight.

2. The vertical circle: The vertical circle is rigidly connected to the transverse axis of the telescope and moves as the telescope is raised or depressed. It is graduated in degrees with graduations at 20°. Graduation in each quadrant is numbered from 0° to 90° in the opposite directions from the two zeros placed at the horizontal diameter of the circle.

3. The index frame or T-frame or Vernier frame: It consists of a vertical portion called dipping arm and a horizontal portion called an index arm. The two verniers of the vertical circle are fixed to the two ends of the index arm. The index arm can be rotated slightly for adjustment purpose, with the help of clip screw.



1.	TELESCOPE	13.	ALTITUDE LEVEL
2.	TRUNNION AXIS	14.	LEVELLING HEAD
3.	VERNIER FRAME	15.	LEVELLING SCREW
4.	VERTICAL CIRCLE	16.	PLUMB BOB
5.	PLATE LEVELS	17.	ARM OF VERTICAL CIRCLE CLAMP
6.	STANDARDS (A-FRAME)	18.	FOOT PLATE
7.	UPPER PLATE	19.	TRIPOD HEAD
8.	HORIZONTAL PLATE VERNIER	20.	UPPER CLAMP
9.	HORIZONTAL CIRCLE	22.	LOWER CLAMP
10.	LOWER PLATE	24.	VERTICAL CIRCLE CLAMP
11.	INNER AXIS	26.	TRIPOD
12.	OUTER AXIS	32.	FOCUSING SCREW

4. The standard or A-Frame: Two standards resembling letter 'A' are mounted on the upper plates. The trunnion axis of the telescope is supported on these. The T-Frame and the arm of vertical circle clamp are also attached to A-Frame.

5. Levelling head: It consists of two parts namely,

   a) Tribrach is the upper triangular plate which carries three levelling screws at three ends of the triangle.

   b) Trivet or the lower plate (foot plate) has three grooves to accommodate three levelling screws.

The leveling head has three main functions namely,

a) To support the main part of the instrument

b) To attach the theodolite to the tripod

c) To provide a means for levelling

6. The two spindles: Inner spindle is conical and fits into the outer spindle which is hollow. Inner spindle is also called upper axis and outer spindle is called lower axis.

7. The lower plate (scale plate): It carries the circular scale which is graduated from 0° to 360°. It is attached to the outer spindle which turns in a bearing within the tribrach of the levelling head. It is fixed using lower clamping screw; lower tangent screw enable slow motion of the outer spindle.

8. The Upper plate (vernier plate): It is attached to the inner axis and carries two verniers with magnifiers at two extremities diametrically opposite. Upper clamping screw and a corresponding tangent screw are used for moving upper plate.

9. The plate levels: The upper plate carries one or two plate levels which can be centered with the help of foot screws.

10. Accessories:

a) Tripod with three solid legs

b) Plumb bob : for centering

c) Compass : tubular or trough

d) Striding level for testing the horizontality of the transit axis or trunnion axis.

Uses of Theodolite

The Theodolite is a most accurate surveying instrument mainly used for:

a) Measuring horizontal and vertical angles

b) Locating points on a line

c) Prolonging survey lines

d) Finding difference of level

e) Setting out grades

f) Ranging curves

g) Tacheometric Survey

Definitions and Terms

1. **Centering:** Setting the theodolite exactly over an instrument station so that its vertical axis lies immediately above the station point is called centering.
2. **Transiting:** It is the process of turning the telescope vertical plane through 180° about the trunnion axis. This process is also known as plunging or reversing.
3. **Swinging the telescope:** It is the process of turning the telescope in horizontal plane. If the telescope is rotated in clock wise direction, it is known as right swing and other wise left

swing.

4. **Face right observation:** If the vertical circle is to the left of the observer, then the observation is called as face left.
5. **Face right observation:** If vertical circle is to the right of the observer, then the observation called as face right.
6. **Telescope normal and telescope inverted:** If the telescope is in such a way that the face is left and bubble is up, then it is said to be in normal position or direct. If the face is right and bubble is down then the telescope is said to be in inverted position or reversed position. Vertical circle to the right of the observer, if originally to the left and vice versa. It is done by first revolving the telescope through 180° in a vertical plane and then rotating it through 180° in the horizontal plane, i.e. first transiting and then swinging the telescope.

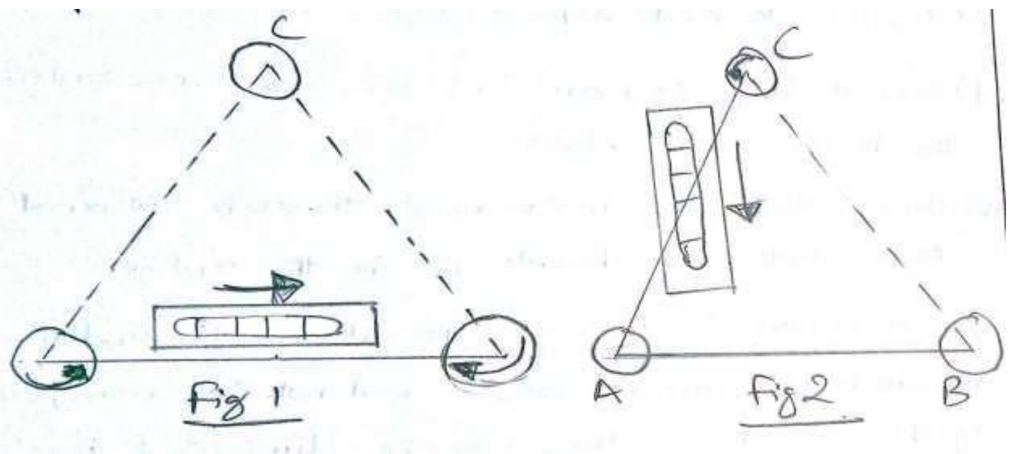
Temporary adjustments of a transit theodolite

The temporary adjustment of a transit theodolite is done by the following three important operations:

1. **Setting up:** The instrument have to be set up properly on the station point. The tripod stand should be approximately levelled before fixing the instrument. This is achieved by moving the legs of the tripod. There is a small spirit level on the tripod head for the leveling of tripod. Centering of the instrument over the station mark is achieved by a plumb bob or by using optical plummet.

2. **Leveling up:** After centering and approximate leveling, accurate leveling is to be carried out with the help of the foot screws and using the plate level tube. In this step the vertical axis of the instrument is made truly vertical. Leveling the instrument depends on the number of foot screws available. For a screw head, the procedure for leveling is as follows:

a) Turn the upper plate until the longitudinal axis of the plate level is parallel to the line joining any two foot screws (let it be A and B)



b) Hold the two foot screws A and B between the thumb and the forefingers of each hand and turn them uniformly so that the thumb move either towards each other or away from each other until the bubble is central. Bubble moves in the direction of the left foot screw.

c) Turn the upper plate through 90° until the axis of the level passes over the position of the third leveling screw C.

d) Turn this levelling screw until the bubble is central

e) Return the upper plate to original position (fig1) and repeat step(b)

f) Turn back and repeat step (c)

g) Repeat steps (e) and (f) for two to three times until the bubble is central.

h) Now rotate the instrument through 180° and check whether the bubble is in thecentre.

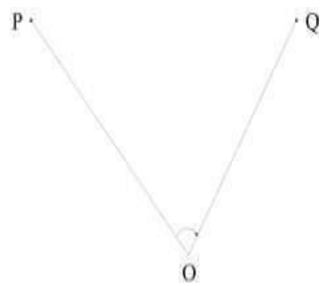
3. Elimination of Parallax: Parallax is a condition in which the image formed does not lie on the plane of the cross hair; this can be eliminated by focusing the eye-piece and the objective. For focusing the eye-piece, hold a white paper in front of the objective and move eye-piece in or out until the cross-hairs are distinctly visible. Objective lens is focused by rotating the focusing screw until the image appears clear and sharp.

Measurement of Horizontal Angles

Theodolites are majorly used to measure horizontal and vertical angles. Horizontal angles are usually, measured by any of these methods.

a) Ordinary method
b) Method of repetition
c) Method of reiteration

a) Ordinary Method



To measure an angle POQ, the following procedure is used:

1. Set up the instrument at ‘O’, Set it up, level it accurately and perform the temporary adjustments.
2. Release the upper clamp screw and lower clamp screw. Turn the upper and lower plates such that the vernier A reads zero and the vernier circle is to the left of the observer. Clamp both the plates and bring the vernier A to zero to coincide with the main scale zero using the upper tangent screw. Check the reading on vernier B, it should read 180°.
3. Loosen the lower clamp and rotate the telescope to view point P. Clamp lower plate and using lower tangent screw sight P exactly. Check the readings on both the verniers to see that it had not changed.
4. Unclamp the upper clamp and rotate the instrument clock-wise until point Q is bisected,

tighten the clamp and using upper tangent screw bisect Q accurately.

5. Reading is observed from verniers A and B. Reading of vernier A gives angle POQ and B vernier gives 180°+POQ
6. Read degrees, minutes and seconds from the vernier scale by observing which line on the vernier scale is having correct coincidence with the reading in the main scale. In a 20" transit theodolite, the least count is 20" or the minimum reading which can be measured from the scale is 20". The reading coinciding with the vernier-zero is considered to be the main scale reading. If there is no exact coincidence for the vernier zero line, then the reading to the immediate left of the vernier scale, on the main scale should be considered. This reading should be added with the vernier reading for the total value.

   Example: At vernier A, Reading on main scale = 128° 40', Reading on vernier scale = 3' 0",
   Thentotalreading=128°40'+3'0" = 128°43'0"

   At vernier B scale, the degree reading is not required, whereas the minutes reading from the main scale is noted and added with vernier reading and this will give the B scale reading.
7. Enter the readings in a field book in the following tabular format,

1	2	3	4						5			6			7		
Instrument Station	Object	Face	Readings on verniers						Mean			Angle			Mean angle		
			A			B											
			°	'	"	°	'	"	°	'	"	°	'	"	°	'	"

8. Change the face by transiting and swinging and repeat the same process.
9. The mean of the two vernier readings gives the angle on faceright.
10. Average horizontal angle is calculated from the mean horizontal angle of face left and face right values.

b) Repetition Method

This method is used for very accurate work. In this method, the same angle is added several times mechanically and the total angle is divided by no of repetitions to obtain the correct value of angle. To measure an angle POQ by the method of repetition, the following procedure is adopted:

1. Setup the instrument at ‘O’ and level it.
2. Loosen the upper clamp and turn the upper plate until the index of vernier ‘A’ nearly coincide with the horizontal circle. Now tight the upper clamp.
3. Turn the upper tangent screw so as to make the two zeros exactly coincide. So that ‘A’ vernier reads 0° and ‘B’ vernierreads 180°.
4. Sight station ‘P’, tighten the lower clamp and bisect station ‘P’ exactly by using the lower tangent screw.
5. Unclamp the upper clamp and swing the telescope, bisect station ‘Q’ by using the upper clamp and upper tangent screw.
6. Read both the verniers take average to get angle POQ.
7. Unclamp the lower clamp and swing the telescope and bisect station ‘P’ accurately by using the lower clamp and lower tangentscrew.
8. Read both the verniers check the vernier reading it should be the same (unchanged) as that obtained in step 6.
9. Release the upper plate using upper clamp and bisect station ‘Q’ accurately using upper tangent screw. The vernier will read twice the angle POQ
10. Repeat the procedure for required number of times say three times and find out the value of angle POQ.
11. Change face and make three more repetitions as described above. Find the average angle with face right by dividing the final reading by the number of repetitions.

Tabular Column:

Instrument Stn	Object Stn	Face:LEFT					Swing: RIGHT				
		Vernier ‘A’			Vernier ‘B’		Vernier Mean			No of Repetition	Horizontal Angle
		°	‘	“	‘	“	°	‘	“		° ‘ “
O	A									1	
	B										
	A									2	
	B										
	A									3	
	B										

Instrument Stn	Object Stn	Face:RIGHT					Swing: LEFT				
		Vernier ‘A’			Vernier ‘B’		Vernier Mean			No of Repetition	Horizontal Angle
		°	‘	“	‘	“	°	‘	“		° ‘ “
O	A									1	
	B										
	A									2	
	B										
	A									3	
	B										

Position of Vertical Circle	Trial No.	Horizontal Angle			Average Horizontal Angle
		°	'	"	
Face Left	1				
	2				
	3				
Face Right	1				
	2				
	3				

12. The average Horizontal angle is then obtained by taking the average of the two angles obtained with face left and faceright.

For high precision surveys, repetition method can be conducted in two ways:

a) The angle is measured respectively for six times, keeping the telescope normal (face left) and then calculating the average.
b) In another way, angle is measured clockwise, with face left for first three observations then, in anticlockwise direction, three face right observations are taken.

Elimination of errors by method of repetition

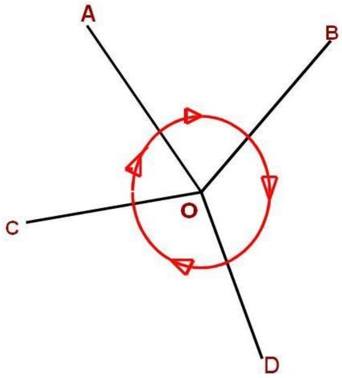
The following errors are eliminated by adopting method of repetition

a) Errors due to eccentricity of verniers and centers by measuring both vernier readings.
b) Errors due to line of collimation not being perpendicular to the horizontal axis of the telescope.
c) Errors due to horizontal axis of telescope not being perpendicular to the vertical axis.
d) Error due to the line of collimation not coinciding with the axis of the telescope. These three errors can be eliminated by changing the face of the theodolite.
e) Errors due to inaccurate graduations can be eliminated by taking both vernier readings.
f) Error due to inaccurate bisection of the object can be eliminated by taking repeated readings.

Reiteration Method

This method is also known as direction method or method of series. Several angles are measured successively and finally the horizon is closed.

To measure a series of angles AOB, BOC, COD etc by reiteration, this procedure is followed:



1. Set up the instrument at 'O' and level it.
2. Set the vernier 'A' to read zero using upper clamp and upper tangent screw.
3. Direct the telescope towards point 'B' and bisect it exactly using the lower clamp and lower tangent screw. Read both vernier 'A' & 'B' and take mean value.
4. Similarly Loosen the upper clamp and bisect the successive points C, D etc using upper tangent screw and keep on observing the readings. Each included angle is obtained by taking the difference of two consecutivereadings. Angle BOC = angle AOC – angle AOB
5. Finally close the horizon by sighting point A. The reading in the vernier should be zero (or 360°). If not, note down the reading and distribute it evenly to all angles.
6. Repeat the same steps in other face.
7. The sets of reading are usually taken first in clockwise direction and then after changing the face in anticlockwise direction.
8. Final average horizontal angles are calculated from the mean horizontal angles of face left and face right values.

Tabular Column:

Insrument Stn	Object Stn	Face : Left Swing : Right								
		Ver 'A'			Ver 'B'		Vernier Mean			Horizontal Angle
		°	'	"	'	"	°	'	"	° ' "
	A									AOB=
	B									BOC=
	C									COD=
	D									DOA=
O	A									

Insrument Stn	Object Stn	Face : Right Swing : Left								
		Ver 'A'			Ver 'B'		Vernier Mean			Horizontal Angle
		°	'	"	'	"	°	'	"	° ' "
	A									AOB=
	B									BOC=
	C									COD=
	D									DOA=
O	A									

MEASUREMENT OF VERTICAL ANGLES

A vertical angle is an angle between the inclined line of sight and horizontal. The instrument has to be leveled with respect to the altitude bubble for measuring vertical angles.

1. Level the instrument with reference to plate level
2. Keep the altitude bubble tube parallel to 2 foot screws and bring the bubble central. Rotate telescope 90° and adjust the bubble using the 3rd foot screw. Repeat the procedure till the bubble is central.
3. Loosen the vertical clamp screw; rotate the telescope in vertical plane to sight the object, use tangent screw for correct bisections.
4. Read vernier C and D. Average gives correct vertical angle.
5. Change the face and continue the procedure.
6. Final average vertical angle is calculated from the mean vertical angles of face left and face right values.

If the vertical angle is measured above the horizontal line, it is called angle of elevation and if the vertical angle is measured below the horizontal line, it is angle of depression.

Tabular Column:

Instrument Stn	Point Sighted	Face : Left Swing : Right								
		Ver 'C'			Ver 'D'		Vernier Mean			Vertical Angle
		°	'	"	'	"	°	'	"	° ' "
O	A									

Insrument Stn	Point Sighted	Face :Right Swing : Left								
		Ver 'C'			Ver 'D'		Vernier Mean			Vertical Angle
		°	'	"	'	"	°	'	"	° ' "
O	A									

Permanent Adjustment of Transit Theodolite

The permanent adjustments are made to establish the relationship between the fundamental lines of the theodolite and, once made; they last for a long time. They are essential for the accuracy of observations.

The permanent adjustments in case of transit theodolites are as follows:

a) Adjustment of Horizontal Plate Levels: The axis of the plate levels must be perpendicular to the vertical axis.
b) Adjustment of line of Collimation Adjustment: The line of collimation should coincide with the axis of the telescope and the axis of the objective slide and should be at right angles to the horizontal axis.
c) Adjustment of Horizontal axis: The horizontal axis must be perpendicular to the vertical axis.
d) Adjustment of Telescope Level or the Altitude Level: The axis of the telescope levels or the altitude level must be parallel to the line of collimation.
e) Adjustment of Vertical Circle Index: The vertical circle vernier must read zero when the line of collimation is horizontal.

For making the adjustments, the instrument should be set up at a fairly level ground where sights of about 100 m can be taken in either direction in the same straight line. Each adjustment involves two steps:

i. A test to determine the error, and
ii. An adjustment of this error

Since certain adjustments upset other adjustments, the adjustments must be made in the order given below:

1) First Adjustment: To make the axis of the plate levels perpendicular to the vertical axis

a) Objective: When this adjustment is made, all the bubbles will remain in the centre of their run during a complete revolution of the instrument.
b) Necessity: It is important in the measurement of horizontal and vertical angle. The vertical axis should be truly vertical. It may be noted that the error due to vertical axis not being truly vertical cannot be eliminated by taking face left and face right observations.
c) Test: The adjustment involves making the vertical axis vertical and then making the level on the upper plate perpendicular to the vertical axis.
    i. Set up the instrument on firm ground, clamp the lower motion and turn the upper plate until the longer plate-level is parallel to any pair of foot screws. Bring each plate level to the centre of its run by means of foot screws as explained in temporary adjustment of a theodolite.
    ii. Rotate the instrument about the vertical axis through 180°. The plate level is again parallel to the pair of foot-screws, but with the ends reversed in direction. If the bubble remains central, the axes of the plate levels are perpendicular to the vertical axis and the vertical axis-is truly vertical.
d) Adjustment:

i. If not, note down the deviation of the bubble (say, n divisions), which is the apparent error and is twice the actual error in the axis of the level and, therefore the correction is half the amount of this error.
ii. Bring each bubble half way (n/2 divisions) back by means of the respective foot-screw and remaining half way back by means of the captain-headed nuts provided at the end of the tube.
iii. Repeat the test and adjustment until both the bubbles remain central during the whole revolution of the instrument.

2) Second Adjustment: To make the line of collimation coincide with the axis of the telescope, and to make it at right angles to the horizontal axis. This adjustment is made in two steps:

(i) Adjustment of the horizontal hair, and

(ii) Adjustment of the vertical hair

First Step: Adjustment of the Horizontal Hair

a) Objective: The objective of this adjustment is to bring the horizontal hair, into the horizontal plane through the optical axis.
b) Necessity: The movement of the object-glass is assumed to be along the optical axis. If this adjustment does not exist, the direction of the line of sight will change when the objective is moved in and out for focusing. This adjustment is necessary when the instrument is used for measuring vertical angles or when it is used for levelling operations. It has no effect in the measurement of horizontal angles.
c) Test:
   i. Set up the theodolite at a convenient point and level it.
   ii. With the face of the instrument left, take a reading on the staff held on peg B driven at a distance of about 100 m from the instrument station (O). Let the staff reading be b_1. Also note the vertical angle (α).
   iii. Change the face of the instrument. Set the vertical vernier to the former angle (α) and again take a reading on B. If the reading is equal to the previous reading b_1, no adjustment is necessary.
d) Adjustment:
   i. If not, note the second staff reading. Let it be b_2.
   ii. Move the horizontal hair by means of the vertical diaphragm screws until the mean 'b' of the two readings b_1 and b_2 is obtained.

Second Step: Adjustment of the Vertical Hair

a) Objective: The objective of this adjustment is to make the line of collimation perpendicular to the horizontal axis.

b) Necessity: If this adjustment exists, the line of collimation will generate a plane when the telescope is transited. But if not, it will generate a cone with the horizontal axis as its axis. The adjustment is necessary for prolonging straight lines and for measuring horizontal angles between points at different elevations.

c) Test:

i. Set up the theodolite at a convenient point O on a fairly level ground and level it. Fix a peg at a point A at a distance of about 100 m from O. Bisect A and clamp the lower and upper screws.

ii. Now transit the telescope and mark a point B in the line of sight at about 100 m from O and at about the same level as A.

iii. Unclamp the upper screw, swing the telescope through 180°, and again bisect A. Then clamp the upper screw.

iv. Transit the telescope again. If now the point B is again bisected by the cross-hairs, the adjustment is correct.

d) Adjustment:

i. If the point B is now not on the line of sight, mark a point C in the line of sight opposite B. The apparent error BC is four times the real error, since the telescope is transited twice.

ii. Mark a point D at one fourth distance from C to B (CD = 1/2CB).

iii. Move the diagram by means of the horizontal diagram screws until the vertical hair is on the point D.

iv. Repeat the process until the adjustment is correct.

3) Third Adjustment: To make the horizontal axis perpendicular to the vertical axis

a) Objective: The objective of this adjustment is to ensure that the line of sight revolves in a vertical plane perpendicular to the horizontal axis.

b) Necessity: The adjustment is necessary for prolonging straight lines and for measuring horizontal angles. This condition can be established by the Spire Test as follows:

c) Test:

i. Set up the instrument near a spire or some other elevated object. Let P be the top point.

ii. Sight the point P, and with both horizontal motions clamped, depress the telescope and mark a point A in the line of sight near the base of the object.

iii. Change the face of the instrument, and again sight on the point P. Depress the telescope. If the line of sight now strikes the point A marked previously, the adjustment is correct.

d) Adjustment:

i. If not, mark another point B in the line of sight near the base of the same object. Mark another point C midway between A and B.

ii. Sight on point C and clamp the upper motion. Raise the telescope. The line of sight will not now strike the point P. Raise or lower the adjustable end of the horizontal axis by means of the screws near the top of the A-frame until the line of sight passes through the point P.

iii. Repeat the test for adjustment until perfect.

4) Fourth Adjustment: To make the axis of the telescope level or altitude level parallel to the line of collimation

a) Objective: The objective of this adjustment is that the line of collimation should remain horizontal when the telescope level is brought in the centre.

b) Necessity: The adjustment is essential when the vertical angles are to be measured and the instrument is to be used as a level.

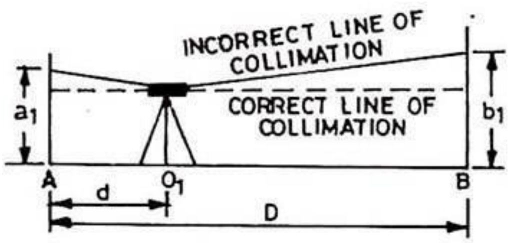
c) Test: The procedure of testing the theodolite for this adjustment is the same as in the 'two peg' method of dumpy level.

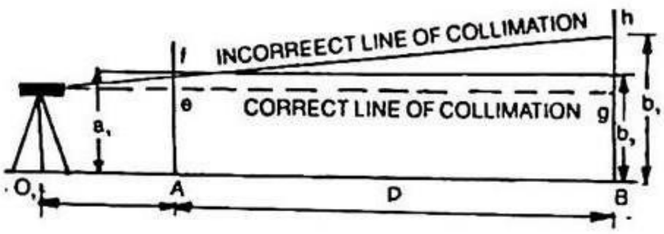
i. Fix two pegs A and B on a fairly level ground about 100m apart. Set up the theodolite at O exactly midway between A and B. Clamp the vertical circle and the telescope level in the centre of its mm by means of the tangent screw of the vertical circle.

ii. With the bubble central, take readings on staff held on A and B and find the difference between these readings, which is the true difference of level between A and B.

iii. Shift the instrument and set it up at O_1 on the line BA produced, at about 20 m from A.

iv. With the bubble central, read the staff first on A and then on B and find the difference between the two readings. If this difference is equal to the first difference, the adjustment is correct.





d) Adjustment:

i. If not, calculate the correct staff readings on A and B as explained in the 'two-peg' adjustment of the dumpy level.

ii. Bring the horizontal hair exactly to the correct reading on B by means of the tangent screw of the vertical circle. The bubble is thus disturbed.
iii. Bring the bubble exactly to the centre of its run be means of the level tube nuts.
iv. Sight the staff on the near peg and note whether the calculated correct reading is obtained.
v. Repeat the process until the adjustment is correct.

5) **Fifth Adjustment:** To make the vertical circle read zero when the line of collimation is horizontal i.e. the telescope bubble is centered.

a) Objective: The objective of this adjustment is that the vertical circle should read zero when the line of collimation is horizontal.
b) Necessity: This adjustment made for convenience only. If the index error i.e. the reading on the vertical circle when the telescope bubble is in the centre is noted and the corresponding correction applied to the observed reading, no error will be introduced. But still this adjustment is preferred while reading vertical angles and prolonging straight lines.
c) Test:
   i. Set up the theodolite at firm ground.
   ii. Centre the plate bubbles and then bring the telescope bubble exactly to the centre of its run by means of the vertical tangent screw as in the first adjustment, and read the vernier of the vertical circle. If it reads zero, then this adjustment of the instrument is correct.
d) Adjustment:

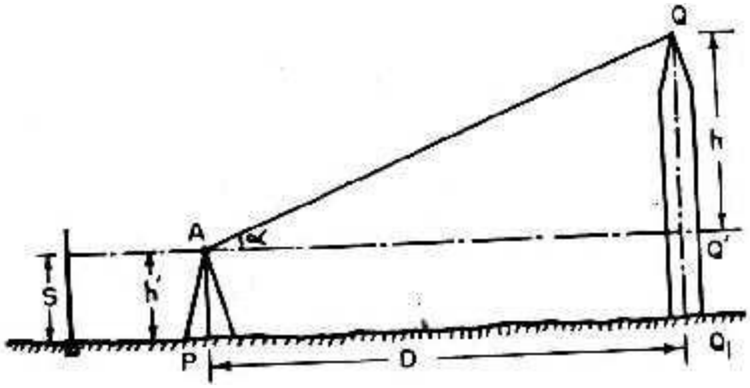
If the vernier does not read zero, loosen it and move it by means of adjusting screw which holds it to the standard until it reads zero.

Trigonometric Leveling

Trigonometric Leveling is the branch of Surveying in which we find out the vertical distance between two points by taking the vertical angular observations and the known distances. The known distances are either assumed to be horizontal or the geodetic lengths at the mean sea level (MSL). The distances are measured directly (as in the plane surveying) or they are computed as in the geodetic surveying.

Trigonometric leveling is an indirect method of leveling in which the difference in elevation is measured using vertical angles, with the help of a transit theodolite. Distances are either measured or computed using trigonometric calculations. Thus this method is also called as a **method of heights and distances**. There are various cases in trigonometric leveling:

Case 1: Base of the object is accessible



It is required to find the elevation (R.L.) of the top of a tower 'Q', whose base is accessible, from the instrument station 'P' as shown in the above diagram.

Let,

P= instrument station

Q= Point to be observed

A= center of the instrument

D= horizontal distance between P and Q

h' = height of the instrument at P

Q'=Projection of Q on horizontal plane

S= Reading on staff kept on B.M, with line of sight on horizontal

α= Angle of elevation from A to Q

Procedure:

a) Setup the Theodolite at P and level it accurately w.r.t. the altitude bubble. See that the vertical circle reads 0°0'0" when the line of sight is horizontal.
b) Direct the telescope towards Q and bisect it accurately, clamp both the plates. Read the vertical angle 'α'.
c) Plunge the telescope and sight to the same point 'Q' and take the vertical angle 'α' calculate the average of the vertical angles measured in both faces.
d) With the vertical vernier set to zero reading and the altitude bubble in the center of its run take the reading on the leveling staff kept at B.M. Let it be 'S'.
e) Measure the horizontal distance between P and Q. let it be 'D'.

Observations and Calculations:

Vertical Angle, α=

Staff Reading, S (m) =

Horizontal Distance, D (m) =

From Triangle AQQ', h = D tanα

R.L. of Q = R.L of B.M + S + h (or)

R.L. of Q = R.L of instrument axis + h (or)

R.L. of Q = R.L of P + h' + h, if R.L of P is known

This method is usually employed when 'D' is small. If 'D' is Large, then the combined correction for curvature and refraction should be applied.

i.e., R.L. of Q = R.L of B.M + S + h ± C_c

Where, C_c=0.06728D^2 meter (D is in kilometer)

Linearly its sign is positive for angles of elevation and negative for angles of depression.

Case 2: Base of the object inaccessible (Single Plane Method)

To find the elevation of the top of a building using the principle of trigonometric leveling with the instrument stations having their vertical axes in the same plane as the object.

Procedure:

It is required to find the elevation (R.L.) of the top of a building 'Q' from the instrument stations P & R as shown in the above diagram.

a) Setup the Theodolite at P and level it accurately with respect to the altitude bubble. See that the vertical circle reads 0°0'0" when the line of sight is horizontal.
b) Direct the telescope towards Q and bisect it accurately clamp both the plates. Read the vertical angle α_1.
c) Transit the telescope so that the line of sight is reversed. Mark instrument station R on the ground along the line of sight. Measure the distance between P & R accurately. Let it be 'b'.
d) Repeat the steps (b) & (c) for both face observations. The mean values should be adopted in the calculations.
e) With the vertical vernier set to zero reading and the altitude bubble in the center of its run take the reading on the leveling staff kept at B.M. Let it be 'S' if both the instrument axis are at same level, and 'S_1' if they are at different levels.
f) Shift the instrument to R and set up the theodolite there. Measure the vertical angle 'α_2' to Q with both face observations.
g) In case of instrument axis at different levels repeat the step (e) and let the staff reading at R be 'S_2'.

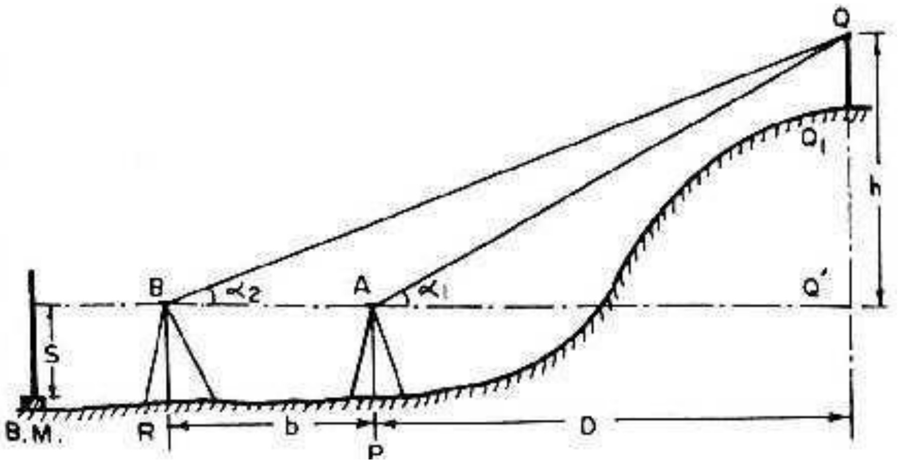
Observations and Calculations:

Vertical Angles, α_1 = ………., α_2 = …………

Staff Readings S_1 = ……….. (or) S = …………, S_2 = ………..

Horizontal distance between P & R = b = ………..

In case of instrument axes at same level:



From triangle AQQ'

$$h = D \tan \alpha_1$$

From triangle BQQ'

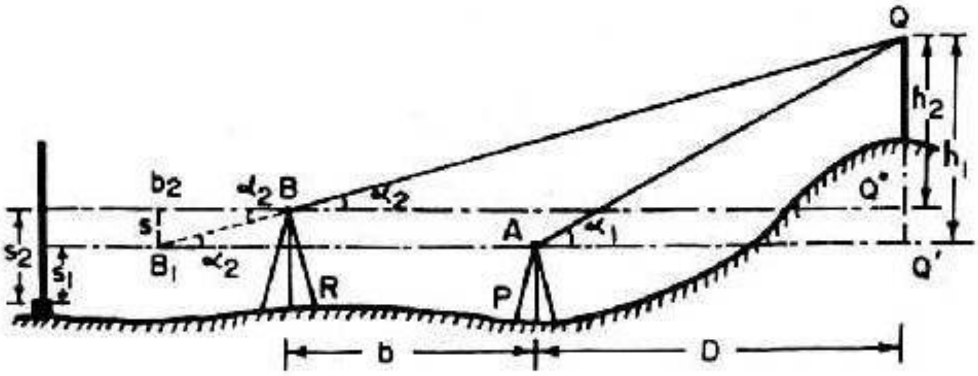
$$h = (b + D) \tan \alpha_2$$

Therefore, $D \tan \alpha_1 = (b + D) \tan \alpha_2$

$$\mathbf{D = \frac{b \tan \alpha 2}{\tan \alpha 1 - \tan \alpha 2}}$$

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

In case of instrument axes at different levels:



From triangle AQQ'

$$h_1 = D \tan \alpha_1$$

From triangle BQQ''

$$h_2 = (b + D) \tan \alpha_2$$

From the above fig., $\mathbf{h_1 - h_2 = S_2 \sim S_1 = S}$

Therefore, $D \tan \alpha_1 - (b + D) \tan \alpha_2 = S$

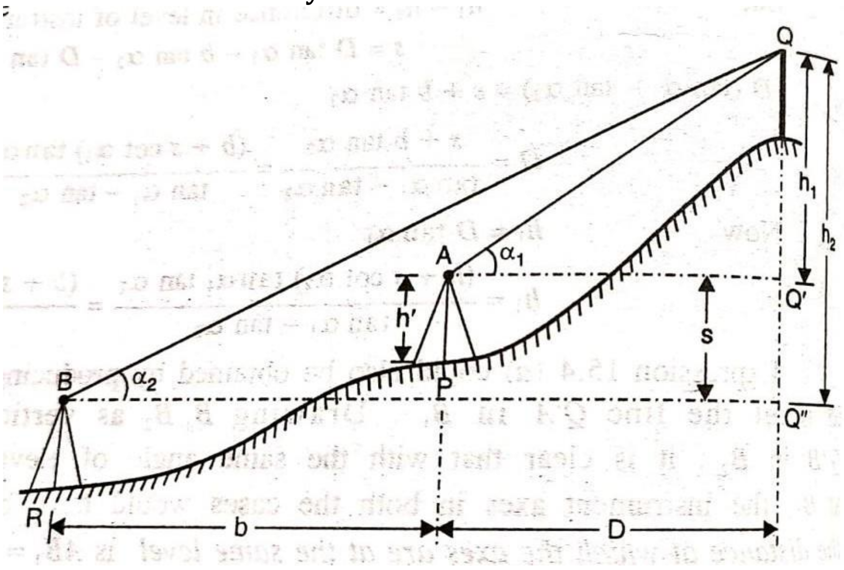
$$D \tan \alpha_1 - D \tan \alpha_2 = S + b \tan \alpha_2$$

Hence, $\mathbf{D} = \dfrac{\mathbf{S+b\tan\alpha 2}}{\mathbf{\tan\alpha 1 - \tan\alpha 2}}$

R.L. of Q = R.L. of B.M. + S_1 + h_1 (or)

R.L. of Q = RL of B.M. + S_2 + h_2

In case of instrument axes at very different levels:



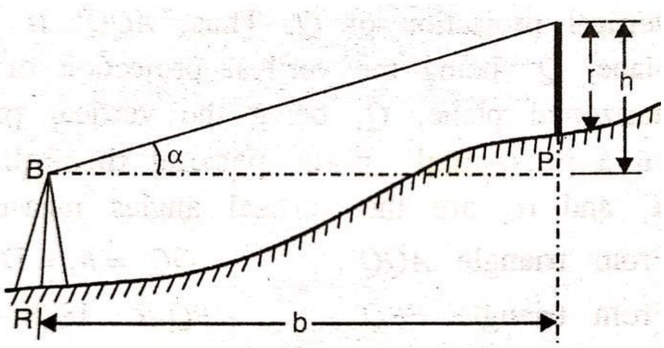
If $S_2 - S_1$ or S is too large to be measured on a staff kept at the B.M., the following procedure is adopted:

a) Set the instrument at P, level it accurately w.r.t. the altitude bubble and measure the angle α_1 to the point Q.

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

c) Shift the instrument to R. set the instrument and level it w.r.t. the altitude bubble and measure the angle α_2 to the point Q.

d) Keep the levelling staff at P and measure the angle α to the reading 'r'.



From the above fig., we have,

Height of station P above the axis at B = h – r = b tan α – r

Height of axis at A above the axis at B = S = b tan α – r + h'

Where, h' is the height of the instrument at P

Let, S = Difference in level between the two axes at A and B.

From triangle AQQ'

$$\mathbf{h_1 = D \tan \alpha_1}$$

From triangle BQQ''

$$h_2 = (b + D) \tan \alpha_2$$

$$h_2 - h_1 = S = (b + D) \tan \alpha_2 - D \tan \alpha_1$$

$$D \tan \alpha_1 - D \tan \alpha_2 = b \tan \alpha_2 - S$$

Hence, $\mathbf{D = \dfrac{b \tan \alpha 2 - S}{\tan \alpha 1 - \tan \alpha 2}}$

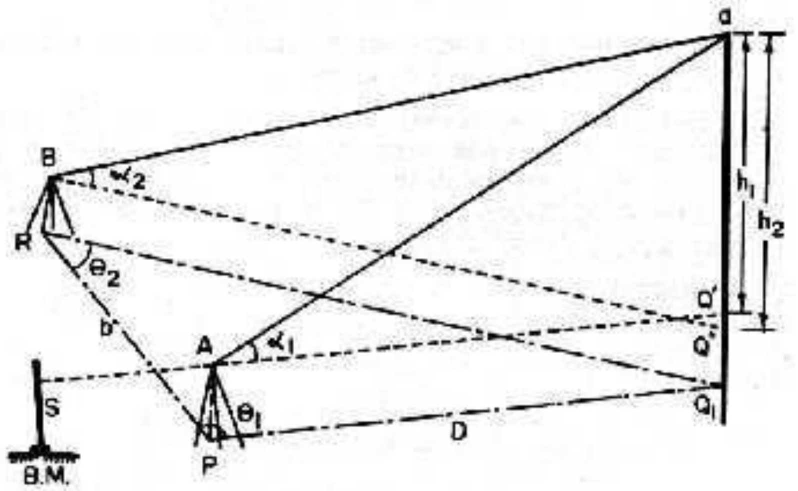
and **R.L. of Q = R.L. of A + h_1 = R.L. of B + S + h_1**

= (R.L. of B.M. + Backsight taken from B) + S + h_1

Where, **S = b tan α – r + h'**

Case 3: Base of the object inaccessible (Double Plane Method)

To find the R.L. of the top of an object, when the base of the object is inaccessible and the instrument stations are not in the same vertical plane as the elevated object.



Procedure:

a) Setup the instruments at P and level it accurately w.r.t. the altitude bubble. Bisect the point Q and measure the angle of elevation 'α_1'.
b) Sight to point R with reading on horizontal circle as zero and measure the horizontal angle RPQ_1 (θ_1) from P.
c) Take a back sight 'S' on the staff kept at B.M.
d) Shift the instrument to R and measure 'α_2'and 'θ_2' from R.
e) Measure the distance 'b' between two instrument stations R & P

Let,

Q'= projection of Q on the horizontal line thought A,

Q''= projection of Q on the horizontal line thought B,

AQ'= horizontal line though A,

BQ''= horizontal line though B,

AQQ' and BQQ'' are the two vertical planes,

PRQ_1 is a horizontal plane

θ_1= Horizontal angle measured at P,

θ_2= Horizontal angle measured at R,

α_1= Vertical angle measured at A,

α_2= Vertical angle measured at B.

From triangle AQQ'

$$\mathbf{h_1 = D \tan \alpha_1}$$

From triangle BQQ''

$$\mathbf{h_2 = RQ_1 \tan \alpha_2}$$

From triangle PRQ_1, angle $PQ_1R = \theta_3 = 180° - (\theta_1 + \theta_2)$

By applying sine rule,

$(PQ_1/\sin \theta_2) = (RQ_1/\sin \theta_1) = (RP/\sin \theta_3)$

Where, $RP/\sin \theta_3 = b/\sin \{180° - (\theta_1 + \theta_2)\} = b/\sin (\theta_1 + \theta_2)$

Therefore, $\mathbf{PQ_1 = D = b \sin \theta_2 / \{\sin (\theta_1 + \theta_2)\}}$

$\mathbf{RQ_1 = b \sin \theta_1 / \{\sin (\theta_1 + \theta_2)\}}$

Observations and Calculations:

Vertical Angles, α_1 = ………., α_2 = …………

Horizontal Angles, θ_1 = ………., θ_2 = …………

Staff Reading on B.M., S = ………..

Horizontal distance between P & R = b = ………..

$\mathbf{h_1 = D \tan \alpha_1}$

$\mathbf{h_2 = RQ_1 \tan \alpha_2}$

R.L. of Q = R.L. of B.M. + $\mathbf{S_1 + h_1}$ **(or)**

R.L. of Q = R.L. of B.M. + $\mathbf{S_2 + h_2}$

Module-02

Unit-02

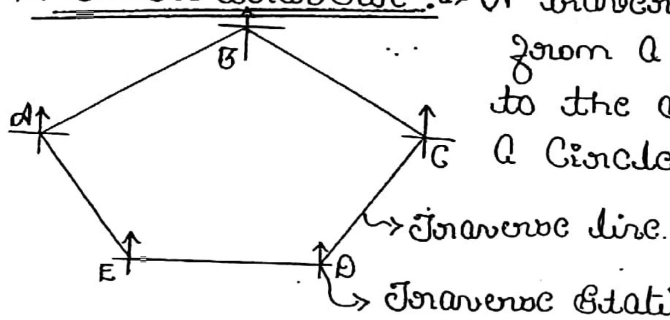
Traversing

"A Series of Connected Straights line each joining two points on the ground." is called travelling.

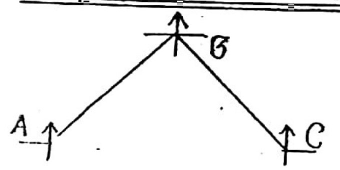
End points of the traverse leg are Called as Traverse Station.

Types of Traverse.

1> Closed traverse :-> A traverse which originates from a station and returns to the same Station Completing a Circle.



2> Open traverse :-> A traverse which neither returns to its starting station nor Closes on any other known station is Called Open traverse.



Based upon the instruments used traverse Can be Classified as,

1> Chain traversing.
2> Compass traversing.
3> Plane table traversing.
4> Theodolite traversing.
5> Tacheometric traversing.

Latitude (L) and Departure (D).

The latitude of a survey line may be defined as its Co-ordinate length measured parallel to an Assumed meridian Direction (i.e true north or magnetic north).

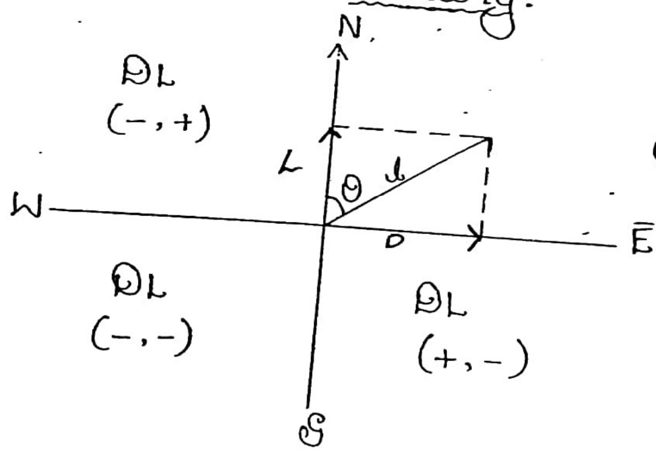
The departure of survey line may be defined as its Co-ordinate length measured at right angles to the meridian direction.

The latitude of the line is positive when measured northward (or upward) and is termed as northing.

The latitude is negative when measured Southward (or downward) and is termed as Southing.

Departure of the line is positive when measured Eastward and is termed as Easting.

Departure is negative when measured Westward and is termed as Westing.



$L = l\cos\theta$

$D = l\sin\theta$

Northern → +ve
Southern → −ve
Eastern → +ve
Western → −ve

W.C.B	R.B	Sign of	
		Latitude	Departure
0° to 90°	NθE	+	+
90° to 180°	SθE	–	+
180° to 270°	SθW	–	–
270° to 360°	NθW	+	–

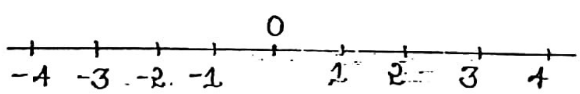
Co-ordinates :- The Co-ordinates of a point are a pair of number that define its exact location on a two-dimensional plane.

Co-ordinate plane has two axes at right angles to each other called X and Y-axis.

The Co-ordinates of a given point represent how far along each axis the point is located.

Imp ** Rectangular Co-ordinates or Cartesian Co-ordinates.

A Co-ordinate is a number which labels a point on a line.

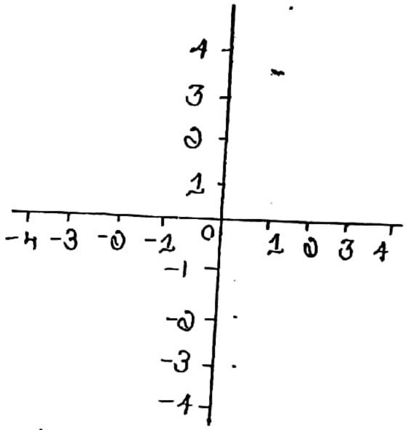


The Co-ordinates zero is called the origin of the Co-ordinates. The distance to the right of zero are labelled with positive Co-ordinates:- 1, 2, 3....

Distance to the left are labelled with negative Co-ordinates :- -1, -2, -3....

Each Co-ordinate is the representation of distance and direction from 0 (zero).

A Co-ordinate axis is a line with Co-ordinates. To label a point in a plane we will need one more axis at right angle to the first.



(To label a point in a plane we will need two Co-ordinates Axis $\perp^r$ to each other)

Distance above origin will have +ve Co-ordinates and distance below ~~bettom~~ will have -ve Co-ordinates. Those Axes are Called rectangular Co-ordinate axis. as they are at right angles (or $\perp^r$ to One another). The Co-ordinates on them are Called rectangular Co-ordinates. They are also Called Cartesian Co-ordinates.

Closing Error.

When a Closed traverse is plotted, the endpoint of the last line may not Co-incide with the starting point. This error is Called "Closing Error."

Closing error may occur due to the Combination of errors in angular and linear measurements.

Error of Closure (or) Closing error may be found out by finding ΣL and ΣD.

If $\Sigma L = 0$ and $\Sigma D = 0$

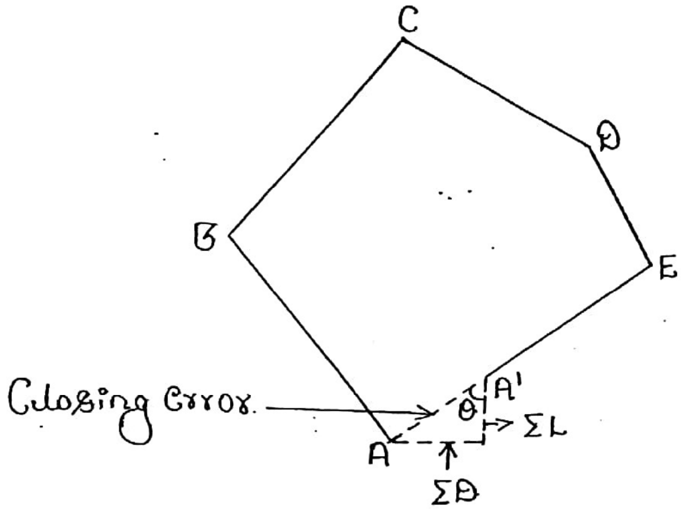
Closing error is zero.

If not

$$\text{Closing error } (C) = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$$

Where, C = value of Closing Error.
Direction of Closing Error either with North (or) South is given by.

$$\tan\theta = \frac{\Sigma D}{\Sigma L} \quad (\text{or}) \quad \theta = \tan^{-1}\left(\frac{\Sigma D}{\Sigma L}\right)$$



Balancing the Traverse.

The term balancing the traverse is generally applied to the operation of applying Correction to latitude and departure. So that $\Sigma L = 0$ and $\Sigma D = 0$.

This applies only When the Survey falls a Closed traverse.

The following were Common methods of balancing (adjusting) the traverse,

1) Bowdish Method.
2) Transit Method.
3) Bowdish Graphical Method.
4) Axis Method.

1. Bowdish Method :→ Correction to latitude of the line is given by,

$$C_L = -\Sigma L \times \frac{l}{\Sigma l}$$

Where,

C_L = Correction to the line.

ΣL = Total error in latitude.

l = Length of the line.

Σl = Perimetre of the traverse.

Correction to departure of the line is given by,

$$C_D = -\Sigma D \times \frac{l}{\Sigma l}$$

Where, C_D = Correction to the line.

ΣD = Total error in Departure.

2. Transit Method :→ According to transit rule, Correction to latitude of line is given by,

$$C_L = -\Sigma L \times \frac{L}{L_T}$$

Where,

ΣL = Total error in latitude.

L = Latitude of the line (+ve value)

L_T = Arithmetic sum of the latitude.

Correction to departure of line is given by,

$$C_D = -\Sigma D \times \frac{D}{D_T}$$

Where,

ΣD = Total error in Departure.

D = Departure of line (+ve value)

D_T = Arithmetic sum of the Departure.

Survey-1
'C' Secⁿ (1)

Tachometry

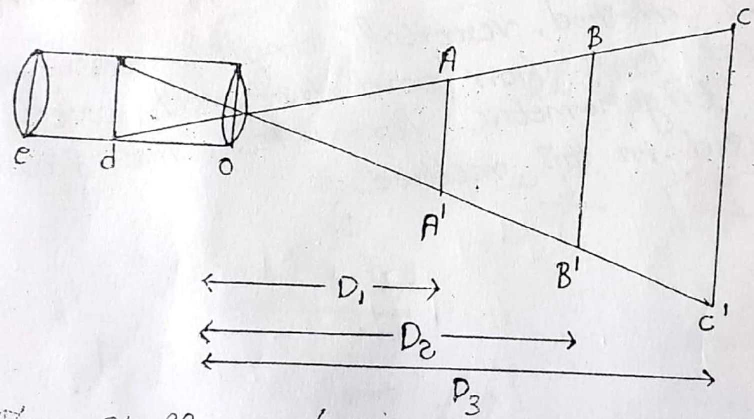
Tachometry, also known as tachymetry, is a branch of Surveying in which both horizontal and vertical distances are measured without the use of a chain or tape

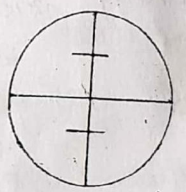
Basic principle of Stadia Tachometry

Tachometry is based on the principal known as Stadia Surveying. The term 'Stadia' comes from the Stadia diaphram of a theodolite which has three horizontal cross hairs.

→ The reading against the middle hair is used for finding difference in elevation.

→ The readings against the top and bottom hairs are used to find horizontal distance.





Stadia hairs

e → Eye piece
d → Diaphragm
o → objective lens

The staff readings at the different distances from the instrument will be proportional to these distances. The triangles formed will be isosceles and similar. The distances D_1, D_2, D_3 to the object from the instrument are proportional to the intercepts AA', BB' &c

In general $D = ks + C$

where k = Multiplying Constant
C = Additive constant

Tachometric methods

There are two basic methods of tachometry.
i) Stadia method and ii) Tangential method.

i) Stadia Method
There are two forms of the stadia method
(a) fixed hair stadia method (b) Movable hair (substance bar) - method

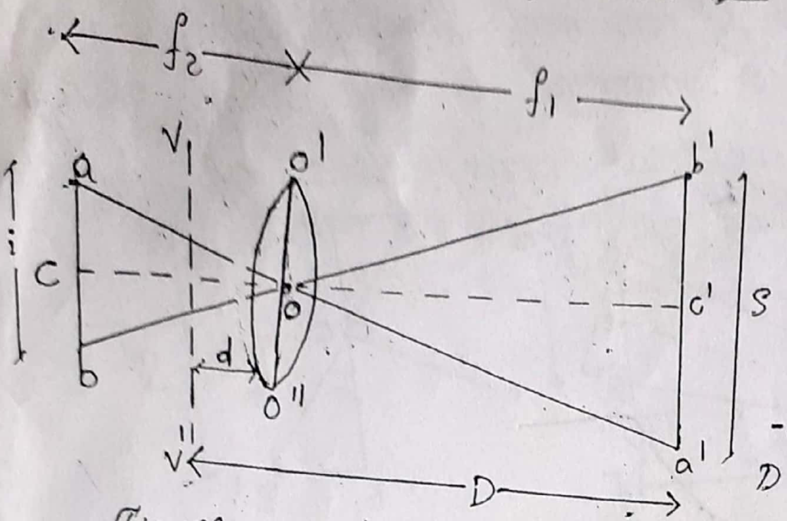
(a) fixed hair stadia method
① In this method the distance among the stadia - hairs is kept constant ② The staff intercept varies as the distance varies. ③ This method is most commonly used as it is convenient to take the staff readings.

(b) Movable hair method
① In this method a special diaphragm which provides the facility of changing the distance among the cross hairs will be used. ie ② the staff intercept is kept constant even though the distance varies ③ This method is generally not used, as it is inconvenient to measure the - staff reading

ii) Tangential Method
In tangential method, vertical angles are measured from the central cross hair and distances are calculated using trigonometric formulae. The stadia hairs are not used in this method.

Distance and elevation formula for horizontal line of sight

(2)



a → Top cross hair
b → Bottom cross hair
c → central cross hair
i → Stadia intercal
V-V' → vertical axis of the instrument
d → Distance b/w objective lens & vertical axis
D → Distance b/w vertical axis and the staff

Triangle oab & triangle oa'b' are similar

f_1 & f_2 → Conjugate focal lengths.

f → focal length of objective lens

S → staff intercept

$$\frac{S}{i} = \frac{f_1}{f_2}$$

from the lens formula

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2}$$

x by f_1

$$\frac{f_1}{f} = 1 + \frac{f_1}{f_2} = 1 + \frac{S}{i} \quad \left\langle \frac{S}{i} = \frac{f_1}{f_2} \right\rangle$$

↓

from similar triangles oab & ob'a'

$$f_1 = \frac{(f \times i) + Sf}{i}$$

$D = f_1 + d$ ⟨from fig⟩

$$D = \frac{(f \times i) + Sf}{i} + d$$

$$D = f + \frac{S \times f}{i} + d$$

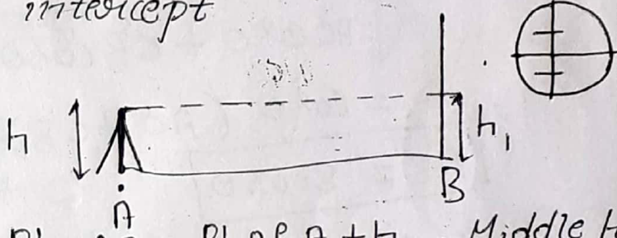
$$D = \left(\frac{f}{i}\right) \times S + (f + d)$$

$k = f/i$ = Multiplying constant

$c = (f+d)$ = Additive constant

$$\boxed{D = kS + c}$$

→ Distance equation



RL of B = RL of A + h − Middle hair reading

Distance and Elevation formula for inclined line of sight

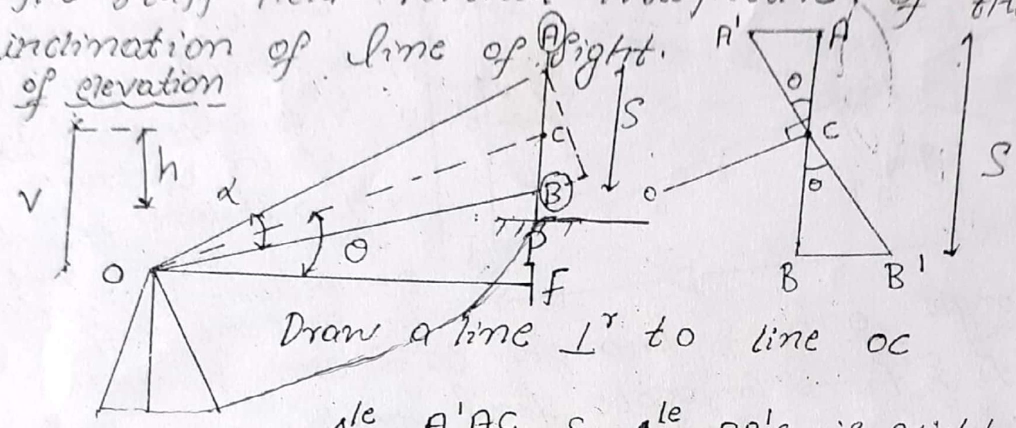
There are two cases of inclined line of sight

case i) The staff held is normal to the line of sight

case ii) The staff held vertical irrespective of the inclination of line of sight.

Angle of elevation

ii)



Draw a line $\perp^r$ to line OC

Δ^{le} A'AC & Δ^{le} BB'C is right angled.

$A'B' = A'C + CB'$

$= AC\cos\theta + CB\cos\theta$

$= \cos\theta\,(AC + CB)$

$\boxed{A'B' = S\cos\theta}$

RL of $O + V - h$ = RL of staff station P.

(L) Inclined length $OC = k(S\cos\theta) + C$

Horizontal Distance $D = L\cos\theta$ [OC = L]

↳ from Δ^{le} OCF [D = OF]

$V = L\sin\theta$

$V = K(S\cos\theta + C)\sin\theta$

$\boxed{D = k \times S \times \cos^2\theta + C\cos\theta}$

$V = KS \times \sin\theta \times \cos\theta + C\sin\theta$

$$\boxed{V = \frac{1}{2} \times KS \times \sin 2\theta + C\sin\theta}$$

Elevation of P can be determined from this expression

Where S = Staff intercept

θ = Angle of elevation

V = vertical distance from initial line of collimation to middle hair reading.

K = Multiplying constant

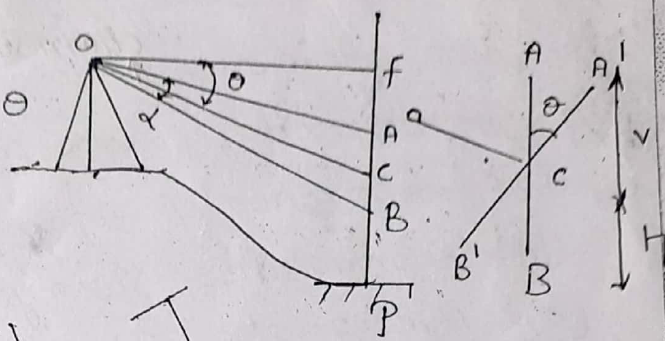
C = Additive constant

3

Angle of depression

$$D = ks\cos^2\theta + C\cos\theta$$

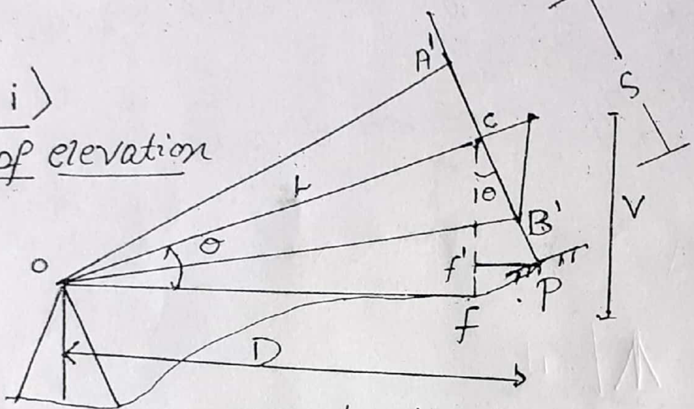
$$V = \left(\tfrac{1}{2}\right) ks\sin 2\theta + C\sin\theta$$



RL of P = RL of O − V − h

i) case i)

Angle of elevation



$OC = L = ks + C$

from Δ^{le} OCf

$Of = L\cos\theta = (ks + C)\cos\theta$

$D = L\cos\theta + f'P$ [from fig]

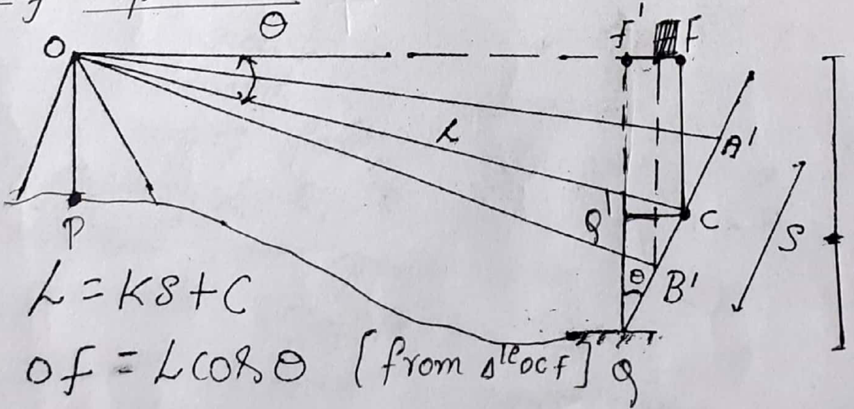
$D = L\cos\theta + PC\sin\theta$

$$\boxed{D = L\cos\theta + r\sin\theta}$$

Also from Δ^{le} OCf r = central hair reading

$$\boxed{V = L\sin\theta = (ks + c)\sin\theta}$$

Angle of depression



$L = ks + C$

$of = L\cos\theta$ [from Δ^{le} OCf]

$of = (ks + c)\cos\theta$

$$\boxed{D = of - f'f = (ks + c)\cos\theta - h\sin\theta}$$

$$\boxed{V = OC\sin\theta = (ks + c)\sin\theta}$$ {from Δ^{le} OCf

Advantage of Tacheometric Surveying

i) Speed of Surveying is very high
ii) Accuracy of Surveying is quite Satisfactory
iii) Not tedious as chain Surveying using chain, tapes, ranging rods
iv) The method is more advantageous in
(a). Preparation of topographical plans
(b) Reconnaisance Surveys for roads & railways
(c) Hydrographic Surveys.
(d) Checking already measured distance.

MODULE - 1

CURVE SURVEYING

Curves:

Curves are regular bends provided in the line of communication like roads, railways etc... and also in canals.

Arc:

Arc is the portion of the circumference of the curve.

Neccesity:

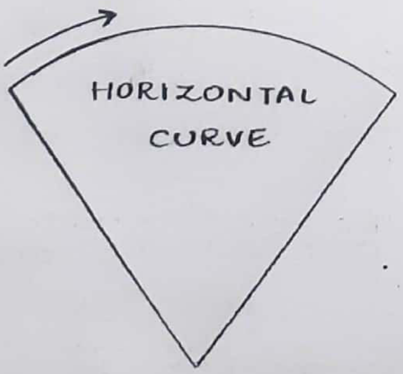
1. To bring about the gradual change of direction in horizontal plane.
2. Curves are provided in vertical plane to avoid abrupt change of grade.

Classification of curves

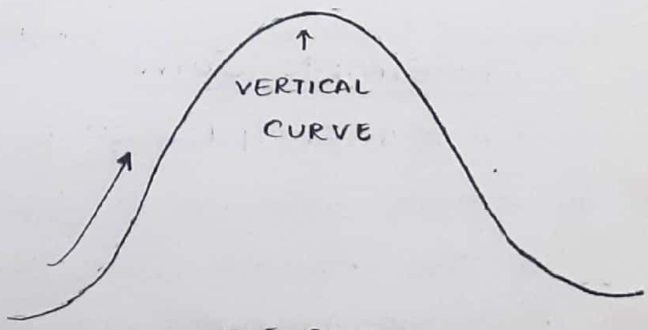
1. Based on Necessity

Horizontal curve: The curves provided in the horizontal plane to have change in direction are known as horizontal curve.

Vertical curve: The curves provided in the vertical plane to obtain the gradual change in the grade are known as vertical curve.



[TOP VIEW]



ELEVATION
[FRONT VIEW]

2. Based upon shape.

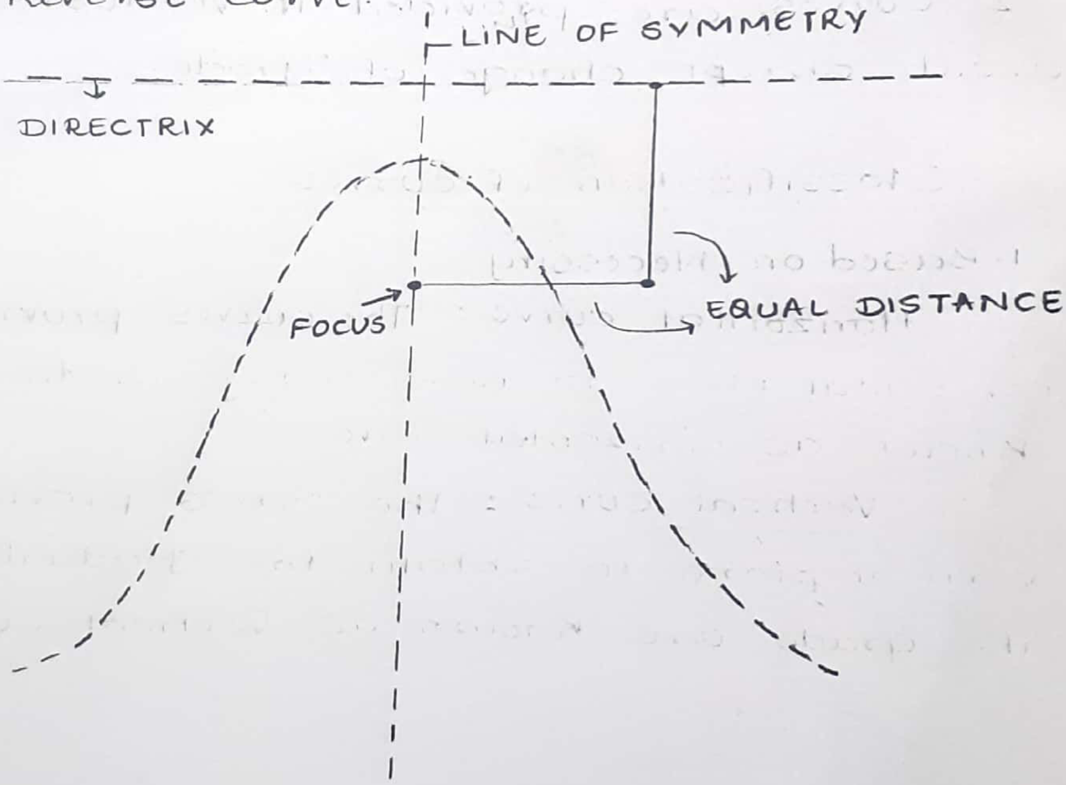
Circular curve : Circular curve having the shape of a circle.

Parabolic : Parabola is a curve where any point is at an equidistant form.

1. A fixed point [focus]
2. A fixed straight line [directrix]

Circular curves are further classified into three classes

1. Simple curve
2. Compound curve
3. Reverse curve.

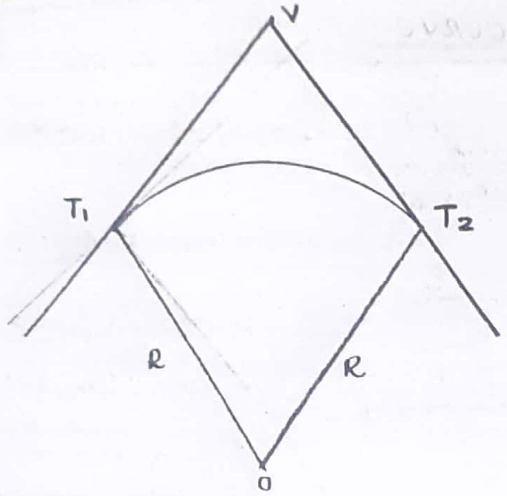


1. Simple Curve.

→ A simple curve is the one which consists of a single arc of a circle.

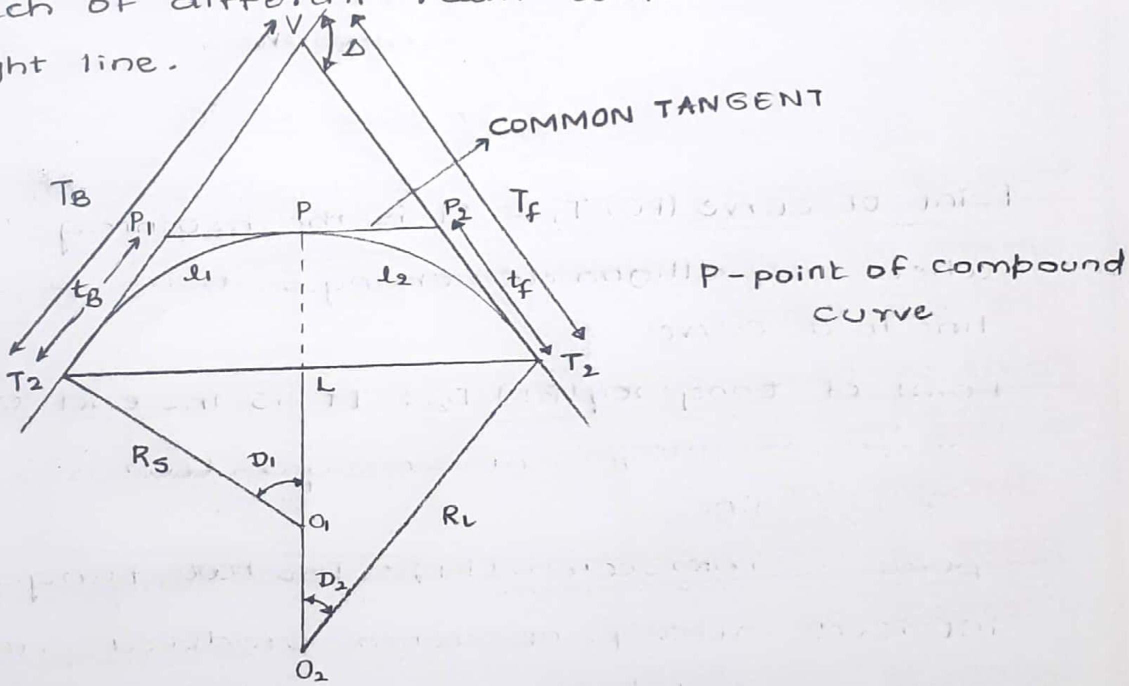
→ The curve will be tangential to both the straight lines

R = Radius of the curve.



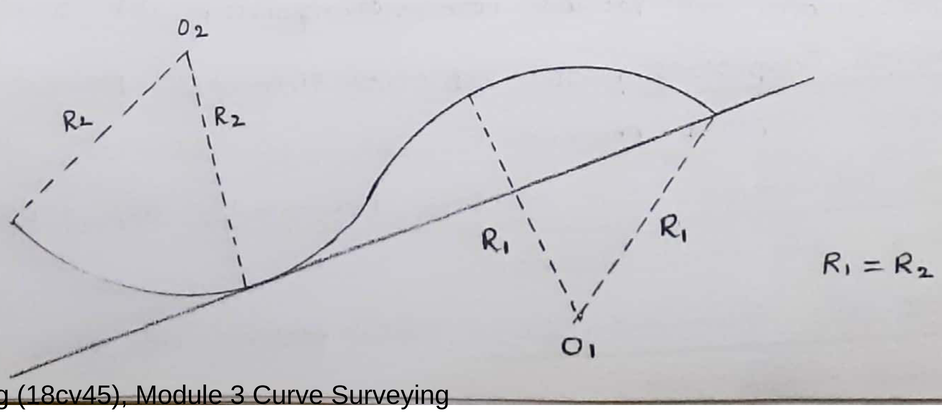
2. Compound curve

A compound curve is made up of 2 (or) more arcs each of different radii which lies between 2 straight line.

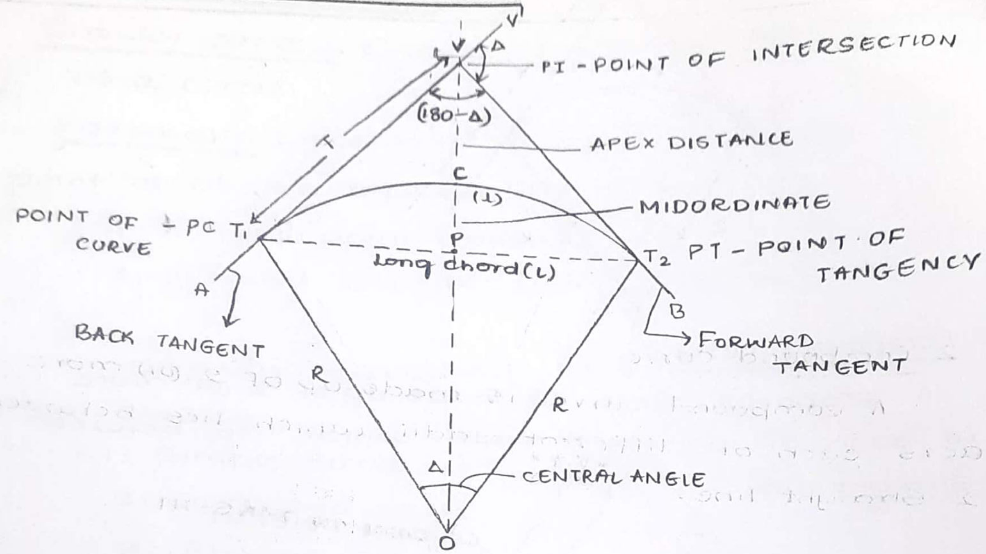


3. Reverse Curve

Reverse curve is the one which consists of two arcs of same (or) diffrent radii having their centers to the different sides of a common tangent.



ELEMENTS OF SIMPLE CURVE :



<u>Point of curve (PC) T_1</u> : It is the begining of the curve where the allignment changes from a straight line to a curve.

<u>Point of tangency (PT) T_2</u> : It is the end of the curve where the allignment changes from curve to the straight line.

<u>point of intersection (PI)</u> : The two tangent (AT_1 & BT_2) intersect at a point when produced and that point is called point of intersection.

<u>Tangent length (T)</u> : It is the distance between the point of curve to the point of intersection.

(or)

It is the distance between the point of intersection to point of tangency.

<u>Forward Tangent</u> : It is the tangent line at the end of the curve.

<u>Back Tangent</u> : It is the tangent line at the begining of the curve

<u>Apex (or) Summit (C)</u> : Mid point of the curve is called apex or summit

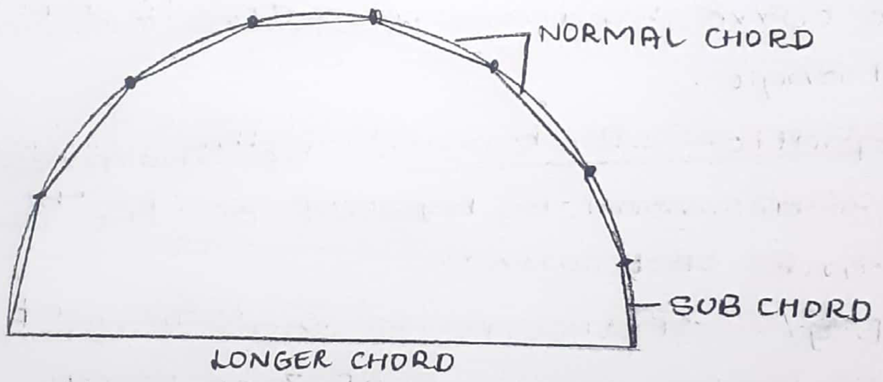
Apex distance (E) (or) External distance : It is the ordinate from mid point of the curve [c] to the point of intersection of two tangent [PI].

Mid ordinate (or) Versit sine. : It is the ordinate from mid point of the long chord (T_1PT_2) to the center of the curve (c).

Long chord (L) : It is the chord joining point of currvature to the point of tangency (L).

Normal chord : The chord between two successive regular stations on the curve is called normal chord.

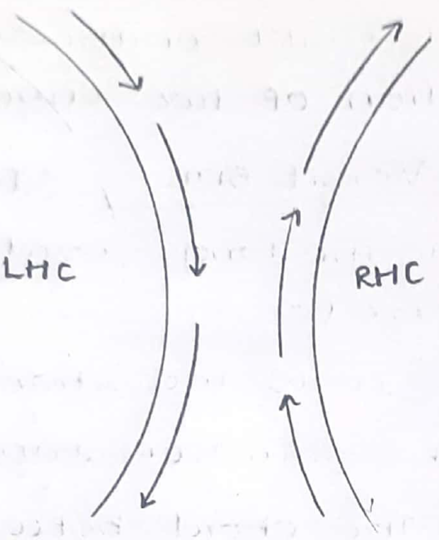
Sub chord : It is any chord shorter than normal chord.



External deflection angle (Δ) : The angle V'VB between the tangent AT_1 and other tangent is called External deflection angle.

Right hand curve : If the curve deflects to the right side in the direction of progress of survey the curve is called right hand curve.

Left hand curve : If the curve deflects to the left side in the direction of progress of survey the curve is called left hand curve.



TOP VIEW

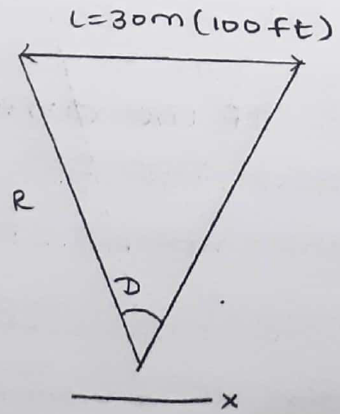
Intersection Angle : Angle between the two tangent is called as intersection angle.

Central Angle : The angle subtended at the centre of the curve by the arc T_1CT_2 is known as central angle.

Designation of the curve : The Sharpness of curve is either designation by radius of the curve (or) by the degree of the curve.

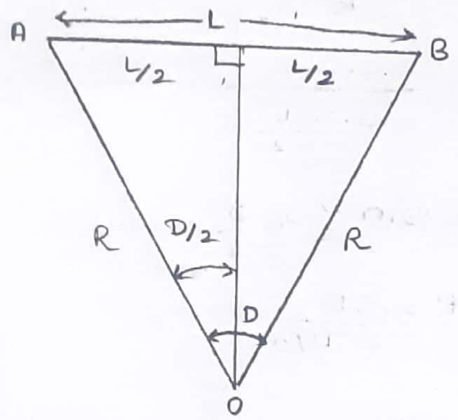
Degree of the curve is angle subtended at the centre by a chord of 30m length.

(100 Feet) 1m → 3.268 feet → 3.33 feet.



Relation between degree and radius of the curve

[D and R]



Let R = Radius of the curve.
D = Degree of the curve.

Arc definition.

For 100 ft,

$2\pi R \rightarrow 360°$

$100\,ft \rightarrow D$

$$\therefore R = \frac{360 \times 100}{2\pi D}$$

$$R = \frac{5730}{D}\ ft$$

For 30m,

$2\pi R \longrightarrow 360°$

$30m \longrightarrow D$

$$\therefore R = \frac{360 \times 30}{2\pi D}$$

$$R = \frac{1719}{D}\ m.$$

For 20m.

$2\pi R \longrightarrow 360$

$20m \longrightarrow D$

$$\therefore R = \frac{360 \times 20}{2\pi D}$$

$$R = \frac{1146}{D}\ m.$$

By chord definition:

For 100 ft, $\sin D/2 = \frac{50}{R}$

$$\therefore R = \frac{50}{\sin \frac{D}{2}}$$

But $\sin \frac{D}{2} \approx \frac{D}{2}$

$$R = \frac{50}{D/2}$$

$$R = \frac{50}{\frac{D}{2} \times \frac{\pi}{180}}$$

$$\left\langle R = \frac{5370}{D} \text{ ft} \right\rangle$$

For 30m $\sin \frac{D}{2} = \frac{15}{R}$

$$\therefore R = \frac{15}{\sin \frac{D}{2}}$$

$\sin \frac{D}{2} \approx \frac{D}{2}$

$$R = \frac{15}{\frac{D}{2}}$$

$$R = \frac{15}{\frac{D}{2} \times \frac{\pi}{180}}$$

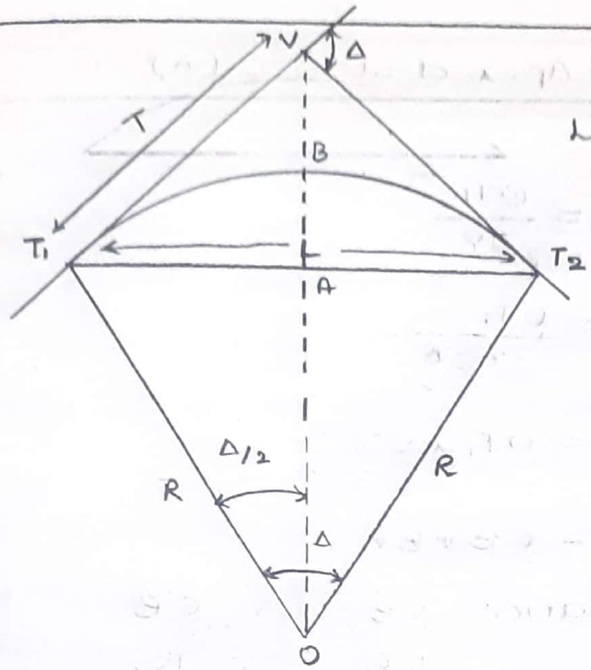
$$\left\langle R = \frac{1719}{D} \text{ m} \right\rangle$$

For 20m $\sin \frac{D}{2} = \frac{10}{R}$

$$R = \frac{10}{\sin \frac{D}{2}}$$

$$R = \frac{10}{\frac{D}{2} \times \frac{\pi}{180}}$$

$$\left\langle R = \frac{1146}{D} \text{ m} \right\rangle$$



L = length of chord

R = Radius of curve

D = Degree of curve

A = midpoint of curve

BA = midordinate

VB = Apex distance.

T = length of tangent

1. Expression For tangent length (T)

From ΔOT_1V,

$$\tan\frac{\Delta}{2} = \frac{T_1V}{OT_1}$$

$$\tan\frac{\Delta}{2} = \frac{T}{R}$$

$$\boxed{T = R \times \tan\frac{\Delta}{2}}$$

2. Expression for length of long chord (L).

Length of long chord, $L = T_1AT_2$

$$L = T_1A + AT_2$$

i.e., $L = 2T_1A$

From ΔOAT_1, $\sin\frac{\Delta}{2} = \frac{T_1A}{OT_1}$

$$T_1A = OT_1 \times \sin\frac{\Delta}{2}$$

$$T_1A = R\sin\frac{\Delta}{2}$$

$$L = 2T_1A$$

$$\boxed{L = 2R\sin\frac{\Delta}{2}}$$

3. Expression For Apex distance (A)

From ΔT_1OV

$$\cos\frac{\Delta}{2} = \frac{OT_1}{OV}$$

$$OV = \frac{OT_1}{\cos\frac{\Delta}{2}}$$

$$OV = OT_1 \times \sec\frac{\Delta}{2}$$

But $OV = OB + BV$

$\therefore$ Apex distance $VB = OV - OB$

$$VB = OV - R$$

$$\therefore VB = OT_1 \sec\frac{\Delta}{2} - R$$

$$= R\sec\frac{\Delta}{2} - R$$

$$\boxed{VB = R\left[\sec\frac{\Delta}{2} - 1\right]}$$

4. Expression For Midordinate (M):

From ΔOT_1A, $\cos\frac{\Delta}{2} = \frac{OA}{OT_1}$

$$OA = OT_1 \cdot \cos\frac{\Delta}{2}$$

$$OA = R\cos\frac{\Delta}{2}$$

From figure. Midordinate $(AB) = OB - OA$

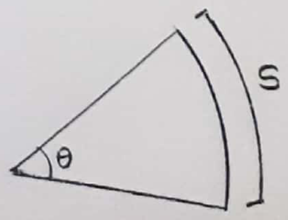
$$\therefore AB = R - R\cos\frac{\Delta}{2}$$

$$\boxed{AB(M) = R\left[1 - \cos\frac{\Delta}{2}\right]}$$

5. Expression for curve length (l):

W.K.T Arc length is given by

$$S = R\theta$$



From the figure.

length of the curve, $T_1BT_2 = l$

$$l = R \times \Delta \quad (\Delta \text{ in radian})$$

$$\boxed{l = R \times \Delta \times \frac{\pi}{180}} \quad (\Delta \text{ in degree})$$

Problems:

1. Calculate the elements of simple curve for the following data.

   Radius of the curve = 300 m

   Angle of intersection = 110 degree

   chainage of point of intersection = 2015.45m.

2. A simple circular curve is to have a radius of 573m. The tangent intersects at the chainage of 1060m. and the angle of intersection is 120° Find

   1. Tangent distance
   2. Chainage at begining and end of the curve
   3. Length of long chord.
   4. Degree of the curve.
   5. Number of Normal & Sub chord.

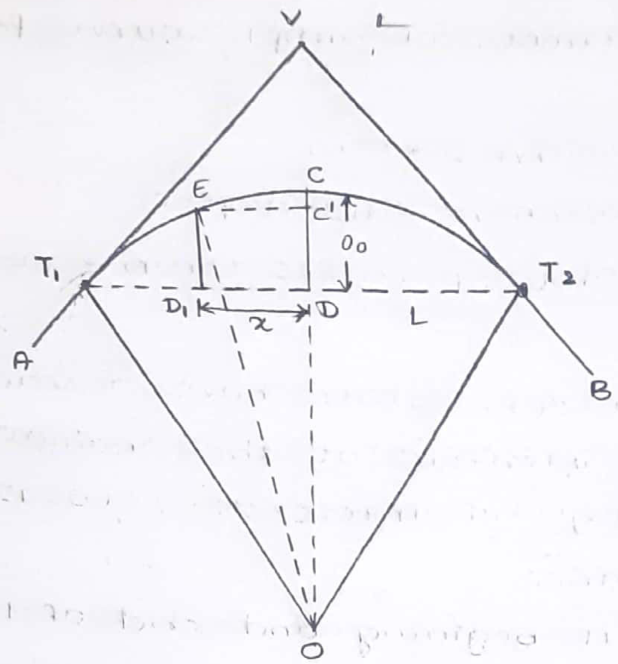
3. An arc of 30m and 2 degree curve conners 2 straight lines. Calculate.

   1. Radius of the curve
   2. Central angle
   3. Tangent length
   4. Length of long chord
   5. Mid ordinate
   6. Apex distance.

Linear methods of setting out simple circular curve.

1. offset from long chord [ordinate]
2. offset [ordinate] from chord produced
3. Succesive bisection of Arc
4. Radial offset Method.
5. Perpendicular offset Method

1. Offsets from long chord:



This method is suitable for setting of simple circular curve of smaller radius.

Let BV & AV — tangents

R – Radius of the curve.

L – length of long chord (T_1DT_2)

C – Apex or Summit

O_0 – mid ordinate [CD]

O_x – other ordinate [ED_1]

To calculate the value of midordinate (O_0)

From the fig. $CD = OC - DO$

where $OC = R$

$O_0 = R - DO$.

To calculate DO,

consider ΔOT_1D.

$$OT_1^2 = OD^2 + DT_1^2$$

$$OD^2 = R^2 - \left(\frac{L}{2}\right)^2$$

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

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

To calculate other ordinate $[O_x]$

From the fig,

$$O_x (ED_1) = OC_1 - OD$$

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

To calculate OC_1

From $\triangle OEC_1$

$$OE^2 = EC_1^2 + OC_1^2$$

$$OC_1^2 = OE^2 - EC_1^2 \quad [EC_1 = DD_1 = x]$$

$$OC_1 = \sqrt{R^2 - x^2}$$

$$\boxed{\therefore O_x = \sqrt{R^2 - x^2} - \sqrt{R^2 - (L/2)^2}}$$

Field procedure for setting out simple circular curve by offset from long chord:

* From the given data calculate radius of the curve (O_0 will be given)
* Set out the chain equal to the length of long chord bisect the long chord.
* At the mid point of long chord erect the perpendicular as offset [midordinate length] mark the point at every 'xm' interval from the centre of long chord on either sides.
* Erect the perpendicular offset at every 'x' meter interval to the distance to the distance calculated by using the formula.

$$O_x = \sqrt{R^2 - x^2} - \left[\sqrt{R^2 - (L/2)^2}\right]$$

Join all the offsets smoothely to get the required curve.

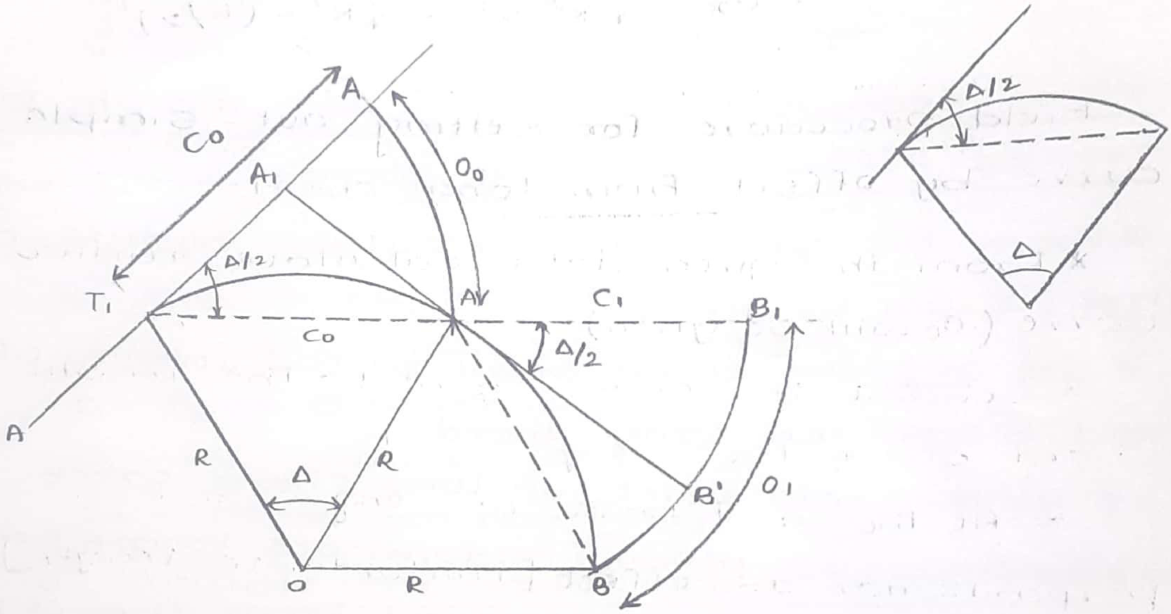
4

Problems:

1. Calculate the ordinates at 10m distances for a circular curve having long chord of 80m and midordinate of 4m.

2. Calculate the Number of Sub chord & Normal chord given Angle of intersection = 120°
Radius of the curve = 573 m
Chainage of point of intersection = 1060 m.

2. Offsets from chord produced.



Let R = Radius of the curve

C_0 = Length of 1st sub chord

$C_1, C_2, C_3 \ldots$ = length of normal chords

$O_1, O_2 \ldots$ = offsets

To calculate O_0

Consider sector A_1T_1A

Arc length $A_1A\,(O_0) = C_0 \times \frac{\Delta}{2}$ [Since $S = R\theta$] —(1)

consider the sector OT_1A

Arc length $T_1A = R \times \Delta$

Arc length T_1A = Chord length T_1A (C_0)

$$\Delta = \frac{C_0}{R}$$

Substituting value of Δ in equation (1)

$$\therefore O_0 = C_0 \times \frac{C_0}{2R}$$

$$\left\langle O_0 = \frac{C_0^2}{2R} \right\rangle \text{---}(2)$$

<u>To Findout the arc length</u>

$$B_1B'B\,(O_1) = \text{Arc } B_1B' + \text{Arc } B'B$$

From the Fig. an eq(2)

$$\text{Arc length } B'B = \frac{C_1^2}{2R}$$

<u>To find the arc length B_1B'</u>

$$\text{Arc length } B_1B' = C_1 \times \frac{\Delta}{2}$$

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

$$O_1 = \frac{C_1}{2}\left[\Delta + \frac{C_1}{R}\right]$$

By using $\Delta = \frac{C_0}{R}$

$$O_1 = \frac{C_1}{2}\left[\frac{C_0}{R} + \frac{C_1}{R}\right]$$

$$O_1 = \frac{C_1}{2R}(C_0 + C_1)$$

$$O_2 = \frac{C_2}{2R}(C_1 + C_2)$$

$$O_3 = \frac{C_3}{2R}(C_2 + C_3)$$

$$\vdots$$

$$\boxed{O_n = \frac{C_n}{2R}\left[C_{n-1} + C_n\right]}$$

Procedure for setting out simple circular curve by offsets from chord produced (or) offset from chord produced (or) deflection distance method.

* Locate tangent point T_1 on the straight T_1V. cut T_1A_1 equal to the length of 1st sub chord (C_0) already calculated

* With T_1 as a centre and T_1A as the radius swing the tape such that an arc of length O_1 is formed. Thus fix the 1st point A' on the curve.

* Keep the chain along T_1A' and pull it in that direction untill the length of normal chord is obtained from point A'.

* With A' as center A'B as radius from an arc of length O_2. Thus fix second point B' on the curve.

* Continue the process by repeating the above procedure till the end of the curve is reached.

Problems:

1. 2 tangents intersect at the chainage of 1190 m. The deflection angle will be 36°. calculate all data necessary for setting out curve.

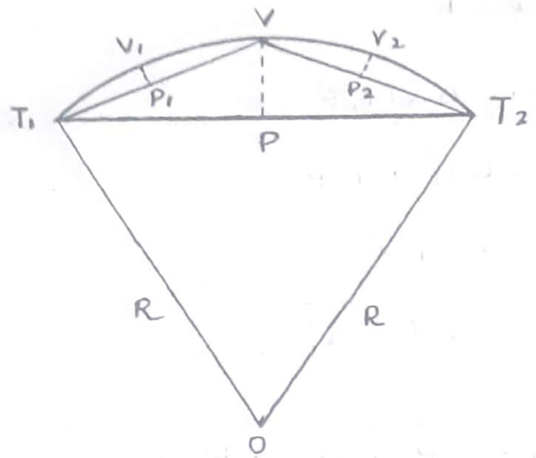
radius of the curve = 300m

peg interval is 30m,

prepare curve table, use offset from chord produced method.

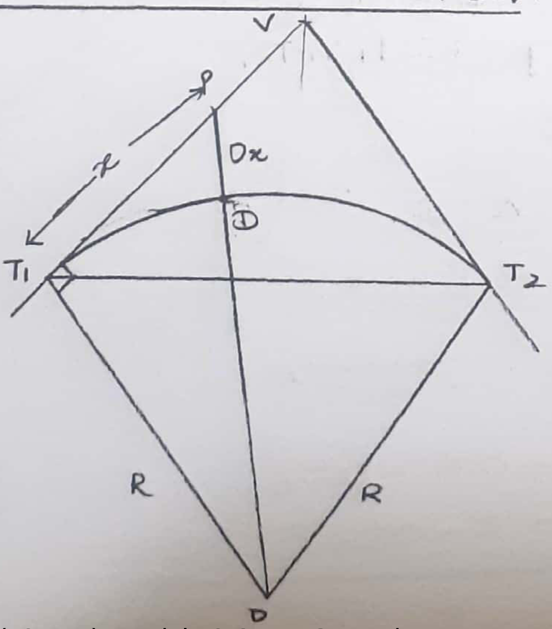
2. 2 strights intersect at a chainage [80+17]. The deflection angle is 28°. calculate all the necessary data for setting out simple curve by the method of offset from chord produced peg interval being

3. By successive bisection of arc :



* This method is also known as versit sine method.
* Stretch the chain for the length of T_1T_2. Bisect the length T_1T_2 and name it as P.
* Set out the offset VP equal to the length of $R(1-\cos D/2)$ Thus fixing the point V of the curve.
* Join T_1V and T_2V and bisect them at P_1 and P_2 set out offset V_1P_1 and V_2P_2 at P_1 and P_2 equal to the length of $R(1-\cos D/2)$. Thus fixing 2 more point V_1 and V_2 of the curve.
* By repeating this process set out as many as points required and join them to form a curve.

4. By Radial offset method.



Radial offset PD (O_x)

$$PD = PO - OD$$

where $OD = R$

To find PO From ΔOT_1P

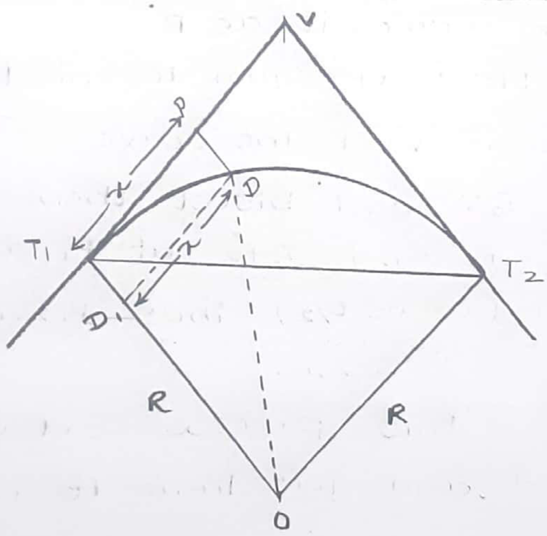
$$OP^2 = OT_1^2 + T_1P^2$$

$$= R^2 + x^2$$

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

$$\boxed{\therefore O_x = PD = \sqrt{R^2+x^2} + R.}$$

5. Perpendicular offset Method.



Perpendicular offset PD (O_x)

From the fig $PD \parallel T_1D_1$

To calculate T_1D_1

From ΔOD_1D

$$R^2 = OD_1^2 + x^2$$

$$OD_1 = \sqrt{R^2 - x^2}$$

But $T_1D_1 = OT_1 - OD_1$

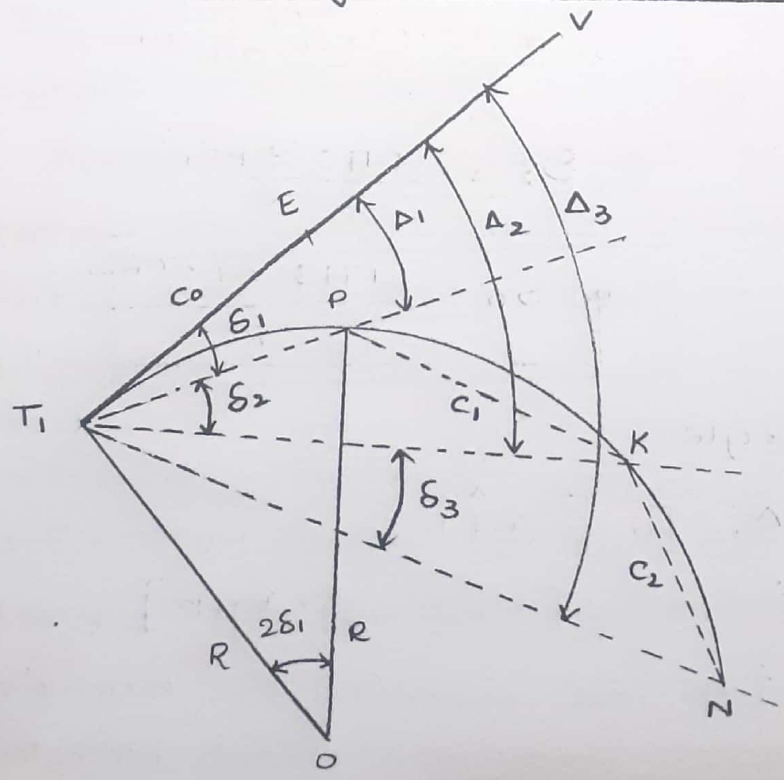
$$T_1D_1 = R - OD_1$$

$$\boxed{\therefore O_x = T_1D_1 = R - \sqrt{R^2 - x^2}}$$

Procedure for setting out simple circular curve by Radial offset method and perpendicular offset method:

* Locate the tangent point T_1 & T_2.
* Lay the chain along the tangent length measure and fix offset interval on the tangent from T_1.
* Set out the offsets (O_x) calculated by any of the above methods, thus, obtaining the required point on the curve.
* continue the process until the apex of the curve is reached.
* Set out other half of the curve from the 2nd tangent point T_2.

Rankine's Method (or) Instrument Method of Setting of simple curve.



Let R - Radius of the curve.

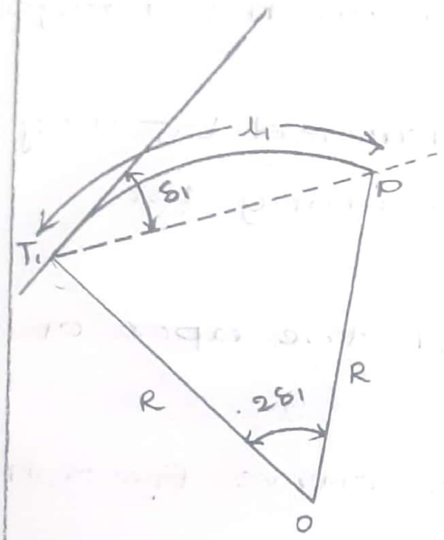
T_1V - length of back tangent

$C_0 \ldots\ldots C_n$ - Sub chord

$C_1, C_2 \ldots\ldots C_{n-1}$ - Normal chord

$\delta_1, \delta_2 \ldots\ldots \delta_n$ - Tangential angles

$\Delta_1, \Delta_2 \ldots \Delta_n$ - Deflection angles.



Consider the sector OT_1P

$$\frac{l_1}{2\delta_1} = \frac{2\pi R}{360}$$

$$\delta_1 = \frac{l_1 \times 360}{4\pi R}$$

$$\delta_1 = \frac{28.64\, l_1}{R} \text{ degree}$$

But $l_1 = C_0$

$$\delta_1 = \frac{28.64\, C_0}{R} \text{ degree.}$$

$$\delta_1 = \frac{28.64 \times 60 \times C_0}{R} \text{ min}$$

$$\delta_1 = \frac{1718.87 \times C_0}{R} \text{ min}$$

$$\delta_2 = \frac{1718.87 \times C_1}{R}$$

$$\boxed{\delta_n = \frac{1718.87 \times C_{n-1}}{R}}$$

Deflection angle

$\Delta_1 = \delta_1 \quad [V\hat{T_1}P]$

$\Delta_2 = \delta_1 + \delta_2 \quad [V\hat{T_1}P + P\hat{T_1}K]$

$\Delta_2 = \Delta_1 + \delta_2.$

$\vdots$

$$\boxed{\Delta_n = \Delta_{n+1} + \delta_n.}$$

Field procedure for Rankine's Method:

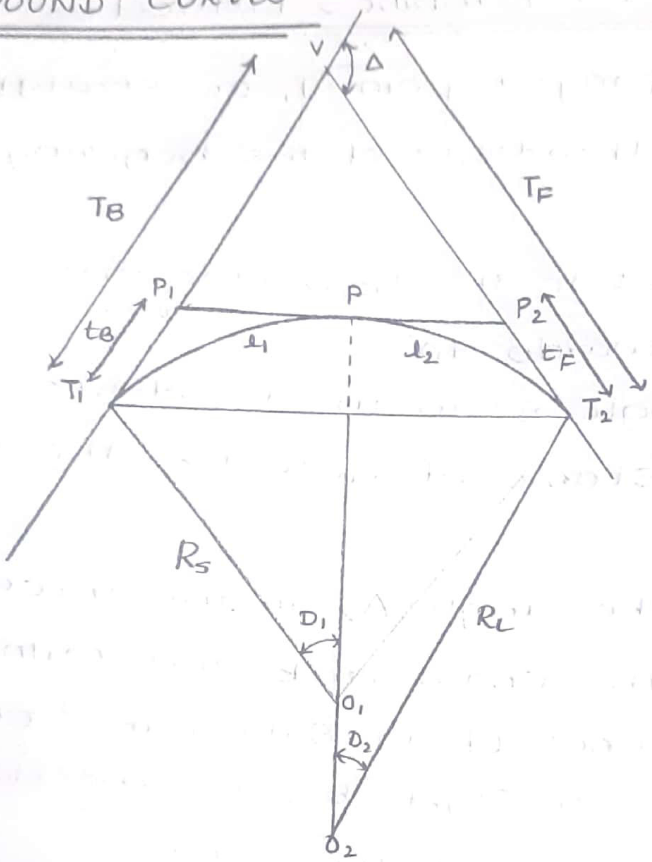
* Locate the tangent point T_1 on straight T_1V
* Set up the theodolite of the begining of the curve T_1.
* With 0°0' set in the horizontal circle direct the telescope towards T_1V
* Set the angle δ_1 in the theodolite and measure the distance of C_0 to fix the 1^{st} point of the curve B.
* Now set the angle Δ_2 in the theodolite and with 0 end of the chain at B and with an arrow head at a distance of C_1 swing the chain about B until the line of sight bisects the arrow. Thus, fix the 2^{nd} point C of the curve.
* Repeat the process untill the last point T_2 is reached.

Problems:

1. 2 tangents intersect at chainage 1190m. The deflection angle being 36°. Calculate all the data neccessary for setting out curve with radius of 300m. The peg interval being 30m. Prepare the curve table use Rankine's method.

2. Two straights intersect at a chainage [80+17]. The deflection angle is 28°. calculate all the necessary data for setting out simple curve by the method of Rankine for setting simple curve. Peg interval being 30m and radius of curve is 200m.

COMPOUND CURVES



P – point of compound curve

Let R_S – Radius of small curve.

R_L – Radius of large curve,

O_1 – center of small curve.

O_2 – center of large curve.

t_B – tangent for smaller curve,

t_F – tangent for large curve.

T_B – Back tangent of compound curve (T_1PT_2)

T_F – Forward tangent of compound curve.

l_1, l_2 – Arc length of smaller & larger curve.

Δ – External deflection angle.

D_1 and D_2 – External deflection angles for smaller and larger curve.

Arc length of 1st curve, $l_1 = \frac{\pi R D_1}{180}$

Arc length of 2nd curve, $l_2 = \frac{\pi R D_2}{180}$.

Compound curve length

$$l = l_1 + l_2$$

$$l = \frac{\pi}{180}\left[R D_1 + R D_2 \right]$$

length of long chord (L)

$$L = L_1 + L_2$$

where $L_1 = 2R_S \sin\left(\frac{D_1}{2}\right)$

$$L_2 = 2R_L \sin\left(\frac{D_2}{2}\right)$$

$$\therefore L = 2\left[R_S \sin\left(\frac{D_1}{2}\right) + R_L \sin\left(\frac{D_2}{2}\right)\right]$$

Tangent length for 1st curve.

$$t_B = R_S \tan\left(\frac{D_1}{2}\right)$$

Back tangent length of compound curve (T_B)

$$T_B = t_B + P_1V$$

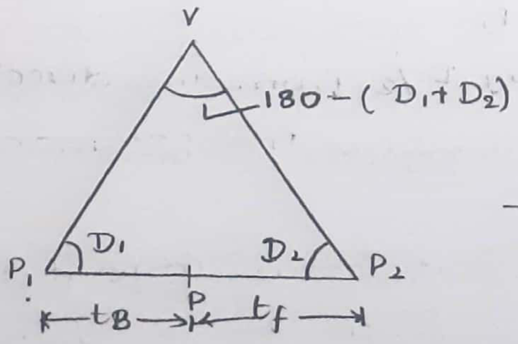
Forward tangent length of compound curve (T_f)

$$T_f = t_f + P_2V$$

Tangent length of 2nd curve

$$t_f = R_L \tan\left(\frac{D_2}{2}\right)$$

<u>To calculate P_1V and VP_2.</u>



Consider ΔVP_1P_2

By sine rule.

$$\frac{P_1V}{\sin D_2} = \frac{P_1P P_2}{\sin(180-(D_1+D_2))}$$

$$P_1V = \frac{(t_f + t_B)\sin D_2}{\sin[180-(D_1+D_2)]}$$

III'y $$\frac{VP_2}{\sin D_1} = \frac{t_B + t_f}{\sin[180-(D_1+D_2)]}$$

$$\therefore VP_2 = \frac{(t_F + t_B)\sin D_1}{\sin[180 - (D_1 + D_2)]}$$

$$\boxed{\begin{aligned} \therefore T_B &= t_B + \frac{(t_F + t_B)\sin D_2}{\sin[180 - (D_1 + D_2)]} \\ T_F &= t_F + \frac{(t_F + t_B)\sin D_1}{\sin[180 - (D_1 + D_2)]} \end{aligned}}$$

If chainage of V (PI) is given by

Chainage of T_1 = chainage of V − Bank tangent length T_B

Chainage of point of compound curve

PCC = chainage of T_1 + length of small curve

chainage of T_2 = chainage of PCC + length of large curve.

Procedure:

* Set up the theodolite at T_1 and set out the 1st arc T_1P by rankein's method of deflection angle.
* Shift the theodolite and set up at point P.
* with the zero degree set in the horizontal circle assembly site point T_1
* Swing the telescope by $D_1/2$ from the direction of PT_1. The telescope will be along the common tangent
* Transit the telescope and now layout the 2nd arc PT_2 in the same manner till T_2 is reached.

Problems:

1\. Two straight lines AB & BC are intersect by line EF. The angle $B\hat{E}F$ and $B\hat{F}E$ are $40° 30'$ and $36° 24'$. The radius of 1st arc is 600m and 2nd arc is 800m. Chainage of PI is 8248.1m. Find the chainage of tangent point and chainage of point of compound curve.

2\. The following data refers to a compound circular curve.

1. Total deflection angle 93°
2. Degree of 1st curve = 4°
3. Degree of 2nd curve = 5°
4. point of intersection chainage [45+61] (20m units)

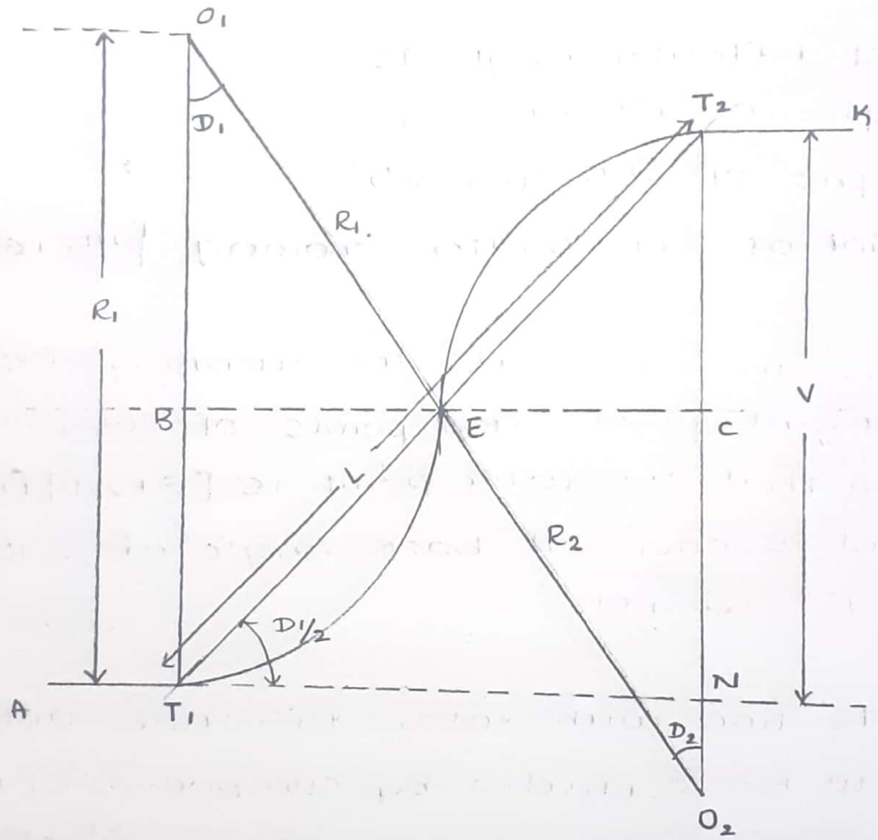
Determine in 20m units the running distance of the tangent points and points of compound curve, given that the later point is [6+24] from the point of intersection at back angle of $290° 36'$ from the 1st tangent.

3\. Two straights line with total deflection angle of $72° 30'$ are to be connected by compound curve of 2 branch of equal length. The radius of 1st arc is 350m and that of 2nd arc is 500m. Find the chainage of 2 tangent point and that of compound curve take chainage of vertex as 1525m.

4\. A compound curve consists of 2 simple curves of radii 350m and 500m is to be laib between 2 straights. The deflection angles are 55° and 45° for 1st and 2nd arc. calculate various elements of compound curve.

5. Two straight lines having deflection angles 76°, are to be connected by a compound curve, Radius of 1st arc is 500m and that of 2nd arc is 800m, The chainage of point of intersection is 7540m. Find the chainage of tangent points and point of compound curve. Deflection angle for 1st arc is 35°

Reverse Curve:



Let l_1 and l_2 be the length of first and second curve

- O_1 and O_2 be the center of first and second curve
- R_1 and R_2 be the radius of first and second curve
- D_1 and D_2 be the central angle corresponding to radii R_1 and R_2
- V - vertical distance b/w two straight lines AT_1 & T_2K
- L - sloping length b/w two tangent T_1 and T_2
- h - horizontal distance b/w two tangent points T_1 and T_2

To calculate Vertical distance (V)

$$T_2CN\ (V) = T_2C + CN \quad \text{——(1)}$$

$$CN = BT_1$$

To calculate BT_1

$$BT_1 = O_1T_1 - O_1B$$

From fig $O_1T_1 = R_1$

To calculate O_1B

From Δ O_1BE

$$\cos D_1 = \frac{O_1B}{O_1E}$$

$$O_1B = O_1E \times \cos D_1$$

$$O_1B = R_1 \times \cos D_1$$

$$\therefore BT_1 = R_1 - R_1\cos D_1$$

$$\boxed{CN = BT_1 = R_1[1 - \cos D_1]}$$

$$T_2C = O_2T_2 - O_2C$$

From the figure.

$$O_2T_2 = R_2$$

$$O_2C = ?$$

From Δ O_2CE

$$\cos D_2 = \frac{O_2C}{O_2E}$$

$$\cos D_2 = \frac{O_2C}{R_2}$$

$$O_2C = R_2 \times \cos D_2$$

$$T_2C = R_2 - R_2\cos D_2$$

$$\boxed{T_2C = R_2[1 - \cos D_2]}$$

$\therefore$ (1) becomes.

$$\boxed{V = R_2[1 - \cos D_2] + R_1[1 - \cos D_1]}$$

If $R_1 = R_2 = R$ and $D_1 = D_2 = D$

then, $V = R[1-\cos D] + R[1-\cos D]$

$$\boxed{V = 2R[1-\cos D]}$$

To find 'h'

$BEC\ (h) = BE + EC$ —— (2)

From ΔBO_1E

$$\sin D_1 = \frac{BE}{R_1}$$

$$BE = R_1 \sin D_1$$

From ΔEO_2C

$$\sin D_2 = \frac{EC}{R_2}$$

$$EC = R_2 \sin D_2$$

∴ (2) becomes

$$\boxed{\therefore h = R_1 \sin D_1 + R_2 \sin D_2}$$

If $R_1 = R_2 = R$ and $D_1 = D_2 = D$

$$h = R\sin D + R\sin D$$

$$\boxed{h = 2R\sin D}$$

To Find 'L'

$T_1ET_2\ (L) = T_1E + ET_2$

$$T_1E\ (L_1) = 2R_1 \sin\frac{D_1}{2}$$

$$ET_2\ (L_2) = 2R_2 \sin\frac{D_2}{2}$$

$$\boxed{\therefore L = 2R_1 \sin\frac{D_1}{2} + 2R_2 \sin\frac{D_2}{2}}$$

If $R_1 = R_2 = R$ then $D_1 = D_2 = D$

$$\boxed{L = 4R\sin\frac{D}{2}}$$

Relationship b/w V and h

From $\Delta\ T_1NT_2$

$$\tan\left(\frac{D_1}{2}\right) = \frac{T_2N}{T_1N} = \frac{V}{h}$$

$$\tan\left(\frac{D_1}{2}\right) = \frac{V}{h}$$

$$\boxed{\therefore V = h \times \tan\left(\frac{D_1}{2}\right)}$$

Relationship b/w V and L

From $\Delta\ T_1NT_2$

$$\sin\left(\frac{D_1}{2}\right) = \frac{T_2N}{T_1T_2}$$

$$\sin\left(\frac{D_1}{2}\right) = \frac{V}{L}$$

$$\boxed{\therefore V = L \times \sin\left(\frac{D_1}{2}\right)}$$

Problems:

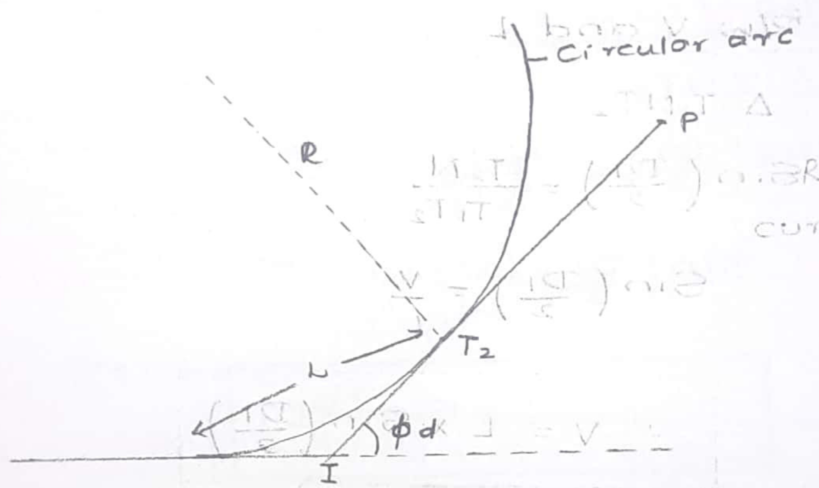
1- Two parallel railway lines are to be connected by a reverse curve each section having same radius. If the lines are 12m apart and maximum distance measured between two tangent point is 48m. Find

1. The maximum allowable radii
2. If both radii are to be different calculate the radius of second branch. if that of first branch is 60m.

Also calculate length of both the curves.

2. A reverse curve lies between two parallel straight lines with are 25m apart. It consists of two arcs of equal radius. If the distance between two tangent points is 220m. Find the radius also calculate offsets at 10m interval from the long chord to set out the first arc of reverse curve

Transition Curve



R - Radius of the curvature of the circular curve

L - Length of transition curve

ϕd - Sprial angle

S - Shift

A transition (or) easement curve is a curve of varying radius introduced between a straight line and a circular curve.

Neccesity of transistion Curve:

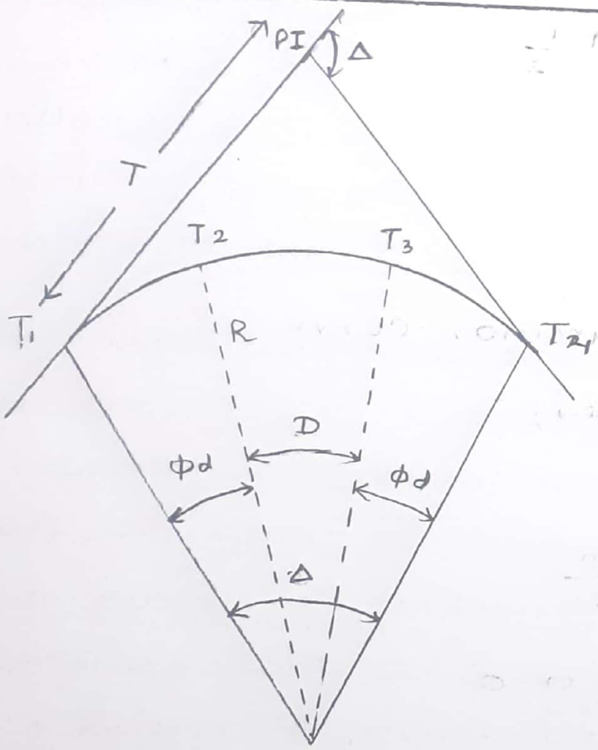
1. To accomplish gradually the transition from the tangent to the circular curve

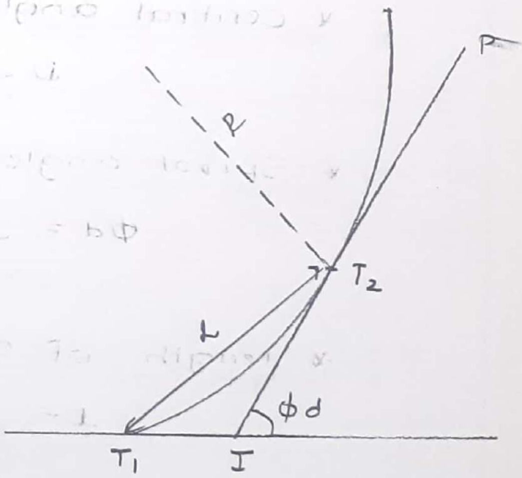
2. To provide medium for the gradual introduction of super-elevation

Requirements of transition curve:

* It should be tangential to the straight line.
* It should meet the circular curve tangentially
* Its curvature should be zero at the origin.
* The rate of increase of curvature along the transition curve should be same as that of increase of cant (or) super elevation.
* The length of transition curve must be such that cant can be provided conveniently.

Elements of transition curve:





1. Deflection angle (Δ)
2. Radius of circular curve (R)
3. Length of transition curve
4. Length of circular curve
5. Spiral angle (φd)
6. Central angle of circular curve
7. Shift
8. Total length of composite curve.

Chainage of begining of transistion curve (T_1)

Chainage of Junction @ begining of circular arc (T_2)

Chainage of other junction @ end of circular curve i.e. T_3

Chainage of end of composite curve (T_4)

* length of transision curve

$$L = \frac{V^3}{(\alpha \times R)}$$

* Tangent length

$$T = (R+S)\tan\Delta + \frac{L}{2}$$

* Shift $S = \dfrac{L^2}{24 \times R}$

* Δ = deflection angle.

* central angle of circular curve

$$D = \Delta - (2\phi_d)$$

* Spiral angle

$$\phi d = \frac{L}{2R} \times \frac{180}{\pi}$$

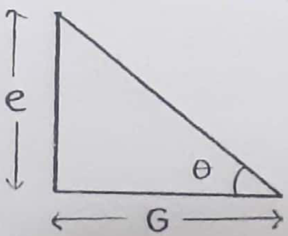
* length of circular arc

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

* length of composite curve = L + l + L

*

$$\frac{e}{G} = \frac{V^2}{gR}$$



where

V = Speed

G = gauge (or) Grade length

α = ratio of radial acceleration (m/s^3)

e = Super elevation (or) cant [m (or) cm]

$\frac{V^2}{gR}$ = Centrifugal Ratio.

g = acceleration due to gravity (m/s^2)

R = Radius of the curve.

Problems:

1. A transition curve is required for circular curve of radius 200m. the gauge being 1.5m and maximum super-elevation is restricted to 15cm. The transition curve is to be designed for a velocity such that no lateral pressure is imposed on the rails and the rate of gain of radial accelera-tion is $30cm/s^3$. Calculate the required length of transition curve and design speed.

2 The road bend which deflects 80° is to be designed for a maximum speed of 100 Km/hr. A maximum centrifugal ratio of 1/4 and maximum rate of gain of radial acceleration is $30cm/s^3$. The curve is consisting of a circular arc combined with cubic sprial [parabola]. calculate.

1. Radius of circular arc
2. The length of transition curve
3. Total length of composit curve
4. The chainage of begining and end of transition curve if chainage of PI is 10500 m.
5. Chainage of junctions of transition curve.

3 Two straight roads intersect at deflection angle $60^\circ 30'$ at chainage 3330m. The maximum speed of vehicle is 120 Km/hr. The centrifugal ratio is 1/4. The rate of change of radial acceleration is to be $0.5 \ m/s^3$.

Design the transition and circular curve. and Also find chainage of points at begining and at end of the curve.

Vertical Curve:

Vertical curve are need to connect two gradient lines to smoothen out the change from one gradient to another gradient.

Vertical curves are provided

1. To improve the visibility.
2. An abrupt Change of gradient is likely to Subject the vehicle to impact which could be dangerous. Thus, vertical curve are essential for Safety and comfort.

Vertical curves are classified as

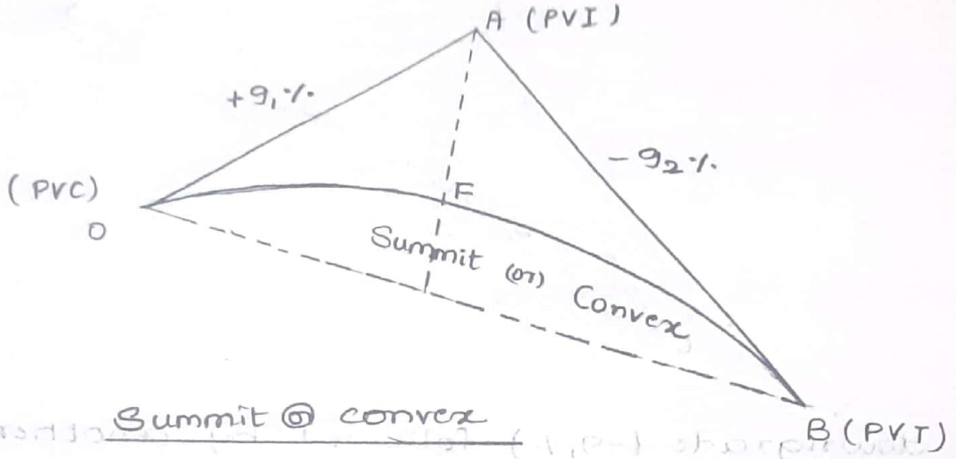
1. Sag curve [Valley curve]
2. Summit curve.

* Sag curves are used where the change in gradient is positive

* Summit curves are used where the change in gradient is negative.

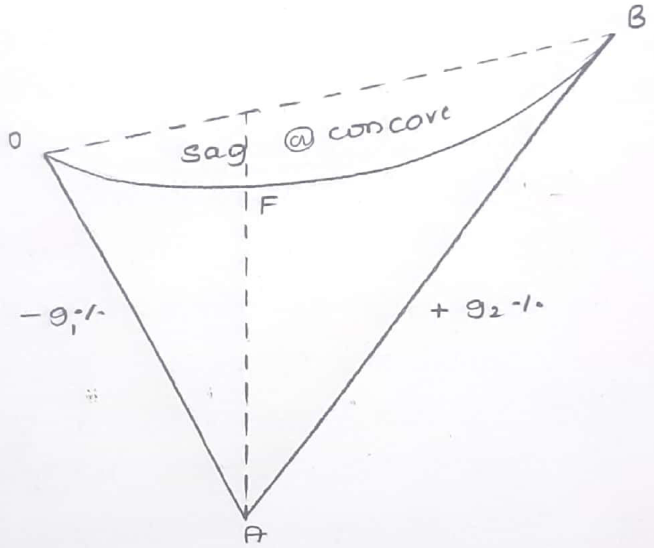
Types of Vertical Curve:

1. An upgrade ($+g_1$%) followed by a downgrade ($-g_2$%)

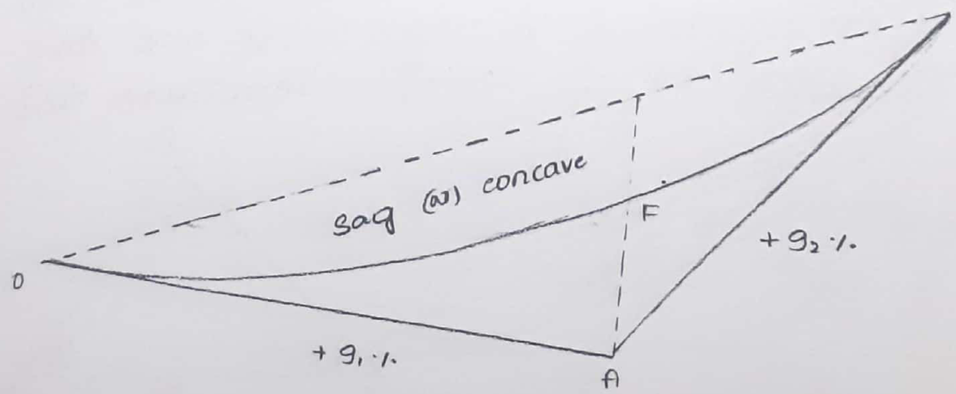


Summit @ convex

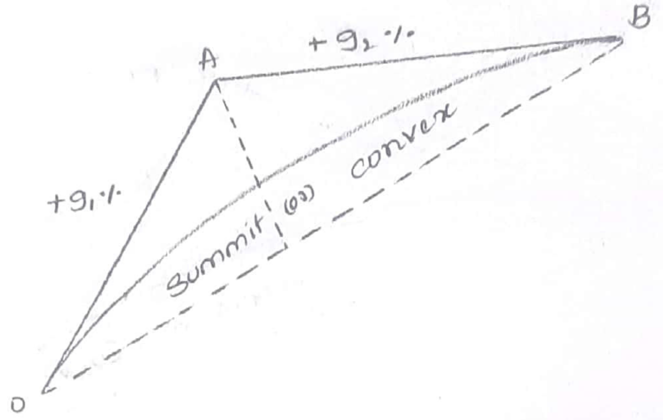
2. A downgrade ($-g_1$%) followed by an upgrade ($+g_2$%)



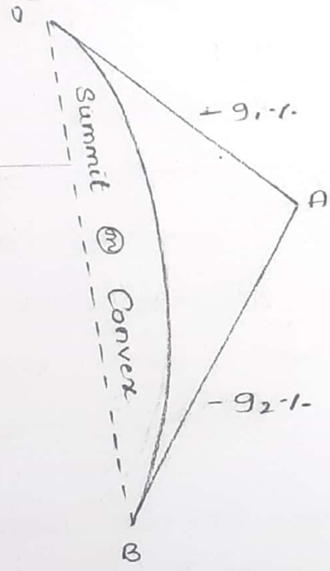
3. An upgrade ($+g_1$%) followed by another upgrade ($+g_2$%) i.e., ($g_2 > g_1$)



4. An upgrade ($+g_1\%$) followed by another upgrade ($+g_2\%$) i.e. ($g_1 > g_2$)



5. A downgrade ($-g_1\%$) followed by another downgrade ($-g_2\%$) i.e. ($g_2 > g_1$)



6. An downgrade ($-g_1\%$) followed by another downgrade ($-g_2\%$) i.e. [$g_1 > g_2$]