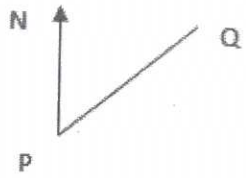
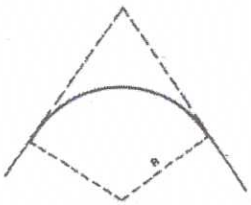
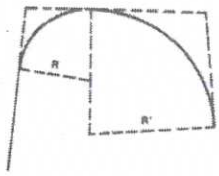
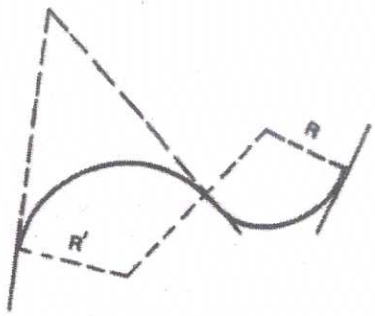
Scheme of Evaluation
(Scoring Indicator)

Revision: 2015 Course Code: 3012
Course Title: Surveying II

Question No	Scoring Indicator	Split up score	Sub Total	Total
I)	PART A			
1.	Line of sight can be reversed by revolving the telescope through 180° in the vertical plane (Transitted) – Transit theodolite Telescope cannot be transitted – Non transit theodolite	2	4	
2.	Departure of survey line – coordinate length measured at right angles to the meridian direction. Departure is positive when measured eastward and is easting. It is negative when measured westward and is termed as westing	2		10
3.	Case 1 : When bearing or length or both of one side is omitted Case 2 : When the length of one side and the bearing of another side are omitted Case 3 : When the lengths of two sides are omitted Case 4 : When the bearing of two sides are omitted	½ ½ ½ ½	2	
4.	By providing annalactic lens, the vertex is formed at the vertical axis and thus the stadia constant c = f+d is reduced to zero.	2	4	
5.	A transition or easement curve is a curve of varying radius introduced between a straight and a circular curve. A curve whose curvature increased gradually from zero to a specified value.	2		
II)	PART B			
1.	a. Axis of telescope : Imaginary line passing through the optic centre of the object glass and the optical centre of eye piece b. Swinging of telescope: Turning the telescope in a horizontal plane. Right swing – telescope is turned clockwise. Left swing – telescope is turned anticlockwise. c. Plunging: Transitting or reversing. The method of turning the telescope about its horizontal axis in a vertical plane through 180° .	2 2 2	6	6
2.	Theodolite should be provided with trough compass or tubular compass • Set the instrument at P and level it accurately • Set accurately the vernier A zero • Loose the lower clamp. Release the needle of the compass. Rotate			

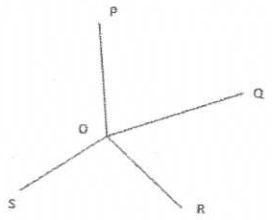
	the instrument about its outer axis till the magnetic needle roughly points to north. Clamp the lower clamp. Using the lower tangent screw, bring the needle exactly against the mark so that it is in magnetic meridian. • Loose the upper clamp and point the telescope towards Q. Bisect Q accurately using the upper tangent screw. Read verniers A and B • Change the face and repeat steps 2,3 and 4. The average of the two will give the correct bearing of the line PQ.  	5 1	6	6
3	Bowditch's Rule • Mostly used to balance a traverse where linear and angular measurements are of equal precision • Errors in linear measurements are proportional to √l, errors in angular measurements are inversely proportional to √l, where l is length of line. • $C_L = \sum L \frac{l}{\sum l}$; $C_D = \sum D \frac{l}{\sum l}$ C_L = Correction to latitude of any side C_D = Correction to departure of any side $\sum L$ = total error in latitude $\sum D$ = total error in departure $\sum l$ = length of perimeter l = length of any side Transit rule • Used when angular measurements are more precise than linear measurements. • Total error in latitudes and departures is distributed in proportion to the latitudes and departures of sides. • $C_L = \sum L \frac{L}{\sum L_T}$; $C_D = \sum D \frac{D}{\sum D_T}$ C_L = Correction to latitude of any side C_D = Correction to departure of any side $\sum L$ = total error in latitude $\sum D$ = total error in departure L = Latitude of any line D = Departure of any line LT = Arithmetic sum of latitudes DT = Arithmetic sum of departure	1 2 1 2	3 3	6
4,	Bearing of AB = 10°12'			

4.	Add <B = 225°13′ = 235°25′ Subtract = 180° Bearing of BC = 55°25′ Add <C = 211° 36′ = 267°1′ Subtract = 180° Bearing of CD = 87°1′ Add <D = 300°26′ = 387°27′ Subtract = 180° Bearing of DE = 207°27′ Add <E = 231°12′ = 438°39′ Subtract = 180° Bearing of EA = 258°39′ Add <A = 291°33′ = 550°12′ Subtract = 180° = 370°12′ Subtract = 360° Bearing of AB = 10°12′ Which agrees with the observed bearing of AB and checks the arithmetical work	1 1 1 1 1 1	6	6
5.	For the first observation: 50 = ks+c 50 = k x 0.49 +c ---- Eqn 1 For the second observation 300 = ks + c 300 = k x 2.99 + c ---- Eqn 2 Solving Equation 1 and 2 -2.5 k = -250 K = 100 Substituting in eqn 1, 50 = k x 0.49 + c 50 = 100 x 0.49 + c C = 1 So, k =100 and c=1	1 1 2 2	6	6
6.	Horizontal circular curves: • Simple curve : The curve which consists of a single are of a circle. It is tangential to both the straight lines • Compound curve : Consists of two or more simple arcs that turn in the same direction and join at common tangent points • Reverse curve : Two circular arcs of same or different radii, having their centres to the different sides of common tangent, both arcs bend in different directions with common tangent at junction	2 2 2	6	6

7.	 SIMPLE CURVE  COMPOUND CURVE  REVERSE CURVE – Zoning, subdivision planning – Land acquisition – Housing renovation programs – Emergency response – Monitoring environmental risk – Modeling storm water runoff – Management of watersheds, floodplains, wetlands, forests, aquifers – Environmental Impact Analysis – Groundwater modeling and contamination tracking – Locating underground facilities – Designing alignment for freeways, transit – Coordination of infrastructure maintenance Write Any 6	6	6	6
	Part C			
III a	Method preferred – Reiteration method 1. Set up and level the theodolite at O. Keep the instrument in the normal posi-tion, i.e., face left. Set the vernier at A to read zero using the upper clamp and upper tangent screw. Check that the vernier at B reads 180 °. 2. Loosen the lower clamp and swing the instrument to bisect the station mark P Tighten the screw and using the lower tangent screw finaly bisect the signal at P. Check that the verniers at A and B read zero and 180 °, respectively. 4. Release the upper plate with the upper clamp, swing the instrument clockwise to bisect the signal at Q. Tighten the clamp and using the upper tangent screw, bisect the mark at Q exactly. 5. Read the verniers at A and B and record both the readings. 6. Release the upper clamp screw, bisect the signal at R. Tighten the			

clamp and bisect the mark at R exactly with the upper tangent screw. Read the verniers at A and B and record the readings.

Continue the procedure with other stations.



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1

b

Method 1

i) Set up the theodolite at A and level it accurately .Bisect the point b correctly. Establish a point C in the line beyond B approximately by looking over the top of the telescope and accurately by sighting through the telescope.

ii) Shift the instrument to B ,take a fore sight on C and establish a point D in line beyond C.

iii) Repeat the process until the last point Z is reached.

3.5

Method 2

i) Set up the instrument at B and level it accurately .

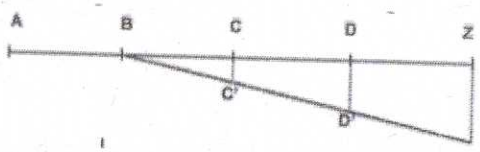
ii) Take a back sight on A.

iii) Tighten the upper and lower clamps, transit the telescope and establish a point C in the line beyond B.

iv) Shift the theodolite to C ,back sight on B transit the telescope and establish a point D in line beyond C. Repeat the process until the last point (Z) is established.

7 7

Now if the instrument is in adjustment, the points A,B,C,D and Z will be in one line, which is straight but if it is not in adjustment i.e. line of collimation is not perpendicular to the horizontal axis ,then C', D' and Z' will not be in a straight line.



3.5

Method 3

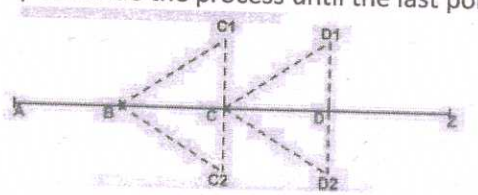
i) Set up the theodolite at B and level it accurately.

ii) With the face of instrument left, back sight on A and clamp both the upper and lower motions.

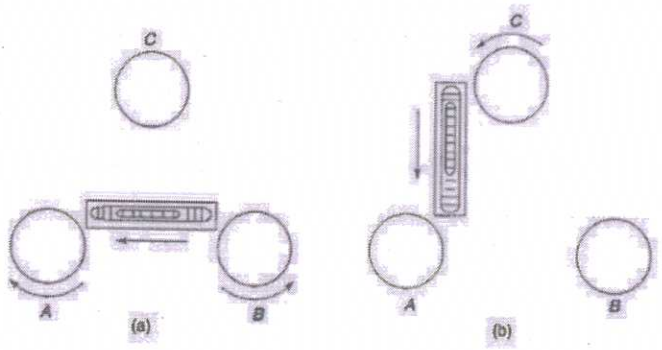
iii) Transit the telescope and set a point C1 ahead in line.

iv) Loosen the lower clamp ,revolve the telescope in the horizontal plane and back sight on A .Bisect A exactly by using the lower clamp and its tangent screw. Now the face of instrument is right.

v) Transit the telescope and establish a point C2 in line beside the point

	C1. vi) The exact position of the true point C must be mid-way between C1 and C2. vii) Measure C1 C2 and establish a point C exactly mid-way, which lies on the true prolongation of AB. viii) Shift the instrument to C, double sight on B ,establish the point D1 and D2 and locate the true point D as before . ix) Continue the process until the last point Z is established.  write Any 2 Methods			
IV a	Fundamental axes : • The vertical axis • The horizontal axis • The line of collimation • Axis of plate level • Axis of altitude level • Axis of the striding level Desired relations: • The axis of the plate level must lie in a plane perpendicular to the vertical axis • The line of collimation must be perpendicular to the horizontal axis at its intersection with the vertical axis • Horizontal axis must be perpendicular to the vertical axis • The axis of altitude level (or telescope level) must be parallel to the line of collimation • The vertical circle vernier must read zero when the line of collimation is horizontal • The axis of striding level (if provided) must be parallel to the horizontal axis	4 4	8	8
b	Temporary Adjustment of transit theodolite: ★ Setting over the station ★ Levelling up ★ Elimination of parallax **Setting over the station:** The operation of setting includes: 1. Centring of instrument over the station mark by plumb Bob. 2. Approximate levelling with the help of Tripod legs. **Levelling up:** After the centring and approximate levelling , The accurate levelling is done with the help of foot screws and with reference to the plate levels.The levelling of instrument by the plate levels depend upon	2 3	5	

whether the instrument has three levelling screws or four leveling screws :



Elimination of parallax :

It is arised when the image formed by the objective is not in the plane of the cross-hairs . Parallax is eliminated in two steps :

1. By focusing the eye-piece .
2. By focusing the objective.

Focusing the eye-piece : To focus the eye-piece for distinct visions of the cross-hairs, hold a sheet of white paper in front of objective and move eyepiece in and out till the cross hairs are seen sharp and distinct.

Focusing the objective : The Telescope is now directed towards the object to be sighted and the focusing screw is turned till the image appears clear and sharp.

2 | 2 | 7

V A

Side	L	D	DMD	Double Area
AB	214.8	124	124	26635.24
BC	-245.4	205.7	453.7	-111201.1
CD	-155	-90	569.4	-88769.40
DA	186.2	-239.7	239.7	44632.21

2 x Area = sum of double area

= 128703.05

Area of traverse = 64351.53 m^2

Students can use any convenient method for finding the area (Area from co-ordinates, area from departures and total latitudes)

7 | 7 | 7

b

1. Adjust the interior angles to satisfy the geometrical conditions, that is sum of interior angles to be equal to (2N-4) right angles and exterior angles (2N+4) right angles.

In the case of a compass traverse, the bearings are adjusted for a local attraction

2. Starting with observed bearings of one line, calculate the bearings of all other lines. Reduce all bearings to the quadrantal system.

3.Calculate the consecutive co-ordinates (that is latitudes and departures)

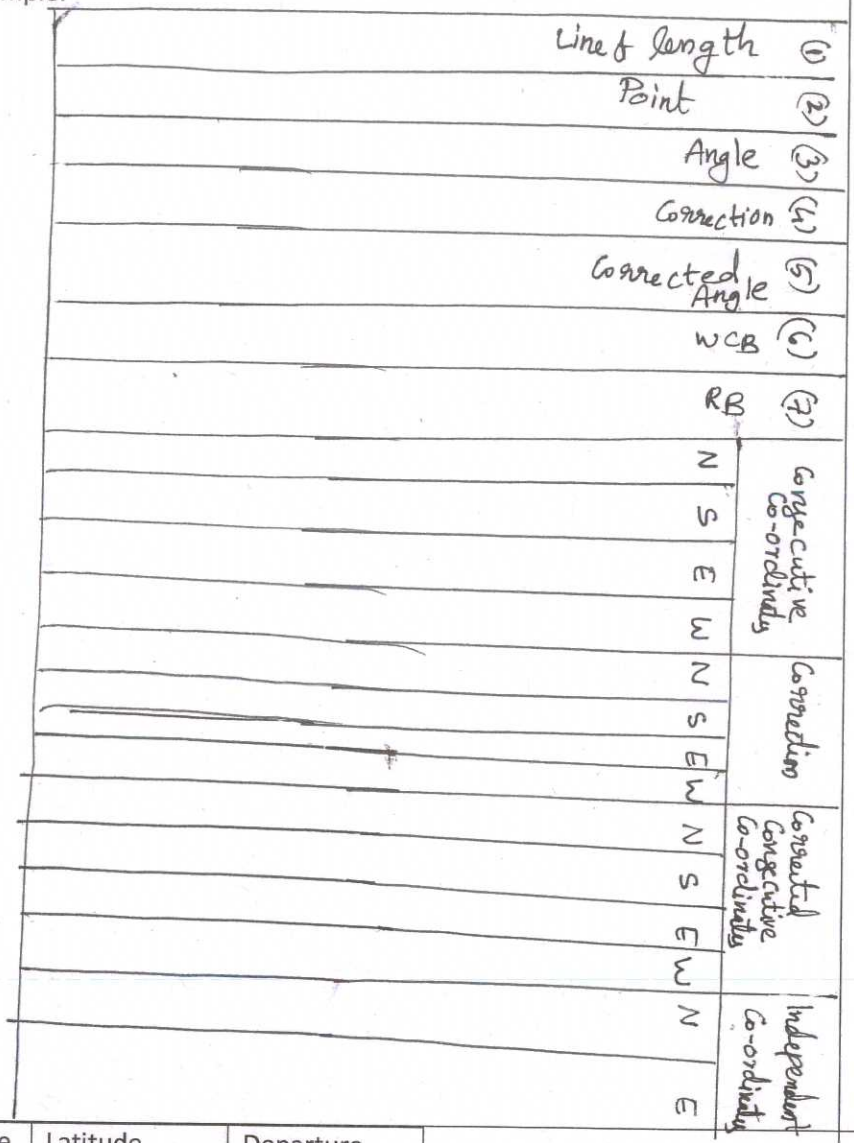
4. Calculate the $\sum L$ and $\sum D$.

5. Apply necessary corrections to the latitudes and departures of the lines

so that ∑L=0 and ∑D=0. The corrections may be applied either by transit rule or by compass rule depending upon the type of traverse.

6. Using the corrected consecutive coordinates, calculate the independent coordinates to the points so that they are all positive, the whole of the traverse thus lying in the northeast quadrant.

Example:



7 | 7 | 7

VI A

Line	Latitude	Departure
AB	-73.91	494.5
BC	535.11	313.11
CD	223.45	-411.29
	εL' = 684.55	εD' = 396.32

Latitude of DA = -684.55

Departure of DA = -396.32

$$tan\theta = \frac{D}{L} = \frac{396.32}{684.55}$$

$$\theta = 30^0 4'$$

Bearing of DA = S30°4'W
= 210°4'
Latitude of DA = lcosθ
684.55 secθ = l
Length of DA = 791.01 m

<ADE = α = Bearing of DE – Bearing of DA
= 230°o' – 210°4' = 19°56'
<DEA = β = Bearing of EA – Bearing of ED
= Bearing of EA – Back Bearing of DE
= 150°10' – (230° – 180°) = 100°10'
<DAE = Y = 180° – (100°10'+19°56')
= 59°54'

$$\frac{DE}{sinY} = \frac{DA}{sin\beta}$$

$$DE = 791.01\ \frac{\sin(59^0 54')}{\sin(100^0 10')} = 695.27\ m$$

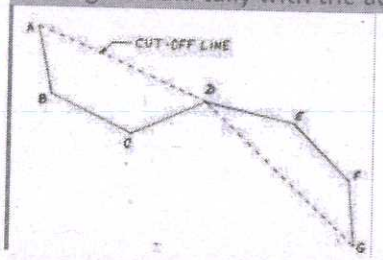
$$\frac{EA}{sin\alpha} = \frac{DA}{sin\beta}$$

$$EA = 791.01\ \frac{\sin(19^0 56')}{\sin(100^0 10')} = 273.99\ m$$

6 6 8

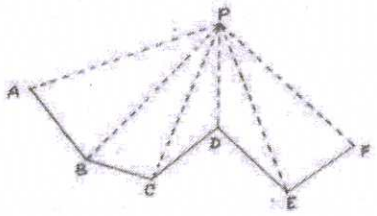
b Check on open traverses:

Taking cut-off lines. Cut-off lines are taken between some intermediate stations of the open traverse. Suppose ABCDEFG represents an open traverse. Let AD and DG be the cut-off lines. The length and the magnetic of the cut-off lines are measured accurately. After plotting the traverse, the distances and bearings are noted from the map. These distances and bearings should tally with the actual records obtained from the field



3.5 3.5

Taking an auxiliary point. Suppose ABCDEF an open traverse. A permanent point P is selected on the side of it. The magnetic bearings of this point are taken from traverse stations A, B, C, D, etc. If the survey carried out accurately and so is the plotting, all the measured bearings of P when plotted should meet at the point P. The permanent point P is known as the 'auxiliary point



CHECK ON CLOSED TRAVERS

1). Check on angular measurements.
a). The sum of measured interior angles should be equal to (2n-4) x 90° where n is the number of sides of the traverse.
b). The sum of measured exterior angles should be equal to (2n+4) x 90°.
c). The algebraic sum of the deflection angles should be equal to 360°. Right hand deflections are considered positive and left-hand deflection negative.
2). Check on linear measurements.
a). A line should be once each of two different days (along opposite direction). Both measurement should tally.
b). Linear measurements should also be taken by stadia method. The measurements by chaining or by other method should tally.

3.5 3.5 7

VII a

RL of instrument axis at A
= 543.074 + 1.578
= 544.653 m 1

RL of instrument axis at B
= 543.075 + 1.269
= 544.344 m 1

Difference in level of instrument axis at two stations A and B
= 1.578 – 1.269
= 0.309 m

$$\alpha_1 = 10^0 12', \alpha_2 = 8^0 20'$$

$$D = \frac{s - b\tan\alpha_2}{\tan\alpha_2 - \tan\alpha_1}$$

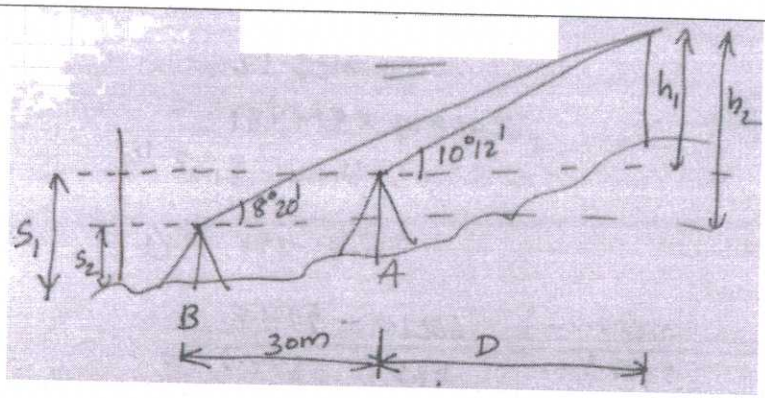
$$= \frac{0.309 - 30\ X\ \tan 8^0 20'}{\tan 8^0 20' - \tan 10^0 12'}$$

D = 122.13 m 2

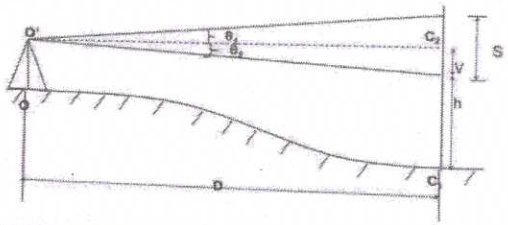
$h_1 = D\tan\alpha_1$
= 122.13 x $\tan 10^\circ 12'$
= 21.97 m 2

RL of top of chimney = RL of BM + s_1 + h_1
= 543.075 + 1.578 + 21.97
= 566.623 m 2

8 8



b



O' -Instrument axis
O – Instrument station
C_1 – Staff station
V – vertical distance between lower vane and axis of instrument
S – distance between the targets
θ_1 - vertical angle by upper targets
θ_2 - vertical angle lower targets
h – height of lower vane above the staff station

From figure we can say that,

$$V = D\tan\theta_2$$

$$S - V = D\tan\theta_1$$

$$D = \frac{S}{\tan\theta_1 + \tan\theta_2}$$

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

RL of station A = RL of instrument axis - V - h

2

2

1

1

1

7

7

VIII

a)

OP = ks $\cos^2\theta$ + C $\cos\theta$
= 100 x 2.25 x $[\cos(9°30'))]^2$
= 218.871 m
S = 2.25 , $\theta = 9°30'$, c =0
V_1 = (ks $\sin 2\theta$)/2 + c $\sin\theta$
= 36.626 m

OQ = ks $\cos^2\theta$ + c $\cos\theta$
= 100 x 2.055 x $[\cos 6°]^2$
= 203.254 m
V_2 = (ks $\sin 2\theta$)/2 + c $\sin\theta$

3

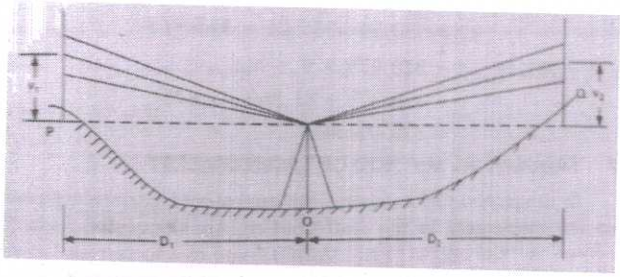
3

6

= 21.363 m

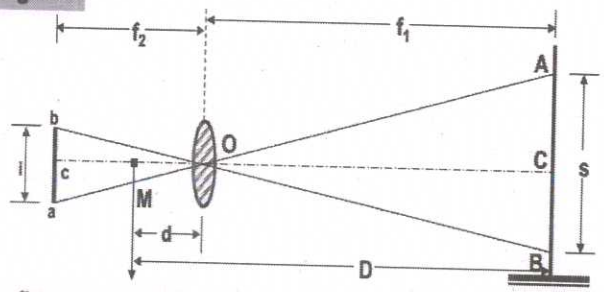
RL of Q = RL of P $+r_1 - v_1 + v_2 - r_2$
= 350.50 + 2.015 – 36.626 + 21.363 – 1.875
= 335.467 m

2 2 8



b)

Horizontal Sights:



2

Consider the figure, in which *O* is the optical centre of the objective of an *external focusing telescope.*

Let A, C, and B **= the points cut by the three lines of sight corresponding to three wires.**

b, c, and a **= top, axial and bottom hairs of the diaphragm.**
ab = i = interval b/w the stadia hairs (stadia interval)
AB = s = staff intercept;
f = focal length of the objective

f_1 = horizontal distance of the staff from the optical centre of the objective
f_2 = horizontal distance of the cross-wires from O.
d = distance of the vertical axis of the instrument from O.
D = horizontal distance of the staff from the vertical axis of the instruments.
M = centre of the instrument, corresponding to the vertical axis.

Since the rays *BOb* and *AOa* pass through the optical centre, they are straight so that *AOB* and *aOb* are similar. Hence,

$$\frac{f_1}{f_2} = \frac{s}{i}$$

Again, since f_1 and f_2 are conjugate focal distances, we have from lens formula,

$$\frac{1}{f} = \frac{1}{f_2} + \frac{1}{f_1}$$

Multiplying throughout by ff_1, we get $f_1 = \frac{f_1}{f_2} f + f$

Substituting the values of $\frac{f_1}{f_2} = \frac{s}{i}$ in the above, we get

$$f_1 = \frac{s}{i} f + f$$

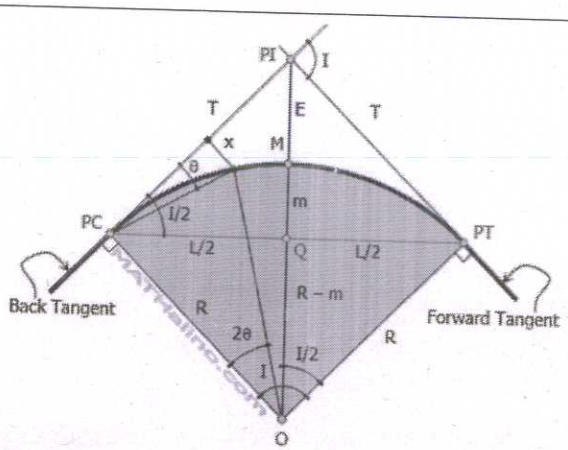
Horizontal distance between the axis and the staff is $D = f_1 + d$

$$D = \frac{f}{i} s + (f + d) = k \cdot s + C$$

5

7

IX A



1

Back Tangent:
The tangent previous to the curve
Forward Tangent:
The tangent following the vurve
Point of Intersection:
If the two tangents are produced, they will meet in a point called point of intersection

Point of curve / commencement (PC):
It is the beginning of the curve where the alignment changes from a tangent to a curve
Point of tangency (PT):
It is the end of the curve where the alignment changes from a curve to tangent
External deflection angle = I
Tangent distance (T):
It is the distance between PC to PI or PI to PT
Length of curve:
Total length of curve from point of curve to the point of tangency
Long chord:
It is the chord joining PC to PT
Mid ordinate (m)
It is the ordinate from the point of long chord to the mid point of the curve
Normal chord:
A chord between two successive regular stations on a curve
Sub chord:
Any chord shorter than normal chord
External distance (E)
Distance from mid point of curve to PI

6

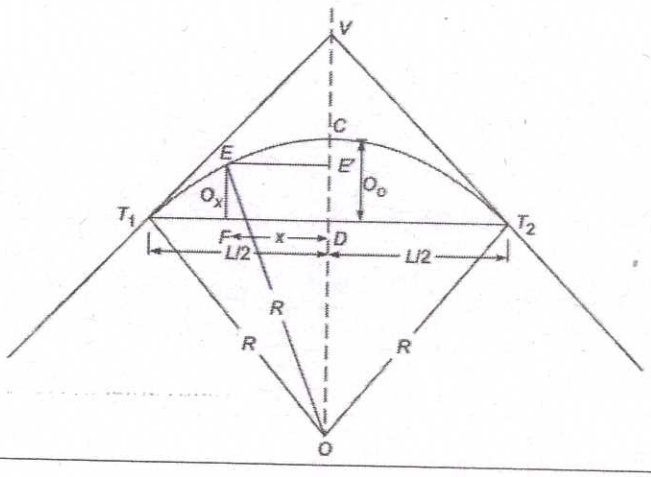
7

b) Write any 4 elements with explanation

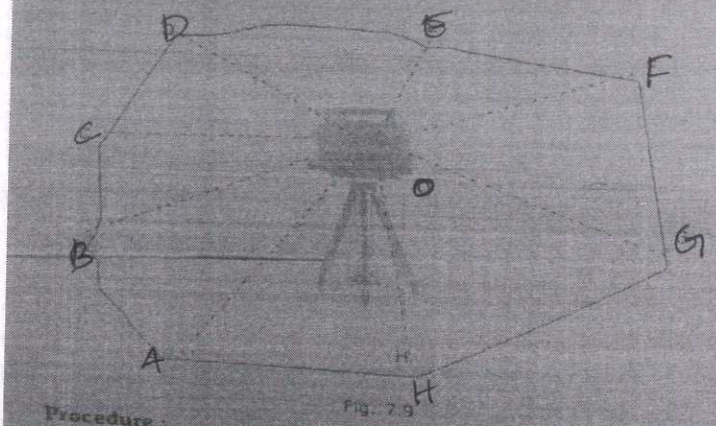
- Erect ranging rods at T1 and T2
- Divide the long chord into even no. of equal parts of suitable length
- Calculate the length of the offsets corresponding to the distances from the midpoint of the chord (if exact equation is used) or else calculate the length of offsets corresponding to the distance from T1 (if approximate equation is used)
- Erect perpendiculars with the help of an optical square or using 3,4,5 rule and measure the calculated offset distances

4

6



2

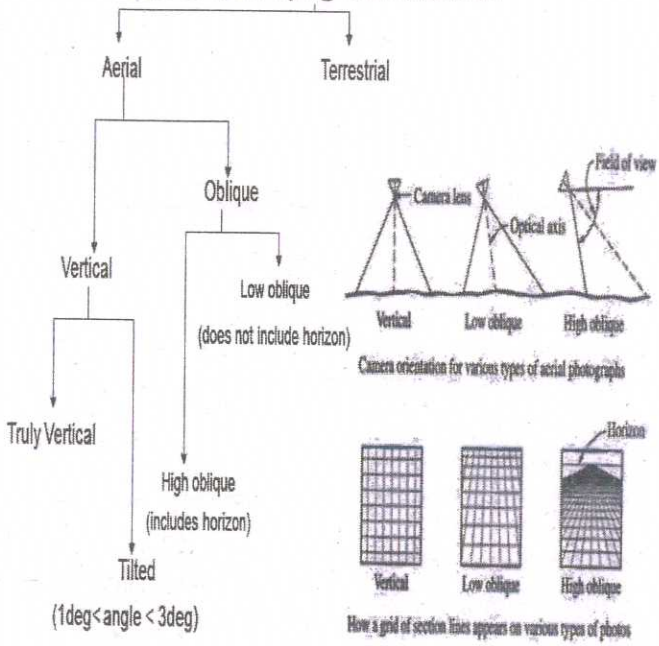
	$O_X = \sqrt{R^2 - x^2} - (R - O_O) Exact\ equation$ $O_x = \frac{x(L-x)}{2R}\ Approximately$	1	1	7
X a)	**7.7 TO MEASURE THE AREA OF GIVEN FIELD BY TOTAL STATION WITH SINGLE STATION :**  Fig. 7.9 **Procedure :** • Set the total station in such a way that the boundaries of the field can be visible at a single station. • After setting the T.S at convenient place, mark the station on the ground. • Perform the temporary adjustments. • Select Measure Topo from 'Survey'. • It shows the sub-window and enter the name of point. 'A' as shown in the Fig. 7.9. • Place the pole on A and check bubble at back of pole for vertically. • Sight to prism at 'A' by telescope of TS and press 'measure'. • After 'measure' press store then and data will be saved in the T.S. • Repeat the above process for number of points to be required BCDEFGH as shown in Fig. 7.9. • If roads, trees, electric poles are to be located, place the prism at appropriate places and then measure it and store it. • Find the area of the field from the Co Go File Menu : select compute + subdivide area • Then all points are selected one by one in an order. • Now the map of field is shown. • The area and perimeter of field are observed in the total station. • We can also divide the total area in to parts as per requirement.	1 7	8	8

b)

Photogrammetry is the science of obtaining reliable information about the properties of surfaces and objects without physical contact with the objects, and of measuring and interpreting this information. 1

In **Terrestrial Photogrammetry** the photographs are taken from a fixed position on or near the ground. 1

In **Aerial Photogrammetry** the photographs are taken by a camera mounted in an aircraft flying over the area 1



- The technical requirements of the Aerial photogrammetry are very high-end; the specially designed planes or drones are just the initials.
- Although there is a much lesser investment in Terrestrial photogrammetry, there is the requirement of high technical skills for operating the tools.
- To map a large surface area of the land, we can use Aerial photogrammetry. But the knowledge of longitudes and latitudes with their calculations are essential in photogrammetry operation. 3
- When there is a need for designers and planners to manage and plan bridges, transportation network, pipeline networks, etc. then Terrestrial photogrammetry is preferred. The adjustments of the camera's axis as per the surface area matter a lot.
- The environmental projects which trace the changes of glaciers, volcanoes, earthquakes, etc., use terrestrial photogrammetry methods.
- The fixed location of the camera creates such kind of data sets to work upon further.
- There are various benefits of aerial photogrammetry in the security field as well. Archaeology also gets numerous advantages through aerial photogrammetry as you can see a big chunk of the area at once.
- Terrestrial photogrammetry is preferable for movement-related and

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	comparatively smaller land masses, unlike the aerial one. • The arena where both the fields play an active role is Urban Planning! That's obvious as with the help of aerial photogrammetry, designers and planners generate an accurate perspective plan of the site or location. An aerial view gives a bright idea of all around the neighborhood, and so it helps in further assessment. • Terrestrial photogrammetry, unlike aerial, creates visuals showing interrelation between civil constructions, safety issues, disaster management and various other problems. Thus, it helps in knowing the creation, prevention, and demolition of any civilization.			