Example 1.19:

The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m: 0, 2.50, 3.50, 5.00, 4.60, 3.20, and 0 m. Compute the area between the chain line, the irregular boundary line and the end of offsets by: (a) the Trapezoidal rule, and (b) Simpson's rule.

Solution:

By Trapezoidal rule:

Here d=10 m

Required area=10 / 2{0 + 0 + 2(2.50 + 3.50 + 5.00 + 4.60 + 3.20)}

= 5*37.60 =188 m^2

By Simpson's rule:

d=10 m

Required area=10/ 3{0 + 0 + 4(2.50 + 5.00 + 3.20) + 2(3.50 + 4.60)}

= 10/3{42.80 +16.20}=10/3*59.00

= 196.66 m^2

Example 1.20:

An embankment of width 10 m and side slopes 1½:1 is required to be made on a ground which is level in a direction transverse to the centre line. The central heights at 40 m intervals are as: 0.90,1.25,2.15,2.50,1.85,1.35, and 0.85 m, calculate the volume of earth work according to (i) Trapezoidal formula, and (ii) Prismoidal formula

Solution:

The cross-sections areas are calculated by

Δ= (b + sh)*h

Δ1= (10 + 1.5*0.90)*0.90 = 10.22 m^2

Δ2= (10 + 1.5*1.25)*0.90 = 14.84 m^2

Δ3= (10 +1.5*1.25)*2.15 = 28.43 m^2

Δ4= (10 +1.5*2.50)*2.50 = 34.38 m^2

Δ5= (10 +1.5*1.85)*1.85 = 23.63 m^2

Δ6= (10 +1.5*1.35)*1.35 = 16.23 m^2

Δ7= (10 +1.5*0.85)*0.85= 9.58 m^2

(a) Volume according to trapezoidal formula

V= 40 / 2{10.22 + 9.58 + 2(14.84 + 28.43 + 34.38 + 23.63 + 16.23)}

= 20{19.80 + 235.02} = 5096.4 m^2

(b) Volume calculated in prismoidal formula:

V = 40 / 3 {10.22 + 9.58 + 4(14.84 + 34.38 +16.23) + 2(28.43 + 23.63)}

= 40 / 3 (19.80 + 261.80 +104.12) = 5142.9 m^2

Example 1.21:

The areas enclosed by the contours in the lake are as follows:

Contour (m)	**Area (m^2)**
270	2050
275	8400
280	16300
285	24600
290	31500

Calculate the volume of water between the contours 270 m and 290 m by Trapezoidal formula.

Solution:
Volume according to trapezoidal formula:
= 5 / 2{2050 + 31500 + 2(8400 + 16300 + 24600)}
= 330,250 m^3

Example 1.22:
The following readings were taken with a tacheometer on to a vertical staff, calculate the tacheometric constants.

Horizontal Distance (m)	**Stadia Readings (m)**
45.00	0.885 1.110 1.335
60.00	1.860 2.160 2.460

Solution:
$D_1 = KS_1 + C$ ....(1)
$D_2 = KS_2 + C$ ....(2)
So, 45 = K (1.335 – 0.885) + C
45 = K (0.45) + C ....(3)
60 = K (2.460 – 1.860) + C
60 = K (0.6) + C ……………….(4)

Equating C from equations 3 and 4, we get-
45 – K (0.45) = 60 - K (0.6)
0.15K = 15
K =100
Now put the value of K in either equation 3 or 4, we get C =0

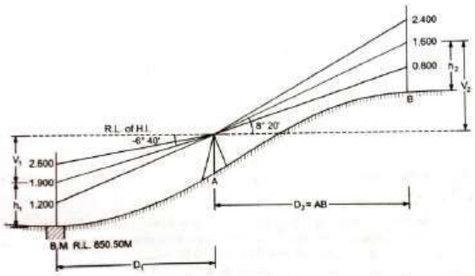
Example 1.23:
The tacheometer instrument was setup over a station P of RL 1850.95 m and the height of instrument above P was 1.475 m. The staff was held vertical at a station Q and the three readings were 1.050, 1.900 and 2.750 m with the line of sight horizontal. Calculate the horizontal distance of PQ and RL of Q point.

Solution:
D = KS + C
Now, S = 2.750 – 1.050 = 1.700 m
D = 100 (1.700) + 0 = 170 m
RL of Q = 1850.95 + 1.475 – 1.900
= 1850.525 m

Example 1.24:
A tachometer was setup at a station A and the following readings were obtained on a staff held vertical. Calculate the horizontal distance AB and RL of B, when the constants of instrument are 100 and 0.15.

Inst. Station	**Staff Station**	**Vertical angle**	**Hair Reading (m)**			**Remark**
A	BM	$- 6^0\ 40'$	1.200	1.900	2.600	RL of BM = 850.500 m
A	B	$+ 8^0\ 20'$	0.800	1.600	2.400	

Solution:



In first observation
$S_1 = 2.600 - 1.200 = 1.400$ m
$\theta_1 = -6^0\,40'$ (Depression)
K = 100 and C = 0.15
Vertical Distance $V_1 = KS_1 \sin 2\theta / 2 + C \sin\theta$
$= 100\,(1.400) \sin(2 \times 6^0 40') / 2 + 0.15 \sin 6^0 40'$
$= 16.143 + 0.0174$
$= 16.160$ m

In second observation
$S_2 = 2.400 - 0.800 = 1.600$ m
$\theta_2 = +8^0\,20'$ (Elevation)
Vertical Distance $V_2 = KS_2 \sin 2\theta/2 + C \sin\theta$
$= 100\,(1.600) \sin(2 \times 8^0 20') / 2 + 0.15 \sin 8^0 20'$
$= 22.944 + 0.022$
$= 22.966$ m
Horizontal distance $D_2 = KS_2 \cos^2\theta + C \sin\theta$
$= 100\,(1.600) \cos^2 8^0 20' + 0.15 \sin 8^0 20'$
$= 156.639 + 0.148 = 156.787$ m
RL of instrument axis = RL of BM + h_1 + V_1
$= 850.500 + 1.900 + 16.160$
$= 868.560$ m
RL of B = RL of Inst. axis + $V_2 - h_2$
$= 868.560 + 22.966 - 1.600$
RL of B = 889.926m

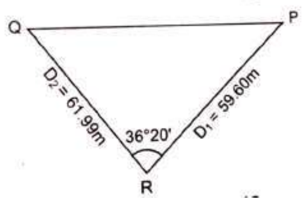
Example 1.25:
To determine the gradient between two points P and Q, a tacheometer was set up at R station, and the following observations were taken keeping the staff vertical at P and Q. If the horizontal angle PRQ is $36^0 20'$ and RL of HI is 100 m, determine the average gradient between P and Q.

Staff station	**Vertical angle**	**Stadia readings (m)**
P	$+4^0 40'$	1.210, 1.510, 1.810
Q	$-0^0 40'$	1.000, 1.310, 1.620

Solution:
In the first observation (From R to P)
$S_1 = 1.810 - 1.210 = 0.6$ m
$\theta_1 = +4^0 40'$
Horizontal distance $D_1 = KS_1 \cos^2\theta + C \sin\theta$
$= 100 \times 0.6 \times \cos^2 4^0 40' + 0$
$= 59.60$ m

Vertical Distance $V_1 = KS_1 \sin 2\theta/2 + C \sin\theta$
$= 100 \times 0.6 \times \text{Sin}\ (2 \times 4^0 40') / 2 + 0$
$= 4.865$ m

In the second observation (From R to Q)
$S_2 = 1.620 – 1.000 = 0.62$ m
$\theta_2 = - 0^0 40'$
Horizontal distance $D_2 = KS_2 \cos^2\theta + C \sin\theta$
$= 100 \times 0.62 \times \text{Cos}^2 0^0 40' + 0$
$= 61.99$ m
Vertical distance $V_2 = KS_2 \sin 2\theta/2 + C \sin\theta$
$= 100 \times 0.62 \times \sin (2 \times 0^0 40') / 2 + 0$
$= 0.721$ m
Avg. Gradient Between P and Q point



Distance $D_1 = PR = 59.60$ m
Distance $D_2 = QR = 61.99$ m
Angle $PRQ = 36^0 20'$
$PQ^2 = PR^2 + QR^2 – 2 \times PR \times QR \times \cos 36^0 20'$
$PQ^2 = (59.60)^2 + (61.99)^2 – 2 \times 59.60 \times 61.99 \times \cos 36^0 20'$
$PQ = 37.978$ m

Difference of elevation between P and Q
RL of P = RL of HI + $V_1 – h_1$
$= 100 + 4.865 – 1.510$
$= 103.355$ m
RL of Q = RL of HI – $V_2 – h_2$
$= 100 – 0.721 – 1.310$
$= 97.969$ m
Difference $= 103.355 – 97.969 = 5.386$ m
Average gradient between P and Q = Difference in RL between P & Q / Distance PQ
$= 5.386 / 37.978$
$= 1 / 7.051$

Exercise 1.26:
The vertical angles to vanes fixed at 1 m and 3 m above the foot of the staff held vertically at station Q were $3^0 20'$ and $6^0 40'$, respectively from instrument station P. If the elevation of the instrument axis at station P is 101.520 m, calculate (i) the horizontal distance between P and Q, and (ii) the elevation of the staff station Q.

Solution:
$S = 3 – 2 = 1$ m
$\theta_1 = 6^0 40'$
$\theta_2 = 3^0 20'$
$h = 1$ m

$D = S / [\tan \theta_1 – \tan \theta_2]$
$= 1 / [\tan 6^0 40' - \tan 3^0 20']$
$= 34.13$ m

$V = S \tan \theta_2 / [\tan \theta_1 – \tan \theta_2]$
$= 2 \times \tan 3^0 20' / [\tan 6^0 40' - \tan 3^0 20']$
$= 1.99$ m
Elevation of staff station Q = RL of HI + V – h
= 101.520 + 1.99 – 1.0
= 102.510 m

Example 1.27:
From the top of a light house, the angles of depression of two ships are 30^0 and 45^0. The two ships, as it was observed from the top of the light house, were 100 m apart. Find the height of the light house.

Solution:
Height = Distance / [cot (original angle) – cot (final angle)]
Height of the light house = $100 / (\cot 30^0 – \cot 45^0)$
= 50 m

Example 1.28:
An instrument was set up at P and the angle of depression to a vane 2 m above the foot of the staff held at Q was 5° 36′. The horizontal distance between P and Q was known to be 3000 metres. Determine the R.L. of the staff station Q given that staff reading on a B.M. of elevation 436.050 was 2.865 metres.

Solution:
The difference in elevation between the vane and the instrument axis = $D \tan \alpha$
= 3000 tan 5° 36′ = 294.153 m
Combined correction due to cuvature and refraction $C = 0.06735 D^2$ metres
when D is in km = 0.606 m.
Since the observed angle is negative, the combined correction due to curvature and refraction is subtractive.
Difference in elevation between the vane and the instrument axis = 294.153 – 0.606 = 293.547 = h
RL of instrument axis = 436.050 + 2.865 = 438.915
RL of the vane = RL of instrument axis – h = 438.915 – 293.547 = 145.368
RL of Q = 145.368 – 2 = 143.368 m

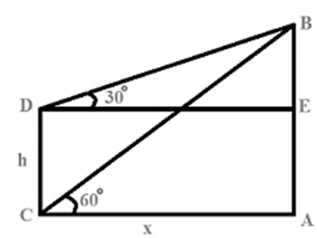
Example 1.29:
In order to ascertain the elevation of the top (Q) of the signal on a hill, observations were made from two instrument stations P and R at a horizontal distance 100 metres apart, the station P and R being in the line with Q. The angles of elevation of Q at P and R were 28° 42′ and 18° 6′, respectively. The staff reading upon the bench mark of elevation 287.28 m were respectively 2.870 m and 3.750 m, when the instrument was at P and at R, the telescope being horizontal. Determine the elevation of the foot of the signal if the height of the signal above its base is 3 metres.

Solution:

Elevation of instrument axis at P = R.L. of B.M. + Staff reading = 287.28 + 2.870 = 290.15 m
Elevation of instrument axis at R = R.L. of B.M. + staff reading = 287.28 + 3.750 = 291.03 m
Difference in level of the instrument axes at the two stations S = 291.03 – 290.15 = 0.88 m
$\alpha_1 = 28° 42$ and $\alpha_2 = 18° 6'$
S cot α_2 = 0.88 cot 18° 6′ = 2.69 m
D = 152.1 m
$h_1 = D \tan \alpha_1$ = 152.1 tan 28° 42′ = 83.272 m
RL of foot of signal = RL of inst. axis at P + h_1- ht. of signal
= 290.15 + 83.272 – 3 = 370.422 m
Check :
(b + D) = 100 + 152.1 m = 252.1 m
$h_2 = (b + D) \tan \alpha_2$ = 252.1 x tan 18° 6′ = 82.399 m
RL of foot of signal = RL of inst. axis at R + h_2 + ht. of signal
= 291.03 + 82.399 – 3 = 370.429 m

Example 1.30:
There are two poles with different heights; one on each side of the road. The higher pole is 54 m high, and from its top, the angle of depression of top and bottom of the shorter pole is 30^0 and 60^0, respectively. Find the height of the shorter pole.

Solution:
Let AB be the higher pole, and CD be the lower pole.
In triangle ABC,
tan 60 = AB / AC
√3 = 54 / AC
AC=54/ √3 = DE
In triangle BED,
tan 30 = BE / DE
BE = tan 30 *DE
= (1/√3) * (54/ √3)
= 18 m
CD = AE = AB – BE
CD = 54 – 18 = 36 m
Therefore, height of the shorter pole = 36 m



Example 1.31:
In the following table, the WCB and the lengths of traverse lines of a closed traverse 1-2-3-4-5-6-7-8-9-10-1 are given. Compute the correct latitudes and departures of traverse lines, if the coordinates of point 1 are (1000, 2000) m:

Side	**WCB (Φ)**	**Length (L)**
1-2	327*56'7"	217.72
2-3	331*53'7"	113.33
3-4	82*39'2"	318.67
4-5	90*28'55"	137.77
5-6	109*53'9"	80.81
6-7	111*30'4"	71.13
7-8	169*58'59"	219.18
8-9	264*51'15"	162.36
9-10	256*55'51"	208.60
10-1	276*9'20"	101.260
Σ		1630.83

Solution:
First compute the latitude and departure of lines taking into account the proper sign of the quadrant.

Side	WCB (Φ)	Length (L)	L cos(Φ)	L sin(Φ)
1-2	327*56'7"	217.72	184.506	-115.582
2-3	331*53'7"	113.33	99.958	-53.405
3-4	82*39'2"	318.67	40.735	316.056
4-5	90*28'55"	137.77	-1.159	137.765
5-6	109*53'9"	80.81	-27.489	75.991
6-7	111*30'4"	71.13	-26.071	66.180
7-8	169*58'59"	219.18	215.839	38.124
8-9	264*51'15"	162.36	-14.562	-161.706
9-10	256*55'51"	208.60	-47.170	-203.197
10-1	276*9'20"	101.260	10.858	-100.676
Σ		1630.83	3.769	-0.456

Determine the closing error = $\sqrt{[(3.769)^2 + (-0.456)^2]}$ = 3.80 m
Apply corrections to each as per Latitude and Departure by Bowditch method
Correction in each Latitude CL =$\sum L \times (L/\sum L)$
Similarly, correction in each Departure CD = $\Sigma D * (L/\Sigma L)$

The corrected Latitude and Departure as well as Coordinates of the points are computed as given below:

Station	Corrected Lcos Φ	Corrected Lsin Φ	Corrected latitude	Corrected departure
1	184.002	-115.522	1000	2000
2	99.696	-53.374	884.478	2184.002
3	39.998	316.144	831.104	2283.698
4	-1.477	137.803	1147.248	2323.696
5	-27.674	76.013	1285.051	2322.219
6	-26.235	66.2	1361.064	2294.545
7	-216.345	38.184	1427.264	2268.310
8	-14.937	-161.661	1465.448	2051.965
9	-47.652	-203.139	1303.787	2037.028
10	10.624	-100.648	1100.648	1989.376
1			1000	2000
	Σ Lcos Φ = 0	ΣLsin Φ = 0	Check	

Example 1.32:
The data for a closed traverse is given below. Balance the traverse by Bowditch method and Transit method.

Line	Length	WCB
AB	89.31	45°10′
BC	219.76	72°05′
CD	151.18	161°52′
DE	159.10	228°43′
EA	232.26	300°42′

Solution:
Perimeter = $\sum L$ = Sum of lengths = 851.61 m

Magnitude of closing error = $\sqrt{(0.51)^2 + (0.224)^2} = 0.557$ m
Direction of closing error = $\tan^{-1}(0.224 / 0.51) = 23^\circ 42' 42.6''$

By Bowditch method
CL =∑L×(L/∑L)
Correction in latitude of line AB = 0.51×(89.31 / 851.61) =0.053
Correction in latitude of line BC = 0.51×(219.76 / 851.61) =0.132
Correction in latitude of line CD = 0.091
Correction in latitude of line DE = 0.095
Correction in latitude of line EA = 0.061

Similarly,
CD = ΣD * (L/ΣL)
Correction in departure of line AB = 0.224×(89.31/851.61) = 0.023
Correction in departure of line BC = 0.224×(219.76/851.61) = 0.058
Correction in departure of line CD = 0.224×(151.18/851.61) = 0.040
Correction in departure of line DE = 0.224×(159.10/851.61) = 0.042
Correction in departure of line EA = 0.224×(232.26/851.61) = 0.061

For corrected coordinates,
Corrected Latitude= Latitude -Correction (since (∑L) is positive, correction will be negative)
Corrected departure= Departure-Correction

Line	Length	WCB	Consecutive coordinate		Correction		Corrected coordinate	
			Latitude (Lcosθ)	Departure (Lsinθ)	Latitude	Departure	Latitude	Departure
AB	89.31	$45^\circ 10'$	62.968	63.335	0.053	0.023	62.915	63.312
BC	219.76	$72^\circ 05'$	67.606	209.103	0.132	0.058	67.474	209.045
CD	151.18	$161^\circ 52'$	-143.672	47.052	0.091	0.040	-143.763	47.012
DE	159.10	$228^\circ 43'$	-104.971	-119.557	0.095	0.042	-105.066	-119.599
EA	232.26	$300^\circ 42'$	118.579	-199.709	0.139	0.061	118.44	-199.77
			∑L=0.51	∑D=0.224	∑L=0.51	∑D=0.224	∑L=0	∑D=0

By Transit method
For correction,
CL = ΣL * L / ΣLt
CD = ΣD * D / ΣDt
ΣL = Total error in latitude (Sign consideration)
ΣD= Total error in departure (Sign consideration)
L, D = Latitude, and Departure of any side
∑Dt = Arithmetic sum of Departures (No sign consideration)
∑Lt = Arithmetic sum of Latitudes (No sign consideration)

Correction in latitude of line AB = 0.51×(62.968 / 497.796) = 0.065
Correction in latitude of line BC = 0.51×(67.606 / 497.796) = 0.069
Correction in latitude of line CD = 0.51×(143.672 / 497.796) = 0.147
Correction in latitude of line DE = 0.51×(104.971 / 497.796) = 0.108
Correction in latitude of line EA = 0.51×(118.579 / 497.796) = 0.121

Similarly,

Correction in departure of line AB = 0.224×(63.335 / 638.756) = 0.022
Correction in departure of line BC = 0.224×(209.103 / 638.756) = 0.073
Correction in departure of line CD = 0.224×(47.052 / 638.756) = 0.017
Correction in departure of line DE = 0.224×(119.557 / 638.756) = 0.042
Correction in departure of line EA = 0.224×(199.709 / 638.756) = 0.070

For corrected coordinates,
Corrected latitude= latitude - correction in latitude (since (∑L) is positive, correction will be negative)
Corrected departure= departure- correction in departure

Line	Length	WCB	Consecutive coordinate		Correction		Corrected coordinate	
			Latitude (Lcosθ)	Departure (Lsinθ)	Latitude	Departure	Latitude	Departure
AB	89.31	45°10′	62.968	63.335	0.065	0.022	62.903	63.313
BC	219.76	72°05′	67.606	209.103	0.069	0.073	67.537	209.03
CD	151.18	161°52′	-143.672	47.052	0.147	0.017	-143.819	47.035
DE	159.10	228°43′	-104.971	-119.557	0.108	0.042	-105.079	-110.5
EA	232.26	300°42′	118.579	-199.709	0.121	0.070	118.458	-199.779
			∑L=0.51 ∑Lt=497.796	∑D=0.224 ∑Dt=638.756	∑L=0.51	∑D=0.224	∑L=0	∑D=0

Example 1.33:
In a closed traverse, calculate the length and bearing of the missing traverse leg BC.

Line	**Length (m)**	**Bearing**
AB	89.31	45°10′
BC	L	θ
CD	151.18	161°52′
DE	159.1	228°43′
EA	232.26	300°42′

Solution:
For a closed traverse,
∑L= 0 (Sum of latitude is zero)
$62.968 + L\cos\theta - 143.672 - 104.971 + 118.579 = 0$
$L\cos\theta = 67.096$ ……….…(i)
∑D=0 (Sum of departure is zero)
$63.335 + L\sin\theta + 47.052 - 119.557 - 199.709 = 0$
$L\sin\theta = 208.879$ ………..(ii)

Dividing equation (ii) by (i)
$\tan\theta = (208.879 / 67.096)$
$\theta = 72°11′31.1″$
Putting value of y on any equation, we get-
L = 219.391 m

Example 1.34:
Calculate the missing data in a closed traverse-

Line	**Length (m)**	**Bearing**
AB	89.31	45°10′

BC	219.76	72º05′
CD	151.18	161º52′
DE	?	228º43′
EA	232.26	?

Solution:

Consider a closed traverse ABCDA, where the length of line AD be x and bearing θ.

For a closed traverse ABCDA,

$\sum L = 0$ (Sum of latitude is zero)

$62.968 + 67.606 + x \cos\theta - 143.672 = 0$

$x \cos\theta = 13.098$ ……..…(i)

$\sum D = 0$ (Sum of departure is zero)

$63.335 + 209.103 + x \sin\theta + 47.052 = 0$

$x \sin\theta = -319.49$ ……...(ii)

From eqn (i) and (ii)

$\tan\theta = -24.392$

$\theta = -87^{o}39'8.58''$ (can't be negative)

$= 360^{o} - 87^{o}39'8.58''$

$= 272^{o}20'57.42''$

$x = 319.758$ m

Now, in triangle AED, using sine formula;

$\{(\sin\angle D) / 232.26\} = \{(\sin\angle A) / ED\} = \{(\sin\angle E) / 319.758\}$

Then,

$\angle D = 272^{o}20'51.42'' - 228^{o}43'$

$= 43^{o}37'51.42''$

$((\sin 43^{o}37'51.42'') / 232.26) = ((\sin\angle E) / 319.758)$

$\sin\angle E = 0.958$

$\angle E = 71^{o}47'48.37''$

Using cosine law,

$\cos D = ((AD^2 + ED^2 - AE^2) / 2(AD)(ED))$

$\cos 43^{o}37'51.42'' = (319.758^2 + ED^2 - 232.26^2) / 2\times319.758\times ED$

$ED = 158.982$ m

Again,

$((\sin\angle D) / 232.26) = ((\sin\angle A) / ED)$

$\sin\angle A = 0.472$

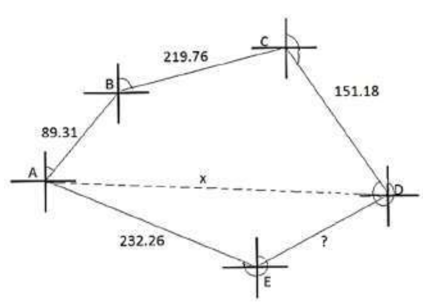
$\angle A = 28^{o}11'4.21''$

Then,

Bearing of DE = $228^{o}43'$

Bearing of ED = $228^{o}43' - 180^{o} = 48^{o}43'$

Bearing of EA = $360^{o} - 71^{o}47'48.37'' + 48^{o}43' = 336^{o}55'11.63''$



Exercises for Practice

(A) Short Questions

1.35. Write the basic principle of surveying.

1.36. Why is it important to have a knowledge of surveying to a civil engineer?
1.37. Discuss various maps and the scale of topographic maps prepared by Survey of India.
1.38. Draw the symbol and write colours of various features; Railway line, Contours, Landslide, BM, Bridge, Temple, Canal, Road, and Underground tunnel.
1.39. Describe, Survey Station, Survey Lines in a traverse.
1.40. Write the criteria used while making an appropriate selection of traverse stations.
1.41. What are the various methods, employed for distance measurement on the Earth surface?
1.42. What do you understand by the term *Ranging* in surveying? How do you carry out ranging process in the field?
1.43. What is Local Attraction in compass measurement? How do you detect the presence of Local Attraction at a station?
1.44. What is the use of a Levelling Staff? List types of staffs used? What is the least count of levelling staff?
1.45. What are the checks applied to the computed RLs by both the methods?
1.46. Define the following terms; Contour, Contour interval, Horizontal equivalent,
1.47. Draw the contours of a Vertical cliff, Overhanging cliff, Steep slope, and valley.
1.48. What is the use of Plane Table in surveying? List the other accessories and equipment used along with the Plane Table for mapping work.
1.49 Define the terms: Axis of the telescope, Vertical axis of a Theodolite, Trunnion axis of a Theodolite.
1.50. What is a Tacheometry? How do you determine the distance by Tacheometry on a flat ground?
1.51. Write the difference between Triangulation and Trilateration in surveying?

(B) Long Questions
1.52. Describe, how the surveying technology has developed in India.
1.53. Describe various types of surveying, based on area covered, based on instruments used, and based on purpose of survey.
1.54. Discuss various sources of errors, likely to be present in survey observations.
1.55. Define the following- True bearing, Magnetic bearing, Whole circle bearing, Quadrantal bearing, Magnetic declination, Fore bearing and Back bearing.
1.56. What is relationship between (i) True bearing and Magnetic bearing, (ii) WCB and QB, (iii) Fore bearing and Back bearing of a line, and (iv) Bearing and included angle.
1.57. Draw neat sketch of Prismatic Compass and explain various components and their function. What is the least count of this compass?
1.58. Define various terms with the help of neat sketches; Levels surface, Level line, Horizontal line, Plumb line, Datum, Mean sea level, Reduced level, Beach mark, Line of sight, Line of collimation, and Height of Instrument, Diaphragm.
1.59. Define Back sight, Intermediate sight, Change point and Fore sight in levelling. Why do you normally keep Back sight and Fore sight distances nearly equal in levelling?
1.60. Discuss various types of levels used in levelling work.
1.61. Write the salient characteristics of an Auto level and a Laser level.
1.62. Explain the steps to be followed for temporary adjustment of a level.
1.63. Discuss the two methods of reduction of levelling observations
1.64. Explain Profile levelling and Cross-section levelling. What is their utility in civil engineering projects?
1.65. Establish a relationship to find the collimation error using Reciprocal Levelling method.

1.66. What are various sources of errors that might be present in levelling observations, and how to minimise them?
1.67. Discuss the factors which decide the contour interval.
1.68. Describe the characteristics of contour maps.
1.69. List various applications of a contour map.
1.70. What are the different method to compute area and volume from a topographic map? Write the relationships.
1.71. List the advantages and disadvantages of Plane Tabling surveying.
1.72. Draw a diagram of Vernier theodolite and write its various parts.
1.73. Differentiate between (i) Transit and Swing of a Theodolite, (ii) Face left and Face right observations, and (iii) Reiteration method and Repetition method of observations.
1.74. Explain the use of a Theodolite (i) For prolonging a straight line, and (ii) as a Level.
1.75. Establish a relationship to compute the distance when the staff is not held vertical.
1.76. What is the purpose of Trigonometrical levelling? Write a simple relationship to determine the height of a building when the base is accessible.
1.77. Determine a relationship to determine the height of a tower when the base is inaccessible and it is not possible to set the instrument in the line of sight of the tower.
1.78. What is the basic purpose of Traversing? Write the steps involved in taking the observations of a traverse and adjusting the observations to compute the coordinates of traverse stations.
1.79. What are the different methods of adjustment of closing error in a closed traverse? Write the relationships.
1.80. Discuss the types of Triangulation schemes.
1.81. Define the terms in triangulation; Centered figure, Base line, Laplace station, Satellite station, Signals, Axis signal correction, Reduction to centre, and Accuracy of triangulation.

(C) Unsolved Examples

1.82. A levelling is a carried out to establish the RL of a point C with respect to BM of 100.00 m RL at A. Compute the RL of point C. The staff readings are given below.

Staff station	BS (m)	FS (m)
A	1.545	-
B	-0.860	-1.420
C		0.835

(Ans: RL of C = 101.27 m)

1.83. The staff reading taken on a point of RL 40.500 m below the bridge is 0.645 m. The inverted staff reading taken at the bottom of the deck of the bridge is -2.960 m. Compute the reduced level of the bottom of the deck point.

(Ans: RL of bottom of the deck = 44.105 m)

1.84. During a theodolite survey the following details were noted:

Line	Length (m)	Back Bearing
AB	550	60^{O}
BC	1200	115^{O}
CD	?	?
DA	1050	310^{O}

Calculate the length and bearing of the line CD.

(Ans: Bearing of CD, θ = S 58^o 49′ W = 238^o 49′, Length of CD = 855.15 m)

1.85. In a closed traverse ABCDE, it is required to find length and bearing of AE.

Following is the record of readings.

Line	Length (m)	Bearing
AB	130.5	N20^{O}30′ E
BC	215.0	N60^{O}15′ E
CD	155.5	N30^{O}30′ E
DE	120.0	N30^{O}30′ E

(Ans: Bearing of EA, θ = S 38^{o} 35′W = 218^{o} 35′, Length of EA = 596.51 m)

1.86. It is not possible to measure the length and fix the direction of a line AB directly on account of an obstruction between the stations A and B. A traverse ACDB was therefore run and the following data was obtained.

Line	Length (m)	Reduced Bearing
AC	45	N 50^{O} E
CD	66	S 70^{O} E
DB	60	S 30^{O} E

Find the length and direction of line BA

(Ans: Bearing of BA, θ = N 70^{o} 9′ W = 289^{o} 51′, Length of BA = 134.32 m)

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