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PART - A

2 Marks Questions

Q.1

Sol. Fundamental principles of land surveying:

1. Measurement of location of a point from atleast two reference points.
2. Working from whole to part: In this, firstly, major control points with greater accuracy is determined and then minor details can be measured with less accuracy. By doing this, error involved in minor measurements won't get reflected in major measurements.

The various examples of land surveying includes:

1. Topographical survey
2. Cadastral survey
3. City survey

Q.2

Sol. Systematic errors versus accidental errors

Systematic errors	Accidental errors
These errors are called as systematic because they always follow a definite pattern or a mathematical/ physical law. These are also called as cumulative errors.	This type of error occurs due to human limitation in reading an observation. These are also called as random errors or compensating errors.
This type of error makes the result either too small	When a large number of observations are made, then they use to cancel out because there is equal probability of the error to be positive or negative.

Q.3

Sol. When the chain is too long, it is adjusted by

- (a) Closing up the joints of the rings,
- (b) Hammering the elongated rings,
- (c) Replacing some old rings by new rings, and
- (d) Removing some of the rings.

Q.4

Sol. Given data:

True length of chain, $L = 20 \text{ m}$
 Error in chain, $e = 3 \text{ cm} = 0.03 \text{ m}$, too long
 $L' = L + e = 20 + 0.03 = 20.03 \text{ m}$
 Measured length = 327 m

$$\begin{aligned} \text{True length of line} &= \frac{L'}{L} \times ML \\ &= \frac{20.03}{20} \times 327 = 327.49 \text{ m} \end{aligned}$$

Q.5

Sol. When one chain length is measured on sloping ground then it shows a shorter distance on the horizontal plane. The difference between the sloping distance and horizontal distance is known as the hypotenusal allowance.

Q.6

Sol. Cadastral surveys are made incident to the fixing of property lines, the calculation of land area, or the transfer of land property from one owner to another. They are also made to fix the boundaries of municipalities and State and Federal jurisdictions.

Q.7

Sol:Magnetic bearing of line in 1940 = $S67^{\circ}30'E$

$$W.C.B. = 180^{\circ} - 67^{\circ}30' = 112^{\circ}30'$$

$$\begin{aligned}\text{True bearing} &= \text{Magnetic bearing} \pm \text{magnetic declination} \\ &= 112^{\circ}30' + 8^{\circ}0' = 120^{\circ}30'\end{aligned}$$

In 2007, declination was $4^{\circ}W$,

$$\text{True Bearing} = \text{Magnetic bearing} - \text{Magnetic declination}$$

 $\Rightarrow$

$$120^{\circ}30' = \text{Magnetic bearing} - 4^{\circ}0'$$

or,

$$\text{Magnetic bearing} = 124^{\circ}30'$$

Q.8

Sol. The line passing through points of equal declination is known as the isogonic line, and the one passing through points of zero declination is called the agonic line.

Q.9

Sol. Reciprocal levelling is a type of levelling used for determination of the difference of elevations of the two points which are situated quite apart & it is not possible to set up the instrument midway between the points and it eliminates error due to curvature & refraction.

Q.10

Sol. These are simple arithmetic calculations which are made to check the correctness of entries in height of instrument method or rise and fall method. The arithmetic calculations can be checked by using the following equation :

$$\Sigma \text{Back sight} - \Sigma \text{Fore sight} = \text{Last RL} - \text{first RL} = \Sigma \text{Rise} - \Sigma \text{Fall}$$

Thus, if the calculations on left hand side as well as on right hand side are equal, then the calculations made in filling the entries are correct.

Q.11

Sol: Let O be the instrument in station and A be staff station,

$$V = 2000 \tan 2^{\circ} = 69.84 \text{ m}$$

Since the distance of 2000 m is quite large, the correction for curvature and refraction must be applied.

Correction, $C = 0.0673 D^2$, where D is in km.

$$= 0.0673 \times \left(\frac{2000}{1000} \right)^2 = 0.2692 \text{ m}$$

Hence, reduced level of staff station

$$\begin{aligned}&= \text{RL of O} + \text{HI} + V - 5 + C \\ &= 300 + 69.84 + 0.2692 - 5 = 365.1092 \text{ m}\end{aligned}$$

Q.12

Sol. Face left observation : If the face of the vertical circle is to the left of the observer during observation the observation of the angle (horizontal or vertical) is known as face left observation.

Face right observation : If the face of the vertical circle is to the right of the observer during observation the observation is known as face right observation.

Q.13

Sol. A horizontal bar used in the substance system of surveying by tacheometry. It is held at a distant point and its distance is calculated from its known length and the angle that it subtends at the observer's eye.



Q.14

Sol. From the figure, it is clear that the required distance will be equal to $D_1 + D_2$.

Now
$$D_1 = \sqrt{\frac{50}{0.0673}} = 27.26 \text{ km}$$

$$D_2 = \sqrt{\frac{10}{0.0673}} = 12.19 \text{ km}$$

Hence, distance between man and object = $D_1 + D_2$
 $= 27.26 + 12.19 = 39.45 \text{ km}$

Q.15

Sol. If it is found after completion of the plane table survey that an important object has not been plotted then the two-point or three point problem is applied to locate a new station. These problems can be applied even if all the station pegs have been removed.

Q.16

Sol. Area of top portion, $A_1 = 10 \text{ m} \times 20 \text{ m}$

Area of bottom portion, $A_2 = 20 \text{ m} \times 40 \text{ m}$

Area of midportion
$$A_m = \left[\frac{(20-10)}{5} \times 2.5 + 10 \right] \times \left[\left(\frac{40-20}{5} \right) \times 2.5 + 20 \right] = 15 \text{ m} \times 30 \text{ m}$$

Depth of pit, $d = 5 \text{ m}$

As per the Prismoidal Rule, the volume of the pit will be given as

$$V = \frac{d}{6} [A_1 + A_2 + 4A_m] = \frac{5}{6} [(10 \times 20) + (20 \times 40) + (4 \times 15 \times 30)]$$

$$= 2333.33 \text{ m}^3$$

Q.17

Sol: A transition curve is provided for following purpose:

1. It allows a gradual transition of curvature from tangent to the circular curve to the tangent.
2. The radius of curvature increase or decrease gradually.
3. It is provided for gradual change in superelevation in a convenient manner.
4. It eliminates the danger of derailment, overturning or side slipping of vehicles and discomfort to the passengers.

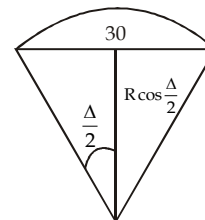
Q.18

Sol.
$$2R \sin\left(\frac{\Delta}{2}\right) = 30$$

$$R \sin \frac{\Delta}{2} = 15$$

$$R \sin 3^\circ = 15$$

$$R = \frac{15}{\sin 3^\circ} = 286.60 \text{ m}$$



Q.19

Sol. When the line of sight is not parallel to the horizontal line of collimation. In this case levels will not be correct. A test for checking the level is known as two peg test.

Q.20

Sol: Let H and f be flying height and f be focal length.

$$\therefore \text{Scale at 500 m} = \frac{10.50}{2350} = \frac{20}{H - 500} = \frac{f}{H - h}$$

$$\text{i.e.,} \quad H - 500 = \frac{20 \times 2350}{10.50}$$

$$\Rightarrow \text{Scale at 1200 m} = \frac{f}{H - h_{\text{new}}} = \frac{20}{(4776.19 - 1200)100}$$

$$= \frac{1}{18880.95} = \frac{1}{18881}$$

Scale is 1 cm = 188.81 m or 1 : 18881

Q.21

Sol: We know,
$$\text{Scale} = \frac{\text{Map distance/photo distance}}{\text{Ground distance}} = \frac{f}{H - h_{\text{avg}}}$$

Where

f = focal length of camera, cm

H = Flying height above MSL

h_{avg} = height of terrain above MSL

$$\Rightarrow \frac{20}{100 \times 10,000} = \frac{f}{(15000 - 5000) \times 100}$$

$$\Rightarrow f = 20 \text{ cm}$$

Hence, the focal length of the camera is 20 cm.

Q.22

Sol:
$$\therefore \frac{\text{Photo scale}}{\text{Map scale}} = \frac{\text{Photo distance}}{\text{Map distance}}$$

$$\Rightarrow \text{Photo scale} = \frac{10 \text{ cm}}{4 \text{ cm}} \times \frac{1}{40000} = \frac{1}{16000}$$

But,
$$\text{Photo scale} = \frac{f}{H - h}$$

$$\Rightarrow \frac{1}{16000} = \frac{22 \times 10^{-2} \text{ m}}{(H - 400) \text{ m}}$$

$$\Rightarrow H = 3920 \text{ m}$$

$$\therefore \text{Flying height of aircraft} = 3920 \text{ m}$$

Q.23

Sol. The vertical distance between two consecutive contours is called a contour interval.

The horizontal distance between two consecutive contours is known as horizontal equivalent.

Q.24

Sol. Most Probable Value : The most probable value (MPV) of a quantity is the value which has more chances of being true than any other value. This value is very close to the true value. It is found by measuring the quantity a number of times. If all the observations are given equal weightage then.

MPV = AM (Arithmetic Mean) of observed values. If observations are of different weightage then.

MPV = Weighted mean of observed values.

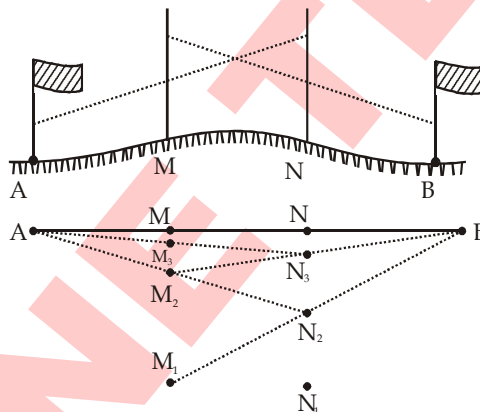


5 Marks Questions

Q.25

Sol. Indirect or Reciprocal ranging is resorted to when both the ends of the survey line are not intervisible either due to high intervening ground or due to long distance between them. In such a case, ranging is done indirectly by selecting two intermediate point M_1 and N_1 very near to the chain line (by judgement) in such a way that from M_1 , both N_1 and B are visible (Figure) and from N_1 , both M_1 and A are visible.

Two surveyors stations themselves at M_1 and N_1 with ranging rods. The person at M_1 then directs the person at N_1 to move to a new position N_2 in line M_1B . The person at N_2 then directs the person at M_1 to move to a new position M_2 in line with N_2A . Thus, the two persons are now at M_2 and N_2 which are nearer to the chain line than the positions M_1 and N_1 . The process is repeated till the points M and N are located in such a way that the person at M finds the person at N in line with MB, and the person at N finds the person at M in line with NA. After having established M and N, other points can be fixed by direct ranging.



(Figure: Reciprocal Ranging)

Q.26

Sol: Horizontal and Vertical controls in land surveying:

Horizontal and vertical controls are developed to create a framework around which other survey can be adjusted. These control surveys are used for accurate mapping projects in land surveying and many other high precision projects.

1. **Horizontal control:** Horizontal control coordinate horizontal positional data. These positions can be referenced by parallels or plane coordinate axes. Because they are used as framework for other surveys, this must be precise and accurate. Most horizontals should be connected to control network.
2. **Vertical Control:** A vertical control determines the elevation w.r.t. sea level. These are used as bench marks upon which land surveys are based and high degree of accuracy is required. In vertical control system atleast two permanent benchmark should be used but more may be required depending upon the needs and complexity of the object. These controls are needed for the construction of water supply and sewer systems, highway, bridges etc. These surveys can be done alone but are often done with horizontal control survey.

Q.27

Sol: Errors in chaining (Linear Measurements) : Errors in chaining are classified as:

- (i) Compensating Error
- (ii) Cumulative Errors.
- (i) **Compensating Error:** These are the errors which are liable to occur in both directions and tend to compensate. Compensating errors are proportional to the square root of the length of line.
The compensating error are caused by the following:
 - (a) Incorrect holding and marking of arrows.
 - (b) Fractional parts of the chain may not be correct i.e. the chain may not be calibrated uniformly.
 - (c) Plumbing may be incorrect while chaining by stepping on slopes.
 - (d) In setting chain angles with a chain.
- (ii) **Cumulative Error:** The cumulative errors are proportional to the length of the line and may be positive or negative.

Positive Cumulative Errors: These errors may be caused due to the following reasons:

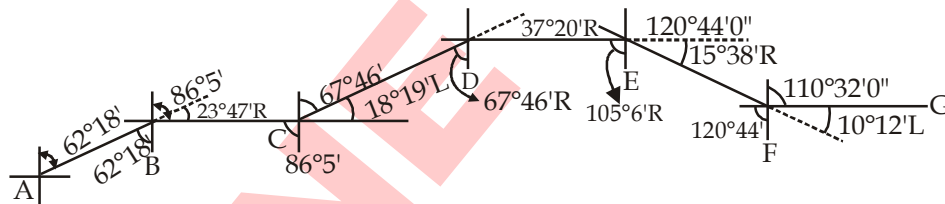
1. The length of chain is shorter than the standard length.
2. Not applying sag correction.
3. Not applying slope correction.
4. Bad ranging, bad straightening and wrong alignment.
5. Bending of links, knots in links, clogging of rings with muds etc.
6. Not applying temperature corrections, when field temperature is less than standard temperature.

Negative Cumulative Errors: These error may be caused due to following reasons:

1. Length of chain more than its standard lengths.
2. Not applying temperature correction when temperature during the measurement is more than standard Temperature.

Q.28

Sol:



The calculations are done based on geometry of open traverse as shown below.

$$\text{True bearing of BC} = 62^\circ 18' + 23^\circ 47' = 86^\circ 5'$$

$$\text{True bearing of CD} = 86^\circ 5' - 18^\circ 19' = 67^\circ 46'$$

$$\text{True bearing of DE} = 67^\circ 46' + 37^\circ 2' = 105^\circ 6'$$

$$\text{True bearing of EF} = 105^\circ 6' + 15^\circ 38' = 120^\circ 44'$$

$$\text{True bearing of FG} = 120^\circ 44' - 10^\circ 12' = 110^\circ 32'$$

Q.29

Sol.

$$l \cos \theta = -2 \quad \dots (i)$$

$$l \sin \theta = -2\sqrt{2} \quad \dots (ii)$$

Square of (i) and (ii) and sum of these

$$l^2 \cos^2 \theta + l^2 \sin^2 \theta = 4 + 8$$

$$l^2 = 12$$

$$l = \sqrt{12} \quad \dots (iii)$$

Since ΣL & ΣD both are negative hence bearing of closing error will be $180^\circ < \theta < 270^\circ$

$$\begin{aligned} \text{then} \quad \tan \theta &= \frac{-2\sqrt{2}}{-2} \\ \theta &= 54.73^\circ \text{ or } 234.73^\circ \end{aligned}$$

Hence W.C.B of closing error = 234.73°

RB of closing error = $S54.73^\circ W$

Q.30

Sol. Bowditch's Method : The basis of this method is on the assumptions that the errors in linear measurements are proportional to $\sqrt{l}$ and that the errors in angular measurements are inversely proportional to $\sqrt{l}$ where l is the length of a line. The Bowditch's rule, also termed as the compass rule, is mostly used to balance a traverse where linear and angular measurements are of equal precision. The total error in latitude, and in the departure is distributed in proportion to the lengths of the sides.

The Bowditch Rule is :

Correction to latitude (or departure) of any side =

$$\text{Total error in latitude (or departure)} \times \frac{\text{Length of that side}}{\text{Perimeter of traverse}}$$

Thus, if

C_L = Correction to latitude of any side

C_D = Correction to departure of any side.

ΣL = Total error in latitude.

ΣD = Total error in departure

Σl = Length of the perimeter

l = length of any side

We have

$$C_L = \Sigma L \cdot \frac{l}{\Sigma l} \quad \text{and} \quad C_D = \Sigma D \cdot \frac{l}{\Sigma l}$$

Q.31

Sol: Natural Errors: The several natural errors in the levelling are as follows:

- (i) Curvature: In case of a long sight, the horizontal line is not a level line due to curvature of the earth. The vertical distance between horizontal line and level line represent effect of curvature of the earth. (always -ve in nature)

$$C_c = 0.0785 D^2; \text{ where } D = \text{km}$$

- (ii) Refraction : Refraction of the ray passing through the atmosphere from the signal to the observer is the main source of external error. The average refraction correction can however taken as 1/7th of the curvature correction. (always +ve in nature)

$$C_R = 0.0112 D^2; \text{ where, } D = \text{Distance in kms}$$

- (iii) Error due to temperature and strong winds: The instrument may vibrate, disturbing the bubble and making difficult to hold the staff vertical in strong winds. Temperature variations do not affect much the staff. Under high temperatures, though the liquid in level tube expands and bubble is shortened but does not produce any error.

Q.32

Sol:

Station	BS	IS	FS	RISE	FALL	RL	Remarks
1	3.150					x	
2	1.770		x		0.700	x	CP
3		2.20			x	x	
4	x		1.850	x		x	CP
5		2.440			0.010	x	
6	x		x	1.100		x	CP
7	1.185		2.010	x		222.00	CP
8		-2.735		x		x	Inverted staff
9	x		1.685		4.420	x	CP
10			1.525		0.805	x	
Σ	12.055		x	x	x		

Station	BS	IS	FS	RISE	FALL	RL	Remarks
1	3.150					220.90	
2	1.770		3.850		0.700	220.20	CP
3		2.20			0.430	219.77	
4	2.430		1.850	0.35		220.12	CP
5		2.440			0.010	220.11	
6	2.800		1.340	1.100		221.21	CP
7	1.185		2.010	0.790		222.00	CP
8		-2.735		3.92		225.92	Inverted held staff
9	0.720		1.685		4.420	221.50	CP
10			1.525		0.805	220.695	
	Σ BS = 12.055		Σ FS = 12.26	Σ Rise = 6.16	Σ Fall = 6.365		Last RL - First RL = 220.695 - 220.90 = -0.2050
	Σ BS - Σ FS = 0.205			Σ Rise - Σ Fall = -0.2050			

Q.33

Sol: Characteristics of Contours : The characteristics are listed below:

1. All the points on a contour line have the same elevation. The elevations are indicated either by inserting the figure in a break in the respective contour or printed closed to contour. When no value is present, it indicates a flat terrain. A zero meter contour line represents the coastline.
2. Two contour lines do not intersect each other except in the cases of an overhanging cliff or cave penetrating a hillside.
3. A contour line must close onto itself not necessarily within limits of the maps.
4. Equally spaced contour represents a uniform slope and contours that are well apart indicate a gentle slope.
5. A set of close contours with higher figures inside and lower figure outside indicate a hillock whereas in the case of depressions, lakes etc. the higher figures are outside and the lower figures are made inside.
6. Irregular contours represent uneven ground.
7. The direction of the steepest slope is along the shortest distance.

Q.34

Sol: The uses of contour maps in civil engineering projects are as follows:

1. Drainage area estimation : The extent of drainage area may be estimated on contour map by locating the ridge line around the watershed.
2. Capacity of Reservoir : Reservoirs are made for water supply and for power or irrigation projects. A contour map is very useful to study the possible location of a dam and the volume of water to be confined. All contour lines are closed within the reservoir area.
3. Site of structures : The most economical and suitable site for the structure such as buildings, bridges, dams etc. can be found from large scale contour maps.
4. Earthwork Estimates : On the contour lines of original surface, the contours of the desired altered surface are drawn. The volume of a cut or a fill is found by multiplying the average by the contour interval.
5. Route location : By inspection a contour map, the most suitable site. For road, railway, canal etc. can be selected. By following the contour lines, steep gradients, cutting and filling etc. may be avoided.

Q.35

Sol:

Here,

 α = angle between line of sights in radians

D = distance of instrument from staff

n = number of divisions through which bubble is moved.

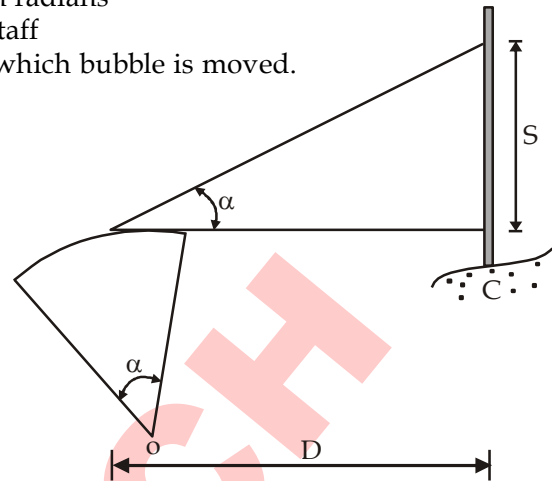
S = staff intercept

$$\therefore \alpha = \frac{S}{D} = n \left(\frac{l}{R} \right)$$

$$\Rightarrow \frac{l}{R} = \frac{S}{nD}$$

$$\Rightarrow \alpha' = \frac{l}{R} \frac{S}{nD} \times 206265 \text{ (in seconds)}$$

$$\therefore S = \frac{\alpha' n D}{206265} \Rightarrow \frac{20'' \times 4 \times 100}{206265} = 0.0388$$



Q.36

Sol:

Instrument at A:

Incorrect level difference between A and B = 2.847 - 1.543 = 1.304m

Instrument at B:

Incorrect level difference between A and B = 2.622 - 1.422 = 1.20m

True difference of level between A and B = Mean of two incorrect readings differences

$$= \frac{1.304 + 1.200}{2} = 1.252 \text{ m (fall from A to B)}$$

Extent of refraction correction:

Let x be extent of curvature and refraction correction,

Reading of A = 1.543 m

Fall from A to B = 1.252 m

Required reading of level line = 2.795 m = 1.543 + 1.252 m

Actual staff reading at B(refraction correction) = 2.795 ± x

Error in collimation adjustment = 0.002 m per 100 m

 $\therefore$ Reading after collimation error = 2.795 ± x + 0.015

$$= 2.81 \pm x$$

$$\therefore x = 2.847 - 2.810 = 0.037 \text{ m}$$

Refraction correction + curvature correction = 0.037 m

Q.37

Sol.

The points to be considered in deciding contour interval are

- (i) **The nature of the ground :** The contour interval depends upon whether the country is flat or highly undulated. For every flat ground, a small interval is necessary. If the ground is more broken, greater contour interval should be adopted, otherwise the contours will come too close to each other.
- (ii) **The scale of the map :** The contour interval should be inversely proportional to the scale. If the scale is small, the contour interval should be large. If the scale is large, the contour interval should be small.
- (iii) **The purpose and extent of the survey :** The contour interval largely depends upon the purpose and the extent of the survey. For example, if the survey is intended for detailed design work or for accurate earth work calculations, small contour interval is to be used. The extent of survey in such cases will generally be small. In the case of location surveys, for lines of communications and for reservoir and drainage areas, where the extent of survey is large, a large contour interval is to be used.
- (iv) **Time and expense of field and office work :** If the time available is less, greater contour interval should be used. If the contour interval is small, greater time will be taken in the field survey, in reduction and in plotting the map.

Q.38

Sol. **Barometric levelling :** Barometric levelling makes use of the phenomenon that difference in elevation between two points is proportional to the difference in atmospheric pressures at these points. A barometer, therefore, may be used and the readings observed at different points would yield a measure of the relative elevations of those points.

At a given point, the atmospheric pressure does not remain constant in the course of the day, even in the course of an hour. The method is, therefore, relatively inaccurate and is little used in surveying work except on reconnaissance or exploratory surveys.

(a) **Trigonometric Levelling (Indirect levelling) :**

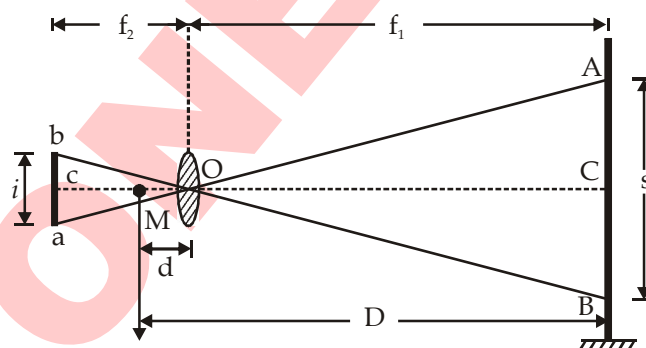
Trigonometric or Indirect levelling is the process of levelling in which the elevations of points are computed from the vertical angles and horizontal distances measured in the field, just as the length of any side in any triangle can be computed from proper trigonometric relations. In a modified form called stadia levelling, commonly used in mapping, both the difference in elevation and the horizontal distance between the points are directly computed from the measured vertical angles and staff readings.

(b) **Spirit Levelling (Direct Levelling) :**

It is that branch of levelling in which the vertical distances with respect to a horizontal line (perpendicular of gravity) may be used to determine the relative difference in elevation between two adjacent points. A horizontal plane of sight tangent to level surface at any point is readily established by means of a spirit level or a level vial. In spirit levelling, a spirit level and a sighting device (telescope) are combined and vertical distances are measured by observing on graduated rods placed on the points. The method is also known as direct levelling. It is the most precise method of determining elevations and the one most commonly used by engineers.

Q.39

Sol.



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 instrument.

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 $\triangle AOB$ and $\triangle aOb$ are similar. Hence

$$\frac{f_1}{f_2} = \frac{s}{i} \quad \dots(i)$$

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

$$\frac{1}{f} = \frac{1}{f_2} + \frac{1}{f_1} \quad \dots(ii)$$

Multiplying throughout by ff_1

$$\text{we get } f_1 = \frac{f_1}{f_2} f + f$$

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

$$\text{we get } f_1 = \frac{s}{i} f + f$$

The horizontal distance between the axis and the staff is

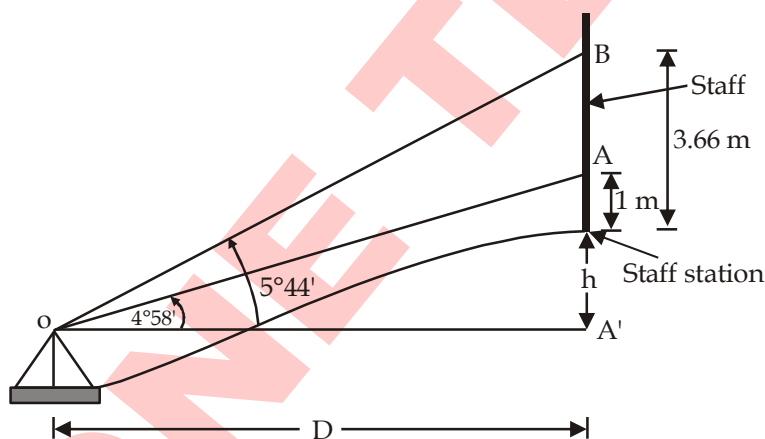
$$D = f_1 + d$$

or

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

Q.40

Sol:



Assuming staff bottom is below the horizontal axis.

Let D be the horizontal distance between the instrument and the staff and h be the height on the staff from the ground level to horizontal axis of the theodolite.

From geometry,

$$\text{In } \triangle AA'O \quad \tan 4^\circ 58' = \frac{1+h}{D} \quad \dots(i)$$

$$\text{In } \triangle BA'O \quad \tan 5^\circ 44' = \frac{3.66+h}{D} \quad \dots(ii)$$

Eq. (ii) - eq. (i) gives,

$$(\tan 5^\circ 44' - \tan 4^\circ 58') = \frac{3.66-1}{D}$$

$\Rightarrow$

$$D = 197.06 \text{ m}$$

$$\text{From eq. (i)} \quad h = (197.06 \tan 4^\circ 58' - 1) = 16.125 \text{ m}$$

$$\therefore \text{RL of the staff station} = (\text{RL of horizontal axis} + h) \\ = 254.30 + 16.125 = 270.425 \text{ m}$$



Q.41

Sol: Major disadvantages of Plane Tabling:

1. The plane table survey is not possible in unfavourable climates such as rain, fog etc.
2. This method of survey is not very accurate and thus unsuitable for large scale or precise work. It is suitable for small scales usually ranging from 20 m : 1 cm to 40 m : 1 cm.
3. The method requires large amount of time to be spent in the field.
4. The equipments used in the survey are heavy and cumbersome.
5. It requires open country for easy sight of the stations.
6. Time spent in this survey is much more than other types of surveying.
7. Quality of the map (final) largely depends upon the drafting capability of the surveyor.
8. As no field book is maintained, plotting at different scale requires full exercise.
9. Plane table survey is not very accurate.
10. For drawing contours in extensive area, it is inferior to photogrammetry

PART - C

20 Marks Questions

Q.42

Sol. There are several types of instruments used to set out a right angle to a chain line, the most common being (i) cross staff (ii) optical square (iii) prism square

1. Cross Staff :

The simplest instrument used for setting out right angles is a cross staff. It consists of either a frame or box with two pairs of vertical slits and is mounted on a pole shod for fixing in the ground. The common forms of cross staff are as follows:

- (a) **Open Cross Staff** : It is provided with two pairs of vertical slits giving two lines of sights at right angles to each other. The cross staff is set up at a point on the line from which the right angle is to run and is then turned until one line of sight passes through the ranging pole at the end of the survey line. The line of sight through the other two vanes will be a line at right angles to the survey line at right angles to the survey line and a ranging rod may be established in that direction. If, however, it is to be used to take offsets, it is held vertically on the chain line at a point where the foot of the offsets is likely to occur. It is then turned so that one line of sight passes through the ranging rod fixed at the end of the survey line. Looking through the other pair of slits, it is seen if the point to which the offset is to be taken is bisected. If not, the cross staff is moved backward or forward till the line sight also passes through the point.
- (b) **French Cross Staff** : It consists of a hollow octagonal box. Vertical sighting slits are cut in the middle of each face, such that the lines between the centres of opposite slits make angles of 45° with each other. It is possible, therefore, to set out angles of either 45° or 90° with this instrument.

2. Optical Square : Optical square is somewhat more convenient and accurate instrument than the cross staff for setting out a line at right angles figure to another line. Figure: Optical square(a) illustrates the principle on which it works.

It consists of a circular box with three slits at E, F and G. In line with the openings E and G, a glass silvered at the top and unsilvered at the bottom, is fixed facing the opening E. Opposite to the opening F, a silvered glass is fixed at A making an angle of 45° to the previous glass. A ray from the ranging rod at Q passes through the lower unsilvered portion of the mirror at B, and is seen directly by eye at the slit E. Another ray from the object at P is received by the mirror at A and is reflected towards the mirror at B which reflects it towards the eye. Thus, the images of P and Q are visible at B. If both the images are in the same vertical line as shown in figure (b), the line PD and QD will be at right angles to each other.



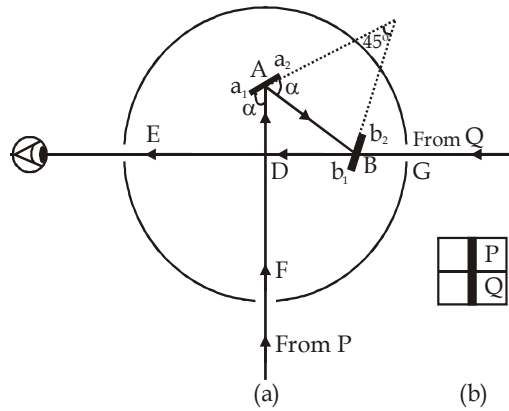


Figure : Optical Square

Let the ray PA make an angle α with the mirror at A,

$$\angle ACB = 45^\circ \quad \text{or} \quad \angle ABC = 180^\circ - (45^\circ + \alpha) = 135^\circ - \alpha$$

By law of reflection

$$\angle EBB_1 = \angle ABC = 135^\circ - \alpha$$

Hence

$$\angle ABE = 180^\circ - 2(135^\circ - \alpha) = 2\alpha - 90^\circ \quad \dots(i)$$

Also

$$\angle DAB = 180^\circ - 2\alpha \quad \dots(ii)$$

From $\triangle ABD$,

$$\begin{aligned} \angle ADB &= 180^\circ - (2\alpha - 90^\circ) - (180^\circ - 2\alpha) \\ &= 180^\circ - 2\alpha + 90^\circ - 180^\circ + 2\alpha = 90^\circ \end{aligned}$$

Thus, if the images of P and Q lie in the same vertical line, as shown in figure (b) the line PD and QD will be at right angles to each other.

3. **Prism Square** : The prism square shown in Figure below works on the same principle as that of optical square. It is a more modern and precise instrument and is used in a similar manner. It has the merit that no adjustment is required since the angle between the reflecting surfaces (i.e. 45°) cannot vary.

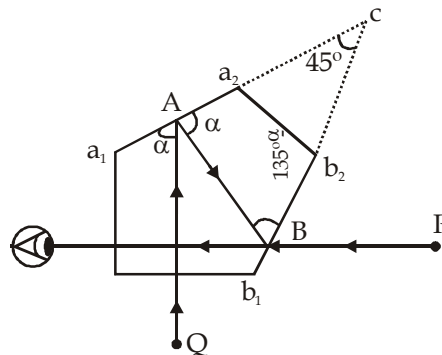
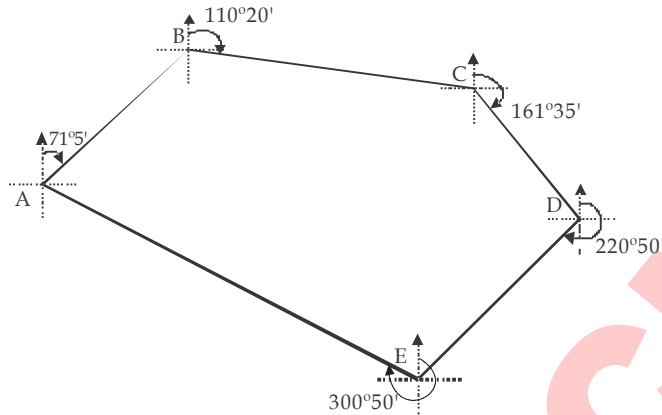


Figure : Prism Square

Q.43

Sol.

By inspection, we find that there is no line whose F.B. and B.B. differ exactly by 180° . However, the F.B. and B.B. of line CD differ by $180^\circ 10'$ the difference being only $+10'$. Hence the correct F.B. of CD is obtained by adding half the difference.



Hence corrected F.B. of CD = $161^\circ 35' + 5' = 161^\circ 40'$

and Corrected B.B. of CD = $341^\circ 45' - 5' = 341^\circ 40'$

Difference = $180^\circ 0'$

$$\angle ABC = 250^\circ 20' - 110^\circ 20' = 140^\circ 0'$$

$$\angle BCD = 290^\circ 35' - 161^\circ 35' = 131^\circ 0'$$

$$\angle CDE = 341^\circ 45' - 220^\circ 50' = 120^\circ 55'$$

$$\begin{aligned} \angle DEA &= 300^\circ 50' - 40^\circ 05' = 260^\circ 45' \text{ (Exterior)} \\ &= 99^\circ 15' \text{ (Interior)} \end{aligned}$$

$$\angle EAB = 121^\circ 10' - 71^\circ 5' = 50^\circ 5'$$

Sum = $541^\circ 15'$

$$\text{Theoretical sum} = (2N - 4) 90^\circ = 540^\circ$$

$$\text{Error} = 541^\circ 15' - 540^\circ = 1^\circ 15'$$

$\therefore$
 $\therefore$

Correction for each angle = $-15'$

Hence the corrected angles are

$$\angle ABC = 140^\circ 0' - 15' = 139^\circ 45'$$

$$\angle BCD = 131^\circ 0' - 15' = 130^\circ 45'$$

$$\angle CDE = 120^\circ 55' - 15' = 120^\circ 40'$$

$$\angle DEA = 99^\circ 15' - 15' = 99^\circ 00'$$

$$\angle EAB = 50^\circ 05' - 15' = 49^\circ 50'$$

Sum = $540^\circ 00'$

The corrected bearings of all the lines are obtained from the included angles and the corrected bearing of lines are as follows:

Corrected	F.B. of DE	= $341^\circ 40' - 120^\circ 40'$	= $221^\circ 00'$
	B.B. of DE	= $221^\circ 00' - 180^\circ$	= $41^\circ 00'$
	F.B. of EA	= $41^\circ 00' + 261^\circ$	= $302^\circ 00'$
	B.B. of EA	= $302^\circ 00' - 180^\circ$	= $122^\circ 00'$
	F.B. of AB	= $122^\circ 00' - 49^\circ 50'$	= $72^\circ 10'$
	B.B. of AB	= $72^\circ 10' + 180^\circ$	= $252^\circ 10'$
	F.B. of BC	= $252^\circ 10' - 139^\circ 45'$	= $112^\circ 25'$
	B.B. of BC	= $112^\circ 25' + 180^\circ$	= $292^\circ 25'$
	F.B. of CD	= $292^\circ 25' - 130^\circ 45'$	= $161^\circ 40'$ (check)

Q.44

Sol.

RL of station 1 = 190.75 m
 BS of station 1 = 0.780 m
 $\therefore$ Height of Instrument = 190.75 + 0.780
 = 191.53 m
 RL of station 2 = HOI - IS of station 2
 or 191.53 - 1.535 = 189.995 m
 Similarly, RL of station 3 = 191.530 - 1.955 = 189.575 m
 RL of station 4 = 191.530 - 2.430 = 189.1 m
 RL of station 5 = 191.530 - 2.985 = 188.545 m
 Now the station 5 is the change point
 So Height of Instrument = 188.545 + 1.155 = 189.7 m
 Now, RL of station 6 = 189.7 - 1.960 = 187.74 m
 RL of station 7 = 189.7 - 2.365 = 187.335 m
 RL of station 8 = 189.7 - 2.895 = 186.805 m
 Now the station 8 is the change point
 So Height of Instrument = 186.805 + 0.935 = 187.74 m
 RL of station 9 = 187.74 - 1.045 = 186.695 m
 RL of station 10 = 187.74 - 1.635 = 186.105 m
 RL of station 11 = 187.74 - 2.545 = 185.195 m
 R.L. of station 1 = 190.75 m
 R.L. of last station = 185.195
 Distance between first & last station = 10 $\times$ 30 = 300 m
 Gradient of line joining first & last point

$$= \frac{190.75 - 185.195}{300} = \frac{1}{54}$$

Station	BS	IS	FS	HOI	RL	Remarks
1	0.780			191.53	190.750	BM
2		1.535			189.995	
3		1.955			189.575	
4		2.430			189.1	
5	1.155		2.985	189.7	188.545	CP
6		1.960			187.74	
7		2.365			187.335	
8	0.935		2.895	187.74	186.805	CP
9		1.045			186.695	
10		1.635			186.105	
11			2.545		185.195	

Now applying check,

$$\text{Last RL} - \text{First RL} = -5.555 \text{ m}$$

$$\Sigma \text{BS} - \Sigma \text{FS} = 2.87 - 8.425 \Rightarrow -5.555 \text{ m}$$

$$\therefore \text{Last RL} - \text{First RL} = \Sigma \text{BS} - \Sigma \text{FS}$$

Q.45

Sol: Case I: Instrument at C and staff at A:

$$\theta_1 = 6^\circ 30' \text{ (Angle of depression); } S = 3.50 - 2.130 = 1.37$$

$$CA = D = KS \cos^2 \theta_1 + C \cos \theta_1$$

$$D = 50 \times 1.37 \times \cos^2 6^\circ 30' + 0.50 \cos 6^\circ 30'$$

$$D = 67.622 + 0.49678 = 68.12 \text{ m}$$

⇒

$$V = \frac{KS \sin 2\theta_1}{2} + C \sin \theta_1$$

$$= \frac{(50 \times 1.37 \times \sin 13^\circ)}{2 + 0.50 \sin 6^\circ 30'}$$

$$= \frac{15.409}{2} + 0.0566 = 7.7045 + 0.0566 = 7.761 \text{ m}$$

Let the R.L of C be X and h be height of instrument,

Then,

$$\text{RL of A} = \text{RL of C} + h - V - r \text{ (central hair reading)}$$

⇒

$$100 = \text{RL of C} + h - 7.761 - 2.815$$

⇒

$$\text{RL of C} + h = 110.576 \text{ m}$$

Case II: Instrument at C and staff at B:

$$\theta_2 = -8^\circ 30'; S = 1.870 - 0.110 = 1.76 \text{ m}$$

$$CB = D = KS \cos^2 \theta_2 + C \cos \theta_2$$

$$D = 50 \times 1.76 \cos^2 8^\circ 30' + 0.50 \cos 8^\circ 30'$$

$$D = 86.078 + 0.494 = 86.573 \text{ m}$$

⇒

$$V = \frac{KS \sin 2\theta_2}{2} + C \sin \theta_1 = \frac{50 \times 1.76 \sin 17^\circ}{2} + 0.50 \sin 8^\circ 30'$$

$$= 12.938 \text{ m}$$

$$\begin{aligned} \text{RL of station B} &= \text{RL of C} + h - V - r \\ &= 110.576 - 12.938 - 0.990 \\ &= 96.648 \end{aligned}$$

The reduced level of station B is 96.648 m.

Q.46

Sol.

Since road level is to be maintained at a constant RL of 197 m and RLs of surface level as observed are greater than 197 m. Thus entire road is in cutting.

Chainage (m)	0	20	40	60	80	100
Surface level along C/L of road (m)	204.6	203.0	200.8	201.6	202.0	200.2
Depth of cutting of (m)	7.6	6	3.8	4.6	5	3.2

Area of cross section is given by ,

$$A = (b + sh) h$$

∴

$$A_1 = (5 + 2 \times 7.6) 7.6 = 153.52 \text{ m}^2$$

$$A_2 = (5 + 2 \times 6) 6 = 102 \text{ m}^2$$

$$A_3 = (5 + 2 \times 3.8) 3.8 = 47.88 \text{ m}^2$$

$$A_4 = (5 + 2 \times 4.6) 4.6 = 65.32 \text{ m}^2$$

$$A_5 = (5 + 2 \times 5) 5 = 75 \text{ m}^2$$

$$A_6 = (5 + 2 \times 3.2) 3.2 = 36.48 \text{ m}^2$$

Chainage interval,

$$d = 20 \text{ m}$$



Volume from trapezoidal rule

$$V = L \left[\frac{A_1 + A_6}{2} + A_2 + A_3 + A_4 + A_5 \right]$$

$$= 20 \left[\frac{153.52 + 36.48}{2} + 102 + 47.88 + 65.32 + 75 \right] = 7704 \text{ m}^3$$

Volume from prismoidal rule

Here number of sections are six and this formula is applicable for odd number sections only, thus this formula is applied from sections one to five and the resulting answer is clubbed with sixth section to give final volume by trapezoidal rule.

$$V_1 = \frac{L}{3} [(A_1 + A_5) + 4(A_2 + A_4) + 2A_3]$$

$$= \frac{20}{3} [(153.52 + 75) + 4(102 + 65.32) + 2 \times 47.88] = 6623.733 \text{ m}^3$$

$$V_2 = L \left(\frac{A_5 + A_6}{2} \right) = \left(\frac{75 + 36.48}{2} \right) \times 20 = 1114.8 \text{ m}^3$$

$$V = V_1 + V_2 = 6623.733 + 1114.8 = 7738.533 \text{ m}^3$$

Q.47

Sol: Precautions in theodolite observations: The following precautions are as follows:-

1. The axis of the plate level must be perpendicular to the vertical axis.
2. The line of collimation must be at right angles to the horizontal axis.
3. The horizontal axis must be perpendicular to the vertical axis.
4. The axis of the telescope level must be parallel to the line of collimation.
5. Make plate bubbles central to their run when the vertical axis of the theodolite is truly vertical.
6. Make the vertical cross hair lie in the plane perpendicular to the horizontal axis.
7. Make the line of sight perpendicular to the horizontal axis.
8. Make the horizontal axis perpendicular to the vertical axis.
9. Make the telescope bubble central when the line of sight is horizontal.
10. Make the vertical circle indicate zero when the line of sight is perpendicular to the vertical axis.

Fundamental lines of a theodolite: The fundamental lines of theodolite are the vertical axis, axis of plate

levels, the line of collimation, the horizontal axis and the bubble line of altitude. The following relationships are desired:

1. The axis of the plate levels must be perpendicular to the vertical axis.
2. The line of collimation must be right angles to the horizontal axis.
3. The horizontal axis must be perpendicular to the vertical axis.
4. The axis of the telescope level must be parallel to the line of collimation.

Errors: The following errors arise if such relationships are not maintained:

1. Vertical axis error : Axis not vertical in an observations either from imperfect plate level adjustment or settlement of instrument.
2. Lateral collimation error : Line of collimation not perpendicular to the horizontal axis.
3. Horizontal axis error : Horizontal axis not perpendicular to the vertical axis.
4. Vertical collimation error : Line of altitude bubble not parallel to the line of collimation when the verniers of vertical circle read zero.



Q.48

Sol:

Objectives of triangulation surveys: The triangulation surveys are carried out

- (i) To establish accurate control for plane and geodetic surveys of large areas, by terrestrial methods.
- (ii) To establish accurate control for photogrammetric surveys of the large areas.
- (iii) To assist in the determination of the size and shape of the earth by making observations for latitude, longitude and gravity.
- (iv) To determine accurate locations of points in engineering works such as fixing of centreline and abutments of long bridges over large rivers. etc.

Criteria for selection of layouts of triangles: The criteria while deciding and selecting a suitable layout of triangles are as follows:

1. Simple triangles should be preferably equilateral.
2. Braced quadrilaterals should be preferably equilateral.
3. Centered polygons are regular.
4. The arrangements should be such that computations can be done through two or more independent routes.
5. The arrangements should be such that at least one route and preferably two routes from well conditioned triangles.
6. No angle of the figure opposite known side should be small.
7. The sides of figures should be of comparable lengths. Very long and very short lines should be avoided.
8. Angles of similar triangles should not be less than 45° and in case of quadrilaterals no angles should be less than 30° .

Well conditioned triangles:

- The triangles of such a shape, in which any error in angular measurement has minimum effect upon the computed lengths is known as well-conditioned triangles.
- The best shape of an isosceles triangle is that triangle whose base angles are $56^\circ 14'$ each. However, from practical considerations. An equilateral triangle may be treated as well condition triangle. In actual practice, the triangles having an angle less than 30° or more than 120° should not be considered. Strength of figure: The strength of figure is a factor to be considered in establishing a triangulation system to maintain the computations within the desired degree of precision. It plays also an important role in deciding layout of a triangulation system.

Q.49

Sol:

Principles of triangulation : It is the process of measuring the angles of a chain or network of triangles formed by stations marked on the surface of the earth.

- (i) In triangulation, all the three angles of each triangle are measured in the field alongwith one baseline.
- (ii) The side of the first triangle whose length is predetermined is called the baseline and the vertices of the individual triangles are known as triangulation stations and the whole figure is called the triangulation system or triangulation figure.
- (iii) The length and azimuth of each line is based on the length and azimuth of the preceding line.
- (iv) To minimize and accumulation of errors in lengths, subsidiary bases at suitable intervals are provided.
- (v) To control errors in azimuth of stations, astronomical observations are made at intermediate stations.
- (vi) The triangulation stations at which astronomical observations for azimuth are made are called Laplace station.

Factors affecting selection of baseline for triangulation :

The measurement of baseline forms the most important part of the triangulation operations.

The baseline is laid down with great accuracy of measurement and alignment as it forms the basis for the computations of triangulation system.

Selection of site for baseline:

1. The site should be fairly level. If however, the ground is sloping, the slope should be uniform and gentle.



2. The site should be free from obstructions throughout the whole of the length. The line clearing should be cheap in both labour and compensation.
3. The extremities of the base should be intervisible at ground level.
4. The ground should be reasonably firm and smooth. Water gaps should be few, and if possible not wider than length of the long wire or tape.
5. The side should suit extension to primary triangulation. This is an important factor since the error in extension is likely to exceed the error in the measurement.

Q.50

Sol: **Types of satellites: (Based on Inclination of orbital plane):**

1. Geostationary or geosynchronous earth orbit satellite: Geosatellites are synchronous with respect to earth. Looking from fixed point from earth, these satellites appear to be stationary. The orbit of these satellites is circular. These satellites must travel in the rotational speed of earth and in the direction of motion of earth, that is eastward. The inclination of satellite wrt earth must be zero. These satellites are used for TV and radio broadcast weather forecast and also, these satellites are operating as backbones for telephone networks.
2. Low Earth Orbit (LEO) Satellites: These satellites are placed 500-1500 km above the surface of earth. As LEO circulate on lower orbit, hence, they exhibit a much shorter period that is 95 to 120 minutes. These satellites are mainly used in remote sensing as providing mobile communication services due to lower latency.
3. Medium Earth Orbit (MEO) Satellite: MEOs can be positioned somewhere between LEOs and GEOs both in terms of their orbit and due to their advantage and disadvantages. Using orbits around 10,000 km the system only requires a dozen satellites which is more than a GEO system but much less than a LEO system.
4. Sun-Synchronous Orbits Satellite: These satellites rise and set with the sun. Their orbit is defined in such a way that they are always facing the sun and hence they never go through eclipse.
5. Prograde Orbit: This orbit is with an inclination of less than 90° . Its direction is the same as direction of rotation of primary (planet).
6. Retrograde Orbit: The orbit is with inclination of more than 90° . Its direction is counter to the direction of rotation of the planet.
7. Polar Orbit: This orbit passes above or nearly above both poles (north and south poles) of the planet on each of its revolutions. Therefore, it has an inclination of (or very close) to 90 degrees. These orbits are highly inclined in shape.

Remote sensing : Remote sensing means sensing of the earth's surface from space by making use of properties of electromagnetic waves emitted, reflected, or defracted by the sensed objects, for the purpose of improving resource management, land use and the protection of environment. Basically, remote sensing is concerned with the detection and recording of electromagnetic radiation. The instruments for recording electromagnetic radiations are called sensors. The sensor systems are required to be placed on suitable observation platforms which can be lifted to a predefined altitude. These may be stationary (like a tripod for field observation) or mobile (aircraft and satellites).

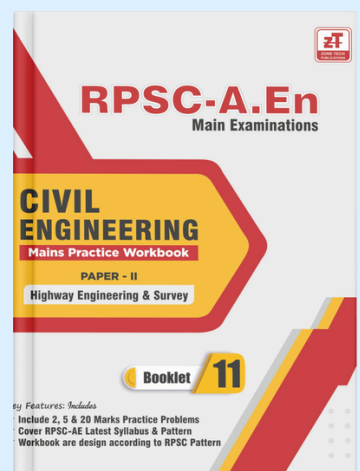
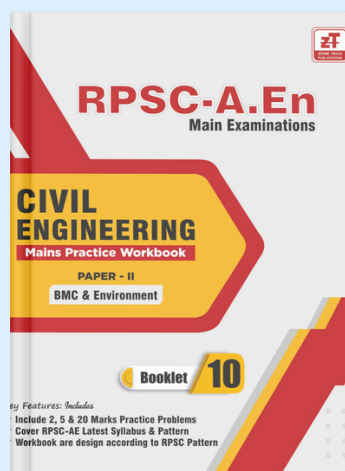
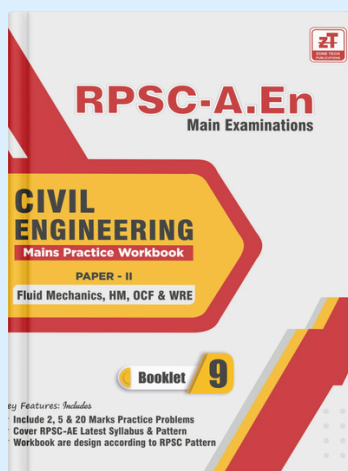
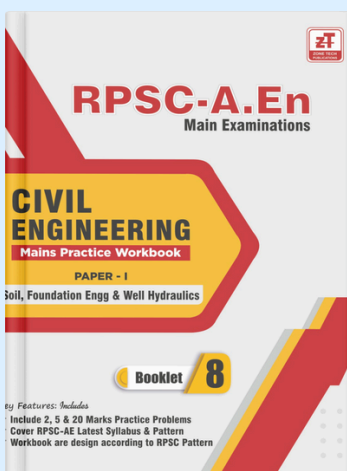
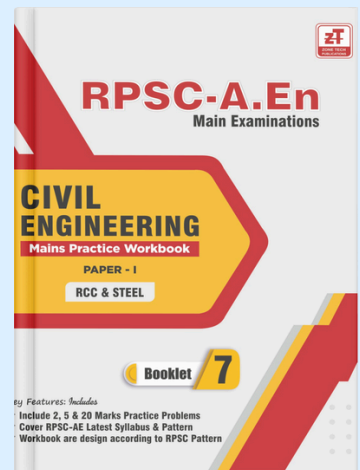
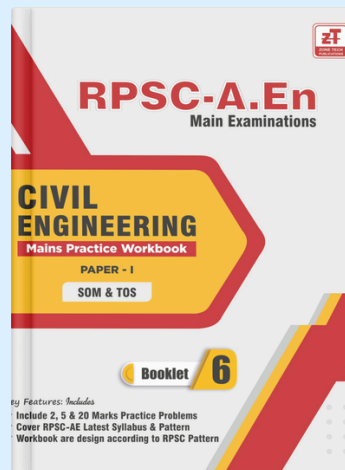
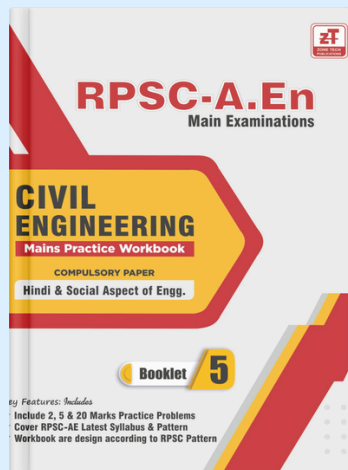
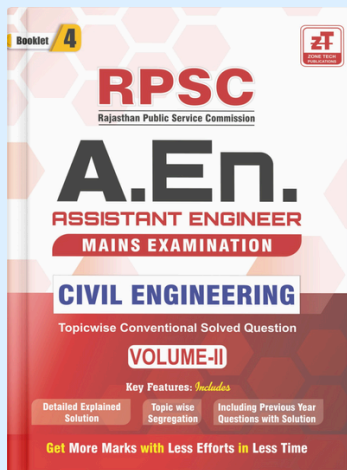
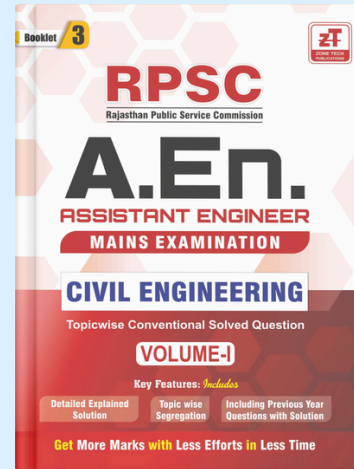
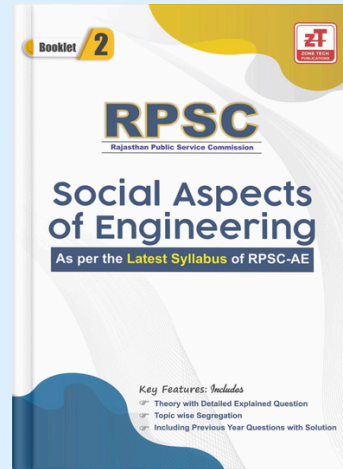
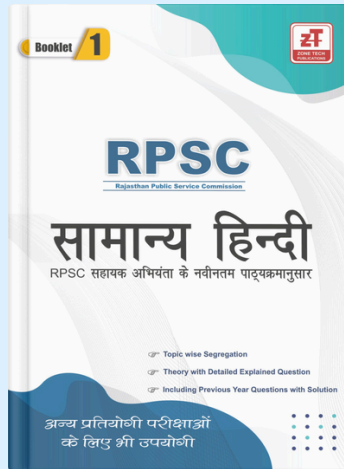
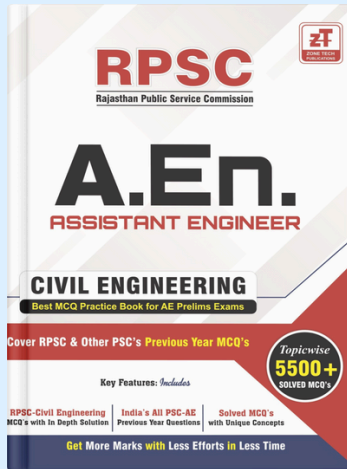
Various Uses of IRS data:

The data from Indian Remote sensing satellite are used for various applications of resources survey and management. Following is the list of those applications:

- (i) Biodiversity characterizations at landscape level.
- (ii) Land use and land cover mapping
- (iii) Urban survey
- (iv) Forest survey
- (v) Wetland mapping
- (vi) Environmental impact analysis
- (vii) Mineral prospecting
- (viii) Coastal studies
- (ix) Flood risk zone mapping and flood damage management(x) Irrigation command area status monitoring
- (xi) Snowmelt runoff estimates for planning water use in downstream projects.

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