LECTURES NOTE OF SURVEY-2

PREPARED BY

B.BEHERA

CONTENT

Chapter No.	Topic Name	Page No
1	TACHEOMETRY	1-10
2	CURVES	11-40
3	BASICS ON SCALE AND BASICS OF MAP	41-45
4	SURVEY OF INDIA MAP SERIES	46-51
5	BASICS OF AERIAL PHOTOGRAPHY, PHOTOGRAMMETRY, DEM AND ORTHO IMAGE GENERATION	52-66
6	MODERN SURVEYING METHODS	67-73
7	BASICS ON GPS & DGPS AND ETS	74-78
8	BASICS OF GIS AND MAP PREPARATION USING GIS	79-83

CHEPTER-1 :- TACHEOMETRY

Tacheometric Surveying

Tacheometric is a branch of surveying in which horizontal and vertical distances are determined by taking angular observation with an instrument known as a tachometer. Tacheometric surveying is adopted in rough in rough and difficult terrain where direct leveling and chaining are either not possible or very tedious. The accuracy attained is such that under favorable conditions the error will not exceed 1/100. and if the purpose of a survey does not require accuracy, the method is unexcelled. Tacheometric survey also can be used for Railways, Roadways, and reservoirs etc. Though not very accurate.

Tacheometric surveying is very rapid, and a reasonable contour map can be prepared for investigation works within a short time on the basis of such survey.

Uses of Tachometry

Tachometry is used for

preparation of topographic map where both horizontal and vertical distances are required to be measured;

survey work in difficult terrain where direct methods of measurements are inconvenient;

reconnaissance survey for highways and railways etc; Establishment of

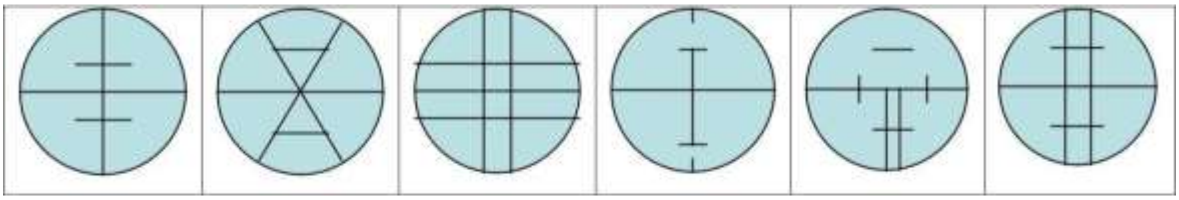
secondary control points.

Instruments used in tachometric surveying

An ordinary transits theodolite fitted with a stadia diaphragm is generally used for tacheometric surveying. The stadia diaphragm essentially consists of one stadia hair above and the other an equal distance below the horizontal cross hair, the stadia hair being mounted in the same ring and in the same vertical plane as the horizontal and vertical cross-hair.

1. The telescope used in stadia surveying are three kinds
2. The Simple external focusing telescope.
3. The external focusing anal lactic telescope
4. The internal focusing telescope.

The first type is known as stadia theodolite, while the second type is known as tacheometer. The tacheometer has the advantage over the first and third type due to fact that the additive constant of the instrument is zero.



The instruments employed in tachometry are the engineer's transit and the leveling rod or stadia rod, the theodolite and the subtense bar, the self- reducing theodolite and the leveling rod, the distance wedge and the horizontal distance rod, and the reduction tacheometer and the horizontal distance rod.

Features of tacheometer or Characteristic of tacheometer

The multiple constant (f/i) should have a normal value of 100 and the error contained in this value should not exceed 1 in 1000.

The axial horizontal lines should be exactly midway between the other two lines. The telescope should be fitted with an anallatic lens to make the additive constant (f + d) exactly to zero.

The telescope should be truly analectic.

The telescope should be powerful having a magnification of 20 to 30 diameters. The Aperture of the object should be 35 to 45 mm in diameter.

Levelling and Stadia Staff Rod

For short distances, ordinary leveling staves are used. The leveling staff normally 4m long, and it can be folded with here parts. The graduations are so marked that a minimum reading of 0.005 or 0.001m can be taken.

Different systems of Tacheometric Measurement

The various systems of tacheometric survey may be classified as follows, The

Stadia Method

i. Fixed Hair Method and

ii. Movable Hair Method

The Tangential System Measurements by means of special instruments.

The principle is common to all system is to calculate the horizontal distance between two points A and B their deference in elevation, by observing 1) the angle at the instrument at A subtended by known short distance a long a staff kept at B and 2) the vertical angle to B from A

Stadia systems

In this systems staff intercepts, at a pair of stadia hairs present at diaphragm, are considered.

The stadia system consists of two methods:

a) Fixed- hair method and

b) Movable-hair method

Fixed-hair method

In this method, stadia hairs are kept at fixed interval and the staff interval or intercept (corresponding to the stadia hairs) on the leveling staff varies. Staff intercept depends upon the distance between the instrument station and the staff.

Movable- hair method

In this method, the staff interval is kept constant by changing the distance between the stadia hairs. Targets on the staff are fixed at a known interval and the stadia hairs are adjusted to bisect the upper target at the upper hair and the lower target at the lower hair. Instruments used in this method are required to have provision for the measurement of the variable interval between the stadia hairs. As it is inconvenient to measure the stadia interval accurately, the movable hair method is rarely used.

Non-stadia systems

This method of surveying is primarily based on principles of trigonometry and thus telescopes without stadia diaphragm are used. This system comprises of two methods:

(i) Tangential method and

(ii) Subtense bar method. Tangential method

In this method, readings at two different points on a staff are taken against the horizontal cross hair and corresponding vertical angles are noted.

Subtense bar method.

In this method, a bar of fixed length, called a subtense bar is placed in horizontal position. The angle subtended by two target points, corresponding to a fixed distance on the subtense bar, at the instrument station is measured. The horizontal distance between the subtense bar and the instrument is computed from the known distance between the targets and the measured horizontal angle.

Principles of Stadia Method

(Figure 23.1) A tacheometer is temporarily adjusted on the station P with horizontal line of sight. Let a and b be the lower and the upper stadia hairs of the instrument and their actual vertical separation be designated as i. Let f be the focal length of the objective lens of the tacheometer and c be horizontal distance between the optical centre of the objective lens and the vertical axis of the instrument. Let the objective lens is focused to a staff held vertically at Q, say at horizontal distance D from the instrument station.

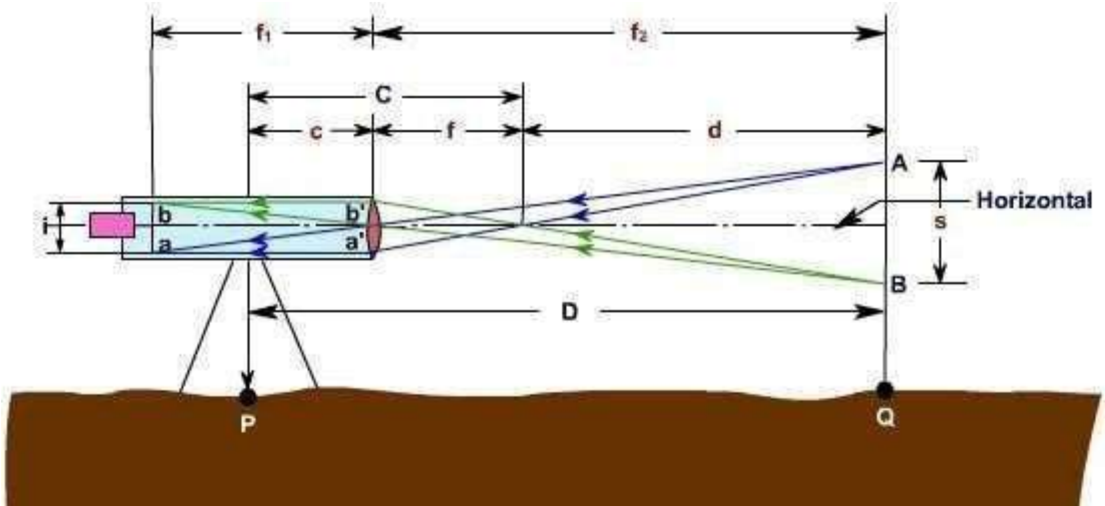


Figure 23.1 Principle of stadia measurement

By the laws of optics, the images of readings at A and B of the staff will appear along the stadia hairs at a and b respectively. Let the staff interval i.e., the difference between the readings at A and B be designated by s. Similar triangle between the object and image will form with vertex at the focus of the objective lens (F). Let the ho rizontal distance of the staff from F be d. Then, from the similar Ds ABF and a' b' F,

$$\frac{AB}{d} = \frac{a'b'}{f}$$

$$\text{Or, } d = \frac{AB}{a'b'} \times f = \frac{s}{i} \times f$$

$$\therefore d = \frac{f}{i} \times s$$

as a' b' = ab = i. The ratio (f / i) is a constant orf a particular instrument and is known as stadia interval factor, also instrument constant. It is denoted by K and thus

d = K.s------------------------------------ Equation (23.1)

The horizontal distance (D) between the center of the instrument and the station point (Q) at which the staff is held is d + f + c. If C is substituted for (f + c), then the horizontal distance D from the center of the instrument to the staff is given by the equation

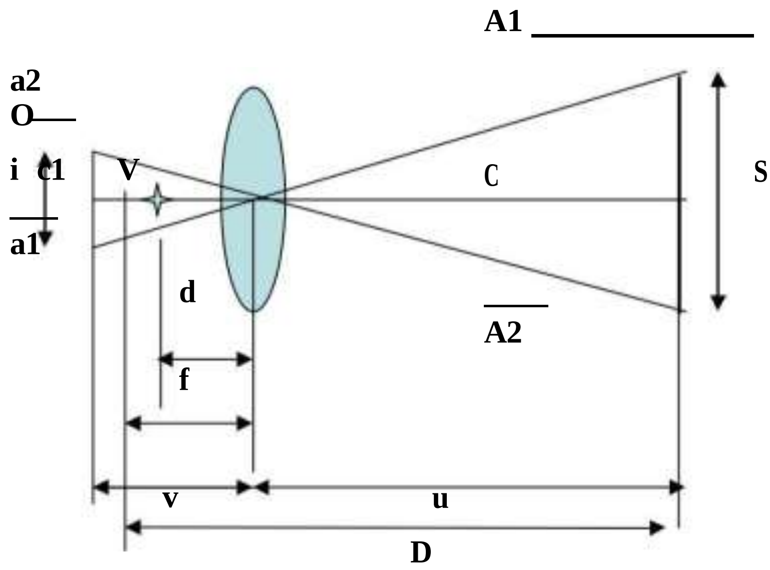
D = Ks + C------------------------------------- Equation (23.2)

The distance C is called the stadia constant. Equation (23.2) is known as the stadia equation for a line of sight perpendicular to the staff intercept.

Theory of Stadia Tacheometry

The following is the notation used in stadia tacheometry

O	=	Optical centre of object glass.
A 1, A 2, C	=	Readings on staff cut by three hairs
a 1, a 2, C	=	Bottom Top, and Central Hair of diaphragm
a 1 a 2, i =	=	length of image
A 1, A 2, = S	=	Staff Intercept
V	=	Vertical axis of instrument



f=Focal length of a object glass

d=distance between optical centre and vertical axis of instrument

u=distance between optical centre and staff v=distance between optical centre and image.

For similar triangles a 1, O a 2 and A 1, OA 2,

$$\frac{i}{s} = \frac{v}{u} \quad \text{or} \quad v = \frac{iu}{s}$$

From the properties of length,

$$\frac{1}{V} + \frac{1}{u} = \frac{1}{f}$$

Putting the value of v in Eq. (2)

$$\frac{1}{iu/s} + \frac{1}{u} = \frac{1}{f}$$

Or

$$\frac{S}{iu} + \frac{1}{u} = \frac{1}{f}$$

Or

$$\frac{1}{U}\left\{\frac{s}{I} + 1\right\} = \frac{1}{f}$$

Or

$$u = \{\frac{S}{f\,i} + 1\}$$

But, $D = u + d$ -------------------------- (3)

$$D = \{\frac{S}{Di} + 1\}\ f + x\ f + f + d = \{\frac{f}{d\,i}\}\ x\ s + (f + d)\ i$$

The quanta ties (f/i) and (f + d) are known as techeometric constants. (f/i) is called the multiplying constant, as already stated, and (f + d) the additive constant. by adopting an anal latic lens in the telescope of a tacheometric, the multiplying constant is made 100, and the additive constant zero. However, in some tacheometers the additive constants are not exactly zero, but vary from 30 cm to 60 cm

Inclined Stadia Measurements

It is usual that the line of sight of the tacheometer is inclined to the horizontal.

Thus, it is frequently required to reduce the inclined observations into horizontal distance and difference in elevation.

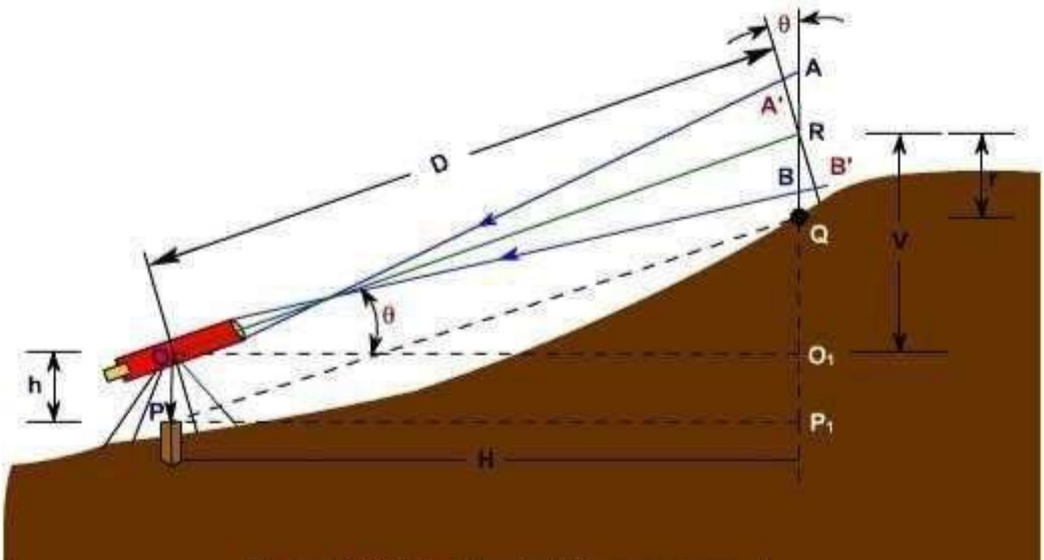


Figure 23.2 Principle of stadia measurement

Let us consider a tacheometer (having constants K and C) is temporarily adjusted on a station, say P (Figure 23.2). The instrument is sighted to a staff held vertically, say at Q. Thus, it is required to find the horizontal distance PP1 (= H) and the difference in elevation P1Q. Let A, R and B be the staff points whose images are formed respectively at the upper, middle and lower cross hairs of the tacheometer. The line of sight, corresponding to the middle cross hair, is inclined at an angle of elevation q and thus, the staff with a line perpendicular to the line of sight. Therefore A'B' = AB cos q = s cos q where s is the staff intercept AB. The distance D (= OR) is C + K. scos q (from Equation 23.2). But the distance OO1 is the horizontal distance H, which equals OR cos q. Therefore the horizontal distance H is given by the equation.

$$H = (Ks \cos q + C) \cos q$$

Or $H = Ks \cos^2 q + C \cos q$ -------------------------------- Equation (23.3)

in which K is the stadia interval factor (f / i), s is the stadia interval, C is the stadia constant (f + c), and q is the vertical angle of the line of sight read on the vertical circle of the transit.

The distance RO1, which equals OR sin q, is the vertical distance between the telescope axis and the middle cross- hair reading. Thus V is given by the equation

$$V = (K s \cos q + c) \sin q$$

$V = Ks \sin q \cos q + C \sin q$ ---------------- Equation (23.4)

-- -

[illegible] Equation (23.5)

Thus, the difference in elevation between P and Q is (h + V - r), where h is the height of the instrument at P and r is the staff reading corresponding to the middle hair.

Uses of Stadia

The stadia method of surveying is particularly useful for following cases:

1. In differential leveling, the back sight and foresight distances are balanced conveniently if the level is equipped with stadia hairs.
2. In profile leveling and cross sectioning, stadia is a convenient means of finding distances from level to points on which rod readings are taken.
3. In rough trigonometric, or indirect, leveling with the transit, the stadia method is more rapid than any other method.
4. For traverse surveying of low relative accuracy, where only horizontal angles and distances are required, the stadia method is a useful rapid method.
5. On surveys of low relative accuracy - particularly topographic surveys-where both the relative location of points in a horizontal plane and the elevation of these points are desired, stadia is useful. The horizontal angles, vertical angles, and the stadia interval are observed, as each point is sighted; these three observations define the location of the point sighted.

Errors in Stadia Measurement

Most of the errors associated with stadia measurement are those occur during observations for horizontal angles (Lesson 22) and differences in elevation (Lesson 16). Specific sources of errors in horizontal and vertical distances computed from observed stadia intervals are as follows:

1. Error in Stadia Interval factor

This produces a systematic error in distances proportional to the amount of error in the stadia interval factor.

2. Error in staff graduations

If the spaces on the rod are uniformly too long or too short, a systematic error proportional to the stadia interval is produced in each distance.

3.Incorrect stadia Interval

The stadia interval varies randomly owing to the inability of the instrument operator to observe the stadia interval exactly. In a series of connected observations (as a traverse) the error may be expected to vary as the square root of the number of sights. This is the principal error affecting the precisious.

CHAPTER-2 :-Curves

Definition of Curves:

Curves are regular bends provided in the lines of communication like roads, railways etc. and also in canals to bring about the gradual change of direction. They are also used in the vertical plane at all changes of grade to avoid the abrupt change of grade at the apex.

Curves provided in the horizontal plane to have the gradual change in direction are known as Horizontal curves, whereas those provided in the vertical plane to obtain the gradual change in grade are known as vertical curves. Curves are laid out on the ground along the centre line of the work. They may be circular or parabolic.

Classification of Curves:

(i) Simple,

(ii) Compound

(iii) Reverse and

(iv) Deviation

(i) Simple Curve:

A simple curve consists of a single arc of a circle connecting two straights. It has radius of the same magnitude throughout. In fig. 11.1 T1 D T2 is the simple curve with T1O as its radius.

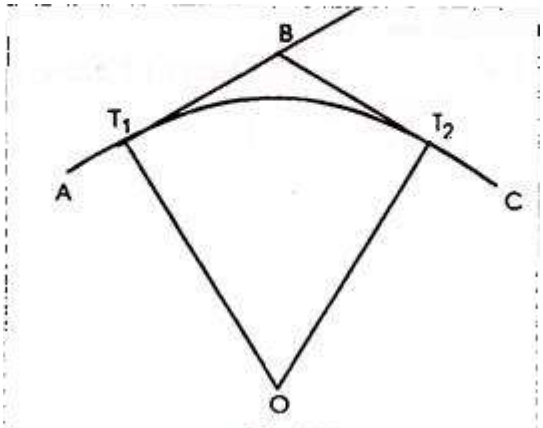


Fig 11.1

(ii) Compound Curve:

A compound curve consists of two or more simple curves having different radii bending in the same direction and lying on the same side of the common tangent. Their centres lie on the same side of the curve. In fig. 11.2, T1 P T2 is the compound curve with T1O1 and PO2 as its radii.

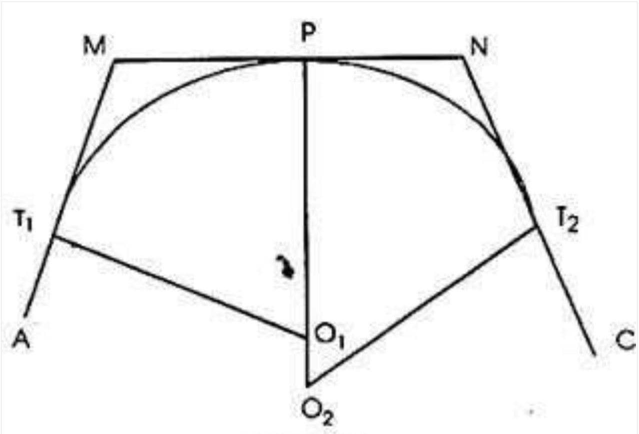


Fig 11.2

(iii) Reverse (or Serpentine) Curve:

A reverse or serpentine curve is made up of two arcs having equal or different radii bending in opposite directions with a common tangent at their junction. Their centres lie of opposite sides of the curve. In fig. 11.3 T1 P T2 is the reverse curve with T1O1 and PO2 as its radii.

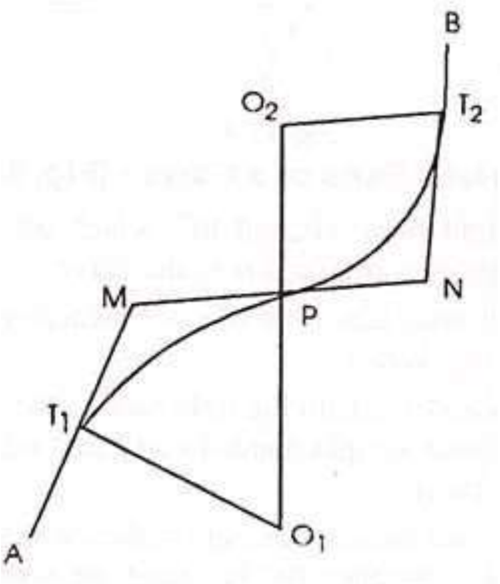


Fig 11 3

Reverse curves are used when the straights arc parallel or intersect at a very small angle. They are commonly used in railway sidings and sometimes on railway tracks and roads meant for low speeds. They should be avoided as far as possible on main railway lines and highways where speeds are necessarily high.

(iv) Deviation Curve:

A deviation curve is simply a combination of two reverse curves. It is used when it becomes necessary to deviate from a given straight path in order to avoid intervening obstructions such as a bend of river, a building, etc. In fig. 11.4. T_1 EDFT_2 is the deviation curve with T_1O, EO_2 and FO_2 as its radii.

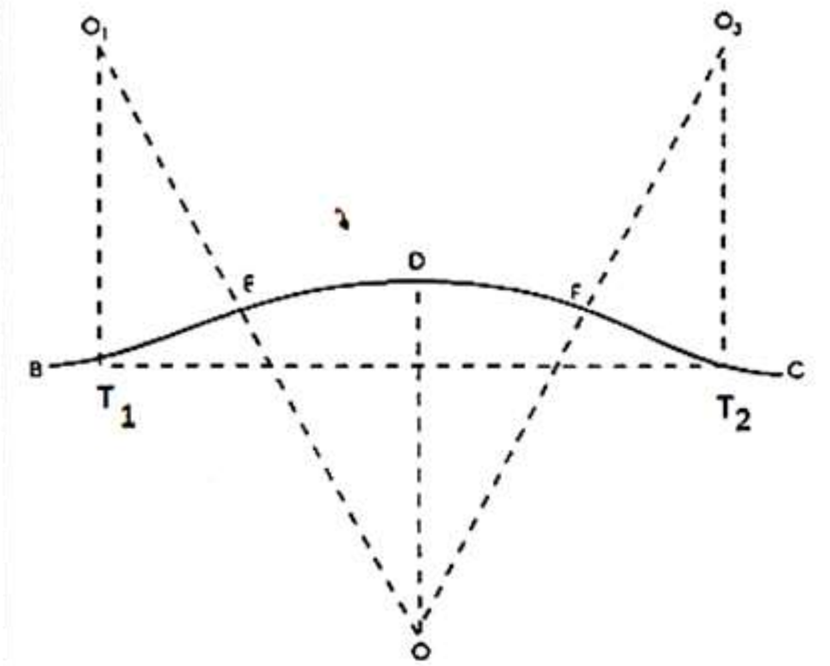


Fig 11.4

Names of Various Parts of a Curve: (Fig. 11.5):

(i) The two straight lines AB and BC, which are connected by the curve are called the tangents or straights to the curve.

(ii) The points of intersection of the two straights (B) is called the intersection point or the vertex.

(iii) When the curve deflects to the right side of the progress of survey as in fig. 11.5, it is termed as right-handed curve and when to the left, it is termed as left-handed curve.

(iv) The lines AB and BC are tangents to the curves. AB is called the first tangent or the rear tangent BC is called the second tangent or the forward tangent.

(v) The points (T_1 and T_2) at which the curve touches the tangents are called the tangent points. The beginning of the curve (T_1) is called the tangent curve point and the end of the curve (T2) is called the curve tangent point.

(vi) The angle between the tangent lines AB and BC (ABC) is called the angle of intersection (I)

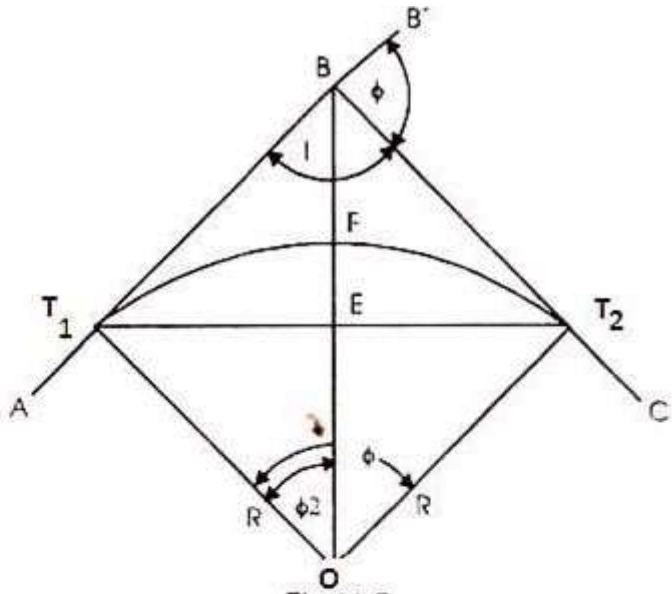


Fig 11.5

(vii) The angle by which the forward tangent deflects from the rear tangent is called the deflection angle (ϕ) of the curve.

(viii) The distance the two tangent point of intersection to the tangent point is called the tangent length (BT_1 and BT_2).

(ix) The line joining the two tangent points (T_1 and T_2) is known as the long-chord

(x) The arc T_1FT_2 is called the length of the curve.

(xi) The mid-point (F) of the arc (T_1FT_2) in called summit or apex of the curve.

(xii) The distance from the point of intersection to the apex of the curve BF is called the apex distance.

(xiii) The distance between the apex of the curve and the midpoint of the long chord (EF) is called the versed sine of the curve.

(xiv) The angle subtended at the centre of the curve by the arc T_1FT_2 is known as the Central angle and is equal to the deflection angle (ϕ).

(*i*) Angle of intersection + Deflection angle = 180°

or $I + \phi = 180°$ ... ...(Eqn. 11.1)

(*ii*) $\angle T_1OT_2 = 180° - I = \phi$ ... ...(Eqn. 11.2.)

(*i.e.* the central angle = the deflection angle).

(*iii*) Tangent length $= BT_1 = BT_2 = OT_1 \tan \frac{\phi}{2}$

$$= R \tan \frac{\phi}{2} \quad \text{... ... ...(Eqn. 11.3)}$$

(*iv*) Length of Long Chord $= 2T_1E = 2 \times OT_1 \sin\left(\frac{\phi}{2}\right)$

$$= 2R \sin \frac{\phi}{2} \quad \text{... ... (Eqn. 11.4)}$$

(*v*) Length of the curve = Length of the arc T_1FT_2

$$= R\,\phi \text{ (in radians)}$$

$$= \frac{\pi R \phi}{180°} \quad \text{... ... (Eqn. 11.5)}$$

(*vi*) Apex distance $= BF = BO - OF$

$$= R \sec \frac{\phi}{2} - R$$

$$= R\left(\sec\left(\frac{\phi}{2}\right) - 1\right) \quad \text{... ...(Eqn. 11.6)}$$

(*vii*) Versed sine of the curve $= EF = OF - OE$

$$= R - R\cos\frac{\phi}{2}$$

$$= R\left(1 - \cos\frac{\phi}{2}\right) = R \text{ versine } \frac{\phi}{2} \quad \text{... ...(Eqn. 11.7)}$$

Designation of Curves:

A curve may be designated either by the radius or by the angle subtended at the centre by a chord of particular length In India, a curve is designated by the angle (in degrees) subtended at the centre by a chord of 30 metres (100 ft.) length. This angle is called the degree of the curve (D).

The relation between the radius and the degree of the curve may be determined as follows: **Refer to fig 11.6:**

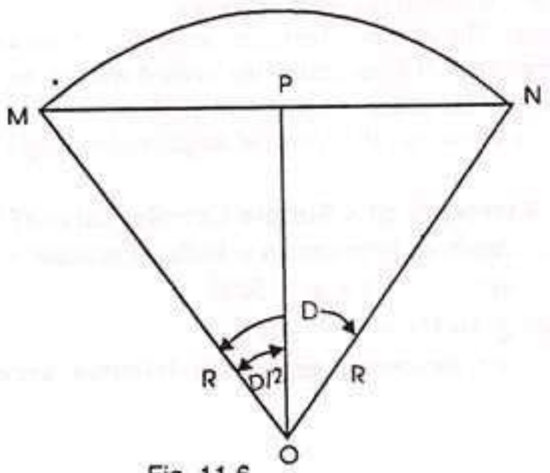


Fig. 11.6

Let R= The radius of the curves in meters

D= The degree of the curve

MN= The chord, 30m long

P= The mid-point of the chord

In Δ OMP, OM = R

$$MP = \frac{1}{2}MN = 15 \text{ m}$$

$$\angle MOP = \frac{D}{2}$$

Then, $\sin \frac{D}{2} = \frac{MP}{OM}, \frac{15}{R}$

or $R = \dfrac{15}{\sin \frac{D}{2}}$ **(Exact)** ... ...**(Eqn. 11.8)**

But when D is small, $\sin \frac{D}{2}$ may be assumed approximately equal to

$= \frac{D}{2}$ in radians.

$$R = \frac{15}{\frac{D}{2} \times \frac{\pi}{180^\circ}} = \frac{15 \times 360}{\pi D}$$

$$= \frac{171.87}{D}$$

or say, $R = \dfrac{1719}{D}$ **(approximate)** ... ... **(Eqn. 11.9)**

The approximate relation holds good up to 5° curves. For higher degree curves, the exact relation should be used.

Methods of Curve Ranging:

A curve may be set out:

1. By linear methods, where chain and tape are used.

2. By angular or instrumental methods, where a theodolite with or without a chain is used.

Before starting setting out a curve by any method, the exact positions of the tangent points between which the curve lies, must be determined.

For this, proceed as follows: (Fig. 11.5)

(i) Having fixed the directions of the straights, produce them to meet at point (B).

(ii) Set up a theodolite at the intersection point (B) and measure the angle of intersection (I). Then find the deflection angle (ϕ) by subtracting (I) from 180°. i.e., $\phi = 180° - I$

(iii) Calculate the tangent length from the Eqn. 11.3:

$$\left(\text{tan lenght} = \text{R} \tan\frac{\phi}{2}\right)$$

(iv) Measure the tangent length (BT_1) backward along the rear tangent BA from the intersection point B, thus locating the position of T_1.

(v) Similarly, locate the position of T_2 by measuring the same distance forward along the forward tangent BC from B,

Having located the positions of the tangent points T_1 and T_2; their changes may be determined. The change of T_1 is obtained by subtracting the tangent length from the known change of the intersection point B. And the change of T_2 is found by adding the length of the curve to the change to T_1.

Then the pegs are fixed at equal intervals on the curve. The interval between the pegs is usually 30 m or one chain length. This distance should actually be measured along the arc, but in practice it is measured along the chord, as the difference between the chord and the corresponding arc is small and hence negligible. In order that this difference is always small and negligible, the length of the chord should not be more than 1/20th of the radius of the curve. The curve is then obtained by joining all these pegs.

The distances along the centre line of the curve are continuously measured from the point of beginning of the line up to the end, i.e., the pegs along the centre line of the work should be at equal interval from the beginning of the line to the end. There should be no break in the regularity of their spacing in passing from a tangent to a curve or from a curve to a tangent.

For this reason, the first peg on the curve is fixed at such a distance from the first tangent point (T_1) that its change becomes the whole number of chains i.e. the whole number of peg interval. The length of the first chord is thus less than the peg interval and is called as a sub- chord. Similarly, there will be a sub chord at the end of the curve. Thus, a curve usually consists of two-chords and a number of full chords. This is made clear from the following example.

Linear Methods of Setting out Curves

The following are the methods of setting out simple circular curves by linear methods and by the use of chain and tape: 1. By ordinates from the Long chord 2. By Successive Bisection of Arcs. 3. By Offsets from the Tangents. 4. By Offsets from Chords Produced.

Method # 1. By Ordinates from the Long Chord (Fig. 11.8):

Let T1T2=L= the length of the Long chord

ED= O0= the offset at mid-point (e) of the long chord (the versed sine)

PQ=Ox= the offset at distance x from E

Draw QQ1 parallel to T1 T2 meeting DE at Q1

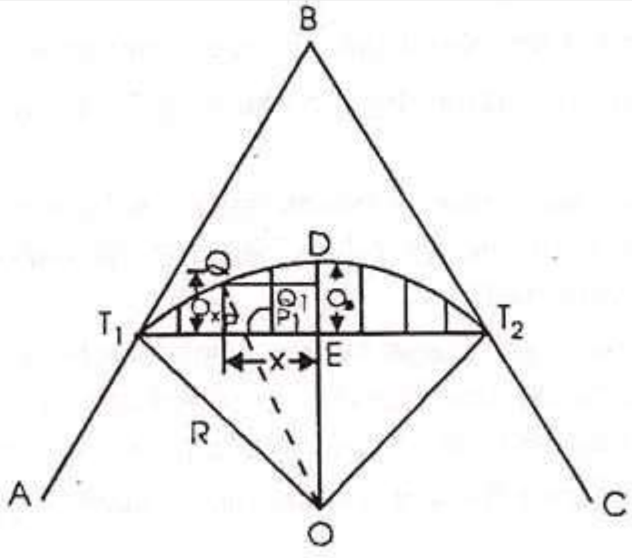


Fig. 11.8

Join OQ cutting T_1T_2 in P_1.

From the ΔOQQ_1, $OQ^2 = QQ_1{}^2 + OQ_1{}^2$

But $OQ = R$; $QQ_1 = x$

and $OQ_1 = OE + EQ_1 = (R — Q_3) + O_x$

$R^2 = x^2 \{(R — O_0) + O_0\}^2$

or $(R—O_0) + O_x = \sqrt{R^2 — x^2}$

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

(Exact) ... ...(Eqn. 11.10)

Where $O_0 = ED = OD — OE$

$$= R — \sqrt{R^2\left(\frac{L}{2}\right)^2} \quad ... \quad ...(\text{Eqn. 11.10, a})$$

When the radius of the curve is large as compared with the length of the long chord, the offset may be equated by the approximate formula which is derived as follows:

Here O_x is assumed to be equal to the radial ordinate QP_1.

$$QP \times 2R = T_1P \times PT_2$$

or $$QP_1 = \frac{T_1P \times PT_2}{2R}$$

Now $T_1P = x$, and $PT_2 = L — x$

$$Q_x = \frac{x(L-x)}{2R} \text{ (approximate)} \quad ... \quad ...(\text{Eqn. 11.11})$$

Note:

In the exact equation (11.1), the distance x of the point P is measured from the mid-point of the long chords; while in the approximate equation (11.11), it is measured from the first tangent point (T1).

Procedure of Setting Out the Curve:

(i) Divide the long chord into an even number of equal parts.

(ii) Calculate the offsets by the equation 11.10 at each of the points of division.

Note:

1. Since the curve is symmetrical on both sides of the middle- ordinate, the offsets for the right-hand half of the curve are the same as those for the left-hand half.

2. If the offsets are found by the approximate equation (11.11), the long chord should be divided into a convenient number of equal parts and the calculated offsets laid out at each of the points of division.

This method is used for setting out short curves e.g., curves for street bends.

Method # 2. By Successive Bisections of Arcs (Fig 11.10):

It is also known as Versine Method. Join T1 T2 and bisect it at E. Set out the offset ED the versed since equal to:

$R(1 - \cos\frac{\Phi}{2}$, thus fixing the point Don the curve

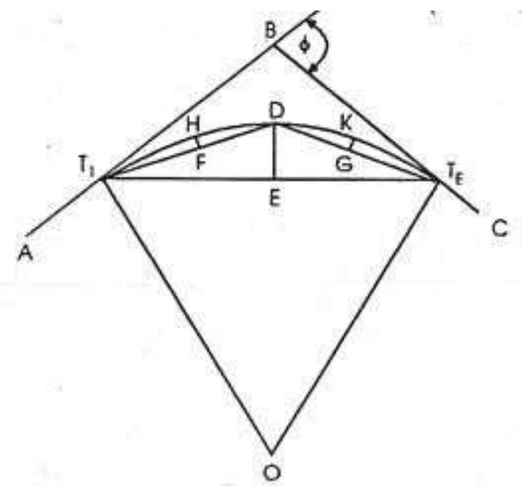


Fig. 11.10.

Join T1D and DT2 and bisect them at F and G respectively. Then set outsets FH and GK at F and G each equal to $R\left(1 - \cos\frac{\phi}{4}\right)$ thus fixing two more points H and K on the curve. Then each of the offsets to be set out at mid points of the chords T1H, HD,

$$R\left(1 - \cos\frac{\phi}{8}\right)$$

DK and KT2 is equal to .By repeating this process, set out as many point as are required.

This method is suitable where the ground outside the curve is not favorable to the tangents.

Method # 3. By Offsets from the Tangents:

The offsets may be either radial or perpendicular to the tangents.

(a) By Radial Offsets (Fig 11.11a):

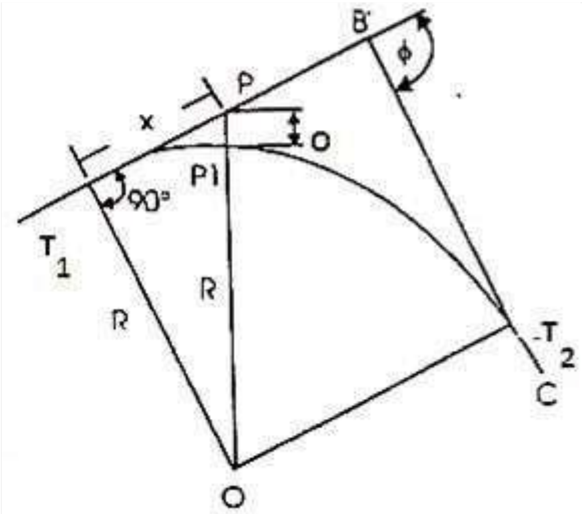


Fig. 11.11 (a)

Let $O_x = PP_1$ = the radial offset at P at a distance of x from T_1 along the tangent AB

$$PP_1 = OP - OP_1, \text{ where } OP = \sqrt{R^2 + x^2} \text{ and } OP_1 = R$$

$$O_x = \sqrt{R^2 + x^2} - R \qquad \textbf{(exact) ... ...(Eqn. 11.12)}$$

When the radius is large, the offsets may be calculated by the approximate formula, which may be derived as under:

$$PT_1^2 = PP_1 \times (2R + PP_1)$$

i.e. $$x^2 = O_x (2R + O_x) = 2RO_x + O_x^2$$

Since O_x^2 is very small as compared with 2R, it may be neglected.

$$x^2 = 2R.O_x$$

or $$O_x = \frac{x^2}{2R} \quad \text{(approximate)} \qquad \text{... ...(Eqn. 11.13)}$$

(b) By Offsets perpendicular to the Tangents (Fig 11.11,b):

Let $O_x = PP_1$ = the perpendicular offset at P at a distance of x from T_1 along the tangent AB.

Draw P_1P_2 parallel to BT_1, meeting OT_1 at P_2

Then $P_1P_2 = PT_1 = x$; $T_1P_2 = PP_1 = O_x$.

Now $T_1P_2 = OT_1 — OP_2$

where $OT_1 = R$, and $OP_2 = \sqrt{R^2 - x^2}$

$$O_x = R — \sqrt{R^2 - x^2} \quad \text{(exact)} \qquad \text{...(Eqn. 11.14)}$$

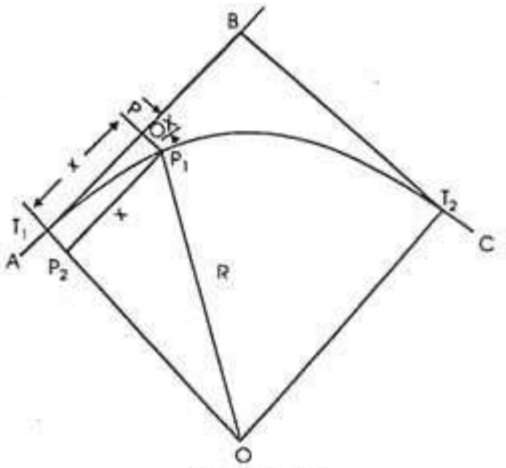


Fig. 11.11 (*b*)

The approximate formula may be obtained similarly as in (a) above ,

$$O_x = \frac{x^2}{2R} \quad \text{(approximate)} \qquad \text{...(Eqn. 11.15)}$$

Procedure of setting out the curve:

(i) Locate the tangent points T1 and T2.

(ii) Measure equal distances, say 15 or 30 m along the tangent from T1.

(iii) Set out the offsets calculated by any of the above methods at each distance, thus obtaining the required points on the curve.

(iv) Continue the process until the apex of the curve is reached.

(v) Set out the other half of the curve from the second tangent.

This method is suitable for setting out sharp curves where the ground outside the curve is favourable for chaining.

Method # 4. By Offsets from Chords Produced (Fig. 11.12):

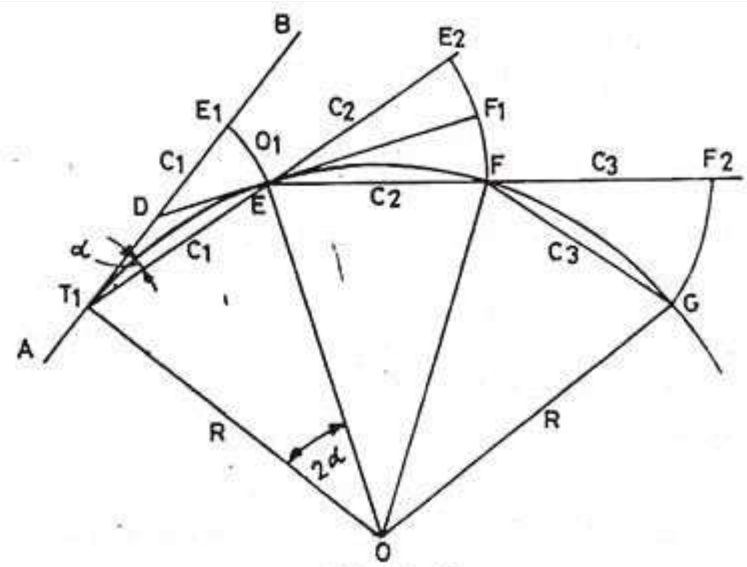


Fig. 11.12

Let AB = the first tangent; T1 = the first tangent point E, F, G etc. on the successive points on the curve T1E = T1E1 = C1 = the first chords.

EF, FG, etc. = the successive chords of length C2, C3 etc., each being equal to the full chord.

∠ BT1E = αin radians = the angle between the tangents BT1 and the first chord T1E.

E1E = O1 = the offset from the tangent BT1

E2F = O2 = the offset from the chord T1E produced.

Produce T1E to E2 such tharEE2 = C2. Draw the tangent DEF1 at E meeting the first tangent at D and E2F at F1.

∠BT1E= α in the radians= the angle between the tangents BT1and the first chord T1E.

E1E=O1= the offset from the tangent BT1

E2F=O2= the offset from the chord T1E produced.

Produce T1E to E2 such that EE2= C2. Draw the tangent DEF1at E meeting the first tangent at D and E2Fat F1.

The formula for the offsets may be derived a under:

∠ BT1E=x

∠T1OE=2x

The angle subtended by any chord at the center is twice the angle between the chord and the tangent

$$\frac{\text{arc}\, T_1E}{\text{Radius}\; OT_1} = 2\alpha$$

But arc T_1E is approximately equal to chord $T_1E = C_1$

$$\frac{C_1}{R} = 2\alpha$$

or $$\alpha = \frac{C_1}{2R}$$

Also $$\frac{\text{arc}\; E_1E}{T_1E} = \alpha$$

But arc E_1E is approximately equal to chord $E_1E = O_1$

$$O_1 = C_1 \times \alpha$$

Putting here the value of α as calculated above.

$$O_1 = C_1 \times \frac{C_1}{2R} = \frac{C_1^2}{2R} \quad \ldots \quad \ldots \quad \ldots\textbf{(Eqn. 11.16)}$$

$$O_2 = \text{offset}\; E_2F = E_2F_1 + F_1F$$

To find out F_2F_1, consider the two triangles T_1EE_1 and EF_1E_2

$\angle E_2EF_1 = \angle DET_1$ (vertically opposite angles) :

$\angle DET_1 = \angle DT_1E$, since $DT_1 = DE$, both being trangents to the circle.

$$\angle E_1EF_1 = \angle DET_1 = \angle DT_1E$$

Both the Δs being nearly isosceles, may be taken as approximately similar.

$$\frac{E_2F_1}{EE_2} \approx \frac{E_1E}{T_1E_1}, \quad \angle \alpha$$

i.e. $$\frac{E_2F_1}{C_2} = \frac{O_1}{C_1}$$

or $$E_2F_1 = \frac{C_2 \times O_1}{C_2}$$

$$= \frac{C_2}{C_1} \times \frac{C_1^2}{2R} = \frac{C_1C_2}{2R}$$

F_1F being the offset from the tangent at E, is equal to

$$\frac{EF^2}{2R} \cong \frac{C_2^2}{2R}$$

the second offset, $O_2 = \frac{C_1C_2}{2R} + \frac{C_2^2}{2R}$,

$$= \frac{C_2(C_1 + C_2)}{2R} \quad \text{... (Eqn. 11.17)}$$

Similarly the third offset, $O_3 = \frac{C_3(C_2 + C_3)}{2R}$

Since $C_2 = C_3 = C_1$.................................etc,

$$O_3 = \frac{C_2^2}{R} \quad \text{... ... ...(Eqn. 11.18)}$$

Each of the remaining offsets O4,O5 etc expect the last one (On) is equal to O3. Since the length of the last chord is usually less than the length of the chord, the last offset,

$$O_n = \frac{C_n(C_{n-1} + C_n)}{2R} \quad \text{... ... ...(Eqn. 11.19)}$$

Procedure of Setting out the Curve (Fig. 11.12):

(i) Locate the tangent points (T1 and T2) and find out their changes. From these changes, calculate lengths of first and last sub-chords and find out the offsets by using the equations 11.16 to 11.19.

(ii) Mark a point E1 along the first tangent T1B such that T1E1 equals the length of the first sub-chord.

(iii) With the zero end of the chain (or tape) at T1 and radius = T1E1, swing an arc E1E and cut off E1E = O1, thus fixing the first point E on the curve.

(iv) Pull the chain forward in the direction of T1E produced until the length EE2 becomes equal to the second chord C2.

(v) Hold the zero end of the chain at E. and radius = C2, swing an arc E2F and cut off E2F = O2, thus fixing the second point F on the curve.

(vi) Continue the process until the end of the curve is reached. The last point fixed in this way should coincide with the previously located point T2. If not, find the closing error. If it is large i.e., more than 2 m, the whole curve are moved sideways by an amount proportional to the square of their distances from the tangent point T1. The closing error is thus distributed among all the points.

This method is very commonly used for setting out road curves.

Angular Methods for Setting out Curves

The following two methods are the methods of setting out simple circular curves by angular or instrumental methods: 1. By Rankine's Tangential Angles. 2. By Two Theodolites.

Method # 1. Rankine's Method of Tangential or Deflection Angles: (Fig. 11.14):

In this method, the curve is set out by the tangential angles (also known as deflection angles) with a theodolite and a chain (or tape). The method is also called as chain and theodolite method.

The deflection angles are calculated as follows:

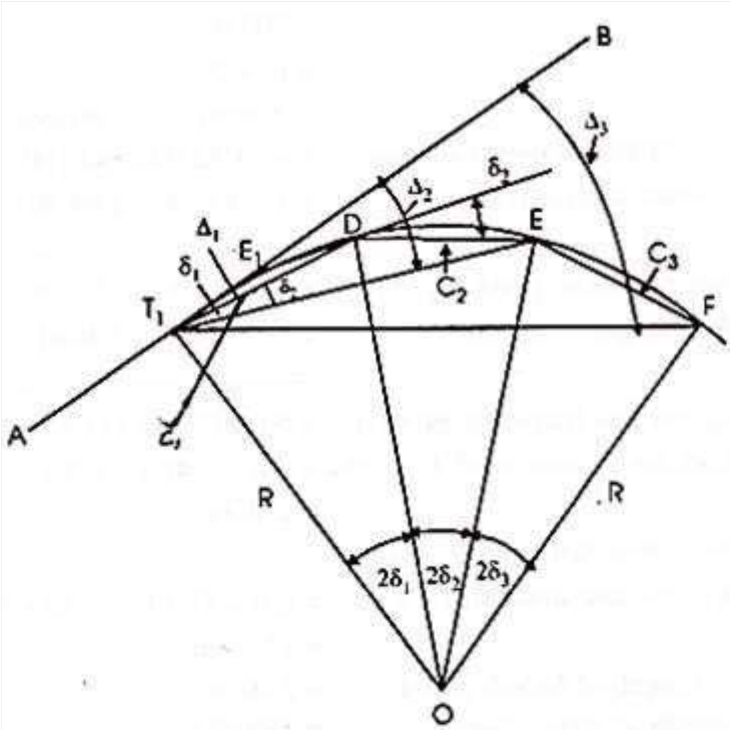


Fig. 11.14

Let T1 and T2 be the tangent points and AB the first tangent to the curve.

D, E, F, etc. =the successive points on the curve,

R = the radius of the curve.

C1, C2, C3 etc. = the lengths of the chords T1D, DE, EF etc., i.e., 1st, 2nd, 3rd chords etc.

$\delta 1$, $\delta 2$, $\delta 3$ etc. = the tangential angles which each of the chords T1 D1, DE, EF, etc., makes with the respective tangents T1, D, E. etc.

$\Delta 1$, $\Delta 2$, $\Delta 3$ etc. = the total tangential or deflection angles which the chords T1D, DE, EF, etc. make with the first tangent AB.

Now the chord T_1D is approximately equal to arc $T_1D = C_1$

$$\angle BT_1D = \delta_1 = \frac{1}{2}\ \angle T_1OD = 2\delta_1\ \ \angle T_1OD = 2\delta_1$$

$$\frac{\text{arc}\, T_1D}{\text{Radius}\, OT_1} = \angle T_1OD \text{ in radians}$$

or $$\frac{C_1}{R} = 2\delta_1 \text{ radians}$$

or $$\delta_1 = \frac{C_1}{R} \text{ radians}$$

$$= \frac{C_1}{2R} \times \frac{180}{\pi} \text{ degrees}$$

$$= \frac{C_1}{2R} \times \frac{180}{\pi} \times 60 \text{ minutes} \quad \ldots \quad \ldots\textbf{(Eqn. 11.20)}$$

Similarly, $\delta_2 = 1718.9\,\frac{C_2}{R}$; $\delta_3 = 1718.9\,\frac{C_3}{R}$; and so on

$$\delta_n = 1718.9\,\frac{C_n}{R} \text{ minutes} \quad \ldots \quad \ldots\textbf{(Eqn. 11.21)}$$

Since each of the chord lengths C2, C3, C4............. Cn-1 is equal to the length of the full chord, $\delta 2 = \delta 3 = \delta 4$…………... δn-1.

The total tangential angle (Δ_1) for the first chord (T_1D)

$$= \angle BT_1D = \delta_1$$

$$\therefore \quad \Delta_1 = \delta_1$$

The total tangential angle (Δ_2) for the second chord (DE) = $\angle BT_1E$

But $\angle BT_1E = \angle BT_1D + \angle DT_1E$

It is well known preposition of geometry that the angle between the tangent and a chord equals the angle which the chord subtends in the opposite segment.

Now $\angle$DT1E is the angle subtended by the chord DE in the opposite segment, therefore, it is equal to the tangential angle (δ2) between the tangent D and the chard DE

$$
\begin{aligned}
\therefore \quad & \Delta_2 = \delta_1 + \delta_2 = \Delta_1 + \delta_2 \\
\text{Similarly,} \quad & \Delta_3 = \delta_1 + \delta_2 + \delta_3 = \Delta_2 + \delta_3 \\
& \Delta_n + \delta_1 + \delta_2 + \delta_3 \ldots\ldots\ldots\ldots + \delta_n \\
& = \Delta_{n-1} + \delta_n \qquad \ldots \qquad \ldots\text{(Eqn. 11.22)}
\end{aligned}
$$

Check:

The total deflection angle BT1 T2 $= \Delta n = \frac{\phi}{2}$

where φ is the deflection angle of the curve.

If the degree of die curve (D) is known, the deflection angle for 30 m chord is equal 1/2D degrees, and that for the sub-chord of length C1,

$$
\begin{aligned}
& = \frac{C_1}{30} \times \frac{D}{2} \text{ degrees} \\
\delta_1 & = \frac{C_1 \times D}{60} \; ; \quad \delta_2 = \delta_3 \ldots\ldots\ldots \delta_{n-1} = \frac{D}{2} \; ; \\
\delta_n & = \frac{C_n \times D}{60} \qquad \ldots \qquad \ldots \qquad \ldots\text{(Eqn. 11.23)}
\end{aligned}
$$

Procedure of Setting out the Curve:

(i) Locate the tangent points (T1 and T2) and find out their changes. From these changes, calculate the lengths of first and last sub-chords and the total deflection angles for all points on the curve as described above.

(ii) Set up and level the theodolite at the first tangent point (T1).

(iii) Set the Vernier A of the horizontal circle to zero and direct the telescope to the ranging rod at the intersection point B and bisect it.

(iv) Loosen the Vernier plate and set the Vernier A to the first deflection angle Δ1, the telescope is thus directed along T1D. Then along this line, measure T1D equal in length to the first sub-chord, thus fixing the first point D on the curve.

(v) Loosen the upper clamp and set the Vernier A to the second deflection angle Δ2, the line of sight is now directed along T1E. Hold the zero end of the chain at D and swing the other end until the arrow held at that end is bisected by the line of sight, thus fixing the second point (E) on the curve.

(vi) Continue the process until the end of the curve is reached. The end point thus located must coincide with the previously located point (T2). If not, the distance between them is the closing error. If it is within the permissible limit, only the last few pegs may be adjusted; otherwise the curve should be set out again.

Note:

In the case of a left-handed curve, each of the values Δ1, Δ 2 Δ 3 etc, should be subtracted from 360° to obtain the required value to which the vernier is to be set i.e. the vernier should be set to (360° – Δ1), (360° – Δ2), (360° – Δ 2) etc. to obtain the 1st, 2n, 3rd etc, points on the curve.

This method gives highly accurate results and is most commonly used for railway and other important curves.

Table of Deflection Angles

Point	Chainage in metres	Length of chord in metres	Deflection Angle (δ)	Total Angle (Δ)	Theodolite vernier Reading	Remarks
T_1	39 + 6.30	° ' "	° ' "	° ' "		
1	40 + 00	23.70	1 58 30	1 58 30	1 58 40	The curve is a
2	41 + 00	30	2 30 00	4 28 30	4 28 40	right-handed
3	42 + 00	30	2 30 00	6 58 30	6 58 40	one.
4	43 + 00	30	2 30 00	9 28 30	9 28 40	The least
5	44 + 00	30	2 30 00	11 58 30	11 58 40	count of the
6	45 + 00	30	2 30 00	14 28 30	14 28 40	instrument in
7	46 + 00	30	2 30 00	16 58 40	16 58 40	20 "
T_2	46 + 12.30	12.30	1 01 30	18 00 00	18 00 00	

Method # 2. Two-Theodolite Method (Fig. 11.16):

This method is very useful in the absence of chain or tape and also when ground is not favorable for accurate chaining. This is simple and accurate method but requires essentially two instruments and two surveyors to operate upon them, so it is not as commonly used as the method of deflection angles. In this method, the property of circle 'that the angle between the tangent and the chord equals the angle which that chord subtends in the opposite segment' is used.

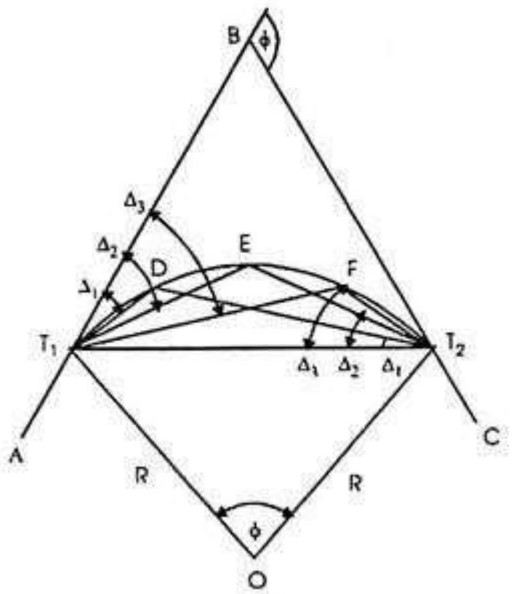


Fig. 11.16

Let D, E, F, etc. be the points on the curve. The angle (Δ1) between the tangent T1B and the chord T1D i.e. ∠BT1 D = ∠T1T2D. Similarly, ∠BT1E = Δ2 = ∠T1T2 E, and
∠BT1F = Δ3 = ∠T1T2F etc. The total deflection angles Δ1, Δ2, Δ3, etc. are calculated from the given data as in the first method (i.e. as in Rankine's method of deflection angles).

Procedure of setting out the curve:

(i) Set up two theodolites, one at T1 and the other at T2.

(ii) Set Vernier of the horizontal circle of each of the theodolites to zero.

(iii) Turn the instrument at T1 to sight the intersection point B and that at T2 to sight T1.

(iv) Set the Vernier of each of the instruments to read the first deflection angle Δ1. Now the line of sight of the instrument at T1 is along T1D and that of the instrument at T2 is along T2D. Their point of intersection is the required point on the curve Direct the assistant to move the ranging rod until it is sighted exactly by both the theodolites, thus fixing the point D on the curve.

(v) Then set the Vernier of each of the instrument to the second deflection angle Δ2, proceed as before to obtained the second point (E) on the curve.

(vi) Repeat the process until the whole curve is set out.

Note:

It may so happen that the point T1 may not be visible from the point T2. In such a case, direct the telescope of the instrument at T2 towards B with the Vernier A set to zero. Now loosen the Vernier plate and set the Vernier A to read an angle of $\left(360^\circ - \frac{\phi}{2}\right)$. The telescope is thus directed along T2 T1. For the first point D on the curve, set the Vernier A to read $\left(360^\circ - \frac{\phi}{2} + \Delta_1\right)$. Similarly, for the second point E, set the Vernier A to $\left(360^\circ - \frac{\phi}{2} + \Delta_2\right)$, and so on.

Transition Curves:

A non-circular curve of varying radius introduced between a straight and a circular curve for the purpose of giving easy changes of direction of a route is called a transition or easement curve. It is also inserted between two branches of a compound or reverse curve.

Advantages of providing a transition curve at each end of a circular curve:

(i) The transition from the tangent to the circular curve and from the circular curve to the tangent is made gradual.

(ii) It provides satisfactory means of obtaining a gradual increase of super-elevation from zero on the tangent to the required full amount on the main circular curve.

(iii) Danger of derailment, side skidding or overturning of vehicles is eliminated.

(iv) Discomfort to passengers is eliminated.

Conditions to be fulfilled by the transition curve:

(i) It should meet the tangent line as well as the circular curve tangentially.

(ii) The rate of increase of curvature along the transition curve should be the same as that of increase of super-elevation.

(iii) The length of the transition curve should be such that the full super-elevation is attained at the junction with the circular curve.

(iv) Its radius at the junction with the circular curve should be equal to that of circular curve.

There are three types of transition curves in common use:

(1) A cubic parabola,

(2) A cubical spiral, and

(3) A lemniscate, the first two are used on railways and highways both, while the third on highways only.

When the transition curves are introduced at each end of the main circular curve, the combination thus obtained is known as combined or Composite Curve.

Super-Elevation or Cant:

When a vehicle passes from a straight to a curve, it is acted upon by a centrifugal force in addition to its own weight, both acting through the centre of gravity of the vehicle. The centrifugal force acts horizontally and tends to push the vehicle off the track.

In order to counteract this effect the outer edge of the track is super elevated or raised above the inner one. This raising of the outer edge above the inner one is called super elevation or cant. The amount of super-elevation depends upon the speed of the vehicle and radius of the curve.

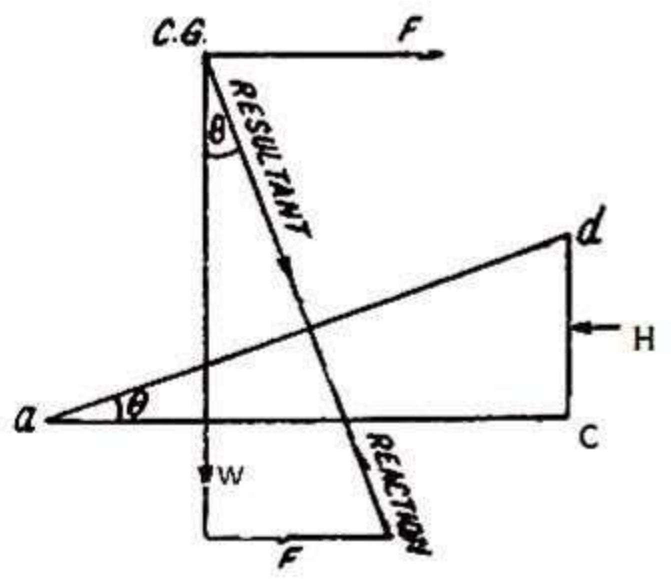


Fig. 11.24

Let:

W = the weight of vehicle acting vertically downwards.

F = the centrifugal force acting horizontally,

v = the speed of the vehicle in meters/sec.

g = the acceleration due to gravity, 9.81 meters/sec^2. R

= the radius of the curve in meters,

h = the super-elevation in meters.

b = the breadth of the road or the distance between the centres of the rails in meters.

Then for equilibrium, the resultant of the weight and the centrifugal force should be equal and opposite to the reaction perpendicular to the road or rail surface.

The centrifugal force, $F = \dfrac{Wv^2}{gR}$

$$\therefore \quad \frac{F}{W} = \frac{v^2}{gR}$$

If θ is the inclination of the road or rail surface, the inclination of the vertical is also θ

$$\tan\theta = \frac{dc}{ac} = \frac{F}{W} = \frac{v^2}{gR}$$

uper-elevation = $b \tan\theta$.

$$= \frac{bv^2}{gR} \qquad \dots \qquad \dots \qquad \textbf{(Eqn. 11.28)}$$

Characteristics of a Transition Curve (Fig 11.25):

Here two straights AB and BC make a deflection angle Δ, and a circular curve EE' of radius R, with two transition curves TE and E'T' at the two ends, has been inserted between the straights.

(i) It is clear from the figure that in order to fit in the transition curves at the ends, a circular imaginary curve ($T_1F_1T_2$) of slightly greater radius has to be shifted towards the centre as (E_1 EF E E_1. The distance through which the curve is shifted is known as shift (S) of the curve, and is equal to $\frac{L^2}{24R}$, where L is the length of each transition curve and R is the radius of the desired circular curve (EFE'). The length of shift (T_1E_1) and the transition curve (TE) mutually bisect each other.

Fig. 11.25:

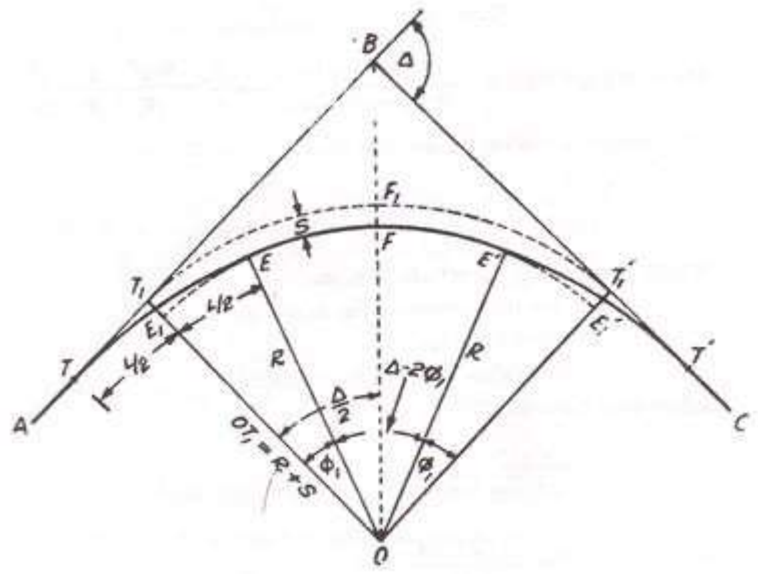


Fig 11.25

(ii) The tangent length for the combined curve

$$= OT_1 \tan\frac{\Delta}{2} + \frac{L}{2}$$

$$= (R + S)\tan\frac{\Delta}{2} + \frac{L}{2}$$

(iii) The spiral angle $\varphi_1 = \frac{\frac{L}{2}}{R} = \frac{L}{2R}\ radians$

(iv) The central angle for the circular curve:

$\angle EOE' = \Delta 2\phi_1$

(v) Length of the circular curve EFE’

$$= \frac{\pi R(\Delta - 2\phi_1)}{180^\circ}, \text{ where } \Delta \text{ and } \phi_1 \text{ are in degrees.}$$

(vi) Length of the combined curve TEE’T”

$$= TE + EE' + E'T'$$

$$= L + \frac{\pi R(\Delta - 2\phi_1)}{180^\circ} + L$$

$$= \frac{\pi R(\Delta - 2\phi_1)}{180^\circ} + 2L$$

(vii) Change of beginning (T) of the combined curve = Change of the intersection point (B)-total tangent length for the combined curve (BT).

(viii) Change of the junction point (E) of the transition curve and the circular curve = Change of T + length of the transition curve (L).

(ix) Change of the other junction point (E’) of the circular curve and the other transition curve-change of E + length of the circular curve.

(x) Change of the end point (T’) of the combined curve = change of E’ + length of the transition curve.

Check:

The change of T thus obtained should be = change of T + length of the combined curve.

Note:

The points on the combined curve should be pegged out with through change so that there will be sub-chords at each end of the transition curve and of the circular curve.

(xi) The deflection angle for any point on the transition curve distant I from the beginnings of combined curve (T),

$$\alpha = \frac{l^2}{6RL} \text{ radians} = \frac{1800 l^2}{\pi RL} \text{ minutes.}$$

$$= \frac{573 l^2}{RL} \text{ minutes.}$$

Check:

The deflection angle for the full length of the transition curve:

$$\alpha = \frac{l^2}{6RL} = \frac{L^2}{6RL} \qquad (\because l = L)$$

$$= \frac{L}{6R} \text{ radians} = \frac{1}{3}\phi_1$$

(xii) The deflection angles for the circular curve are found from:

$$\delta_n = 1718.9 \frac{C_n}{R} \text{ minutes.}$$

Check:

The deflection angle for the full length of the circular curve:

$\Delta_n = \frac{1}{2} \times$ Central angle

i.e., $\Delta_n = \frac{1}{2} \times (\Delta - 2\emptyset_1)$

(xiii) The offsets for the transition curve are found from:

Perpendicular offset, $y = \frac{x^3}{6RL}$, where x is measured along the tangent TB

Tangentail offset , $y = \frac{l^3}{6RL}$, where I is measured along the curve

Check : *(a)* The offset at half the length of the transition curve,

$$y = \frac{l^3}{6RL} = \frac{(L/2)^3}{6RL} \quad (\because l = L/2)$$

$$= \frac{L^2}{48R} = \frac{1}{2}S$$

(*b*) The offset at junction point on the transition curve,

$$y = \frac{l^3}{6RL} = \frac{L^3}{6RL} = \frac{L^2}{6R} \quad (\because l = L)$$

$$= 4S$$

(xiv) The offsets for the circular curve from chords producers are found from:

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

Method of Setting Out Combined Curve by reflection Angles (Fig. 11.25):

The first transition curve is set out from T by the deflection angles and the circular curve from the junction point E. The second transition curve is then set out from T' and the work is checked on the junction point E' which has been previously fixed from E.

(i) Assume or calculate the length of the transition curve.

(ii) Calculate the value of the shift by:

$$S = \frac{L^2}{24R}$$

(iii) Locate the tangent point T by measuring backward the total tangent length BT (article 11.14, ii) from the intersection point B along BA, and the other tangent T by measuring forward the same distance from B along BC.

(iv) Set up a theodolite at T, set the Vernier A to zero and bisect B.

(v) Release the upper clamp and set the Vernier to the first deflection angle (x_1) As obtained from the table of deflection angles, the line of sight is thus directed along the first point on the transition curve. Place zero end of the tape at T and measure along this line a distance equal to first sub chords, thus locating first point on the transition curve.

(vi) Repeat the process, until the end of the curve E is reached.

Check:

The last deflection angle should be equal to $\varphi_1/3$, and the perpendicular offset from the tangent TB for the last point E should be equal to 4S.

Note:

The distance to each of the successive points on the transition curve is measured from T.

(vii) Having laid the transition curve, shift the theodolite to E and set it up and level it accurately.

(viii) Set the Vernier to a reading($360°-2/3\ \varphi1$) for a right-hand curve (or $2/3\ \varphi1$) for a left-hand curve and lake a back sight on T. Loosen the upper clamp and turn the telescope clockwise through an angle $2/3\ \varphi1$ the telescope is thus directed towards common tangent at E and the Vernier reads 360°. Transit the telescope, now it points towards the forward direction of the common tangent at E i.e. towards the tangent for the circular curve.

(ix) Set the Vernier to the first tabulated deflection angle for the circular curve, and locate the first point on the circular curve as already explained in simple curves.

(x) Set out the complete circular curve up to E' in the usual way

Check:

The last deflection angle should be equal to $\frac{1}{2}(\Delta - 2\phi_1)$

(xi) Set out the other transition curve from T as before. The point E' to be set from T should be the same as already set out from E.

Method of Setting Out a Combined Curve by Tangential Offsets (Fig. 11.25):

(i) Assume or calculate the length of the transition curve.

(ii) find the value of the shift train, $S = \frac{L^2}{24R}$.

(iii) Locate the tangent points T and T as in article (11.15, iii),

(iv) Calculate the offset for the transition curve as in article (11.14 xiv)

(v) Locate die points on the transition curve as well as on the circular curves by setting out the respective offsets.

CHEPTER-3 :-BASICS SCALE AND BASICS OF MAP

MAP

We know that the Earth is large and is fully covered by natural (physical) and cultural (man-made) features. Hence it is impractical to communicate the picture of the whole Earth and its features verbally. To overcome this problem, a projection has been developed to represent the Earth along with the location, distribution and relationship of its features through pictorial representations known as maps. A ***map*** is the graphical two dimensional representations of all or part of the Earth's surface and its features on a plane surface drawn to a specific scale. Since, it is also impractical to represent all features of the Earth's surface in their actual sizes and forms on a map, maps are drawn at reduced scales. This attempt to show the surface of the Earth or a portion of the Earth on a flat surface is known as map. Map projections help to represent the three-dimensional features of the Earth's surface in two-dimensional forms. With the help of scales and map projections a correspondence of every point on the map to the actual ground position can be established. In a map, the real-world features have been replaced by symbols, signs, shades and colours. In other words, maps are really selective, symbolised and generalised representation of real-world phenomena on a plane surface at a reduced scale. Maps, therefore, are the paper storehouses of geospatial information.

It is very important for you to understand the difference between maps and sketches. A ***sketch*** is simple arrangement of lines, polygons, etc., with no scale and spherical coordinates. A ***map*** on the other hand always shows scale and coordinate system. Maps are mainly prepared and published by the national mapping organisation of every country of the world.

TYPES OF MAPS

There are several types of maps. Each map may show one or different kinds of information. Maps are generally made according to the function and scale. For example, some maps are used for forecasting the weather, while others are used to plot the population in an area. Depending upon the function and scale, maps are classified into various types as shown in Fig.

Types of maps

- **Based on scale**
  - **Large scale maps**
    - Cadastral maps
    - Topographical maps
  - **Small scale maps**
    - Wall maps
    - Atlas maps
- **Based on function**
  - **Physical/Natural maps**
    - Relief maps
    - Geological maps
    - Climatic maps
    - Weather maps
    - Soil maps
    - Natural vegetational maps
    - Astronomical maps
  - **Cultural maps**
    - Political maps
    - Population maps
    - Economic maps
    - Transportation maps
    - Agricultural maps
    - Industrial maps
    - Historical maps

According to Scale

According to the scale, maps may be classified into large scale and small scale maps.

Large Scale Maps

A map which shows a small area with detailed information is referred to as a ***large scale map***. Large scale maps represent an area in higher detail; thus, map may contain more information but coverage of an area is small. US Geological Survey maps are drawn at a scale of 1:62,500, the Canadian Centre for Topographic Information maps at a scale of 1:50,000, the Survey of India maps at a scale of 1:25,000 are few examples of large scale maps. Large scale maps are further sub-divided into cadastral and topographical maps.

Cadastral Maps

The term 'cadastral' is derived from the French word 'cadastre' which means 'register of property' are related to land. These maps are drawn on a very large scale such as 1:3,960, 1:1,900 or even more. They are used to show the ownership of land properties by demarcating the boundaries of agricultural fields and buildings. Cadastral maps are prepared and compiled by the government agencies and are used for revenue and taxation purposes. Village maps prepared by *Patwari* may be put under this type.

Topographical Maps

Topographical maps are also drawn on a fairly large scale but their scale is relatively smaller than cadastral maps. Topographic maps are based on precise topographical surveys and are published in the form of series of maps by the national mapping agencies. These maps are drawn at different scales, like 1:250,000; 1:50,000; 1:25,000 by the various mapping agencies. A topographical map, for example, shows only natural and cultural features and does not show the boundaries of individual agricultural lands These maps follow a standard system of symbols, signs, contours and colours to show topographic information.

Small scale Maps

Small scale maps refer to maps on which objects are relatively small. For instance, a map depicting a large area, such as an entire country is considered to be a small scale map. They are drawn to show large areas e.g. a map of the world that fits on one/two page would be a small scale map. Small scale maps are further sub-divided into wall and atlas maps.

Wall Maps

Wall maps are usually drawn on large size papers or plastic sheets. After unfolding these maps you can hang on the wall. Wall maps look like posters (Fig. 9.3). Wall maps are small and simple, and used for many purposes. They are used in classrooms, lecture halls, and also help to navigate highways, roads, and locations. Scale for these maps is smaller than those of topographical maps.

Atlas Maps

These maps are generally drawn on a very small scale. Therefore, they represent large areas and also provide highly generalised information of physical and cultural features of different regions of the world. Scale of atlas maps is smaller than wall maps

According to Function

Maps can also be classified according to their functions and purposes. For example, a political map shows an administrative arrangement of a nation or a state and a climate map shows different types of climatic zones. Based on the functions and purposes, maps are also classified into physical and cultural maps.

Physical Maps

Physical maps are especially prepared to show natural features such as heavenly bodies, relief, soils, rocks, vegetation, drainage, weather, climate, etc. These maps further sub-divided into following types:

Astronomical Maps

Astronomical maps are prepared to show heavenly bodies, like stars, galaxies, or surfaces of the planets and the Moon. Modern astronomical maps are based on a coordinate system similar to geographic latitudes and longitudes. These maps have both large and small scales.

Relief Maps

Relief maps are drawn to show the actual relief (topography) features of surface of the Earth, like mountains, valley, plains, plateaus, drainage, slopes, river systems, etc.

Geological Maps

These types of maps are prepared to show different types of rocks, minerals, and geological structures.

Climatic Maps

Climatic maps depict different types of climate zones of an area

Weather Maps

These maps are drawn to show the average conditions of weather's elements (temperature, pressure, rainfall, direction and velocity of winds, etc.) over a short period, which may range from one day to one season.

Soil Maps

Maps are also drawn to show different soil types found in a region or continent using different shades and colours.

Vegetation Maps

These maps are prepared to show distribution and types of vegetation in an area, nation or the whole Earth.

Cultural Maps

Cultural maps are drawn to represent man-made features such as buildings, canals, dams, rails and road networks, etc.

Political Maps

These maps represent boundaries between various political regions, such as tehsil, district, state and country. For example, political map of India shows 28 states and 7 union territories of India

Population Maps

These maps are drawn to show distribution and density of population, and related features such as growth rate, age and sex composition, religious distribution, occupation, etc.

Economic Maps

These maps depict distribution and production of agricultural, mining and industrial products. In addition, economic maps also show location of industries and markets, trade routes, commercial centres, etc.

Transportation Maps

These maps are drawn to show the network of roads, railway lines, air and shipping routes.

Agricultural Maps

These maps represent production and distribution of different types of crops in an area, nation, or the whole world.

Industrial Maps

These maps are drawn to show location of industrial hubs in a region, nation or continent.

Historical Maps

Historical maps are drawn to show past events.

MAP SCALE

From the last section you would have realised that maps can be drawn that are region specific e.g. map of a district, a state, a country, a continent or the whole world. It is impossible to represent the whole surface of the Earth on a single plane piece of paper in its actual size because for this we need a very long paper of the same size as the Earth. To overcome this problem, we make use of a scale to represent a part or entire surface of the Earth. A scale helps us to reduce whole or a portion of the Earth to a size which is convenient and handy to represent the portion to be shown as well as has scientific and logical values. Therefore, maps are scaled down so that they fit on available paper.

Definition

We can define a map scale as the relationship between distance on a map and distance on the ground. In other words, map scale can also be expressed as the ratio of distances between any two points on map and their corresponding distance between the same two points on the ground. Thus, map scale is a method for expressing relationship between map distance and ground distance or distance on the surface of the Earth. Understanding map scale is very important to know the size of the land features represented on a map and relative distances between them. During the process of scaling down of a map, every part of the map should be scaled down by the same amount. This will ensure that every feature on the map is in same proportion. Suppose, if a city is twice as large as a neighbouring town, an accurate scaled map will show the same relationship on paper. The drawn city will be twice as large as the drawn town.

Methods of Representation

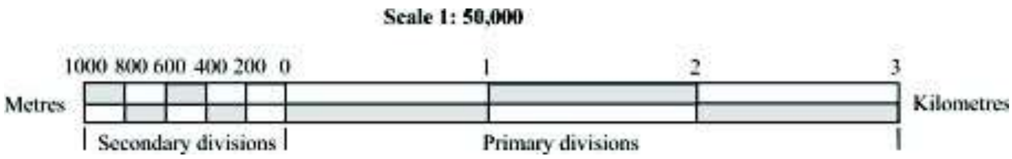
There are three ways to represent a map scale:

Statement of Scale

It is the simplest method of scale representation. In this method, scale of a map is expressed in written statement like 1 centimetre (cm) equals to 10 kilometre (km). It means 1 cm on map is representing 10 km of the corresponding distance on the ground. The main demerits of this method are a) people who are not familiar with the used system of measurements, find it difficult to understand the units and b) when a map is reduced or enlarged, the statement of scale will change.

Graphical Scale

The second way of expressing relation between a distance on a map and corresponding distance on the ground is the graphical scale. Graphical scale is also known as linear scale, bar scale, or scale bar. In graphical scale, distance is shown graphically along a line with primary and secondary divisions marked on it (Fig. 9.6). Length of a segment of line represents some distance on the Earth. In a graphical scale, zero point is located on the left end of the scale. To the right of zero point, the scale is graduated in primary divisions and from left of zero scale is graduated in secondary divisions. The advantage of this method is that when a map is reduced or enlarged scale will get proportionally reduced or enlarged.



Representative Fraction

We may also specify scale as a representative fraction (R.F.). R.F. shows a relationship between map distance and corresponding distance on the ground in units of length. For example, if we have a R.F. of 1/1000 or 1:1000. It means one unit of measurement on a map represents 1000 units on the ground.

R.F. is always written with map distance as 1 and is independent of any unit of measurement e.g., yards, meters, inches, cms, etc. But it is important to remember that both values of the fraction must have same units of measurement, like 1 inch represents 1000 inches, or 1cm represents 1000 cms. In this method, numerator shows distance on the map while denominator represents distance on the ground and its value changes according to the scale.

CHAPTER-4 :-SURVEY OF INDIA MAP SERIES

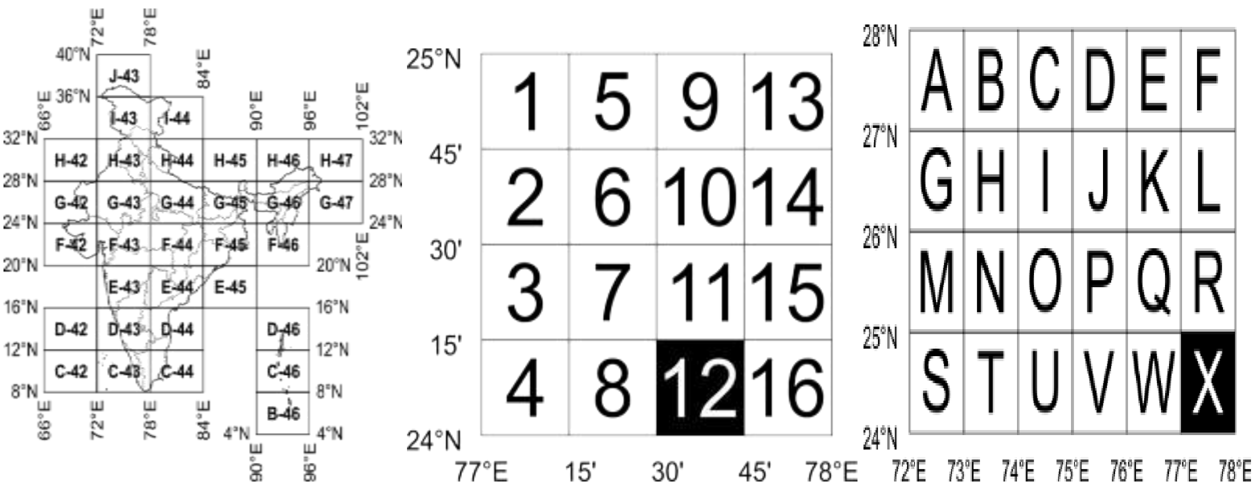
OPEN SERIES MAP

'Open Series Map' have been introduced as per the National Map Policy of 2005 by Survey of India. For the same a new map numbering system has been adopted instead of the previous India and Adjacent Countries (IAC).The map series is based on Transverse Mercator projection on WGS-1984 datum. A numbering system based on International Map of the World (IMW) is used. Map numbering is of the form 'A-12A-1'.

1. The IMW numbering system with minor modification is used upto 1°×1°/1:250,000 scale.

a) Since the IMW map number for India will always start with 'N' (India being in the northern hemisphere), the first letter is omitted.
b) The next alphabet and number of the IMW map number denotes the 6°×4° region of the IMW series. So sheet with Kalyanpur (77.65489°E 24.11981°N) would be in 'G-43' (from NG-43).
c) Each 6°×4° rectangle is further subdivided into 24 squares of 1°×1°. Each square is indicated serially by an alphabet increasing first towards east and then towards south, starting with 'A'. So sheet for Kalyanpur (77.65489°E 24.11981°N) falls within 'G-43X'

2. Each 1°×1° square is further divided into 16 squares of 15'×15' (15 minutes×15 minutes). Each square is indicated serially by a number increasing first towards south and then towards east, starting with '1' (similar to the system adopted in India and Adjacent Countries). So for the map sheet for Kalyanpur (77°39.293'E 24°7.187'N) would be 'G-43X-12'



DEFENSE SERIES MAP

These are prepared on 1:250,000; 1:50,000 and 1:25,000 scales for the use of defence forces of India for supporting national security requirements. Technically maps of this series are based on WGS-84 Datum and LCC Projection. They contain full features of map with grid, contours and other classified information without any dilution of accuracy; therefore, they are kept under restricted category. Survey of India is only authorized for preparation and printing of DSM.

MAP NOMENCLATURE

1.QUDRANGLE NAME:- A "quadrangle" is a topographic map produced by the United States Geological Survey (USGS) covering the United States. The maps are usually named after local physiographic features. The shorthand "quad" is also used, especially with the name of the map. On 1947-1992, the USGS produced the 7.5 minute series, with each map covering an area one-quarter of the older 15-minute quad series, which it replaced. A 7.5 minute quadrangle map covers an area of 49 to 70 square miles (130 to 180 km2).Both map series were produced via photogrammetric analysis of aerial photography using stereoplotters supplemented by field surveys. On a quadrangle map, the north and south limits are not straight lines, but are actually curved to match Earth's lines of latitude on the standard projection. The east and west limits are usually not parallel as they match Earth's lines of longitude.

2.LATITUDE :-

Latitude is the measurement of distance north or south of the Equator. It is measured with 180 imaginary lines that form circles around the Earth east-west, parallel to the Equator. These lines are known as parallels. A circle of latitude is an imaginary ring linking all points sharing a parallel. The Equator is the line of 0 degrees latitude. Each parallel measures one degree north or south of the Equator, with 90 degrees north of the Equator and 90 degrees south of the Equator. The latitude of the North Pole is 90 degrees N, and the latitude of the South Pole is 90 degrees S. Like the poles, some circles of latitude are named. The Tropic of Cancer, for instance, is 23 degrees 26 minutes 21 seconds N—23° 26' 21" N. Its twin, the Tropic of Capricorn, is 23° 26' 21" S. The tropics are important geographic locations that mark the northernmost and southernmost latitudes where the sun can be seen directly overhead during a solstice. One degree of latitude, called an arcdegree, covers about 111 kilometres (69 miles). Degrees of latitude are divided into 60 minutes. To be even more precise, those minutes are divided into 60 seconds. One minute of latitude covers about 1.8 kilometres (1.1 miles) and one second of latitude covers about 32 meters (105 feet). The points of latitude and longitude are called coordinates, and can be used together to locate any point on Earth.

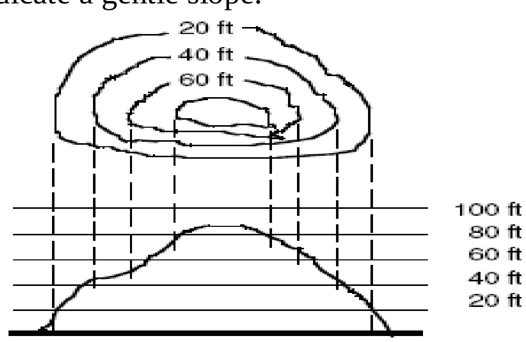
LONGITUDE:-

Longitude is a geographic coordinate that specifies the east west position of a point on the Earth's surface, or the surface of a celestial body. It is an angular measurement, usually expressed in degrees. Because of the earth's rotation, there is a close connection between longitude and time. Local time (for example from the position of the sun) varies with longitude, a difference of 15° longitude corresponding to a one-hour difference in local time. Longitude is given as an angular measurement ranging from 0° at the Prime Meridian to +180° eastward and −180° westward. Each degree of longitude is sub-divided into 60 minutes, each of which is divided into 60 seconds. A longitude is thus specified in sexagesimal notation as 23° 27′ 30″ E. For higher precision, the seconds are specified with a decimal fraction. An alternative representation uses degrees and minutes, where parts of a minute are expressed in decimal notation with a fraction, thus: 23° 27.5′ E. Degrees may also be expressed as a decimal fraction: 23.45833° E.

UTM:-The Universal Transverse Mercator (UTM) is a map projection system for assigning coordinates to locations on the surface of the Earth. Like the traditional method of latitude and longitude, it is a horizontal position representation, which means it ignores altitude and treats the earth as a perfect ellipsoid. However, it differs from global latitude/longitude in that it divides earth into 60 zones and projects each to the plane as a basis for its coordinates. Specifying a location means specifying the zone and the x, y coordinate in that plane. The projection from spheroid to a UTM zone is some parameterization of the transverse Mercator projection. The parameters vary by nation or region or mapping system. Most zones in UTM span 6 degrees of longitude, and each has a designated central meridian. The scale factor at the central meridian is specified to be 0.9996 of true scale for most UTM systems in use.

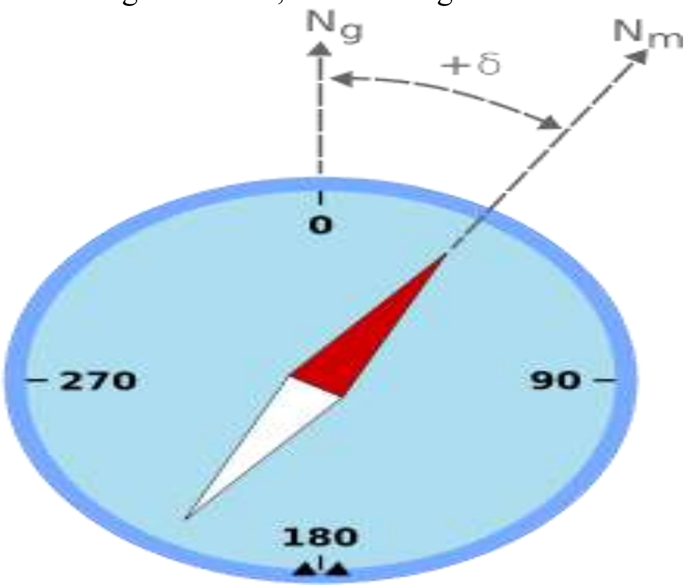
3.**CONTOUR LINE**:-

A contour line is a line drawn on a topographic map to indicate ground elevation or depression. A contour interval is the vertical distance or difference in elevation between contour lines. Index contours are bold or thicker lines that appear at every fifth contour line. If the numbers associated with specific contour lines are increasing, the elevation of the terrain is also increasing. If the numbers associated with the contour lines are decreasing, there is a decrease in elevation. As a contour approaches a streamthe contour lines turn upstream. Widely separated contour lines indicate a gentle slope.



4.MAGNETIC DECLINATION :-

Magnetic declination, or magnetic variation, is the angle on the horizontal plane between magnetic north (the direction the north end of a magnetized compass needle points, corresponding to the direction of the Earth's magnetic field lines) and true north (the direction along a meridian towards the geographic North Pole). This angle varies depending on position on the Earth's surface and changes over time. By convention, declination is positive when magnetic north is east of true north, and negative when it is to the west. Isogonic lines are lines on the Earth's surface along which the declination has the same constant value, and lines along which the declination is zero are called agonic lines.Example of magnetic declination showing a compass needle with a "positive" variation from geographic north. Ng is geographic or true north, Nm is magnetic north, and δ is magnetic declination



5.**PUBLIC LAND SURVEY SYSTEM**:-

The Public Land Survey System (PLSS) is the surveying method developed to plat, or divide, real property for sale and settling. Also known as the Rectangular Survey System, it was created by the Land Ordinance of 1785 to survey land ceded to the United States by the Treaty of Paris in 1783.

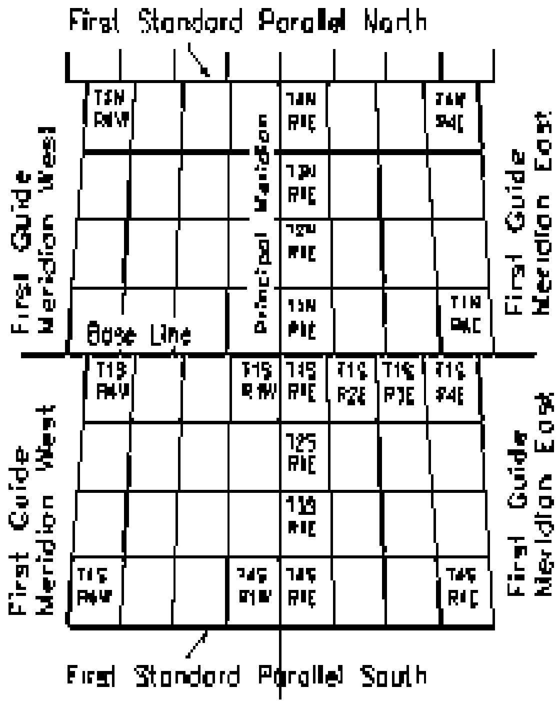
There are some difficulties with this system:

- Irregular shapes for properties make for much more complex descriptions.
- Over time, these descriptions become problematic as trees die or streams move by erosion.
- It was not useful for the large, newly surveyed tracts of land being opened in the west, which were being sold sight unseen to investors.

Common used term

- Base Line:-A parallel of latitude, referenced to and established from a designated initial point, upon which all rectangular surveys in a defined area are based. Also spelled baseline
- Cadastral:-Having to do with the boundaries of land parcels.
- Corner: -The point of intersection of any two actual or potential survey lines, defining one corner of a rectangular land parcel.
- Range :- A measure of the distance east or west from a referenced principal meridian, in units of six miles.

- Section:-An approximately one-square-mile block of land. There are 36 sections in a survey township.
- Monumentation:- Placement and/or marking of physical objects on the ground to mark survey points and lines.
- Principal meridian (PM): A true meridian running through an initial point, which together with the baseline form the highest level framework for all rectangular surveys in a given area.



6.**<u>FIELD NOTES</u>** :-

These are the notes made by the researcher in the course of qualitative fieldwork, often observations of participants, locations or events. These may constitute the whole data collected for a project.Fieldnotes refer to qualitative notes recorded by surveyors in the course of field survey, during or after their observation of a specific land they are studying. The notes are intended to be read as evidence that gives meaning and aids in the understanding of the land. Field notes allow the surveyors to access the subject and record what they observe in the field.One major disadvantage of taking fieldnotes is that they are recorded by an observer and are thus subject to memory and possibly, the conscious or unconscious bias of the observer. It is best to record fieldnotes while making observations in the field or immediately after leaving the site to avoid forgetting important details.

<u>**CHEPTER-5 :-BASICS OF AREAL PHOTOGRAPHY,PHOTOGRAMMETRY,DEM AND ORTHO IMAGE GENERATIONDIGITAL PHOTOGRAMMETRY**</u>

Introduction to Photogrammetry:

- Photogrammetry as a science is among the earliest techniques of remote sensing.
- The word photogrammetry is the combination of three distinct Greek words: ‘Photos’ - light; ‘Gramma’ -to draw; and ‘Metron’ –to measure. The root words originally signify "measuring graphically by means of light."
- The fundamental goal of photogrammetry is to rigorously establish the geometric relationship between an object and an image and derive information about the object from the image.
- For the laymen, photogrammetry is the technological ability of determining the measurement of any object by means of photography.

Digital Photogrammetry

With the advent of computing and imaging technology, photogrammetry has evolved from analogue to analytical to digital (softcopy) photogrammetry.

The main difference between digital photogrammetry and its predecessors (analogue and analytical) is that it deals with digital imagery directly rather than (analogue) photographs.

Digital photogrammetry invovles processing of imagery of all types, including passive (e.g., optical sensing) or active (e.g., radar imaging), and taken from any platform (e.g., airborne, satellite, close range, etc.).

The unique advantages of Digital Photogrammetry in terms of precision and accuracy offers opportunities for automation of DEM/DTM and integration of images acquired on a multi-platform

Techniques of photogrammetry:

1. <u>Depending on the lenses setting:</u>

   A. Far range photogrammetry (with camera distance setting to indefinite).

   B. Close range photogrammetry (with camera distance settings to finite values).

2. <u>on the basis of type of surveying:</u>

   A. Terrestrial or ground photogrammetry.

   B. Aerial photogrammetry.

A. Terrestrial or Ground Photogrammetry:

In terrestrial photogrammetry maps are prepared from terrestrial (or ground) photographs or terrestrial photogrammetry employees take photographs from different points on the earth surface for measurement purposes.

The terrestrial photographic surveying considered as the further development of plane table surveying.

B. Aerial Photogrammetry:

In aerial photogrammetry maps are produced from air photographs (photographs taken from the air). Aerial Photogrammetry encompasses two major areas of specialization:

Fig 1: Aerial Photography

- Metrical
- Interpretive

The first area is of principal interest to surveyors since it is applied to determine distances, elevations, areas, volumes, cross-sections and to compile topographic maps from measurement made on photographs.

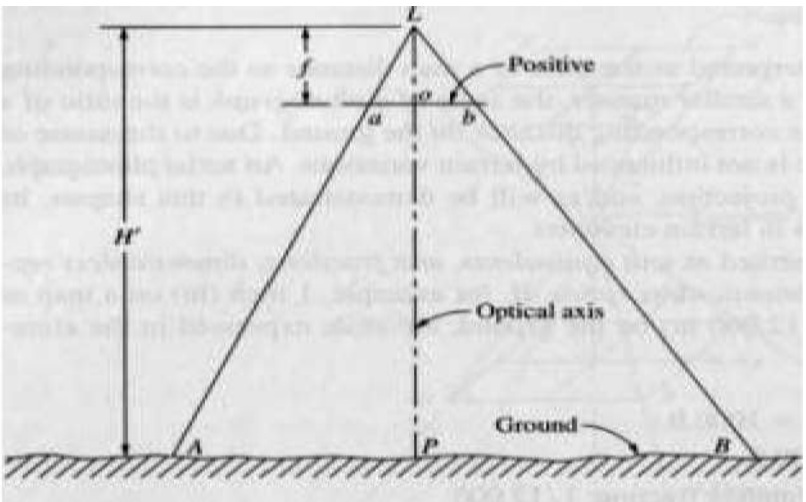
Interpretive photogrammetry involves objects from there photographic images and judging their significances. Critical factors considered in identifying objects of shape, sizes, patterns,
shadow.

1. **Determining the scale of a vertical photograph and estimating horizontal ground distances from measurements made on a vertical photograph.**

<u>**Scale**: </u>the ratio of the distance between two points on a photo to the actual distance between the same two points on the ground (i.e. 1 unit on the photo equals "x" units on the ground).

If a 1 km stretch of highway covers 4 cm on an air photo, the scale is calculated as follows:

$$\frac{\text{PHOTO DISTANCE}}{\text{GROUND DISTANCE}} = \frac{4\text{ cm}}{1\text{ km}} = \frac{4\text{ cm}}{100\,000\text{ cm}} = \frac{1}{25\,000} \quad \text{SCALE: } 1/25\,000$$



1. **To determine the equivalent areas in a ground coordinate system using area measurements made on a vertical photograph.**

The only difference is that whereas ground distances and photo distances vary linearly , ground areas and photo areas vary as the square of the scale (S).

Ground area=Photo area * 1/(S*S)

2. Determination of object heights from relief displacement measurement.

Relief displacement: The images of the tops of objects appearing in a photograph are displaced from the images of their bases this is known as relief displacement. The magnitude of relief displacement depends on:

Flying height: When the flying height increased, the relief displacement will be increased.

The distance from the object from the nadir point: hen the distance of object is more from nadir point, the relief displacement will be more.

The height of the feature: When the distance of objects from the nadir point is remain same. But the object height increased or decreased. Higher object is more displaced.

Focal Length: When the focal length of camera lens is increased, the relief displacement will be more.

Relief displacement is expressed mathematically as:

$$d = hr/H \qquad (1)$$

Where, d = Relief Displacement h = Height of the object r = Radial distance from nadir point H = Total altitude of the camera or flying height

from equation (1)

$$\text{Height of the object}(h) = dH/r$$

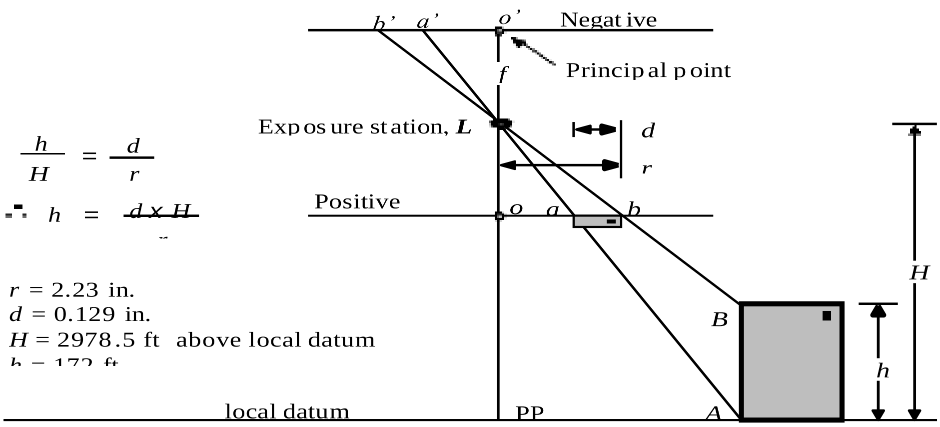


Fig 3: Measurement of height of the object from relief displacement

3. Determination of object heights and terrain elevations by measurement of image parallax.

Image parallax: The term parallax refers to the apparent change in relative position of stationary objects caused by a change in viewing position.

The absolute stereoscopic parallax is the algebraic difference, in the direction of the flight, of the distance of the two images of the object from their respective principal points. The parallax difference can be used to determine the height of the objects and the dip and slope from the stereo pairs.

To measure the height of an object above or below a reference point from stereo-pair of aerial photograph following data is required: -

i) Flying height above the reference point.

ii) Photo base – Which can be measured from the stereo-pair

iii) Parallax difference – It is measured by the use of a set of measuring marks (sometimes called locating marks.

Now by using following (parallax) formula the height of the objects can be measured.

$$h = (Z^*\Delta P) / (b + \Delta P)$$

For smaller heights e.g. trees, embankments, buildings the formula is further simplified to:

$$h = (Z^*\Delta P) / b$$

Where h = height of object , Z = flying height above the reference point, b = photo base,

ΔP = Parallax difference.

This formula gives correct result when the photographs are truly vertical.

Orthophotos and Digital Orthophotography: An orthophoto is an aerial photograph that has been geometrically corrected or 'ortho-rectified' such that the scale of the photograph is uniform.

- Orthophotos are photographs that have been corrected for distortions due to tilting of the camera during the photographic survey, distortions from the camera lens, and relief distortions.
- Orthophotos display all the valuable information of a photograph, but unlike a photograph, true distances, angles and areas can be measured directly (Rossi,2004).
- An orthophoto is an accurate representation of the Earth' s surface. Orthophotos have the benefits of high detail, timely coverage combined with the benefits of a map including uniform scale and true geometry.

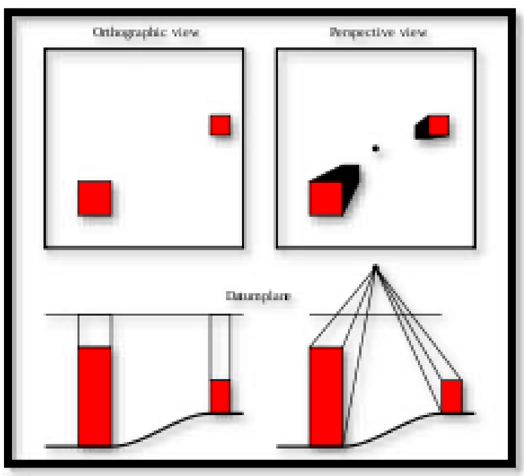


Fig 6: Orthographic views project at a right angle to the data plane. Perspective views project from the surface onto the datum plane from a fixed location.

Digital Orthophotograph:

A digital orthophoto is a digital image of an aerial photograph with displacements caused by the camera angle and the terrain removed. It, thus, combines the image characteristics of a photograph with the geometric qualities of a map.

Requirements:

Photogrammetry is the technique of determining the geometric properties of objects from photographic images where the 3D coordinates of points, features or objects can be determined

by measurements made in stereopair using the principles of triangulation. The following are essential elements required to produce a digital orthophoto:

1) Photo identifiable ground control points.
2) Camera calibration and orientation parameters.
3) A digital elevation model (DEM).
4) A digital image produced by scanning an aerial photograph with a precision high- resolution scanner.
5) Softcopy Photogrammetric Workstations: Processing the imagery to derive image and vector products using Digital Photogrammetric Workstation (DPW).

A DPW combines computer hardware and software i.e., a graphics workstation with, in most but not all cases, a stereo viewing device and a 3-D mouse. Software configuration includes Erdas Imagine with Leica Photogrammetric Suite (LPS)

For modern DPWs, there's no specific requirement for the host computer. Often a DPW can be built on a high-end desktop PC with at least 256RAM, one or two 19- or 21-inch monitors and a high-performance graphics card.

Ortho Image generation using Analytical Stereo plotter i.e. Leica Photogrammetric suite:

Leica Photogrammetric suite (LPS): IMAGINE Photogrammetry (formerly LPS and Leica Photogrammetry Suite) is a software application for performing photogrammetric operations on imagery and extracting information from imagery. IMAGINE Photogrammetry is significant because it is a leading commercial photogrammetry application that is used by numerous national mapping agencies, regional mapping authorities, various DOTs, as well as commercial mapping firms. Aside from commercial and government applications, IMAGINE Photogrammetry is widely used in academic research. Research areas include landslide monitoring, cultural heritage studies, and more.

Ortho Image generation involves the following steps to be followed:

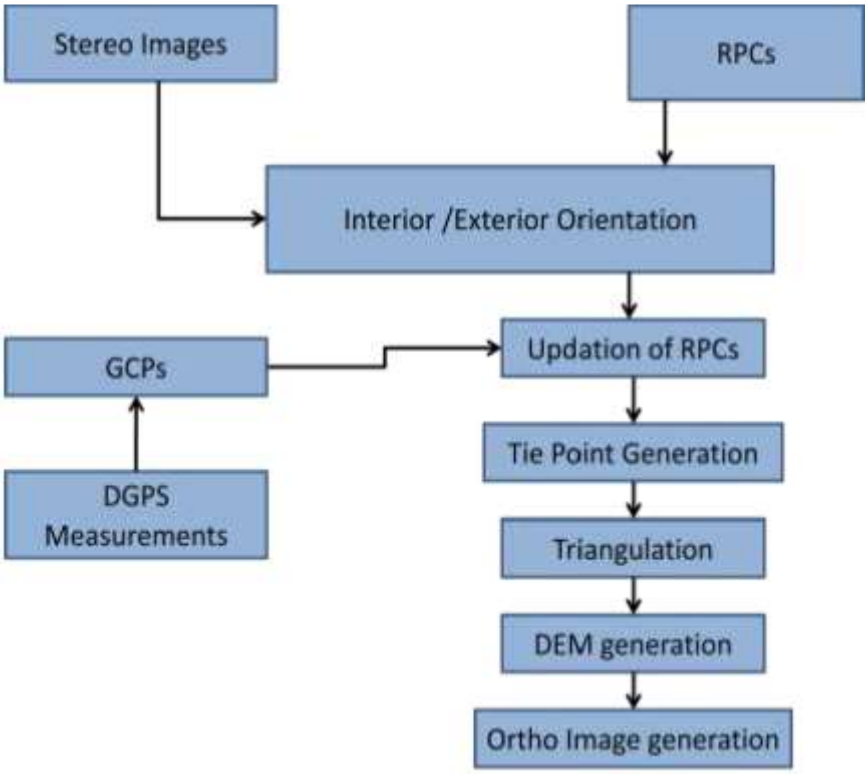


Fig 7: Flowchart for orthoimage generation

.

In the course of a photogrammetrical project we can provide:

- GPS Control
- Aerial Photography
- Airborne GPS and Triangulation
- Triangulation & Adjustments
- Precise Photogrammetric Observations
- Contour and Feature Mapping

Using a computer called a stereoplotter, the stereo pair can be viewed as a single image with the appearance of depth or relief. Ground control points are established based on ground surveys or aerial triangulation and are viewed in the stereoplotter in conjunction with the stereo pair. In this setting, the image coordinates of any (x,y,z) point in the stereo pair can be determined and randomly selected and digitized. These points, in conjunction with the control points, comprise the data points for the DTM. The accuracy of the final digital orthophoto will depend in large part on the point density of the DTM.

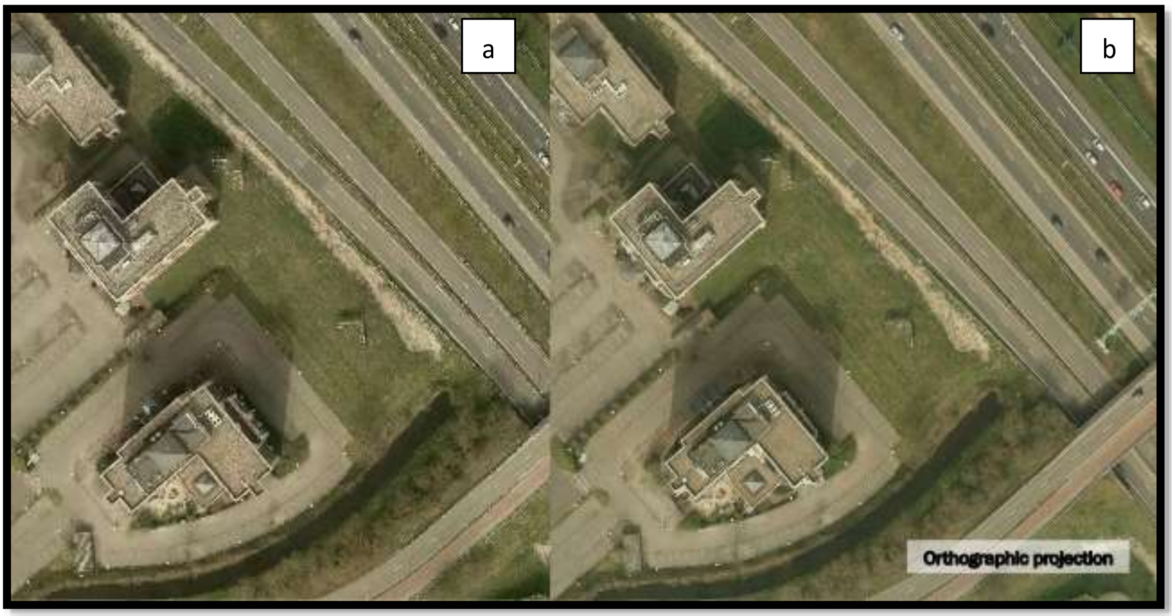


Fig 8: image before(a) and after(b) Orthorectification.

Advantages of Photogrammetry

- Cover areas quickly.
- Low costs.
- Easy to obtain/access information from air.
- Illustrates great detail.
- 3D Visualization.

Applications of Photogrammetry:

Photomapping Services provides solutions to the highly specialised needs of today's world to compile datasets from photogrammetry such as the following:

- **Land Surveying:** Surveying or land surveying is the technique, profession, and science of determining the terrestrial or three-dimensional position of points and the distances and angles between them. it involves land use land cover planning urban planning wasteland mapping, etc.

Fig 9: Area demarcation for land surveying

- **Topographic mapping**: One of the most widespread use of Photogrammetry is topographical mapping, which is considered the primary approach to GIS base data collection and updating. it provides topographical information of the area.

Fig 10: Contours draped over the topography of the area

- **Terrain Models:** Digital Elevation Models (DEMs) / Digital Terrain Models (DTMs), Spot Heights, Contours, and Breaklines, which in turn useful to:
    - superimposed over an orthophoto.

- useful to determine irrigation requirements
- useful to determine drainage requirements
- ideal for rural property mapping

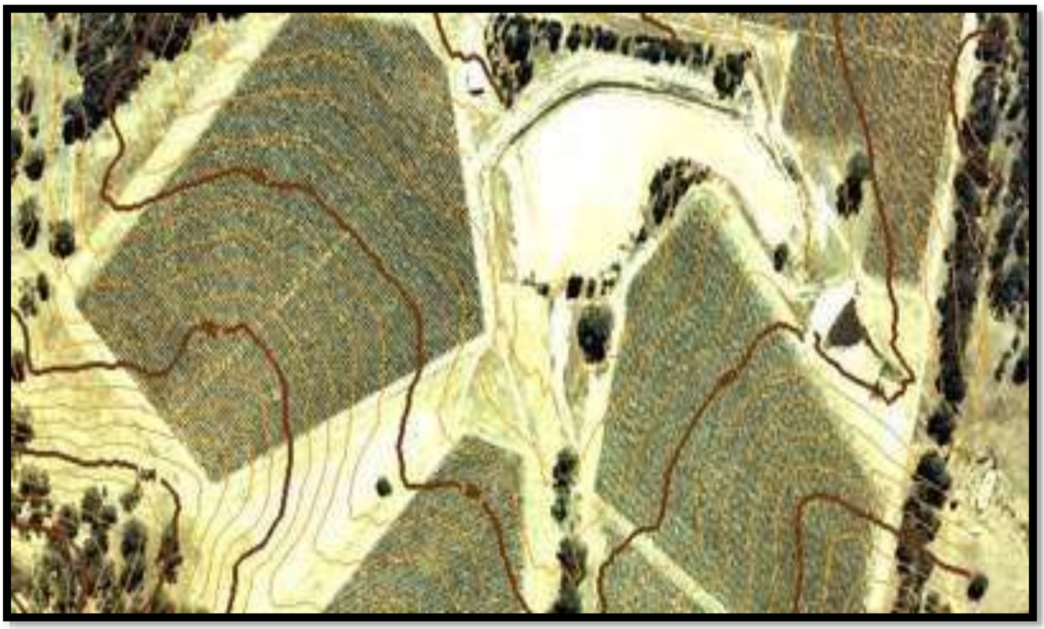
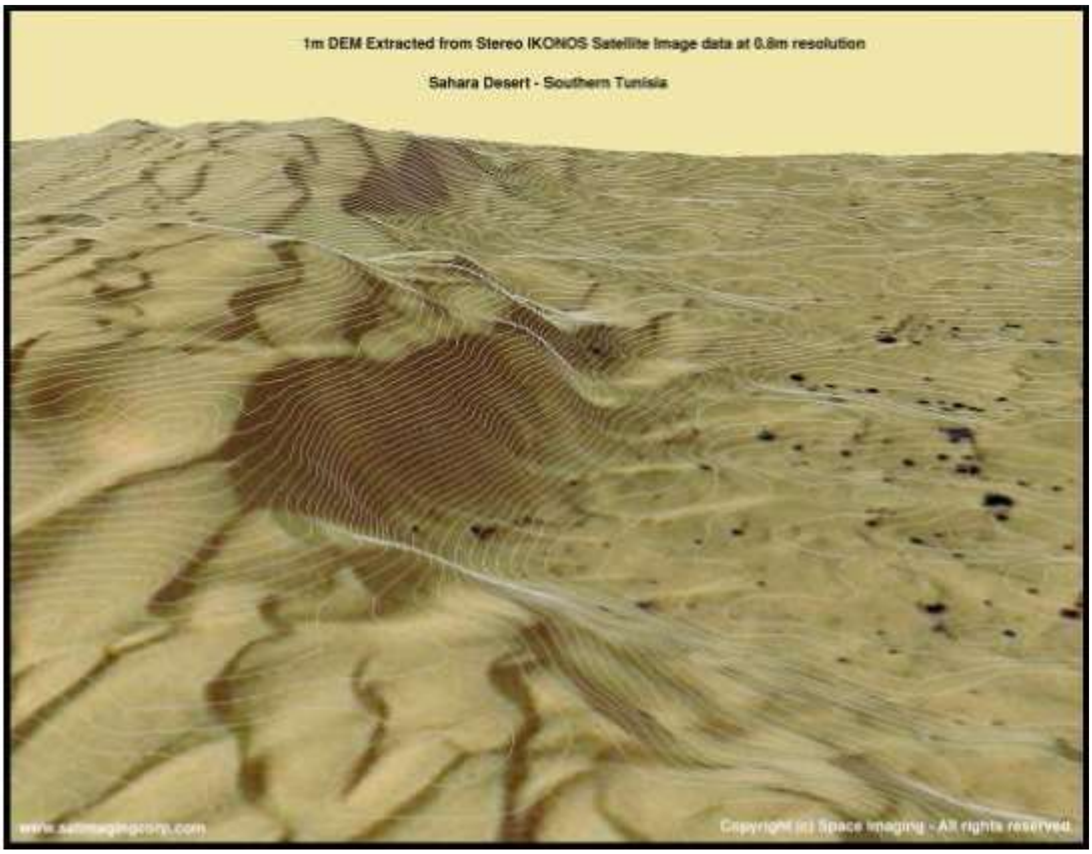


Fig 11: Contour superimposed on orthophotos.



➢ Engineering design:

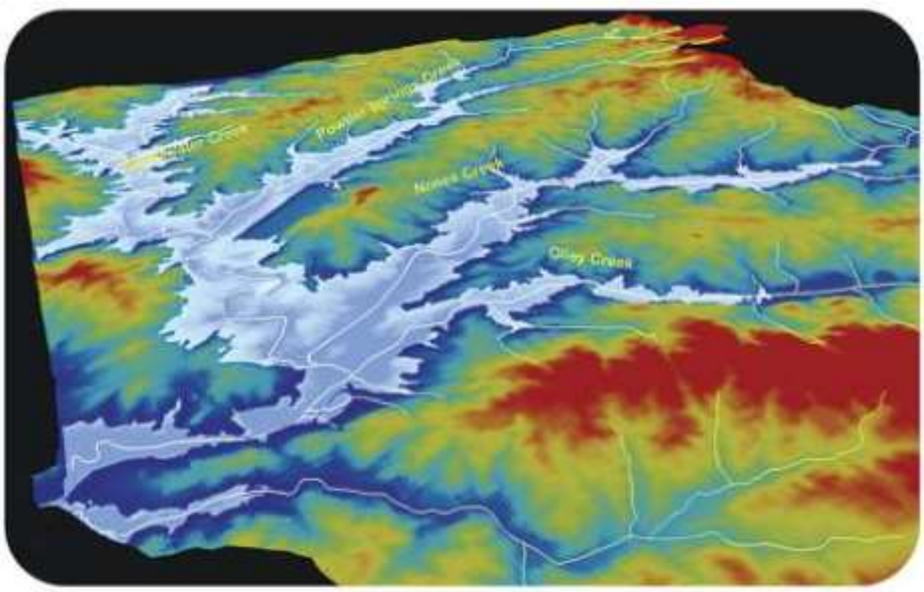
- 3D model of a busy intersection and railway junction.
- useful for urban infrastructure planning.
- useful for infrastructure design and modelling.

Fig 13: Engineering design planning

➢ **Natural resource and environmental inventory:** Photogrammetric products are being used to monitor natural and cultural ecosystems.

➢ Hydrographic survey:

- Detailed Drainage Studies
- 3D model of extensive drainage network
- useful to optimize drainage systems
- ideal for high density urban mapping



- **Geological Mapping:** This technique allows to obtain a highly accurate 3D picture of the visible outcrop. The spatial pattern of joints in nature can be investigated using the software. This might help to understand how physical rock properties influence the spatial complexity of fracture systems and develop constitutive scaling relationships for certain rock types.

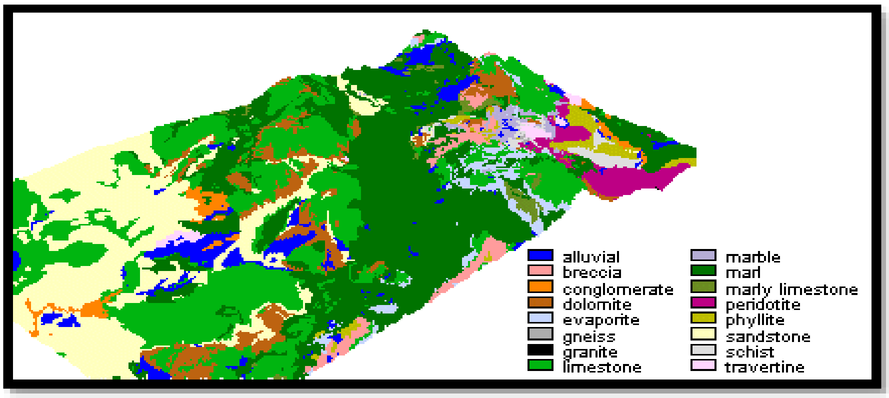


Fig 15: Lithological map draped over the topography

- **Architectural photogrammetry**: Photogrammetric techniques are being used for the representation of the facades or elevations of historic buildings and structures. The most common product is the line drawing which delineates architectural form. Such surveys are needed by the various disciplines involved in building repair and conservation.

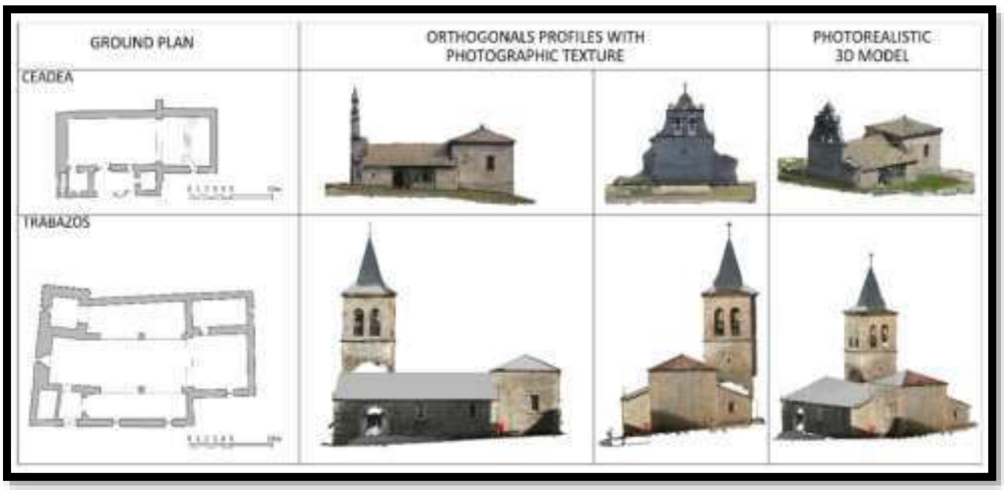


Fig 18: Final graphic documentation of the churches of Ceadea (**top**) and Trabazos (**bottom**): ground plan, orthogonal profiles with photographic texture and photorealistic 3D model.

- **Archaeological mapping:** Using Terrestrial photogrammetry archaeologists can produce photographic plans of sites and their stratigraphy, take accurate measurements directly from the photo, and import photographic data into other computerized technologies for mapping and visualizing archaeological features. The production of a photogrammetric image involves the combination of a number of technologies:
    - Total station surveying.
    - Traditional archaeological photography.
    - Geospatial rectification.

  By combining these technologies, we are able to produce a hybridized documentation technique that can serve many purposes.

Fig 19: 3D model of archaeological site

- **Industrial machinery:** Photogrammetry has been increasingly applied as a precise 3D measuring tool in industrial and engineering works. Analytical photogrammetry is now routinely employed in tasks of measurement:
    - Machine tool inspection.
    - Fixture checking.
    - Structural deformation monitoring.
    - Provision of control databases to guide
    - Industrial robots
    - Measurement of structures in earth orbit.

  photogrammetric method has advantages of a non-contacting 3D object reconstruction by means of spatial rays. It provides, a short recording time on-site nearly independent from the amount of object points to be measured, and the possibility to choose the recording stations in a very flexible way, If a dynamic or kinematic process has to be recorded, photogrammetry seems to be the only way to measure a whole object simultaneously.

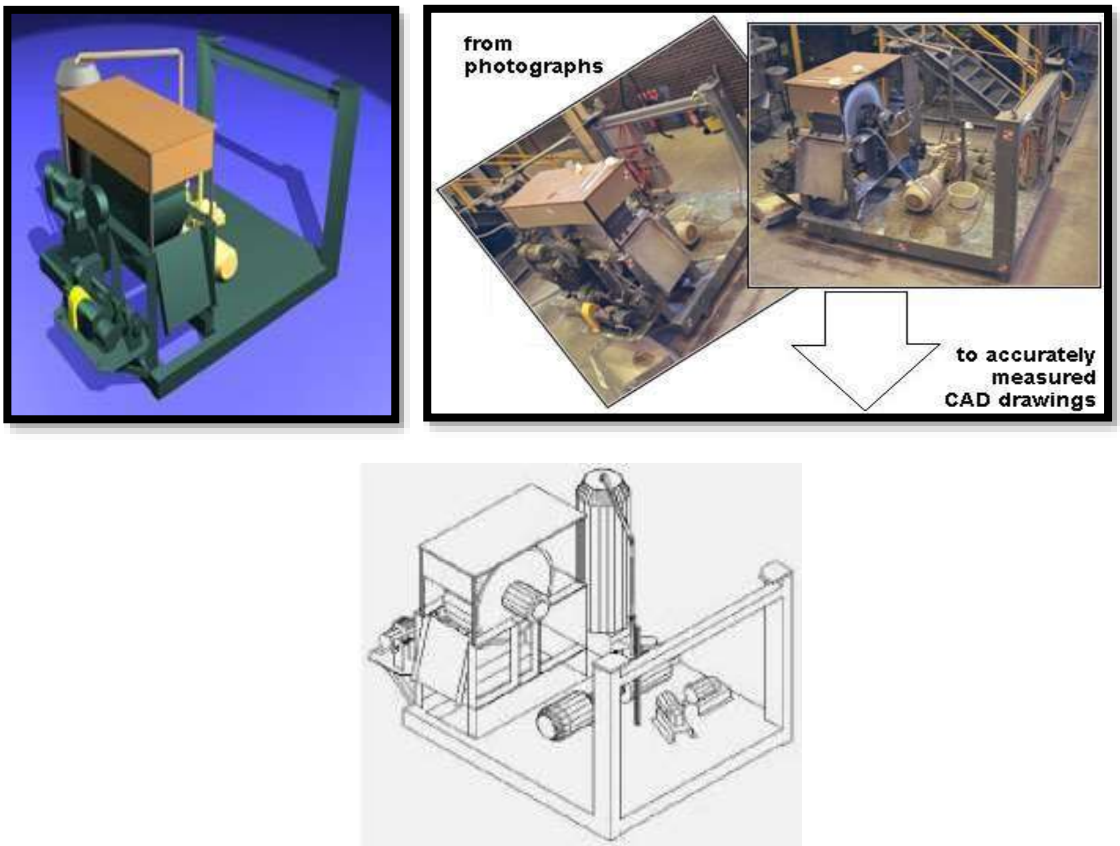


Fig 19: 3D CAD design

- **Military applications:** Mosaic is a aerial photograph of a large area, made by carefully fitting together aerial photographs of smaller areas so that the edges match in location, and the whole provides a continuous image of the larger area. Mosaics are intensely being used by military projects because they provide:
  - Synoptic view all over the target area.
  - Target planning.
  - Topography of the area.
  - Decision making, Etc.

CHEPTER -6 :-BASICS ON GPS & DGPS AND ETS

GPS

GPS ,which stands for Global Positioning System, is the only system today able to show you your exact position on the Earth anytime, in any weather, anywhere.

Applications of GPS

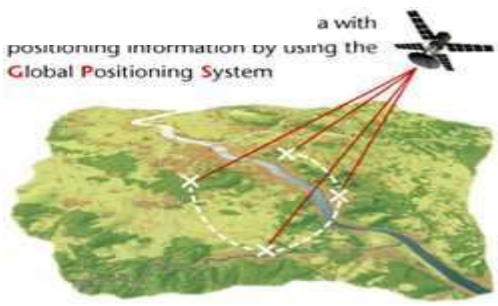
4. Providing Geodetic Control
5. Photogrammetry
6. Finding out location of offshore drilling
7. Pipe line and power line survey
8. Navigation of civilian ships and planes
9. Crustal movement studies

The History of GPS

1. Feasibility studies begun in 1960 .
2. Pentagon appropriates funding in 1973.
3. First satellite launched in 1978.
4. System declared fully operational in April, 1995

Receivers and Satellites

GPS units are made to communicate with GPS satellites (which have a much better view of the Earth) to find out exactly where they are on the global scale of things



GPS Satellites

The GPS Operational Constellation consists of 24 satellites that orbit the Earth in very precise orbits twice a day. GPS satellites emit continuous navigation signals.

GPS Signaling

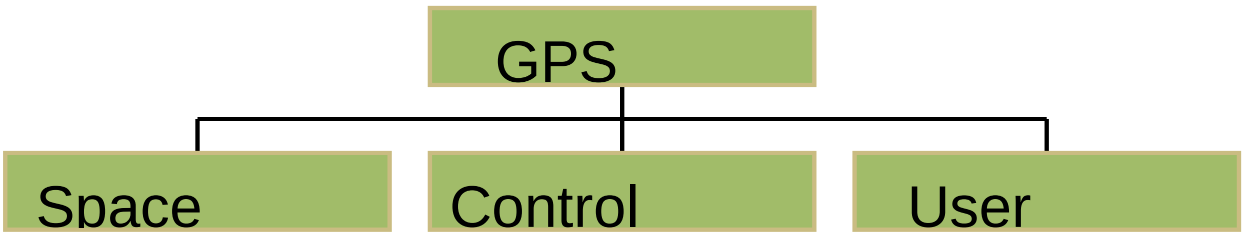
Each GPS satellite transmits data that indicates its location and the current time. All GPS satellites synchronize operations so that these repeating signals are transmitted at the same instant

POSITION CONCEPTS OF GPS

1. Latitude
2. Longitude
3. Altitude

Latitude and Longitude

Latitude and Longitude are *spherical coordinates* on the surface of the earth. Latitude is measured North or South of the Equator. Longitude is measured East or West of Greenwich. GPS uses Latitudes and Longitudes to reference locations.



Control Segment:

The control segment comprises of 5 stations.They measure the distances of the overhead satellites every 1.5 seconds and send the corrected data to Master control.Here the satellite orbit, clock performance and health of the satellite are determined and determines whether repositioning is required.This information is sent to the three uplink stations.

User Segment:

It consists of receivers that decode the signals from the satellites.The receiver performs following tasks:

a) Selecting one or more satellites
b) Acquiring GPS signals
c) Measuring and tracking
d) Recovering navigation data

GPS signal errors

Factors that can affect the GPS signal and thus affect accuracy includes as follows :

1. Ionspheric and Troposphere Delays
2. Signal Multipath
3. Receiver clock Errors
4. Orbital errors/ Ephemeris error
5. Satellite Geometry / Shading

1.Ionspheric and Troposphere delays :

The satellite signal slows as it passes through the atmosphere. The GPS system uses a ‘built in model’ that calculates an average amount of delay to partially correct for this type of error.

2.Signal Multipath

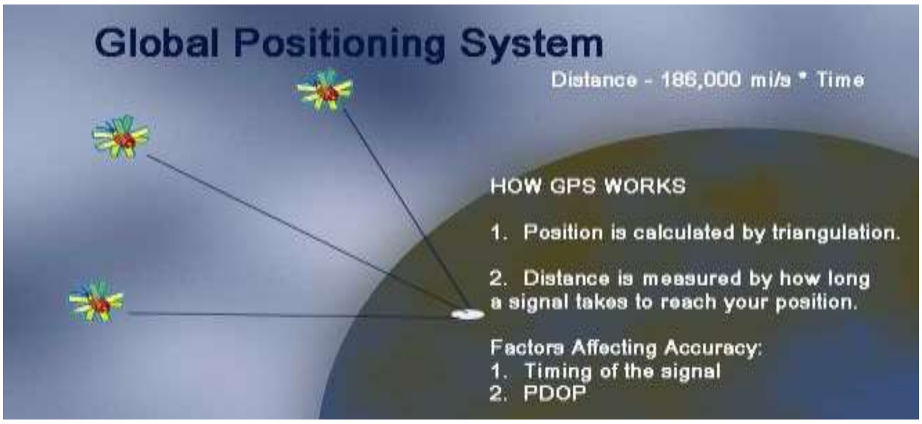
This occurs when the GPS signal is reflected off objects such as tall buildings ,large rock surfaces etc. before it reaches the receiver. This increases the travel time of the signal thereby causing errors.

3.Receiver Clock Errors

A receiver’s built –in clock is not as accurate as the atomic clocks on board the GPS satellites. Therefore, it may have very slight timing errors.

4.Orbital Errors

Also known as ephemeris errors. These are inaccuracies of the satellite reported location



Differential GPS(DGPS)

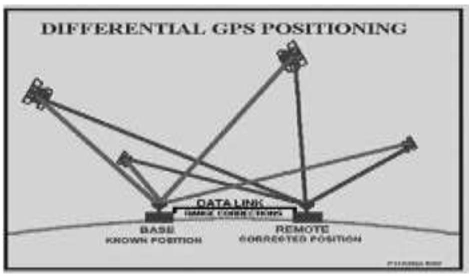
Differential GPS(DGPS) is a system in which differences between observed and computed co-ordinates ranges(known as differential corrections) at a particular known point are transmitted to users(GPS receivers at other points) to upgrade the accuracy of the users receivers position.

Differential GPS Positioning

Differential positioning user finds the point position derived from the satellite signals and applies correction to that position. These corrections, difference of the determined position and the known position are generated by a Reference Receiver ,whose position is known and is fed to the instrument and are used by the second Receiver to correct its Internally generated position. This is known as Differential GPS positioning.

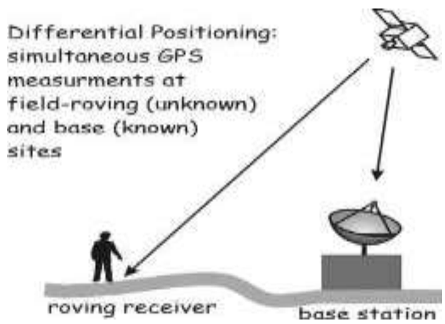
Limitation & Errors of DGPS

1. International Limitation of Accuracy
2. Receiver Independent Exchange Format
3. Reference System Co-ordinates



<u>Differential Correction</u>

Differential correction is a technique that greatly increases the accuracy of the collected DGPS data. It involves using a receiver at a known location - the "base station"- and comparing that data with DGPS positions collected from unknown locations with "roving receivers."



<u>Electronic Total Station (ETS)</u>

The total station is an electronic/optical instrument used in modern surveying**.** The total station is an electronic theodolite integrated with an electronic distance meter (EDM) to read slope distances from the instrument to a particular point

<u>Distance Measuring (Electronic Distance Meters)</u>

In the early 1950's the first Electronic Distance Measuring (EDM) equipment were developed.

Primarily consisted of electro-optical (light waves) and electromagnetic (microwave) instruments & Bulky, heavy and expensive.The typical EDM today uses the electro-optical principle. They are small, reasonably light weight, highly accurate, but still expensive.

<u>Electronic Total station & Prism</u>

CHEPTER-7 :- BASICS OF GIS AND MAP PREPARATION USING GIS PRINCIPLES OF GIS

INTRODUCTION

GIS is a computer-based system that is used in input, output, storage, manipulation, retrieval and analysis of spatial data. These systems consist of computer hardware and software. GIS are increasingly being used in applications in natural resources, tourism, transportation, trade and commerce etc. GIS is also integrated with modern technology of remote sensing and GPS. GIS are now integral part of hydrological models.

History

GIS has basis in manual overlay operations done as early as in 1912 to 1969. In 1958, computer based cartography initiated in University of Washington, which culminated in development of first general purpose mapping software in 1960s. Canada GIS (CGIS) is also cited as first GIS and was developed.

Present day commercial/ open source GIS are Arc GIS, GRASS, MapInfo, ERDAS, IDRISI, ILWIS etc. GRASS (Geographic Resource Analysis Support System) is high-end open source software. Arc GIS is modular high-end commercial software. ERDAS, ILWIS and IDRISI have image processing and GIS capabilities.

Advantages and limitations

Using GIS, it is possible to overlay large number of maps. Conventionally, manually overlays are prepared. This process is cumbersome and error prone. In GIS retrieval of information is faster as it is done through computer. Conventionally, maps are browsed to retrieve information. In GIS information retrieval is much easier and is done automatically. In GIS, interactive/ virtual output may be prepared. The virtual output is automatically updated, if the component maps are revised. In convention method, hardcopy output is prepared. Updating of such maps is difficult. The map is required to be redrawn. Annotation is clumsy in hardcopy maps. Thus, while retrieving information ambiguity may arise. Also all features may not be annotated in paper maps. Thus, attribute information e.g. names etc. for some of the features are lost in paper products. In GIS, information is stored in tables and is linked to geographic features and thus is not limited by availability of annotation space/ color/ symbol etc. If multiple maps are prepared for same area e.g. watershed, land use, geomorphology, common boundaries are drawn manually and may not match in different maps. In GIS common boundaries are once digitized and are available to all layers. Once GIS map layers are prepared, any number of maps can be designed. Handling of paper maps is difficult.

Data capture or input is costly in GIS. Commercially available paper maps may be cheaper than GIS layer. This is because of high cost of data capture in GIS. Use of GIS requires investment in computers, software and training. GIS handling requires trained manpower. In GIS data are required to be converted in to native format of GIS software. In suitable import/ export functionality is not available or the format is obscure/ unknown, data may be unusable.

GIS software should have proper functionality as desired in an application. For example, in transportation applications, network analysis function should be available. For hydrological modeling, DEM analysis functions should be available.

SPATIAL DATA

In GIS, maps are called spatial data. Information on paper maps can be input is GIS as spatial data. Example of spatial data are stream network, well locations, villages, cities, topographic contours, spot elevations, roads, land use, soil, geology, hydrological investigation locations, hydrological response units etc. Spatial data are classified in to three types namely area, line and point. Areas are spatial data that are represented as closed figures e.g. forests, lakes, Thiessen polygons etc. Lines are spatial data that are represented as lines e.g. forest boundaries, lake boundaries, contours, stream network, roads etc. Point spatial data are represented as points on maps e.g. well locations, rain gauge stations, spot elevation, villages etc. The data is scale dependent in some instances
e.g. on small-scale maps a city will be represented as point data, where as on large-scale map, it will be represented as area data. A closed line data can be converted easily to area data in GIS. For example forest boundary data can be converted to forest land-use map. Point data cannot be converted in to area data. But reverse is true i.e. area data can be converted as point data. For example, if city point data is captured, city polygons cannot be obtained from this point data. City polygon data will be required to be captured separately.

Spatial Data Representations

Spatial data are represented in two mainly ways in GIS namely raster and vector. These data representations can be translated in to one another, albeit with some information loss. In raster, spatial data are structured as grid of cells or pixels. Their row and column numbers addresses the cells. In many distributed hydrological models, processing is spatial data are represented in this form and hydrological computations are also done in this form. This is a native representation for remotely sensed data. Satellite data are captured/ re sampled as pixel (picture elements) or grid cell. Thematic maps are prepared from these data through digital image processing. These maps are available in raster representation. In vector model, spatial data are represented as coordinate points. For example, point data is represented as a pair of coordinates. A straight-line data is represented as two pair of coordinates, representing end points of the straight line. A curved line is represented as finite line segments. Area data are represented as line data with some additional information e.g. centroid, adjacent areas etc.

Thematic maps prepared from remotely sensed data are available is raster form and are often processed as such. Many hydrological models use both the representation. For example, thematic maps of catchment variables and hydro meterolgical measurements are prepared in raster form. Stream network is processed in vector form etc. In raster form, value of many catchment variables is scale dependent. For example, average slope of catchment reduces with increase in raster grid size. Thus, results of un calibrated hydrological models will differ at different raster grid size used in parameter derivation. In most of GIS, the representations coexist. For example, it is better to capture spatial data from conventional thematic maps, through visual interpretation of remotely sensed data etc. in vector form. Thematic maps from digital processing of satellite data may be obtained in raster form. Data can be transformed in to one another as and when desired.

Topology

Method of representing vector data is called its topology. A line consists of two nodes and one or more vertices. Nodes are end points of the line. Lines also have directions. Thus, nodes are referred as ‘from node’ and ‘to node’ depending on direction of the line. Areas are represented by ‘left area’ and ‘right area’ of each line.

DATA FORMAT

There are many formats prevalent for images, raster and vector GSI data. Image file formats are normally used by non GIS applications. For use in GIS, data in these formats are required to be imported. The image file formats may or may not have projected coordinate system.

GIF (Graphics Interchange Format): This is copyrighted format by ‘CompuServe’ for image data. A data compression scheme similar to lossless LZW compression is used. Georeference information of raster data is not stored.

TIFF (Tagged Image File Format): The format is designed by Adlus and Microsoft. For types of images are supported namely bitlevelm gray scale, color- mapped (256 color) and RGB- based (three layer or band for full color). The compression type can be uncompressed, RLE or modified Huffman. A format for storing georeferenced images is also designed and is called ‘geo- tiff’.

JPEG (Joint Photographic Expert Group): The format uses lossy compression technique. Compression of 20:1 can be achieved. The format is designed for grey- scale and color images. Georeference information of raster data is not stored.

PNG (Portable Network Graphics): This format uses lossless compression and the compression technique used is not a patented technique. This format is suitable for images with line drawings rather than true color images.

GIS file formats

NTF (National Transfer Format): This is developed in United Kingdom 9UK) for transfer of digital cartographic data in 1987 and updated in 1989. It is format for both raster and vector data. Levels are defined from 0 to 5 for raster, simple vector, complex vector, topology and user defined formats.

SDTS (Spatial Data Transfer Specifications): The data exchange format standard is developed by US Digital Cartographic Data Standards Task Force (DCDSTF) in 1988 for both raster and vector data.

e00 (Arc Exchange Format): This is exchange format developed by ESRI for vector data.

Arc coverage: This is a format for vector data developed by ESRI. The format is also used in ERDAS imagine.

Shape: This is a format for vector data developed by ESRI. The format is also used in ERDAS imagine. A shape file is a collection of files with extension shp, shx, dbf, prj etc. File with shp extension stores actural geographc data. File with shx extension stores index. The file with dbf extension is a dbase III file which stores attribute data. The prj file stores projection information and is a ASCII file. Shape files do not store topological information.

Img: This is a format for raster data developed by ERDAS Incorporated (now Lyca).

DXF (Drawing Exchange Format): The format was designed for data exchange between Autocad and other Cad packages. This is a de facto standard for exchange of vector data.

TIGER (Topographically Integrated Geocoding and Referencing) and GBF-DIME: US Bureau of Census developed these formats in respectively 1990 and 1980. Former is a topologically structured format.

DLG (Digital Line Graph): US Geological Survey (USGS) developed the format for transfer of fully structured data from National Cartographic Database. DLG-3 and DLG-E are types of this format.

BSQ/ BIL and BIP: These are generic data formats and are mainly used for remotely sensed data and some times for GIS data e.g. in ILWIS software. In BSQ (Band Sequential) format, each band of remotely sensed data is stored in separate files. In BIL (Band Interleaved by Line) format each band of a scan line are stored in alternate records in single file. In BIP (Band Interleaved by Pixel) group of pixels (e.g. two pixels) are stored for each band, are stored alternately.

DIGITAL ELEVATION MODEL (DEM)

Topographic elevation data in GIS are called DEM. These are represented in GIS in various manners namely contours, raster, TIN (Triangulated Irregular Network). Contours are conventional representations of DEM and are used in topographic maps. Contours are equal elevation lines. Normally, equal interval contours are drawn in topographic maps to represent topography. For example in 1: 250,000, 1:50,000 and 1: 25,000 scale Survey of India (SOI) maps, contours are at 100, 20 and 10 m elevation interval. Ridges, valleys can be interpreted from these maps. DEM in Raster and TIN representations can be used in deriving topographic information such as slope, aspect and can also be used in hydrological calculations e.g. stream network delineation, topographic index, flow routing, up steam contributing area etc. and in turn in hydrological modeling.

TIN

In TIN model, elevations at the vertices of triangles are used to compute elevation at interior points of the triangles. Using elevation of the vertices of a triangle, a planner or higher order surface can be fitted. The surface can be used to derive elevation at points inside the triangle. TIN model requires Delaunay triangulation. In this, constituent triangles are as equilateral as possible. Circumcircles of the triangles include no other point of the triangulation. Triangulation is performed first by constructing Voronoi diagram (Thiessen polygons). Points included in adjacent polygons are joined to create Delaunay triangulation. Voronoi diagram is drawn using proximity analysis.

Interpolation

Interpolation is a technique of determining unknown value of a variable at location from known values at other locations. Interpolation can be used for any spatial variable e.g. topographic elevation, pH, SAR, pollutant concentration, groundwater depth and level, population etc. Known values can be at point, line of area locations. Point data can be spot heights, pH, pollutant concentration etc. Line data can be topographic contours etc. Area data can be population density in regions etc.

Voronoi diagram or Thiessen polygons or nearest neighbor

To determine basin wide average rainfall, this method of interpolation is widely used. The diagram is prepared by proximity analysis. The Thiessen polygon map is intersected with the catchment map. Area of a Thiessen polygon corresponding to a raingauge station in this intersected map is used as weight in finding weighted average rainfall for the catchment.

Distance weighted averaging

In distance weighted averaging, a weight of inverse of distance function is used. Distance function is nth power of distance. Thus, a higher weight is assigned to values closer to the interpolation location. At any point values are estimated as weighted sum of known values at selected locations. The selected point can be as follows:

- All points within a given range
- Specified number of closest points
- Specified number of closest points within quadrants/ Octants etc.

Surface fitting

A n- degree polynomial surface can be fit between selected known values. The points on this surface represent interpolate values. The points to be used for interpolation can be selected in similar way as that in distance weighted averaging method.

Krigging

Kriging is a statistical technique called best linear unbiased estimator (BLUE). Spatial variables have three components namely drift or structure, small variations and random noise. First component depicts general trend of the data. Second component represents small variations from the general trend. These variations are random but spatially autocorrelated. Third component depict random values that are not spatially autocorrelated. Kriging technique is best suited for interpolation of pollutant concentration, geological and mining variables e.g. grade of ores etc. In these data, single smooth mathematical equations are not suitable. The technique is based on assumption that values in neighbourhood have generally higher correlated. For example elevation in plain area is generally lower than that for hills and varies less abruptly. Apart from the estimate of values, error estimates are also provided in kriging technique. In presence of large random noise in data, good semivariogram is not obtained and this results in deterioration in interpolation quality.

Semivariogram

Semivariogram is a plot of semivariances and distances of the samples for which the semivariance corresponds. Semivariogram is also referred as variogram. When variogram for all separation distances are plotted, the resulted variogram plot is called raw variogram. For 'n' data points, the resulted points in the raw variogram will be 'n*(n- 1)/2'. Raw variogram show cloud of points. A representative variogram or experimental variogram is more useful for fitting theoretical models and doing kriging. In computation of this, the separation distances are grouped. Plotting positions for these groups are mean, median, or middle of the class intervals. Distance intervals for these groups at smaller distances can be smaller.

For mathematical formulation of kriging interpolation, a theoretical models is fit for experimental variogram. Models e.g. Gaussian, exponential, power etc. are used. Gaussian model has parabolic nature at origin. Exponential and power models are linear at the origin. Parameters of the model are sill, range and nugget. The sill is equal to the variance of the data. Due to experimental error and micro level variations, experimental variogram may not pass through origin. This property is modeled byNugget. It is modeled through a discontinuity at the origin.

Validation

'Jack- knifing' can validate the model. A subset of data is used in kriging. The complementary datasets of these subsets are used in validation of selected model. Residuals and standard errors are estimated at all discrete locations. The mean and variance of these residuals should be theoretically 0 and 1 respectively. Normally acceptable deviations of these measures from their true values may be utmost 0.15 to 0.20.

Linear contour interpolation

In linear interpolation from contours, distance map is estimated from contours. Based on distances at an interpolation point towards two nearest points, value is interpolated linearly. The value at the contours is retained in the final map. Distance function can be used in estimating distances.

Ray method

In this method rays are drawn in four, eight or sixteen directions from interpolation points. A value of the contours at points where the rays intersects the contours is determined. Using two nearest values, interpolation is done. Average of all interpolated values provide required interpolation.

GIS OPERATIONS

Input

Digitization

Digitization is done on- screen to create/ edit GIS objects in vector format.Digitization errors for area objects Following errors occur while digitization of area objects:

Dangle: Additional lines are some times digitized that are not areas. These lines appear as dangles.

Overshoot: Areas are closed figures. Sometimes, line is extended beyond closed figure. These are called overshoots.

Undershoot: Sometimes, line stop before closing of the area. These are called undershoots.

Self-overlap: A line crossing itself is called self-overlap.

Intersection: Lines cross each other without a node at the intersection.
Other errors are missing lines, incorrect labels etc.

<u>Data import</u>
Input data are, some times, available in GIS, image formats. These data are converted to native format of GIS through import utility.

Storage

Geographic data are stored in GIS is native format of GIS. For one data, many computer files are created. These files are copied, renamed, deleted wihin GIS. These operations can also be done outside GIS. Attribute data are stored in DBMS. Attribute data are managed within GIS or through DBMS software. Attribute data are linked to geographic objects. External databases can also be connected to geographic data. Data types of geographic objects and their attributes are bit, byte, integer, real, double, text etc.

Analysis

Data analysis involves operations with geographic data and their attributes to obtain derived information, generate query, statistics etc.

Statistics: Statistics e.g. count, length, area, perimeter, shape, centroid, rose diagram etc. of geographic objects can be derived in GIS. For continuous surfaces, average, standard deviation, maximum, minimum etc. are derived. Summary operation produces zonal statistics for a map. For example, land use statistics for watersheds in a basin can be generated.
Mathematical operations: Mathematical operations e.g. addition, subtraction, multiplication, division, exponential, logarithm, absolute, truncation, round off, negative, trigonometric operations can be performed in GIS. For example various component maps in USLE namely R, K, L, S, C and P can be prepared and multiplied using multiplication operation. This operation will multiply these factors for all cells and provide long-term average annual soil erosion for each cell.

Logical operations: Logical operations namely or, and, not, xor can be performed on maps. For example, landuse= agriculture and pH>=8 will result in salt affected agriculture area.

Conditional: If- then-else conditional operational can be performed on maps. For example, 'if 50 < return period <=100 and land use= residential, then vulnerability= high else vulnerability=low' condition gives flood vulnerability map.

Overlay: In this operation, all combinations of classes in two maps are obtained in the resulted map. For example, overlay of soil hydrological soil group and land use/ cover map will provide soil- cover complex map.

Reclassification: Information of geographic object is changed in reclassification. For example soil series map may be changed to soil pH map.

Classification: Classification converts values in to interval. A continuous surface is input and area map is output for the operation. In the output area map, isolines, i.e., line of equal values, enclose the area. Examples of various isolines are contours, isobath, isohyete, isotherm, isobar etc., which represent topographic elevation, groundwater table, rainfall, temperature, pressure etc.

Distance: Distance from a geographic object is estimated. Diagonal distances are nearly consider

Search/ buffer: The operation is similar to distance, except that at a specified distance an area geographic object is created.

Neighborhood: Information in eight neighbor, their locations and statistics e.g. mean, mode, median, predominant, minimum, maximum, standard deviation, coefficient of variation etc. are extracted.

Aggregate: Cell size of raster maps can be changed in fractions of half, one fourth etc. using functions e.g. mean, predominant, minimum and maximum.

Query: Query is done by attributes or geometry. In query by attribute, a logical expression is written in attributes and result is obtained. For example land use=agriculture will select/ display agriculture areas. In query by geometry, objects are selected on screen to view their attributes.

Output

The output maps may contain various cartographic elements namely title, legend, graticules or grids, north arrow, scale, annotations, notes etc. In one output more than one GIS layer may be included apart from cartographic elements. When design is saved, it only contains only reference to the layers. Thus, if a layer is modified and designed output map is opened at a later time, the changes are reflected in to the output.

APPLICATIONS

Groundwater studies

Groundwater depth and quality is studied in GIS through kriging interpolation technique. Water quality variables e.g. EC, RSC, HCO3 etc. can be interpolated using the technique. In these applications, sample locations can be important. For example, samples are taken from working tube wells, which may be in general of good quality and poor water-quality is under represented. Range can be of order of 10 to 100 km. Models can be exponential, spherical etc.

Groundwater potential and quality can be mapped in GIS environment. Various layers namely slope, geology, hydromorphogeology, distances to drainage channel, tanks and lineaments, depth to water table, depth of weathered zone can be overlaid and integrated on GIS environment to obtain groundwater potential map. Similarly, layers namely magnesium, incrustation problem, TH, TDS can be integrated to obtain quality map in incrustation and corrosion problem areas.

Land degradation

Irrigated agriculture areas often face problems of water logging and salinity. The problems are refereed as twin problem as waterlogging leads to soil salinization in long run. Identification of extent of the affected area is pre requisite for reclamation. Criteria adopted for water logging are given in Table 1.1. Criteria for salinity and sodicity are listed in Table 1.2.

Area with surface pondage and moist soil can be delineated easily using remote sensing data. Water has black tone in standard FCC in visible and near IR bands. Most soil has dark signature in these imageries. Shallow water table conditions often are not detected using optical remotely sensed data unless its expression is visible on the surface of the earth. Areas where yield is affected can be monitored.