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LECTURE NOTES ON

LAND SURVEYING - II

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Surveying II (CEng 2902) Lecture Note

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1 Triangulation and Trilateration

1.1 Introduction

The determination of the precise position of a number of stations, usually spread over a large area, is referred to as control surveying. Control surveys can be horizontal or vertical.The objective of horizontal control surveys is to establish a network of control stations whose position is specified in terms of latitude and longitude, whereas the objective of vertical control survey is to provide the elevations of fixed benchmarks with respect to the mean sea level datum. These control surveys are located where other surveys can be conveniently and accurately tied in to them.

The results of the control surveys are used as a basis from which surveys of smaller extent can be originated. Boundary surveys, construction, route surveys, topographic and hydrographic surveys and other may be involved. Horizontal control can be carried out by precise traversing, by triangulation, by trilateration, and perhaps by some combination of these methods. Intersection, resection, and satellite positioning are also the other methods in horizontal control surveys. The exact method used depends of the terrain, equipment available, information needed and economic factors.

With traversing, a series of horizontal distances and angles are measured. This is generally cheaper due to shorter sights and convenience to carry it out under less favorable weather conditions than the other methods. Its disadvantage is that, there are fewer checks available for locating mistakes in the work and the whole system can rather easily sway or bend. To check a traverse it is necessary to form a loop returning to its starting point or to tie it in to previously established control points.

A triangulation consists of a series of joined or overlapping triangles in which an occasional line (called the base line) is measured and all other sides of the triangles are calculated from angles measured at the vertices of the triangles. The lines of a triangulation system form a network that ties together all the triangulation stations at the vertices of the triangles. A triangulation has the following advantages:

1. More redundancies or checks are available i.e. more than one route can be followed to compute the length of a line.
2. There is little tendency for the system to sway or bend i.e. azimuths can be easily and accurately carried or established throughout system.
3. Outstanding landmarks such as steeples, water tanks, etc can be located by establishing directions from different stations.

Its disadvantages are it needs long-range intervisibility, which in turn requires the erection of special towers and signals, making the system the most expensive. Moreover, a good weather is required to attain intervisibility.

With trilateration, the lengths of the sides of a series of joined or overlapping triangles are measured (usually with the EDM equipment) and the angles are computed from the lengths. It has the following advantages:

1. It is more accurate than the other two types due to the fact that distances can be measured more accurately than angles.
2. It is generally less expensive than triangulation.
3. More checks are available.

Unlike triangulation, it is not easy to position transmission towers, steeples, water tanks, etc. by the EDM because to do so requires reflectors on these landmarks. These landmarks can be located if angular measurements are made to them.
In combined triangulation and trilateration systems all sides and angles in the joined or over lapping triangles are measured. This method provides the strongest control network.

Triangulation as a form of horizontal control is applied when a large area is to be surveyed and when the methods of traversing would not be expected to maintain a uniformly high accuracy over the entire area. The methods of triangulation require a maximum number of precise angle measurements and a minimum number of distance measurements. The triangles are developed in to a net of interconnected figures, and certain lines, called base lines, must be measured in order to compute the other sides in the net.

Triangulation is necessary to control the location of large bridge structures, state and federal highways, dams, canals, and other engineering works of a massive nature. A large project, such as boundary location, power development, water resources development, flood control, irrigation, or reclamation, requires triangulation in order to maintain the necessary accuracy throughout the system.

Once a triangulation system, whether large or small, has been developed, measured and adjusted, the points in the system then furnish control for subsequent traversing, minor triangulation, trilateration, intersection, and resection needed for day-to-day engineering operations.

Survey or accuracy standards are generally defined as being the minimum accuracies deemed necessary to obtain certain specific objectives. Survey specifications can be defined, as the field operations or "recipes" needed to achieve the particular standards desired.

Accuracies required for horizontal control depend on the type of survey and the ultimate use of the control points. There are three orders of triangulation based on their uses: first order (primary), second order (secondary) and third order (tertiary) triangulation.
First order is the highest accuracy and is required for developing the national network of horizontal control, for the study of small crustal movements in areas of seismic activity, and for large metropolitan control expansion. Since it covers a large area, the effect of earth's curvature is to be taken in to account geodetic triangulation. It may cover the whole country primary grids may be provided.
Second order (secondary) triangulation provides point at greater density than first order triangulation. This network is adjusted to fit its parent primary triangle or its surrounding primary control. Less refinement is needed as the network is surrounded by the primary control. It is recommended for controlling extensive land subdivision and construction.
Third order (tertiary) triangulations used to establish control for local developments and improvements, topographic and hydrographic surveys or other such projects for which they provide sufficient accuracy. They are not carried out for rural areas. They may not be adjusted to fit the national network.

A triangulation system can be converted to a pure trilateration system by measuring the lengths of the lines directly, using the EDM' s, without measuring any horizontal angles in the network. The net can be adjusted to obtain the co-ordinates of the stations. However, in order to maintain accuracy of the azimuths of the lines in the net, astronomical observations are made to selected stations. These measured azimuths impose conditions that must be satisfied in the adjustment process.

Trilateration is frequently combined with triangulation in order to strengthen a net that may have serious deficiencies in geometric conditioning, or trilaterated lines are used as check bases in a chain of figures to meet the criteria of strength of figures. A hybrid net of triangulation and trilateration imposes several rigid geometric conditions that must be satisfied in the adjustment of the net. In certain instances, the rigidity is necessary for example; the measurement of very small displacements due to earthquake fault movement must be made over fairly large areas. Structural deformation or displacement of dams is also made with the highest possible accuracy. These measurements must be duplicated periodically with the same degree of reliability in order to reflect the actual condition and to avoid measuring errors, as errors of smaller magnitude may cause a large destruction. The internal accuracy and reliability of such a network is greatly enhanced by a hybrid triangulation trilateration system.

Before lengths of lines in the trilateration net can be used in any subsequent computations, their slope lengths determined by the EDM instrument and corrected for atmospheric conditions must be reduced to corresponding sea level distances. Auxiliary data required for determining slope corrections are either the vertical or zenith angles measured at the two ends of each line or elevations of the two ends of each line. The use of the later will be discussed in the following section.

Although the areas involved in construction are relatively small compared with national surveys the accuracy required in establishing the control is frequently of a very high order, e.g. for long tunnels or for dam deformation measurements. There are two useful elements from triangulation that still remain applicable. If it is not possible to set a target over the point being observed then a distance cannot be measured. If the inaccessible point is to be coordinated from known points then the process is one of intersection. If the inaccessible point has known coordinates and the instrument station is to be coordinatedthen the process is one of resection.

1.2 Intersection

This involves sighting in to P from **known positions.** If the azimuths of the rays are used, then using the rays in combinations of two, the coordinates of P are obtained as follows:

In Figure 1-1 it is required to find the coordinates of P, using the azimuths α and β to P from known points A and B whose coordinates are EA, NA and EB, NB.

PL = EP − EA AL = NP − NA

PM = EP − EB MB = NP − NB

Now as PL = AL tan α

then EP − EA = (NP − NA) tan α

Similarly PM = MBtan β

then EP − EB = (NP − NB) tan β

Figure 1-1Intersection by Azimuth

$EB - EA = (NP - NA)\tan\alpha - (NP - NB)\tan\beta$
$= NP\tan\alpha - NA\tan\alpha - NP\tan\beta + NB\tan\beta$
$\therefore NP(\tan\alpha - \tan\beta) = EB - EA + NA\tan\alpha - NB\tan\beta$

Thus $N_P = \dfrac{E_B - E_A + N_A\tan\alpha - N_B\tan\beta}{\tan\alpha - \tan\beta}$

Similarly $N_P - N_A = (E_P - E_A)\cot\alpha$

$N_P - N_B = (E_P - E_B)\cot\beta$

Subtracting $N_B - N_A = (E_P - E_A)\cot\alpha - (E_P - E_B)\cot\beta$

Thus $E_P = \dfrac{N_B - N_A + E_A\cot\alpha - E_B\cot\beta}{\cot\alpha - \cot\beta}$

*Using equations the above equation*the coordinates of *P* are computed. It is assumed that *P* is always to the right of *A* → *B*, in the equations.

If the observed angles α and β, measured at A and B are used (Figure 1-2) the equations become

$$E_P = \frac{N_B - N_A + E_A\cot\beta + E_B\cot\alpha}{\cot\alpha + \cot\beta}$$

$$N_P = \frac{E_A - E_B + N_A\cot\beta + N_B\cot\alpha}{\cot\alpha + \cot\beta}$$

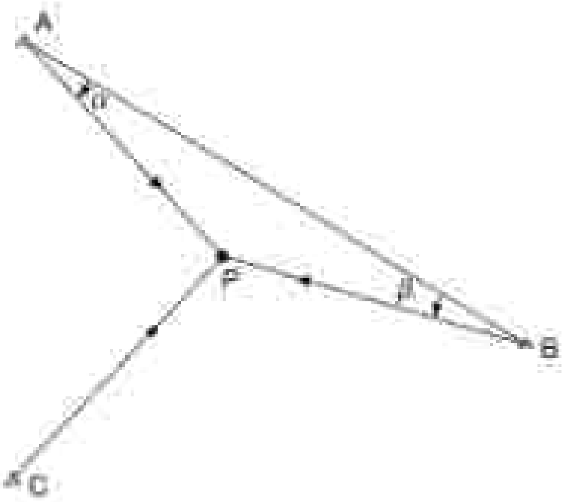


Figure 1-2 Intersection by angles

The above equations are also used in the direct solution of triangulation. The inclusion of an additional rayfrom C, affords a check on the observations and the computation.

1.3 Resection

This involves the angular measurement from P out to the known points A, B, C (Figure 1-3). It is an extremely useful technique for quickly fixing position where it is best required for setting-out purposes.Where only three known points are used a variety of analytical methods is available for the solution of P.

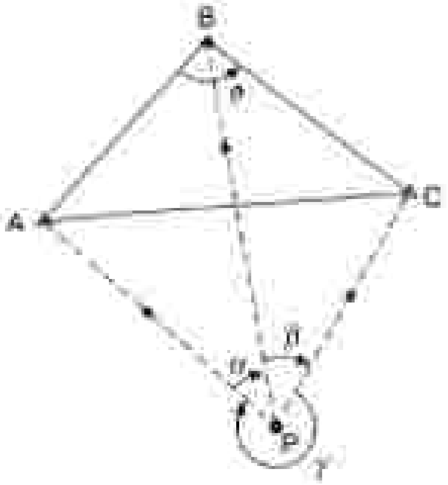


Figure 1-3 Resection

Method I

Let $BAP = \theta$, then $PCB = (360° - \alpha - \beta - \phi) - \theta = S - \theta$
where ϕ is computed from the coordinates of stations A, B and C, thus S is known.
From $\triangle PAB$ $PB = BA \sin\theta / \sin\alpha$

From $\triangle PBC$ $PB = BC \sin(S - \theta)/\sin\beta$

$$\frac{\sin(S-\theta)}{\sin\theta} = \frac{BA \sin\beta}{BC \sin\alpha} = Q \text{ (known)}$$

then $(\sin S \cos\theta - \cos S \sin\theta)/ \sin\theta = Q$
$\sin S \cot\theta - \cos S = Q$
$\therefore \cot\theta = (Q + \cos S)/ \sin S$

Thus, knowing θ and $(S - \theta)$, the triangles can be solved for lengths and bearings AP, BP and CP, and three values for the coordinates of P obtained if necessary.
The method fails, as do all three-point resections, if P lies on the circumference of a circle passing through A, B and C because it has an infinite number of possible positions which are all on the same circle.

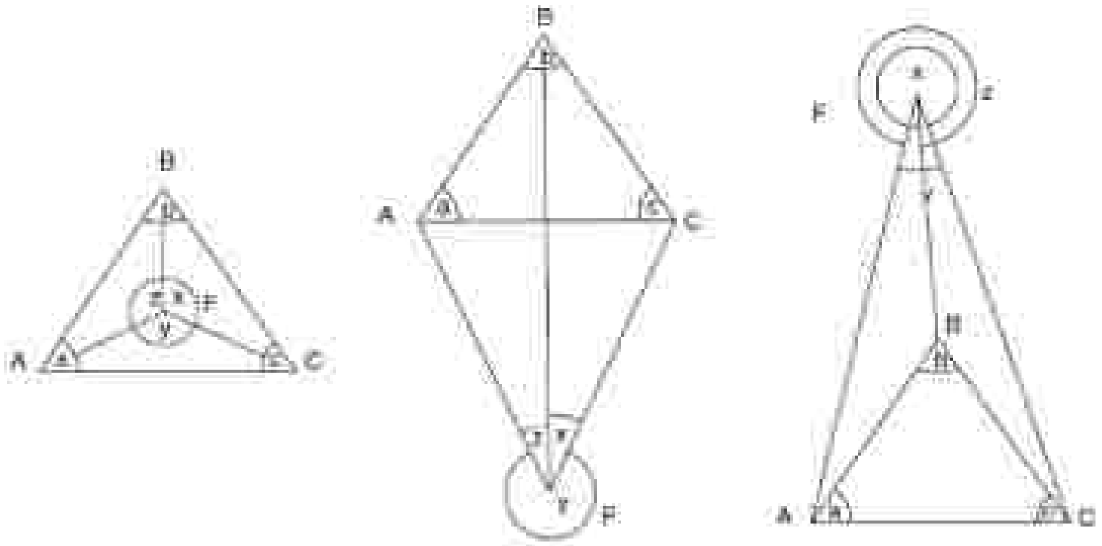


Figure 1-4 Resection figures

Method 2

A, *B* and *C* (*Figure 6.27*) are fixed points whose coordinates are known, and the coordinates of the circle centres *O1* and *O2* are:

$$E_1 = \tfrac{1}{2}[E_A + E_B + (N_A - N_B)\cot\alpha]$$

$$N_1 = \tfrac{1}{2}[N_A + N_B - (E_A - E_B)\cot\alpha]$$

$$E_2 = \tfrac{1}{2}[E_B + E_C + (N_B - N_C)\cot\beta]$$

$$N_2 = \tfrac{1}{2}[N_B + N_C - (E_B - E_C)\cot\beta]$$

Thus the bearing δ of $O_1 \rightarrow O_2$ is obtained in the usual way, i.e.

$$\delta = \tan^{-1}[(E_2 - E_1)/(N_2 - N_1)]$$

then $$E_F = E_B + 2[(E_B - E_1)\sin\delta - (N_B - N_1)\cos\delta]\sin\delta$$

$$N_F = N_B + 2[(E_B - E_1)\sin\delta - (N_B - N_1)\cos\delta]\cos\delta$$

Method 3 'Tienstra's method'

In the three point resection, angles are observed at the unkown stationbetween each of three known stations. Angles at each of the known stations, between the other two knownstations, are calculated from coordinates. Three intermediate terms, *K1*, *K2* and *K3* are also computed.

These are then used in conjunction with the coordinates of the known stations to compute the coordinatesof the unknown station, as in the formulae below.

The coordinates of stations *A*, *B* and *C* are known (*Figure 1-4*). The angles *x* and *y* are measured. Angle *z* is calculated from the sum of angles in a circle. The angles *a*, *b* and *c* are computed from coordinates ofstations *A*, *B* and *C*. The process is then to compute:

$$K_1 = \frac{1}{\cot a - \cot x} = \frac{\sin a \sin x}{\sin(x - a)}$$

$$K_2 = \frac{1}{\cot b - \cot y} = \frac{\sin b \sin y}{\sin(y - b)}$$

$$K_3 = \frac{1}{\cot c - \cot z} = \frac{\sin c \sin z}{\sin(z - c)}$$

and then compute the coordinates of P from

$$E_P = \frac{K_1 E_A + K_2 E_B + K_3 E_C}{K_1 + K_2 + K_3}$$

$$N_P = \frac{K_1 N_A + K_2 N_B + K_3 N_C}{K_1 + K_2 + K_3}$$

The notation in the diagram is all important in that the observed angles x, y and z, the computed angles a, b and c, and the stations A, B and C must all go in the same direction around the figure, clockwise oranti-clockwise. x must be the angle between the known stations A and B, measured from A to B, clockwiseif lettering is clockwise, anti-clockwise if lettering is anti-clockwise. The point P need not lie within thetriangle described by the known stations A, B and C but may lie outside, in which case the same rules forthe order of the angles apply.

Intersection and resection can also be carried out using observed distances.
Although there are a large number of methods for the solution of a three-point resection, all of them fail if A, B, C and P lie on the circumference of a circle. Many of the methods also give dubious results if A, B and C lie in a straight line. Care should be exercised in the method of computation adopted and independent checks used wherever possible. Field configurations should be used which will clearly eliminate either of the above situations occurring; for example, siting P within a triangle formed by A, $B$and C, is an obvious solution.

2 Contour Lines and digital terrain model

2.1 Contours and Contour Lines

A contour is an imaginary line that connects points of equal elevations on the ground surface. A line joining several closely spaced ground points of equal elevation on a drawing is called a contour line. Thus contours on the ground may be represented be contour lines on the map. On a given map successive contour lines represent elevations differing by a fixed vertical distance called the contour interval. The choice of this contour interval depends upon the following factors.

i. **The nature of the ground**:In a flat and uniformly sloping country, the contour interval is small, but in mountainous region the contour interval should be large otherwise the contours will come too close to each other.

ii. **The purpose and extent of the survey**:Contour interval is small if the area to be surveyed is small and the maps are required to be used for the design work or for determining the quantities of earth work etc. Wider interval shall be used for large area and comparatively less important works.

iii. **Scale of the maps**: The contour interval should be in the reverse ratio to the scale of the map i.e. the smaller the scale the greater the contour interval.

iv. **Time and expense of field and office work**: The smaller the contour interval the greater is the amount of field work and plotting work.

The choice of map scale also depends upon:

i. The clarity with which features can be shown.
ii. The cost: - the larger the scale the higher the cost.
iii. The contour interval

Typical map scales, map uses ad corresponding contour interval are shown in the table below.

Map Scale	Typical Use	Contour Interval for mountainous terrain
1:1000	Design	0.25m
1:2000	Design	0.50m
1:5000	Planning	1m
1:10000	Planning	2m
1:25000	Regional Planning	2.5m
1:50000	Regional Planning	5m
1:100000	Regional Planning	5 or 10m
1:250000	Regional Planning	10m
1:500000	State/National	20 or 50m

2.1.1 General Rules for Contour

The principal characteristics of contour lines are as follows:

i. All points on contour lines have same elevation.

ii. Flat ground is indicated where the contour lines are widely separated and steep ground is indicated where contour lines run close together.

iii. The contour lines connecting points on uniform slopes are spaced uniformly.

iv. Contours crossing a human-made horizontal surface (roads, railroads) will be straight parallel lines as they cross the facility.

v. A series of closed contour lines on the map represent a hill if the higher values are inside and a depression if the higher values are outside.

vi. Contour lines across ridge or valley lines are at right angles. If the higher values are inside the bend or loop, it indicates a ridge and if the higher values are outside the bend, it represents a valley.

vii. Contour line cannot end anywhere but close on themselves either within or outside the limits of the map.

viii. Contour lines cannot merge or cross one another.

2.1.2 Methods of Contouring

There are mainly two methods of contouring:

a. Direct Method

- In this method the contours to be located are directly traced out in the field by locating and making a number of points on each contour.

- These method is very slow and tedious as a lot of time is wasted in searching points on the same contour. Compared to the other, it is very accurate and suitable for small areas where great accuracy is required.

b. Indirect Method

- In this method, the points located and surveyed are not necessary on the contour lines. Spot elevations will be taken along a series of lines laid out over the area, e.g. grid or cross section. The contours are then determined by interpolation based on the spot elevations.

- This method is less tedious, cheaper & quicker than the direct method but they are less accurate when compared to the direct method.

Interpolation of Contour

The process of spacing the contours proportionally between the plotted ground points established by indirect method is termed as interpolation of contour.

Measuring Slope from Contour

By measuring the distance between two points (ΔH) on two contour lines, whose elevations are, of course, known, the slope of the line connecting those two points can be calculated.

To lay out a highway, railway, canal or any other communication line at a constant gradient, the alignment can easily plotted on a map.

2.1.3 Uses of Contour Lines (Contour Map)

- Enable an engineer to approximately select the most economical and suitable site for an engineering work such as reservoir, dam, runways, highways etc.
- Drawing of a section/profile.
- Help in computation of quantities of earthwork.
- Determination of catchment areas
- Determination of Reservoir Capacity

2.2 Digital Terrain Model (DTM)

2.2.1 Overview

- A digital terrain model represents the relief of the earth's surface by a set of x, y, z date points where x is the horizontal x-position (e.g. x longitude, y is the horizontal y-position (e.g. latitude) and z is the altitude of the earth's surface (e.g. elevations above see level). It is collection of terrain data as a sequence of discrete (x, y, z) data points.
- The points are usually horizontally regular spaced in the form of a square grid.
- Just as there are engineering design criteria for selecting a contour interval to represent terrain for a given application, so to similar criteria are used to select point spacing so that the DTM adequately represents the terrain. These criteria depend on the potential uses for the data, accuracy requirements, the terrain character and other factors. The advantages of the regular grid layout a simplified data collection routine and ease of data access by subsequent programs.

- As the scale done for maps the grid cell size determines the resolution and degree of generalization of the DTM
- The grid cell size of DTM varies from a few meters (high resolution DTM, mostly available only for small areas) to medium resolution (20 to 100m) and low resolution DTM such as the 30 arc sec DTM with925mm grid cell size which covers the entire world
- Digital terrain model should not be confused with digital elevation model (DEM).
- DEM is only the more general expression for digital surface data but one must define the kind of surface the elevation date are for e.g. DEM of the vegetation surface are for: e.g. DEM of the vegetation surface, DEM of groundwater surface or DEM of the relief of the earth surface which is also called digital terrain model.

2.2.2 DTM Creation

There are two steps necessary for the creation of a DTM:

1. Collecting original x, y, z coordinates.
2. Interpolations of a regular grid DTM

1. Collecting original x, y, z co-ordinates the altitude data of the earth's surface can be collected from the following data source.
   I. Digitalization of contour lines from topographic maps.
   II. Stereoscopic measurements from aerial photos
   III. Stereoscopic measurements from (optical) satellite data
      - DTM of regulation surface gives already regular spaced data
   IV. Radar satellite data
      - gives already regular spaced data
   V. Laser scanning measurements
   VI. Field measurements or ground surveys
2. Interpolation of a regular grid DTM.
   1. The x, y, z co-ordinates collected from the above sources are usually irregularly spaced.
   2. The data have to be transformed into regular space grid data using different interpolation algorithms.

3. The chosen grid cell size usually depends mostly on the density of the collected irregular spaced data.
4. The quality of the DTM is mainly determined by the density and accuracy of the collected original altitude data.

2.2.3 Derivation from DTM

i. Calculation of local morphometric values for each grid cell of DTM such as slope angle and curvature.
ii. Calculation of complex morphometric values for each grid cell of the DTM such as measures of drainage basins, distance to flow lines and water sheds etc.
iii. Derivation of morphographic features: linear features such as crest lines, flow lines and edges; area features such as valley grounds summit areas subdividing of slopes etc.
iv. Visualization of the relief, computing contour lines, analytic hill shading and 3d-views.

2.2.4 Application of DTM

- ✓ DTM were first used for engineering purposes such as planning of rods, railroads, channels and water dams or for calculation of mass (soil) movements for bigger flooding areas.
- ✓ DTM is a base component of all modern geographical information systems or spatial information systems.

2.2.5 Uncertainty and Errors in DTMs

1. The accuracy of a DTM depends on the accuracy of its sources data and on the model resolution.
2. Two DTMs produced from the same data will not contain the same information if their resolution and sampling strategies are different.
3. A DTM may contain badly formed links or spot height errors which must be corrected by hand or automatically prior to use.

3 Curves

3.1 General

In the geometric design of motorways, railways, pipelines, etc., the design and setting out of curves is an important aspect of the engineer's work. The initial design is usually based on a series of straight sections whose positions are defined largely by the topography of the area. The intersections of pairs of straights are then connected by horizontal curves. In the vertical design, intersecting gradients are connected by curves in the vertical plane.

3.2 Types of curves and their use

In general curves can be listed under three main headings, as follows:

(1) Horizontal curve
 (i) Circular curves of constant radius.
 (ii) Transition curves of varying radius (spirals).
(2) Vertical curves of parabolic form.

Curves used in horizontal planes to connect two straight tangent sections are called *horizontal curves.* Two types are used: *circular arcs* and *spirals*. Both are readily laid out in the field with surveying equipment. A *simple curve* [Figure 3.1(a)] is a circular arc connecting two tangents. It is the type most often used. A *compound curve* [Figure 3.1(b)] is composed of two or more circular arcs of different radii tangent to each other, with their centers on the same side of the alignment. The combination of a short length of tangent (less than 100 m) connecting two circular arcs that have centers on the same side [Figure 3.1(c)] is called a broken-back curve.A *reverse curve* [Figure 3.1(d)] consists of two circulararcs tangent to each other, with their centers on opposite sides of the alignment.

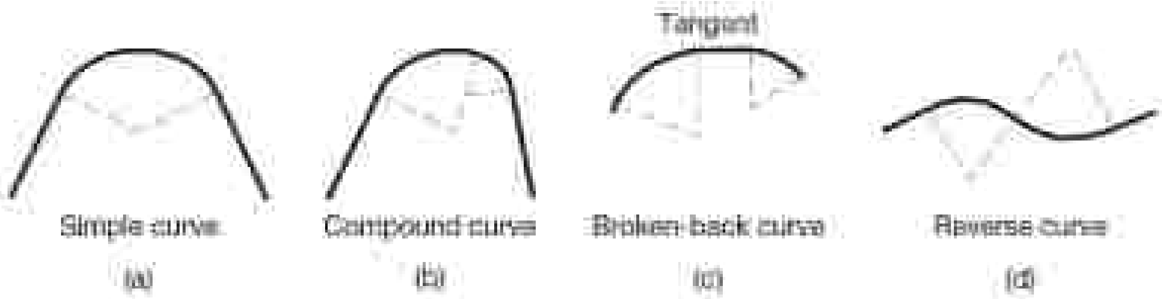


Figure 3-1 Types of Circular Curve

Compound and reverse curves are unsuitable for modern high-speed highway, rapid transit, and railroad traffic and should be avoided if possible. However, they are sometimes necessary in mountainous terrain to avoid excessive grades or very deep cuts and fills. Compound curves are often used on exit and entrance ramps of interstate highways and expressways, although easement curves are generally a better choice for these situations.

Easement curves are desirable, especially for railroads and rapid transit systems, to lessen the sudden change in curvature at the junction of a tangent and a circular curve. A *spiral* makes an excellent easement curve because its radius decreases uniformly from infinity at the tangent to that of the curve it meets. Spirals are used to connect a tangent with a circular curve, a tangent with a tangent (double spiral), and a circular curve with a circular curve. Figure 3.2 illustrates these arrangements.

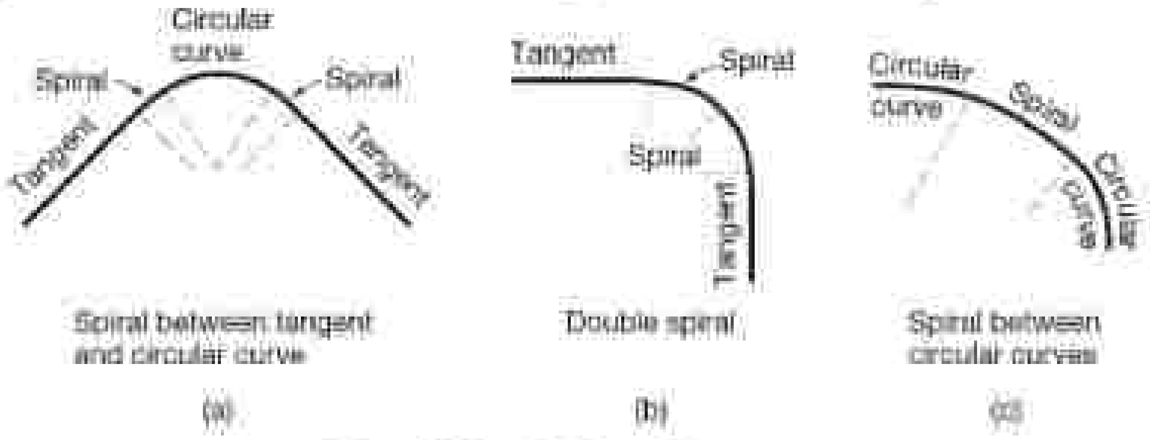


Figure 3-2 Use of spiral transition curves

The effect of centrifugal force on a vehicle passing around a curve can be balanced by *superelevation*, which raises the outer rail of a track or outer edge of a highway pavement. Correct transition into superelevation on a spiral increases uniformly with the distance from the beginning of the spiral and is in inverse proportion to the radius at any point. Properly superelevated spirals ensure smooth and safe riding with less wear on equipment. As noted, spirals are used for railroads and rapid-transit systems. This is because trains are constrained to follow the tracks, and thus a smooth, safe, and comfortable ride can only be assured with properly constructed alignments that include easement curves. On highways, spirals are less frequently used because drivers are able to overcome abrupt directional changes at circular curves by steering a spiraled path as they enter and exit the curves.

3.3 Circular Curve

A simple circular curve shown in Fig. 3.3, consists of simple arc of a circle of radius R connecting two straights AI and IB at tangent points T1 called the point of commencement (P.C.) and T2 called the point of tangency (P.T.), intersecting at I, called the point of intersection (P.I.), having a deflection angle Δ or angle of intersection ϕ. The distance E of the midpoint of the curve from I is called the external distance. The arc length from T1 to T2 is the length of curve, and the chord T1T2 is called the long chord. The distance M between the midpoints of the curve and the long chord, is called the mid-ordinate. The distance T1I which is equal to the distance IT2, is called the tangent length. Thetangent AI is called the back tangent and the tangent IB is the *forward tangent*.

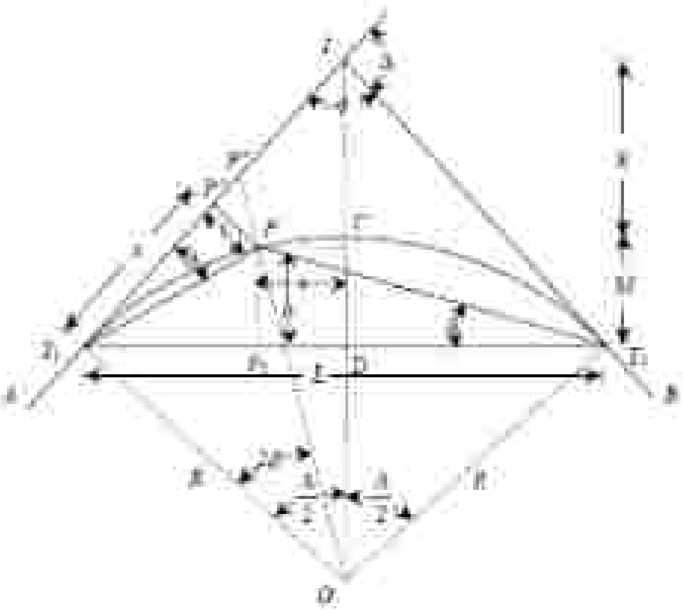


Figure 3-3 Elements of circular curve

Formula to calculate the various elements of a circular curve for use in design and setting out, are as under.

Tangent length (T) $= \boldsymbol{R\tan\frac{\Delta}{2}}$

Length of curve $(l) = \frac{\pi R\Delta}{180}$

Long chord (L) $= \boldsymbol{2R\sin\frac{\Delta}{2}}$

External distance (E) $= \boldsymbol{R(\sec\frac{\Delta}{2} - 1)}$

Mid-ordinate (M) $= \boldsymbol{R(1 - \cos\frac{\Delta}{2})}$

Chainage of T_1 = chainage of P.I. $-$ T

Chainage of T_2 = chainage of T_1 + $\boldsymbol{l}$

Degree of the circular curve

The rate of curvature of circular curves can be designated either by their *radius* (e.g., a 1500-m curve) or by their *degree of curve*. There are two different designations for degree of curve, the *arc definition* and the *chord definition*. By the arc definition, degree of curve is the central angle subtended by a circular *arc* of 100 m (see Figure 1.4). This definition is preferred for highway work. By the chord definition, degree of curve is the angle at the center of a circular arc subtended by a chord of 100 m. This definition is convenient for very gentle curves and hence is preferred for railroads. The formulas relating radius *R* and degree *D* of curves for both definitions are shown by

$$\frac{D}{360^\circ} = \frac{100}{2\pi R}$$

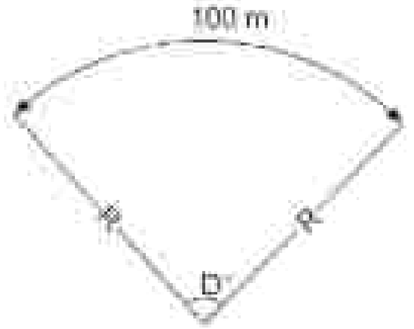


Figure 3-4 Degree of a curve using arc definition

3.4 Setting Out of Circular Curves

One approach for setting out circular curves is to locate the tanged points and intersection points that define the construction lines and then proceed with the setting out of the pegs along the curve by taking measurements from these positions.

The pegs that mark the center line of the curve can be set out from the tangent points:

1. By tape and off set
2. By angle and distance measurement
3. By angle measurement only
4. By Coordinates

1. Methods Using Tape

I. Offset from Long Chord: the method is suitable for curves of short radius.

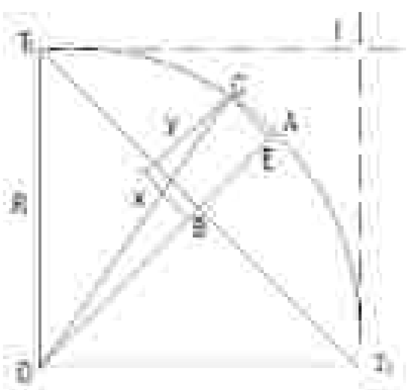


Figure 3-5 Offset from long chord

$$AB = AO - OB$$

$$= AO - \sqrt{OU^2 - UB^2}$$

$$= R - \sqrt{R^2 - (L/2)^2}$$

Draw CE parallel to TU, then

$$y = EB = EO - BO$$

$$EO^2 = CO^2 - CE^2 \rightarrow EO = \sqrt{R^2 - x^2}$$

$$y = \sqrt{R^2 - x^2} - \sqrt{R^2 - (L/2)^2}$$

II. Offset from Tangent: the method is suitable for short curve

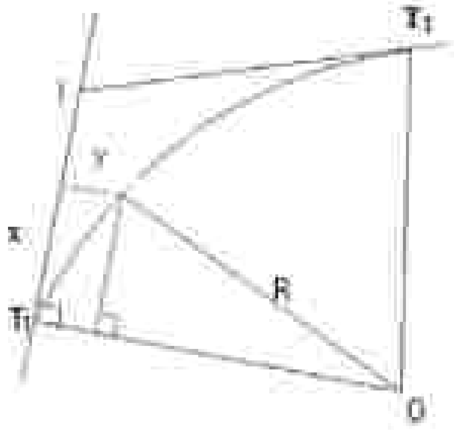


Figure 3-6 Offset from tangent

Therefore

$$x = R - \sqrt{R^2 - y^2} = R - R(1 - \frac{y^2}{R^2})^{\frac{1}{2}}$$

Expanding by using Binomial theorem,

$$x = R - R(1 - \frac{y^2}{2R^2} + \ldots)$$

$$= \frac{y^2}{2R} \text{ Approximately}$$

$$R^2 = AB^2 + BO^2 = y^2 + (R - x)^2$$

2. Setting out using one theodolite and tape by deflection angle method

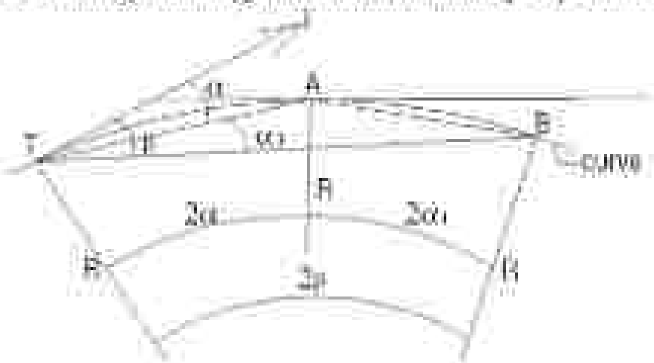


Figure 3-7 Setting out by deflection angle method

Arc TA = 2Rα (α in radians)

$$= 2R\alpha \frac{\pi}{180}$$

= chord TA (if chord not greater than (R/20))

Therefore $\alpha = \frac{TA * 180}{2\pi R}$ deg

$$\alpha = 1718.9 \frac{TA}{R} \text{ min}$$

3. Setting out using two theodolite

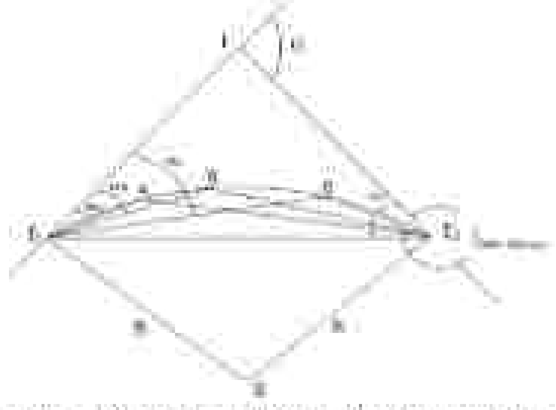


Figure 3-8 Setting out by two theodolite method

4. Setting out using Coordinate

Today, because of the availability of total station and GPS/GNSS instruments with data collectors, circular curves are often staked using the coordinate method. For this procedure, coordinates of the points on the curve to be staked must first be determined in some reference coordinate system. In Figure bellow, assume that the azimuth of the back-tangent going from *A* to *V* is known, the coordinates of the PI (point *V*) are known, and that the defining parts of the curve have been computed using the following equations. Using the tangent distance and azimuth of the back tangent, after the departure and latitude are computed, the coordinates of the point of commencement (A) calculated by

$$X_A = X_V + T \sin Az_{VA}$$
$$Y_A = Y_V + T \cos Az_{VA}$$

where Az_{VA} is the back azimuth of line *AV*.

With the coordinates of the PC known, coordinates of points on the curve can be computed using the same deflection angles and subchords used to stake out the curve by the total chord method. Deflection angles are added to the azimuth of AV to get azimuths of the chords to all stations to be set. Using the totalchord length and chord azimuth for each station, departures and latitudes arecalculated, and added to the coordinates of A (the PC) to get the station coordinates.

With coordinates known for all curve points, they can be staked with the total station occupying any convenient point whose coordinates are also known in the same system. The PC, PT, PI, or curve midpoints are points that are often used.

$$Az_{AO} = Az_{AV} + 90°$$
$$X_O = X_A - R \sin Az_{AO}$$
$$Y_O = Y_A - R \cos Az_{AO}$$
$$X_P = X_O + R \sin Az_{OP}$$
$$Y_P = Y_O + R \cos Az_{OP}$$

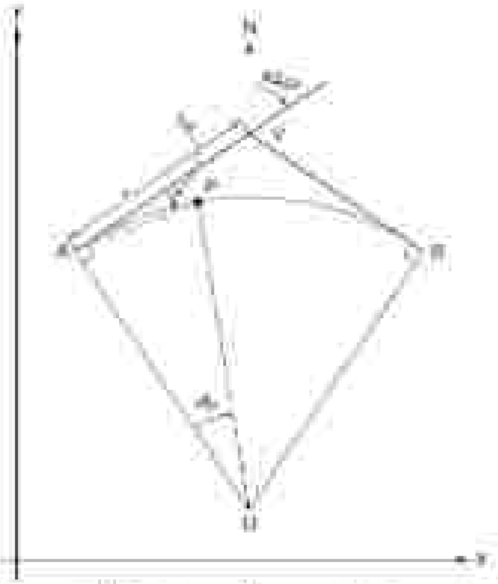


Figure 3-9 Setting out by coordinate method

To stake the curve points, the total station instrument is set on the curve's center point, a backsight taken on point *A*, and the azimuth of line *OA* indexed on the horizontal circle. To stake any point such as *P*, the azimuth of *OP* is placed on the instrument's circle and the stake placed on the line of sight at a distance *R* from the instrument.

5. Setting out with inaccessible intersection point

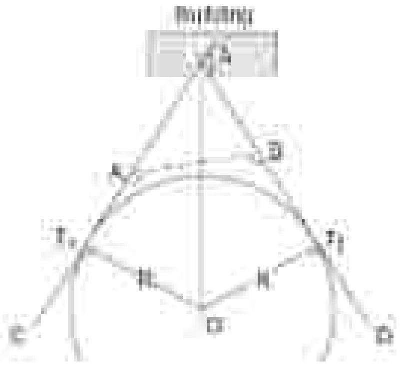


Figure 3-10 Setting out with inaccessible intersection point

3.5 Transition Curves

The *transition curve* is a curve of constantly changing radius. It used to connect a straight to a curve of radius *R*, then the commencing radius of the transition will be the same as the straight (∞), and the final radius will be that of the curve *R*.

The centrifugal force (i.e. $P=WV^2/gR$) increases as R decreases, and thus, since the radius of the straight is infinity, the centrifugal force would increase instantaneously from zero to its maximum value (assuming no change in V), as the vehicle moved from the straight to curve. Passengers in the vehicle would thus experience a lateral shock as the tangent was passed. To avoid this curve variable radius is inserted between the straight and the circular curve in order that the centrifugal force may build up in a gradual and uniform manner. This curve is called a transition curve.

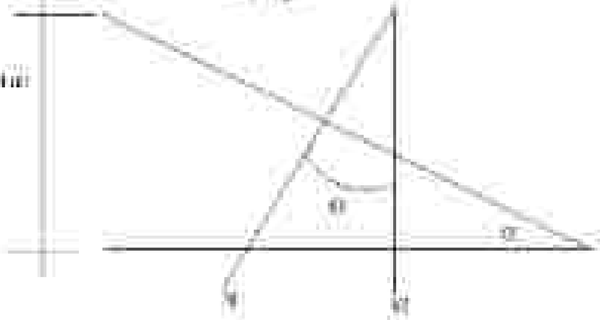


Figure 3-11 Super elevation

Derivation of Transition Curve Equations

- P increases uniformly with distance l from the beginning of the transition curve, i.e. $P \propto l$
- At the point where the radius of the transition curve = r

$P = \frac{WV^2}{gr}$ Hence $P \propto \frac{1}{r}$ for constant velocity

And so $l \propto 1/r$ since $P \propto l$

- $lr = K = LR$

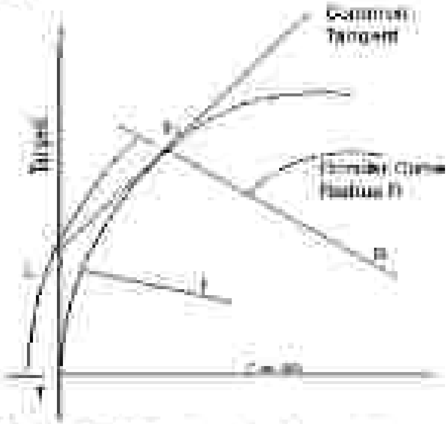


Figure 3-12 Derivation of transition curve

Length of Transition Curve

Let L = total length of the transition curve, R = radius of circular curve entered and V = uniform velocity of the vehicle.

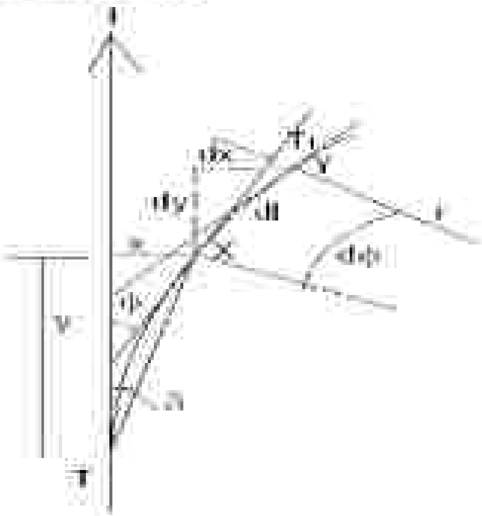
The radial acceleration at the circular curve $= f = V^2/R$

Therefore if the time taken to travel along the transition curve be t seconds,

t = L / V

Rate of change radial acceleration = $f/(L/V) = \alpha = \frac{V^2}{R} / \frac{L}{V} = \frac{V^3}{LR}$

Note: A rate of change of radial acceleration of **1/3** m/s³ is a **COMFORT LIMIT** above which side-throw will be noticed



Consider points X and Y which is near together on the transition curve, X being l from T.

$$dl = r\,d\phi = \frac{K}{l}\,d\phi \Rightarrow d\phi = \frac{l}{K}\,dl$$

On integrating, $\phi = \frac{l^2}{2K} + A$, A=0, since φ=0 when l = 0

$\phi = \frac{l^2}{2K} = \frac{l^2}{2LR} = \frac{l}{2r}$

Now we can write

$dy = dl\cos(\phi + d\phi/2)$

$dx = dl\sin(\phi + d\phi/2)$ Integrating, $x = \frac{l^3}{6K}$

and $y = l$

Relation Between δ and ϕ

$$\tan\delta = \frac{x}{y} = \frac{l^2(1-\phi^2/14+\phi^4/440)}{6RL(1-\phi^2/10+\phi^4/216)} \approx \frac{\phi}{3}$$

For small angle δ $\tan\delta = \delta$ therefore, the maximum $\alpha = \phi_1 = L/2R$

Shift

Where transition curves are introduced between the tangents and a circular curve of radius R, the circular curve is "shifted" inwards from its original position by an amount BP = S so that the curves can meet tangentially. This is equivalent to having a circular curve of radius (R+S) connecting the tangents replaced by two transition curves and a circular curve of Radius R, although the tangents are not the same, being B and T respectively.

Referring the figure below <NMO = <RTO = 90°

BM = NT_1 = Maximum offset on transition curve

Also shift = S = $BP = BM - MO = NT_1 - (PO - MO)$

$$= \frac{L^2}{6LR} - (R - R\cos\phi_1) = \frac{L^2}{6LR} - \left[R - R\left(1 - \frac{\phi_1^2}{2!} + \frac{\phi_1^4}{4!} - \dots\right)\right] = \frac{L^2}{6LR} - R\frac{\phi_1^2}{2}$$

$$= \frac{L^2}{6LR} - \frac{R}{2}\left(\frac{L^2}{2LR}\right)^2$$

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

Also $Q_1T_1 \approx PT_1 \approx R\phi_1 = \frac{RL^2}{2LR} = \frac{L}{2}$

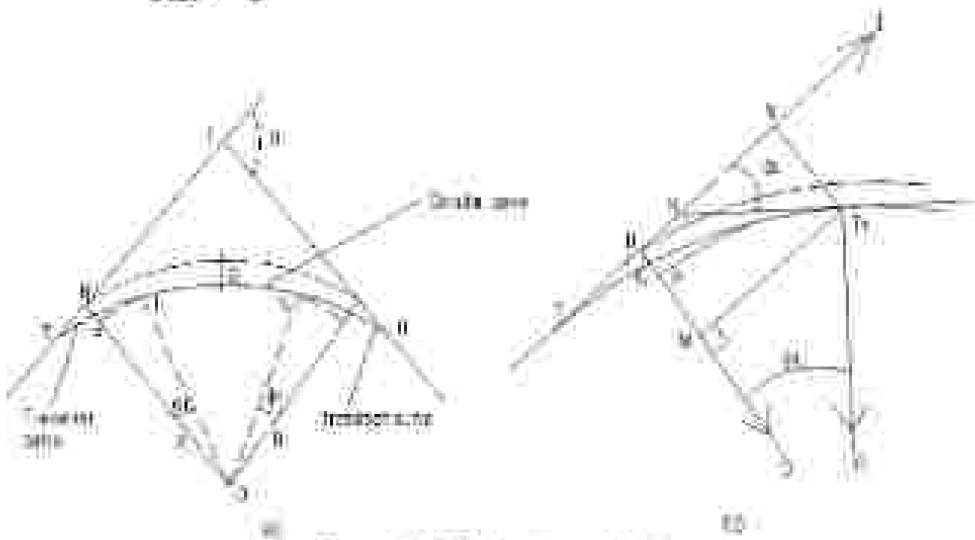


Figure 5-13 Derivation of shift

3.6 Setting Out of Transition Curves from the Tangent Point

Step 1

Calculate the shift S from the expression $S = \frac{L^2}{24R}$

Step 2

Calculate IB = $(R + S)\tan\frac{\theta}{2}$

Step 3

BT = L/2 then IT = $(R + S)\tan\frac{\theta}{2} + \frac{L}{2}$

Step 4

Either calculate offset from

$y = \frac{l^3}{6RL}$ or $y = \frac{x^3}{6RL}$

OR

Calculate the deflection angles δ from particular distances l from T using the fact that $\delta = \phi/3 = \frac{l^2}{6RL}$ rad

$= \frac{1800}{\pi}\frac{l^2}{RL}$ min $\delta_{max} = L/6R$ rad OR $= \frac{1800\,L}{\pi R}$

Step 5

Setting out the circular curve

Calculate the deflection angles for the circular curve from

$\delta_{(min)} = 1718.9\frac{c}{R}$ min

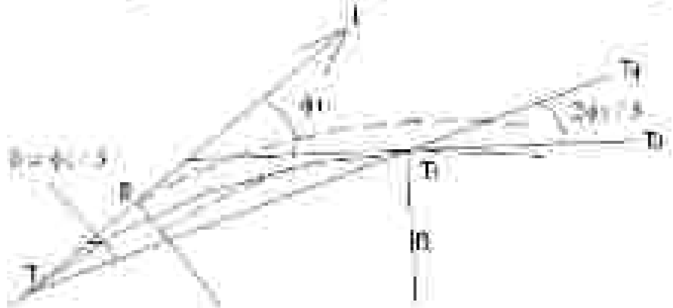
Where c is the chord length for the circular arc

The angle subtended by the circular arc T_1T_2 at the center of that arc in $(\theta - 2\phi_1)$

Length of the arc $T_1T_2 = R(\theta - 2\phi_1)\frac{\pi}{180}$

Step 6

Set up the theodolite at T_1 and sight back on T; then, transit the telescope and locate T_1T_3 by setting off an angle $2/3\ \phi_1$. Set out the circular curve by the deflection angle method, from this tangent.



Step 7

Setting the other transition curve

Point U is located from I using the relationship IU = IT. The transition is then set out from the tangent point U and tangent UI by either of method given in step 4.

3.7 Vertical Curves

Curves are needed to provide smooth transitions between straight segments (tangents) of grade lines for highways and railroads. Because these curves exist in vertical planes, they are called vertical curves. An example is illustrated in Figure below, which shows the profile view of a proposed section of highway to be constructed from A to B.A grade line consisting of three tangent sections has been designed to fit the ground profile. Two vertical curves are needed: curve *a* to join tangents 1 and 2, and curve *b* to connect tangents 2 and 3. The function of each curve is to provide a gradual change in grade from the initial (back) tangent to the grade of the second (forward) tangent. Because parabolas provide a constant rate of change of grade, they are ideal and almost always applied for vertical alignments used by vehicular traffic.

Two basic types of vertical curves exist: *crest* and *sag*. Curve *a* is a crest type, which by definition undergoes a negative change in grade; that is, the curve turns downward. Curve *b* is a sag type, in which the change in grade is positive and the curve turns upward. There are several factors that must be taken into account when designing a grade line of tangents and curves on any highway or railroad project. They include (1) providing a good fit with the existing ground profile, thereby minimizing the depths of cuts and fills, (2) balancing the volume of cut material against fill, (3) maintaining adequate drainage, (4) not exceeding maximum specified grades, and (5) meeting fixed elevations such as intersections with other roads. In addition, the curves must be designed to (a) fit the grade lines they connect,(b) have lengths sufficient to meet specifications covering a maximum rate of change of grade (which affects the comfort of vehicle occupants), and (c) provide sufficient sight distance for safe vehicle operation.

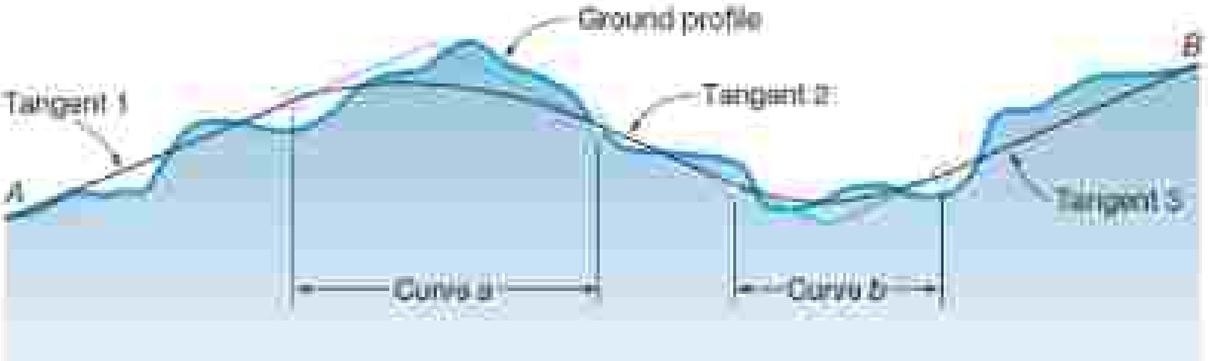


Figure 3-14 Vertical curve: summit and sag

Elevations at selected points (e.g., at 20, 30, or 40 m) along vertical parabolic curves are usually computed by the tangent offset method. It is simple, straightforward, conveniently performed with calculators and computers.

and self-checking. After the elevations of curve points have been computed, they are staked in the field to guide construction operations so the route can be built according to plan.

Curve Design

For flat gradient it is sufficiently accurate to treat the length along the tangents, the length along the curve, a chord AC as equal to length 2l so as to be in accordance with the survey information.

The basic equation for a simple parabola is:

$$Y = C.Xs^2$$

Where Y is the vertical offset from gradient to curve, Xs is the distance from the start of the curve, and C is a constant.

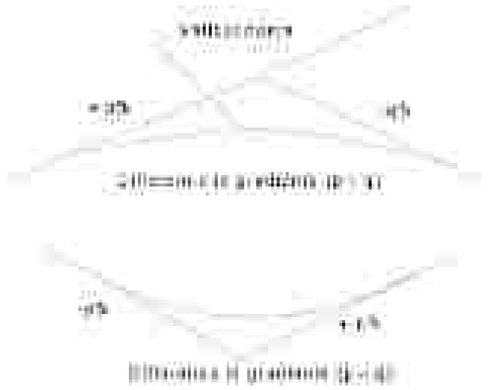


Fig. (a)

Fig. (b)

where p and q are in %

The properties of the parabola give: -

- The vertical through the intersection of the tangents B is a diameter and bisects AC.
- BD=DE. D is the vertex of the parabola.
- The offsets from the tangent AB are proportional to the square of distance from A.
- Offsets from tangent at D are proportional to the square of distance from D.

Fig. (c)

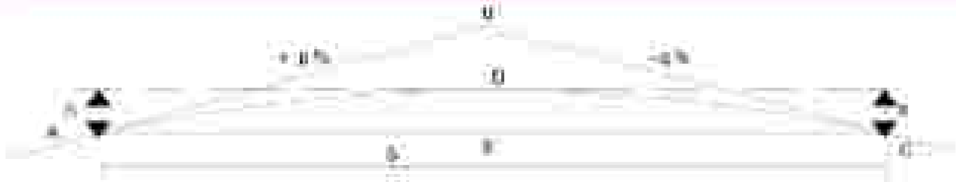


Fig. 10c

From Fig (c)

$y = a_x^2$

$y = C + a_x^2$

$y = C(x \pm \theta)^2$

$y = Kx^2$

When x=2L

Y=B'C=PL/100 + qL/100

$y = Kx^2 = K(2L)^2 = \frac{(p+q)L}{100}$

$K = \frac{p+q}{400L}$

$y = Kx^2 = \frac{(p+q)}{400L}x^2$

When x = L, Y= BD $= \frac{(p+q)L^2}{400L} = \frac{(p+q)L}{400}$

Height of B above A $= \frac{pL}{100}$

Height of C above A $= \frac{pL}{100} - \frac{qL}{100}$

The angle between AC and the horizontal

$= \frac{L}{100} \cdot \frac{(p-q)}{2L} = \frac{p-q}{200}$ radians

$H_C = \frac{pL}{100} - \frac{(p+q)}{200}L = \frac{(p-q)}{200}L$

$\Rightarrow$ [illegible]

For other case where the difference in gradients = p-q

$y = \frac{(p-q)x^2}{400L}$

Height of a point

$H_x = \frac{xp}{100} - \frac{(p-q)x^2}{400L}$

For maximum value of H_x,

$\frac{dH_x}{dx} = 0$

$\frac{p}{100} - \frac{2x(p-q)}{400L} = 0$

$x = \frac{2p}{p-q}L$

4 Photogrammetry

4.1 Introduction

Photogrammetry is the science of making measurements on photographs. Terrestrial photogrammetry deals with photographs taken from a known ground position. Aerial photogrammetry deals with those photographs taken from the air. Measurements made from photographs can be used to obtain horizontal distances between points, elevation of points, compilation of topographic and planimetric maps, preparation of profiles and cross-sections, construction of mosaics, production of orthophotos, etc. Photographs can also be interpreted for geological, agricultural and engineering investigation, forestry, ecology and environmental analysis, resource inventory, military intelligence, etc.

4.2 Types of aerial photographs

Photographs taken with the optical axis of the camera held essentially vertical or perpendicular to the ground surface are called vertical aerial photographs. These have simple geometry and hence they are preferred in photogrammetric applications. Some tilt angle (up to 5^0) is tolerated. Oblique aerial photographs are taken with the camera axis tilted intentionally. A low oblique is that photograph which bears no image of the horizon due to small tilt angle. A high oblique shows the image of the horizon with the ground image.
Most planimetric and topographic mapping, mosaic construction and orthophoto production are done by using vertical aerial photographs although high obliques are sometimes used in the preparation of small-scale planimetric maps and charts due to large ground coverage.

4.3 Scale of Aerial Photographs

The scale of a photograph is the ratio of the distance on the photograph to the corresponding distance on the ground. A vertical aerial photograph resembles a planimetric map in that it shows both the planimetric and cultural features on the ground in their relative positions. It is different from planimetric maps, however, in two respects: (1) the photograph does not contain the standard map symbols, which are essential to the map;(2) the scale of the photograph varies in different portions of the photograph due to the ground relief. Consider fig 4.1

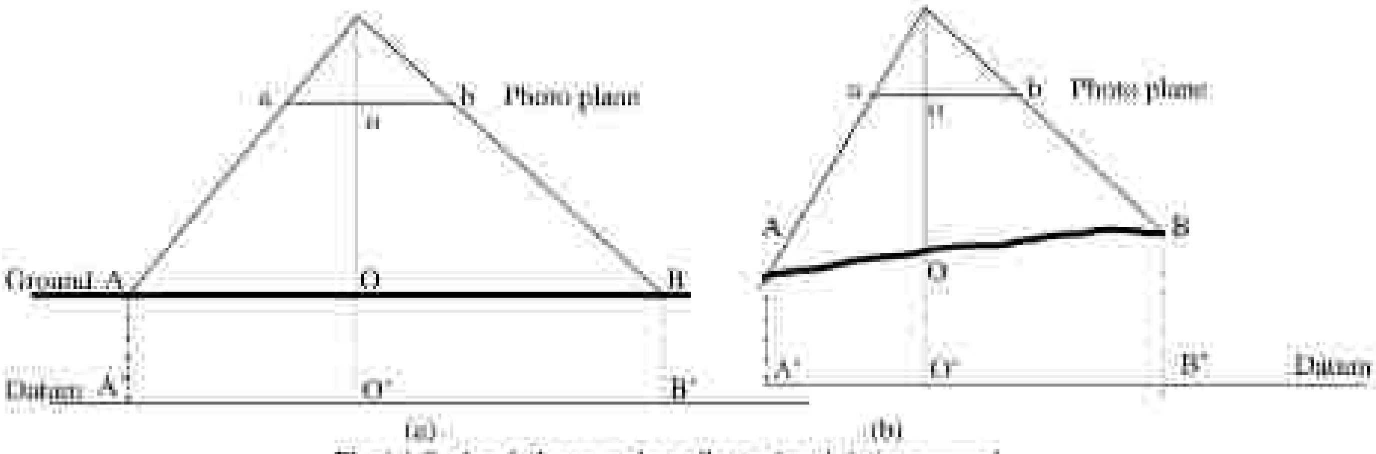


Fig 4.1 Scale of photograph on flat and undulating ground

In fig 4.1 (a) points A, O and B all lie at the same elevation. The horizontal distances AO and OB are equal to A'O' and O'B' on the reference datum. A truly vertical photograph taken with the camera at L would show the images of A, O and B at a, o and b. The ratio ao/ A'O' equals the ratio ob/O'B', and the scale of the photograph is uniform across the photograph.

In fig 4.1 (b) points A, O and B are at different elevations and the horizontal distances between A and O, and O and B are A'O' and O'B' on the datum. If a vertical photograph is taken with the camera axis at L, the points A, O and B are shown on the photograph as a, o and b. The ratio ao/ A'O' does not equal the ratio ob/O'B', and the scale of the photograph varies across the photograph due to variation in elevations of the ground.

Referring to fig 4.1(b)

$$Scale = \frac{Dis\ tan\ ce\ on\ photo}{Dis\ tan\ ce\ on\ ground} = \frac{ao}{A'O'} = \frac{ob}{O'B'}$$

$$Scale = \frac{ao}{A'O'} = \frac{f}{H - h_E}, Scale = \frac{ob}{O'B'} = \frac{f}{H - h_E}$$

$$\therefore Scale, S_E = \frac{f}{H - h_E} \qquad (4.1)$$

Where S_E is the scale of a vertical photograph for a given elevation E;

f is the focal length of the camera in inches or millimeters;

H is the flying height above datum, in feet or meters, and

h is the elevation of the point, line or area above datum, in feet or meters.

Provided that the relief is not extremely variable, the average scale of a single photograph or a set of photographs may be desired to be able to measure distances in any are of the photograph(s). It is given by

$$S_{av} = \frac{f}{H - h_{av}} \qquad (4.2)$$

in which S_{av} is the average scale of the photograph (s) and h_{av} is the average elevation of the area covered by the photograph, in ft or m, above datum.

When the scale of the photograph is not given it can be determined by measuring a distance between known landmarks on the photograph and comparing to the corresponding distance on the ground, as for instance the distance between two road intersections.

The scale of he photograph can also be determined by comparing a distance measured on the photograph with the corresponding distance measured on a map with a known scale. The photo scale is then found from the following relation.

$$\frac{photo\ scale}{Map\ scale} = \frac{photo\ dis\ tan\ ce}{Map\ dis\ tan\ ce} \qquad (4.3)$$

Solved Examples:

1.A camera with a focal length of 152.35mm and a picture size 230* 230mm is used to photograph an area from an altitude of 2400m above sea level. The average ground elevation is 420m above sea level. What is the average scale of the photography?

Solution

1) From eq. (4.2)

$$S_{avg} = \frac{f}{H - h_{avg}} = \frac{152.35\ mm}{2400\ m - 420\ m} = \frac{1mm}{13\ m}$$

$\Rightarrow S_{avg} = 1 : 13{,}000$

2) The actual ground area covered by a single photograph

A= 230*13000*230*13000mm^2

A= 8,940,100m^2

2. Determine the amount of ground area covered by a single photograph of example 1.

Solution

1mm on photo= 13m on ground

1mm^2 on photo= 169m^2 of area on ground

230*230mm^2 of photo area= 230*mm^2*169m^2/mm^2

=8,940,100m^2 of ground area is covered by a single photograph.

3. Two points lying at an elevation of 268.5m appear on the photograph taken in example1 above. The distance between these points scales 22.78mm on the photograph. What is the ground distance?

Solution

At the given elevation of 268.5m,

$$S = \frac{f}{H - h} = \frac{152.35\ mm}{2400\ m - 268.5m} = 1 : 13{,}999$$

and also
$$S = \frac{Distance\ on\ photo}{Distance\ on\ ground} \Rightarrow \frac{22.78}{G.D} = \frac{1}{13{,}09}$$

$$\therefore Ground\ distance = 138.71\ m$$

Exercise: The distance between two road intersections on an aerial photograph is 6.50cm, and the same distance measures 2.30cm on a map with a scale of 1:50,000. What is the approximate scale of the photograph?

4.4 Relief Displacement

Relief displacement is the displacement of the image of a ground point on the photograph from the position the image would have if the point were on the datum, or it is the condition in which a point on the ground is displaced from its true map position.
In fig 4.2 points B and C are located at elevations of h_B and h_C above datum. On a vertical photograph their images appear at the same point. The datum positions B and C would have images at b and c on the photograph. Point p is the principal focus of the photograph formed by joining opposite fiducial marks at the corners or the edges of the photograph. Because of the elevations of B and C above datum, both b and c have been displace outward along the radial lines pb and pc respectively.
The relief displacement d_r for a point is give by

$$d_r = \frac{rh}{H} \qquad (4.4)$$

where r is the radial distance from the principal point to the image point (vb);
h is the elevation of the point above datum; and
H is the flying height above datum, in same units as h.

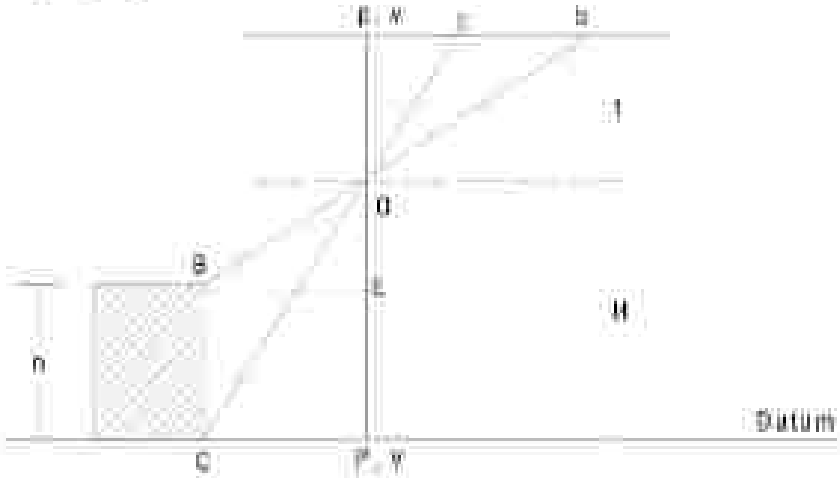


Fig 4.2 Relief displacement

From eq 4.4 it can be see that the relief displacement of a point depends on the position of the point on the photograph. It increases outward toward the edge of the photograph. At the principal point it is zero. Also the relief displacement increases as the elevation of the point above datum increases and decreases with the flying height.
Relief displacement can be used to calculate the height of objects in certain situations where the relief displacement can be measured from the photograph, for instance, for features such as flagpoles, utility poles, some transmission towers, corners of buildings, water tanks, etc. where both the top and bottom of the object can be observed on the photograph and the top occurs vertically above the bottom.
Relief displacement can also be used to determine the actual length of lines, angles between lines, areas and volumes from photographs after reducing all points of the photograph to a common datum.

Solved Example:
On an aerial photograph taken at an altitude of 1500m, the relief displacement of the of the image of a flagpole was measured as 1.6cm. The distance from the center of the photograph to the image of the top of the flagpole is 11.0cm. If the base of the flagpole is at an elevation of 200. 0m A.O.D, what is the height of the flagpole?

Solution:

From eq.44

$$d_r = rh/H \Rightarrow 1.6cm = 11.0cm * h / 1500$$
$$h = 218.0m$$

The height of the flag pole above ground is therefore 218.0-200.0=18.0m

4.5 Stereoscopy

Stereoscopy is the ability of the individual to perceive the object space in three dimensions through using both eyes. Each human eye represents a single camera, and monocular viewing results in flat perspective and the person's ability to perceive depth is hampered. Binocular viewing allows the person to view the object from two different locations due to the separation between the eyes. The pair of camera stations when taking overlapping photographs is therefore similar to viewing with both eyes.

Viewing with one eye fixes one direction from the eye to the object only, which is in sufficient for fixing the object's distance from the viewer. On the other hand viewing with two eyes fixes two directions from both eyes, hence giving the location of the object at their intersection. The two directions from both eyes form a convergence angle (parallactic angle) ϕ. The closer the point to the eyes, the larger the convergence angle between the two directions (refer fig 4.3 a). Since $\phi_1 > \phi_2$, the observer perceives P_1 as closer than P_2, and h is a function of $\phi_1 - \phi_2$

Fig 4.3 Stereoscopy

In fig 4.3(b) an idealized tower is photographed from two camera stations. If it were possible to see the left photograph with the left eye and the right photograph with the right eye only, the observation will perceive the tower in three dimensions as shown. The lines joining the eyes and the two images of the top of the tower will intersect at an angle ϕ_1, while those passing through the images of the tower base make an angle ϕ_2. Since $\phi_1 > \phi_2$, the top of the tower will appear closer to the viewer than its base and the tower will then be perceived in three dimensions.

Stereoscopic Viewing of Aerial Photographs

When aerial photographs are viewed stereoscopically, they produce the three dimensional image of the terrain in the photograph. The three-dimensional view helps in the study of the terrain as it relates to engineering projects and also provides the basis for topographic mapping by photogrammetric techniques. There are three requirements for stereoscopic viewing: (1) the two aerial photographs must provided two views of the terrain taken from two different camera positions;(2) the two aerial photographs must be oriented properly for viewing; and (3) the viewer must have normal binocular vision.

1. Photograph overlap: When aerial photographs for project areas are to be taken, flight lines are aid out on a flight map with a spacing that will cause photographs to cover an overlapping strip of ground both laterally and consecutively. The lateral overlap between photographs amounts to about 25% of the width of the photograph of each side. The actual spacing on the flight map may be determined from the scale at which the photographs are to be taken (eq.4.3). The photographic distance 75% of the width of the photograph if a 25% overlap is to be maintained between strips (fig 4.4)

Fig. 4.4 Photograph overlap

Along the flight line each photograph overlaps an area that overlaps the area taken by the previous photograph by about 60%. Therefore, the time interval between exposures is selected to achieve so during the flight. The large overlap between successive photographs helps to:

(1) Provide coverage of the entire ground area from two view points, such coverage being important for stereoscopic viewing; (2) allow only the central portion of the photograph to be used for mosaic construction, thus eliminating to a great extent errors due to relief displacement.

2. Stereoscopic Viewing Devices: The primary purpose of a stereoscopic viewing device is to ensure that the right eye sees only the photograph on the right and the left eye sees only the photograph on the left. Two common devices are used: the pocket stereoscope and the mirror stereoscope.

3. Orientation of Photographs for Stereoscopic Viewing: The photographs being viewed stereoscopically are oriented in the same relative position as the camera existed in the field at the time of photography. The principal point of each photograph is located by joining opposite fiducial marks at the four corners of each photograph, and the principal point on the right-hand photograph is identified and marked on the left-hand photograph and the principal point of the left-hand photograph identified and marked on the right-hand photograph. The photographs are then overlapped along the flight line with the overlapping images superimposed. Then the two overlapping images are separated along the flight line with the overlap inwards until a satisfactory fusion is obtained through the stereoscope. Vision in both eyes is necessary so that the different images of the terrain contained in the separate photographs can be transferred to the brain by the two eyes of the viewer. The stereo model of the terrain is then perceived by the brain based on the slight differences in the two views of the terrain recorded on the photographs.

4.6 Parallax

The algebraic difference of the distances of two images of a ground point from their perspective principal point, measured parallel to the air base is called parallax.

Referring to fig 4.5 below, a point A at an elevation of h above datum appears at a and a' on two consecutive exposures. It can be seen from the figure that the image of A has moved through a total algebraic distance of x-x' on the principal plane. This apparent movement of the image of an object between successive exposures is called parallax of the object.

The parallax p= x-x' where x and x' are the coordinates on the left and right photographs respectively, with respect to the base line. For convenience the position of the image a' is shown on the left photograph. From the figure, triangles $L_1a''a$ and AL_1L_2 are similar. Hence,

$$\frac{f}{H-h}=\frac{p}{B} \Rightarrow p=\frac{fB}{H-h} \qquad (4.5)$$

Where p= parallax of the object (A), mm

f= the focal length of the lens of the camera, mm

B= the distance between the two camera stations (known as the air base), m

H= the flying height above datum, m

h=elevation of the object above datum, m.

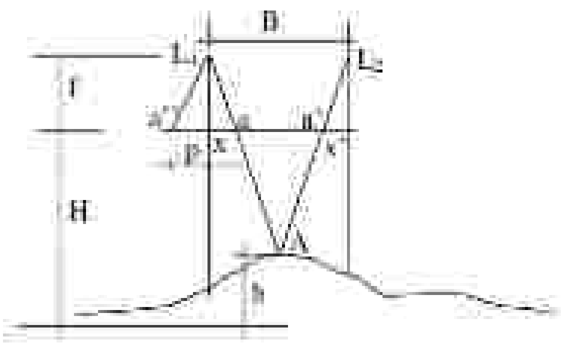


Fig 4.5 Parallax

We know $s_{avg} = \frac{f}{H - h_{avg}} = \frac{b}{B} \Rightarrow B = \frac{b(H - h_{avg})}{f}$

Introducing a new quantity $H_a = H - h_{avg}$

$$B = \frac{bH_a}{f}$$

Substituting for B in equation 4.5

$$p = \frac{bH_a}{H - h} \qquad (4.6)$$

Where b= the average distance between the principal points of the overlapping photographs (also called photo base), mm

H_a= the flying height above mean terrain of the area, m.

Parallax is a direct indication of elevation and can be measured on a pair of overlapping aerial photographs by means of a parallax bar.

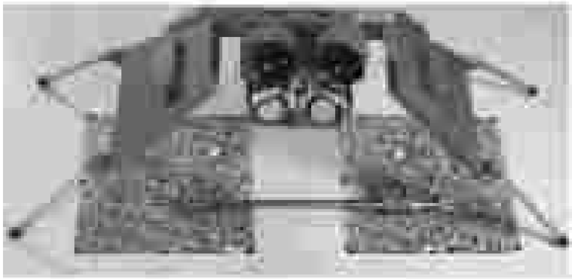


Fig 4.6 Parallax bar

As shown in fig 4.6 above, two measuring marks are located on the bottom surfaces of two pieces of glass attached to the bar. The left mark is fixed to the bar while the right mark is moved to the left or to the right by turning the micrometer screw. A movement to the left is an increase in parallax whereas a movement to the right is a decrease. If the bar is placed under a mirror stereoscope, the two measuring marks will appear as a single mark. This is called the floating mark. By turning the micrometer screw one way or the other, this single mark appears to move up or down according to the increase or decrease in the parallax of the marks.

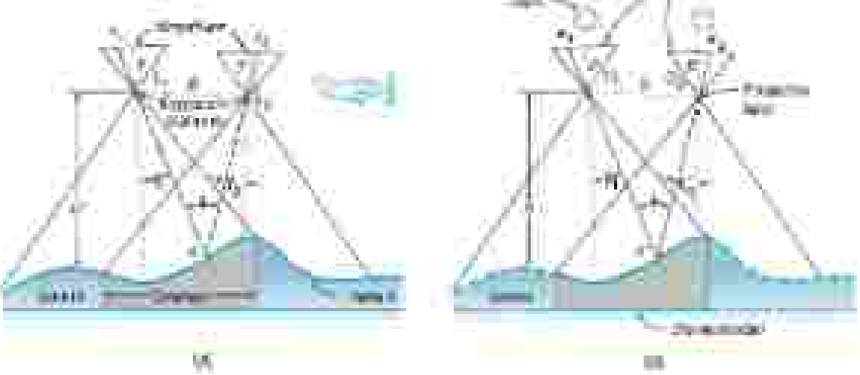


Fig 4.7 Adjusting photographs for stereoscopic viewing

Suppose that it is required to measure the difference in parallax between points a and b shown on the stereo pairs in fig. 4.7b. The photographs are first superimposed as shown in fig 4.7a and then separated by a suitable distance under the stereoscope until satisfactory fusion is obtained (fig 4.7b). The parallax bar is moved over the photographs to point a, and the micrometer screw is turned until the floating mark appears to touch the stereoscopic image at point a. The micrometer is read, say as 12.10mm. Then the bar is moved to point b, the floating mark is again brought to the surface of the stereoscopic image at b. The micrometer is read as, say, 9.65mm. The difference in parallax, dp_{ab} from a to b is thus 9.65-12.10=-2.45mm. In this manner, the difference in parallax between any two points, such as the top and bottom of a tree or a building, or between the top and bottom of a mountain can be measured. The distance o-o is the average photo base b.

All parallax measurements are made parallel to the base line.

If the parallax of point a is given as p_a , the parallax of b will be $P_b=p_a+dp_{ab}$

If only elevation difference between two points, for which no parallax is given, is required, it can be estimated from

$$dh = \frac{dp\,H}{dp + b} \qquad (4.7)$$

in which dh is the elevation difference(m), dp and b are in mm, H_c is the flying height above the ground(m).

Coordinates in the ground system can also be obtained in terms of parallax.

From equation 4.1

$$S_x = \frac{f}{H - h}$$

Substitution in to equation 4.5 gives

$$p = S_x B \Rightarrow S_x = \frac{p}{B} \qquad (4.8)$$

in which p is the parallax of the point in question and B is the air base. If the coordinate in the terrain is established with the X-axis parallel to the photographic axis of flight x-axis, then

$$X = \frac{x}{S_x} = x\frac{B}{p}\ \text{and}$$
$$Y = \frac{y}{S_y} = y\frac{B}{p} \qquad (4.9)$$

In which X and Y are the ground coordinates in the specific system described above and x and y are the coordinates of the image point in the left photograph with respect to the axis of flight coordinate system. On the basis of equation 4.9, the distance between two points could be determined if their elevation are known, or at least the elevation of one point is known in which case the measured parallax is used together with the image coordinates from equation 4.8. Then, the distance between the two points in given by

$$D = \sqrt{(X_a - X_b)^2 + (Y_a - Y_b)^2} \qquad (4.10)$$

Solved Examples

1. A rectangular area 130km×120km is to be mapped from aerial photographs taken to a scale of 1/20000. The focal length of the lens of the camera to be used is 152mm and each print is to be 230mm square. Provision is to be made for a 60% overlap between successive exposures and a 25% lateral overlap. Find (a) the average height above ground at which the plane must operate; (b) the time interval between exposures in any one strip if the operating speed of the plane is 200km/h; and (c) the minimum number of photographs required.

Solution

a) From equation 4.2.

$$S_{av} = \frac{f}{H - h_{av}}, \qquad but \; h_{av} = 0 \; at \; ground \; level$$

$$Hence \; , H = \frac{f}{S_{av}} = \frac{152 \; mm}{1/20000} = 152 * 20000 * 10^{-3} = 3040 \; m$$

b) Let the flight line be parallel to the 130km length. Since there is 60% overlap between successive exposures, the effective length of each photograph is 40% of 230mm.

=0.4*230=92mm

The ground distance covered by this photo length is

92mm*20000*10^{-3}=1840m.

Number of photograph per strip $= \frac{130,000}{1840} = 70.65 \cong 71 \; photo$

The operating speed of the plane is 200km/h. To cover the length of 130km, the plane needs 130/200=0.65 hour.

Since the exposures are at regular intervals

Time interval between exposures$= \frac{0.65}{70.65} hour = 33.12 \; sec$

c) First find the number of strips required. The width of the area to be photographed is 120km. A 25% lateral overlap results in an effective photo distance of 0.75*230=172.5mm. The ground distance covered by this width is 172.5*20000*10^{-3}=3450m.

$$\text{Number of strips} = \frac{120,000}{3,450} \cong 35 \; strips$$

Minimum number of photograph required =71*35=2485

2. On the overlap of a pair of vertical aerial photographs taken at a height of 2500m AOD with a 152mm focal length camera are shown two points A and B. Point B is a point on a pass through a range of hills, while point A is the center of a bridge in a valley. In order to estimate the amount of rise between these points, parallax measurements were made as follows: Point A: mean reading 5.90mm

Point B: mean reading 11.43mm

The mean level of the valley containing the principal points of the two photographs is 82.00m AOD, while a BM on the bridge near A was 74.55m AOD. If the respective photo bases are 89.1mm and

91.4mm, calculate the height of B above A.

Solution

From equations 4.5 and 4.6

$$p = \frac{fB}{H-h} = \frac{bH_m}{H-h}$$

H_m= 2500-82.0=2418 m above mean terrain.

Since the elevation of A can be sufficiently estimated to be 74.55m AOD,

$$p_a = \frac{bH_m}{H-h_a} \qquad b = \frac{b_1 + b_2}{2} = \frac{89.1 + 91.4}{2} = 90.25\ mm$$

$$\therefore p_A = \frac{90.25 * 2418}{2500 - 74.55} = 89.97\ mm$$

From the parallax bar readings, the difference in parallax is found as

$dp_{ba}=p_b-p_a$= 11.43-5.90=5.53mm

$\therefore p_b=p_a+dp_{ba}$=89.97+5.53=95.50mm

$$\Rightarrow p_b = \frac{bH_m}{H-h_b} \Rightarrow h_b = H - \frac{bH_m}{p_b} = 2500 - \frac{90.25 * 2418}{95.50} = 214.93\ mAOD$$

Therefore, the height of B above A is 140.40m

3. Given the photo coordinates of A and B as A (40mm, 75mm) and B (-10mm, 80mm) of the left photograph of example 2 above, determine the ground distance between these points.

Solution

From equation 4.2

$$S_m = \frac{f}{H-h_m} = \frac{152\ mm}{2500 - 82.0} = 1mm : 15.908\ m$$

The air base B= b/S_m= 1435.69m.

Then from equation 4.9

$$X_a = x\frac{B}{p_a} = 40\ \frac{1435.69}{89.97} = 638.30\ m$$

$$Y_a = y\frac{B}{p_a} = 75\ \frac{1435.69}{89.97} = 1196.81\ m$$

$$X_b = x\frac{B}{p_b} = -10\ \frac{1435.69}{95.50} = -150.33\ m$$

$$Y_b = y\frac{B}{p_b} = 80\ \frac{1435.69}{95.50} = 1202.67\ m$$

$$\therefore D = \sqrt{(X_a - X_b)^2 + (Y_a - Y_b)^2} = 788\ m$$

4.7 Use and Products of Aerial photographs

Products of aerial photographs

A variety of products are derived from aerial photographs when they are utilized for various operations. Some of thee products are listed below:

1. Paper- print copies of aerial photographs themselves, which may be used for general planning and for field operations.
2. Mosaics, which are composed of segments of photographs assembled to give the appearance of a continuous picture of the terrain. Since each photograph is a perspective, the mosaic contains distortions and should not be considered as accurate map.
3. Orthophotos specially prepared photographic representations of the terrain without distortions (relief and tilt) which can be used as a plan metric map. In some cases, contour lines are superimposed on an orthophoto, resulting in orthophoto maps, which are the equivalent of topographic maps.
4. Planimetric maps, photogrammetrically prepared, contain only the horizontal positions information of terrain features.
5. Topographic maps, which show both the planimetric detail and contour lines, compiled photogrammetrically by a stereo plotter.
6. Digital data, to produce a system of ground coordinates of a limited number of control points or a dense network representing the total terrain surface- digital terrain model (DTM)- in the form of the XY- grid system.

Mosaics

A mosaic is an assembly of overlapping aerial photographs to form one continuous picture of the terrain. In an uncontrolled mosaic, overlapping photographs are placed down consecutively and stapled to a board. No horizontal control is used during the assembly. All but the central portion of each photograph is trimmed away, leaving a small amount of overlap. Use of the central portion of each photograph decreases distortions due to relief and tilt displacements. A controlled mosaic is constructed to give both high pictorial quality and good accuracy. This is horizontally controlled by picture points whose horizontal positions on the ground are established by ground surveys. Each control point has to be identified on the photographic segment to be used. The accuracy of such a mosaic depends on minimizing the effects of relief and tilt displacements. As for relief, the smaller the area around the center of each photograph to be used in the mosaic, the less the relief displacement due to lesser value of r. Therefore, long focal length higher – altitude flying and increased side and end overlaps are preferred. With regard to tilt the regular aerial photographs are usually replaced by rectified prints produced by rectifiers. Rectificación produces an equivalent truly vertical photograph from the same exposure station. The rectification process also allows for bringing all rectified prints to the same scale. Once the assembly is finished, the mosaic is then photographed with a large copy camera and reproductions are made at the desired scale.

Orthophotos

An orthophoto is an improvement over a mosaic where the images on the aerial photograph are manipulated optically mechanically or electronically to remove the perspective aspect and the tilt displacement. The resulting photograph is an orthophoto. A series of orthophotos are placed together to form orthomosaics to cover a large area.

Con tour lines can also be plotted during the stereo plotting process of an orthophoto and these can be combined with the imagery of the area covered. The result is an orthophoto map.

Topographic Information

Topographic information from aerial photographs is obtained by measuring parallax.

$$p = \frac{f * B}{H - h}$$

In addition to determining vertical information on a point to basis, it is possible to plot contour lines, in perspective, using a combination mirror stereoscope and parallax bar. If f, B and H are known in the above formula, the parallax value p for each contour h (i.e. the plane at elevation h in the terrain) can be calculated. The corresponding reading on the parallax bar is then set, fixing the floating mark in space where the floating mark remains in contact with the perceived terrain surface, this motion would describe a contour line. A drawing pencil attached to the assembly would draw such a contour line. These contours are not orthographic since they are drawn in relation to one of the photographs, which is perspective. This principle is applied in the case of large-scale topographic mapping from aerial photographs using stereo plotter. Contour lines produced photogrammetrically are superior to those produced by ground methods due to the avoidance of interpolation between spot heights in the former case.

4.8 Applications of Aerial photographs

The most extensive use of aerial photographs is in topographic mapping and orthophotographs. Other application includes.

- Design, location construction and maintenance of modern highways at all phases, eg., Earthwork quantities are calculated from photogrammetric models, pavement condition and possible erosion of embankment can be interpreted from aerial photographs.
- Dam settlement, wave action structural deformation and channel sedimentation can be analyzed from photographic products.
- In hydrology, analysis of slopes, ground coverage, watershed areas and snow depth can be determined from photographs these are all important in run off estimation.
- In agriculture, crop inventory and crop diseases analyses are performed by using aerial photographs. Colored prints are prepared to distinguish among diseases.
- In land surveying photographs are used for identification and location of boundary lines and corners, and determination of soil and vegetal covers.

5 Introduction to Remote Sensing, GPS and GIS

5.1 Introduction to Remote Sensing

Introduction

In principle there are, there are two main categories of spatial data acquisition.

- **Ground based methods** such as making field observation, taking in situ measurements and performing land surveying. Using ground based methods you operate in the real world environment.

Figure 5-1 Ground based measurements

- **Remote sensing methods,** which are based on the use of image data acquired by a sensor such as aerial camera, scanner or radar. Taking a remote sensing approach means that information is derived from the image data, which form a limited representation of the real world.

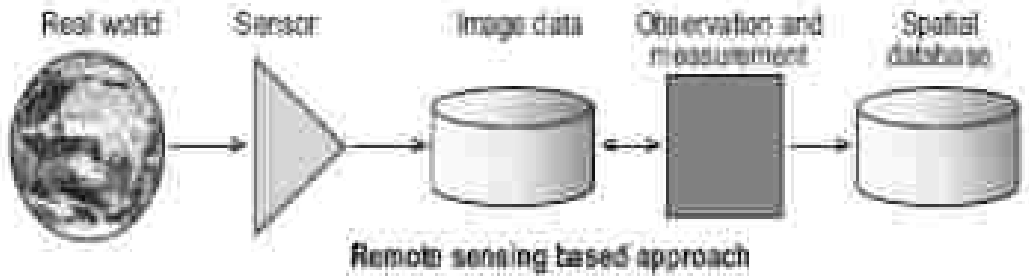


Figure 5-2 Remote Sensing based measurements

Definitions of remote sensing:

So, what exactly is **remote sensing?** For the purposes of this tutorial, we will use the following definition:

Remote Sensing is the science (and to some extent, art) of acquiring information about the Earth's surface without actually being in contact with itIn the present context, the definition of remote sensing is restricted to mean the process of acquiring information about any object without physically contacting it in any way.

regardless of whether the observer is immediately adjacent to the object or millions of miles away. Human eye is perhaps the most familiar example of a remote sensing system. In fact, sight, smell and hearing are all elementary forms of remote sensing.However, the term remote sensing is restricted to methods that employ electromagnetic energy (such as light, heat, and microwave) as a means of detecting and measuring target characteristics. Aircraft and satellites are the common platforms used for remote sensing. Collection of data is usually carried out by highly sophisticated sensors (i.e. camera, multispectral scanner, radar etc.). The information carrier or communication link is the electromagnetic energy. Remote sensing data basically consists of wavelength intensity information by collecting the electromagnetic radiation leaving the object at the specific wavelength and measuring its intensity. Photo interpretation can at best be considered as the primitive form of remote sensing. Most of the modern remote sensing methods make use of the reflected infrared bands, thermal infrared bands and microwave portion of the electromagnetic spectrum.

Principles of Electromagnetic radiation

In much of remote sensing, **the process** involves an interaction between incident radiation and the targets of interest. This is exemplified by the use of imaging systems where the following seven elements are involved. Note, however that remote sensing also involves the sensing of emitted energy and the use of non-imaging sensors

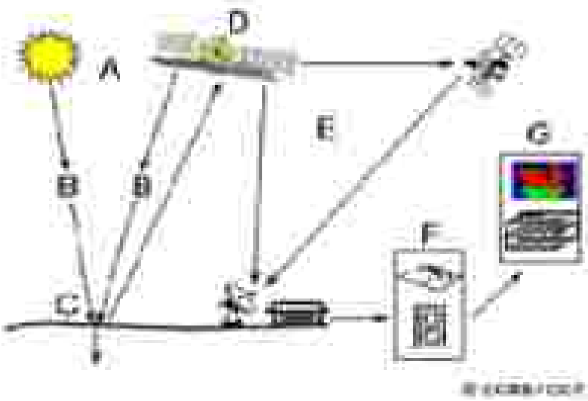


Figure 5-3 Ideal Remote Sensing Process

1. Energy Source or Illumination (A) - the first requirement for remote sensing is to have an energy source, which illuminates or provides electromagnetic energy to the target of interest.

2. Radiation and the Atmosphere (B) - as the energy travels from its source to the target, it will come in contact with and interact with the atmosphere it passes through. This interaction may take place a second time as the energy travels from the target to the sensor.

3. Interaction with the Target (C) - once the energy makes its way to the target through the atmosphere, it interacts with the target depending on the properties of both the target and the radiation.

4. Recording of Energy by the Sensor (D) - after the energy has been scattered by, or emitted from the target, we require a sensor (remote - not in contact with the target) to collect and record the electromagnetic radiation.

5. Transmission, Reception, and Processing (E) - the energy recorded by the sensor has to be transmitted, often in electronic form, to a receiving and processing station where the data are processed into an image (hardcopy and/or digital).

6. Interpretation and Analysis (F) - the processed image is interpreted, visually and/or digitally or electronically, to extract information about the target, which was illuminated.

7. Application (G) - the final element of the remote sensing process is achieved when we apply the information we have been able to extract from the imagery about the target in order to better understand it, reveal some new information, or assist in solving a particular problem. These seven elements comprise the remote sensing process from beginning to end. We will be covering all of these in sequential order throughout the five sub-sections of this tutorial, building upon the information learned as we go.

Electromagnetic radiation

As was noted, the first requirement for remote sensing is to have an **energy source to illuminate the target** (unless the target is emitting the sensed energy). This energy is in the form of electromagnetic radiation.

All electromagnetic radiation has fundamental properties and behaves in predictable ways according to the basics of wave theory. **Electromagnetic radiation** consists of an electrical field (E) which varies in magnitude in a direction perpendicular to the direction in which the radiation is traveling, and a magnetic field (M) oriented at right angles to the electrical field. Both these fields travel at the speed of light (c).

Two characteristics of electromagnetic radiation are particularly important for understanding remote sensing. These are the **wavelength and frequency.**

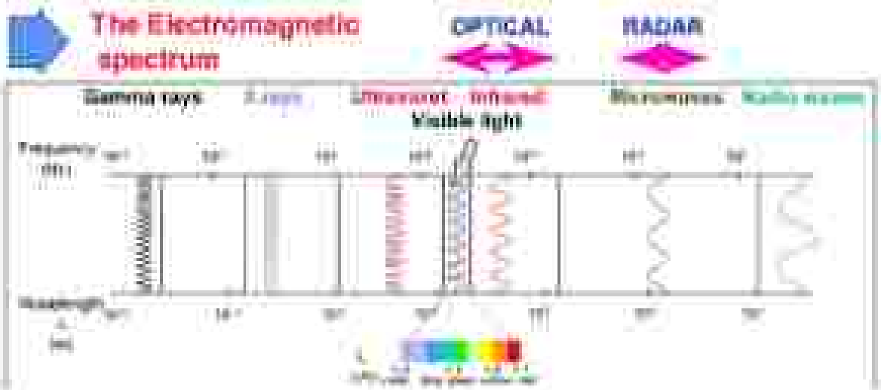
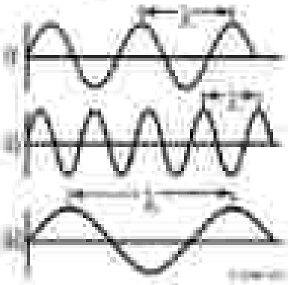


Figure 5-4 Electromagnetic Spectrum

The wavelength is the length of one wave cycle, which can be measured as the distance between successive wave crests. Wavelength is usually represented by the Greek letter lambda (λ). Wavelength is measured in meters (m) or some factor of meters such as **nanometers** (nm, 10^{-9} meters), **micrometers** (mm, 10^{-6} meters) (mm, 10^{-3} metres) or centimeters (cm, 10^{-2} metres). Frequency refers to the number of cycles of a wave passing a fixed point per unit of time. Frequency is normally measured in **hertz** (Hz), equivalent to one cycle per second, and various multiples of hertz. Wavelength and frequency are related by the following formula:



$$c = \lambda v$$

where

λ = wavelength (m)
v = frequency (cycles per second, Hz)
c = speed of light (3×10^8 m/s)

Figure 5-5 Three waves

Therefore, the two are inversely related to each other. The dynamics here are that, shorter the wavelength, the higher the frequency and the longer the wavelength, the lower the frequency. Understanding the characteristics of electromagnetic radiation in terms of their wavelength and frequency is crucial to understanding the information to be extracted from remote sensing data. The following subsection will be examining the way in which we categorize electromagnetic radiation for just that purpose.
Several regions of the electromagnetic spectrum, which are useful for remote sensing For most purposes, the **ultraviolet or UV** portion of the spectrum has the shortest wavelengths, which are practical for remote sensing. This radiation is just beyond the violet portion of the visible wavelengths, hence its name. Some Earth surface materials, primarily rocks and minerals, fluoresce or emit visible light when illuminated by UV radiation.
The light, which our eyes - our remote-sensors" - can detect, is part of the **visible spectrum**. It is important to recognize how small the visible portion is relative to the rest of the spectrum. There is a lot of radiation around us, which is "invisible" to our eyes, but can be detected by other remote sensing instruments and used to our advantage. The visible wavelengths cover a range from approximately 0.4 to 0.7 μm. The longest visible wavelength is red and the shortest is violet. Common wavelengths of what we perceive as particular colours from the visible portion of the spectrum are listed below. It is important to note that this is the only portion of the spectrum we can associate with the concept of **colours**.
Violet: 0.4 - 0.446 μm
Blue: 0.446 - 0.500 μm
Green: 0.500 - 0.578 μm
Yellow: 0.578 - 0.592 μm
Orange: 0.592 - 0.620 μm
Red: 0.620 - 0.7 μm
Blue, **green**, and **red** are the **primary colours** or wavelengths of the visible spectrum. They are defined as such because no single primary colour can be created from the other two, but all other colours can be formed by combining blue, green, and red in various proportions. Although we see sunlight as a uniform or homogeneous colour, it is actually composed of various wavelengths of radiation in primarily the ultraviolet, visible and infrared portions of the spectrum. The visible portion of this radiation can be shown in its component colours when sunlight is passed through a **prism**, which bends the light in differing amounts according to wavelength.

The next portion of the spectrum of interest is the infrared (IR) region which covers the wavelength range from approximately 0.7 μm to 100 μm - more than 100 times as wide as the visible portion! The infrared region can be divided into two categories based on their radiation properties - the **reflected IR**, and the emitted or **thermal IR**. Radiation in the reflected IR region is used for remote sensing purposes in ways very similar to radiation in the visible portion. The reflected IR covers wavelengths from approximately 0.7 μm to 3.0 μm. The thermal IR region is quite different from the visible and reflected IR portions, as this energy is essentially the radiation that is emitted from the Earth's surface in the form of heat. The thermal IR covers wavelengths from approximately 3.0 µm to 100 µm.

The portion of the spectrum of more recent interest to remote sensing is the **microwave region** from about 1 mm to 1 m. This covers the longest wavelengths used for remote sensing. The shorter wavelengths have properties similar to the thermal infrared region while the longer wavelengths approach the wavelengths used for radio broadcasts.

Wave Length Regions and their Applications in Remote Sensing

Figure 4.3 shows the EM spectrum which is divided into discrete regions on the basis of wavelength. Remote sensing mostly deals with energy in visible (Blue, green, red), infrared (near-infrared, mid-infrared, thermal-infrared) regions. Table 4.1 gives the wave length region along with the principal applications in remote sensing. Energy reflected from earth during daytime may be recorded as a function of wavelength. The maximum amount of energy is reflected at 0.5 µm, called the reflected energy peak. Earth also radiates energy both during day and night time with maximum energy radiated at 9.7 µm, called radiant energy peak.

Table 5.1 Wave length regions and their applications in remote sensing

Region	Wave length (µm)	Principal Applications
(a) Visible Region		
1. Blue	0.45 – 0.52	Costal morphology and sedimentation study, soil and vegetation differentiation, coniferous and deciduous vegetation discrimination.
2. Green	0.52 – 0.6	Vigor assessment, Rock and soil discrimination, Turbidity and bathymetry studies.
3. Red	0.63 – 0.69	Plant species differentiation
(b) Infrared Region		
4. Near Infrared	0.76 – 0.9	Vegetation vigor, Biomass, delineation of water features, land forms/geomorphic studies
5. Mid-infrared	1.55 – 1.75	Vegetation moisture content, soil moisture content, snow and cloud differentiation
6. Mid-infrared	2.08 – 2.35	Differentiation of geological materials and soils
7. Thermal IR	3.0 – 5.0	For hot targets, i.e. Fires and volcanoes
8. Thermal IR	10.4 – 12.5	Thermal sensing, vegetation discrimination, volcanic studies

Characteristics of Solar Radiation

All objects above 0°K emit EM radiation at all wavelengths due to conversion of energy into EM energy. All stars and planets emit radiation. Our chief star, the Sun, is almost a spherical body with diameter of 1.39×10^{6} km. The continuous conversion of hydrogen to helium which is the main constitute of the Sun, generates energy that is radiated from the outer layers. Passive remote sensing uses Sun as its source of EM radiation. Sun is the strongest source of radiate energy and can be approximated by a body source of temperature 5750 - 6000°K. Although sun produces EM radiation across a range of wavelengths, the amount of energy it produces is not evenly distributed along this range. Approximately 43% is radiated within the visible wavelength (0.4 to 0.7μm), and 48% of the energy is transmitted at wavelength greater than 0.7μm, mainly within infrared range.

If the energy received at the edge of earth's atmosphere were distributed evenly over the earth, it would give an average incident flux density of 1367 W/m^2. This is known as the solar constant. Thirty five percent of incident radiant flux is reflected back by the earth. This includes the energy reflected by clouds and atmosphere. Seventeen percent of it is absorbed by the atmosphere while 48% is absorbed by the earth's surface materials.

EM Radiation and the Atmosphere

In remote sensing, EM radiation must pass through atmosphere in order to reach the earth's surface and to the sensor after reflection and emission from the earth's surface features. The water vapor, oxygen, ozone, CO_2, aerosols, etc. present in the atmosphere influence EM radiation through the mechanism of (i) scattering, and (ii) absorption.

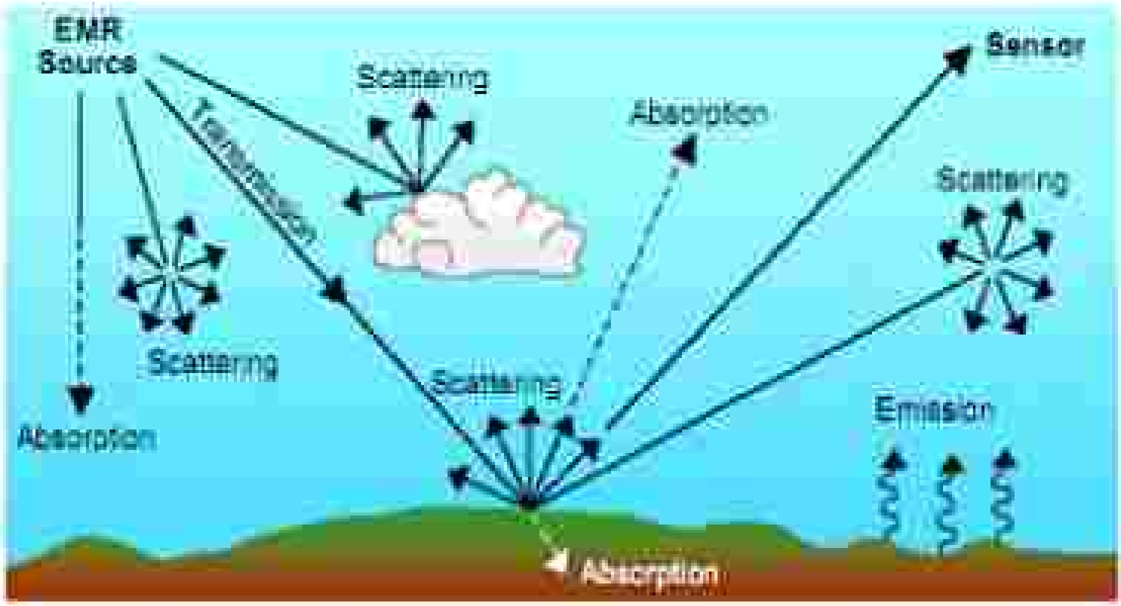


Figure 5-6-Electromagnetic Interaction with Atmosphere

Scattering: It is unpredictable diffusion of radiation by molecules of the gases, dust and smoke in the atmosphere. Scattering reduces the image contrast and changes the spectral signatures of ground objects. Scattering is basically classified as (i) selective, and (ii) non-selective, depending upon the size of particle with which the electromagnetic radiation interacts.

Absorption: In contrast to scattering, atmospheric absorption results the effective loss of energy as a consequence of the attenuating nature of atmospheric constituents, like molecules of ozone, CO_2 and water vapor. Oxygen absorbs in the ultraviolet region and also has an absorption ban centered on 6.3μm. Similarly CO_2 prevents a number of wavelengths reaching the surface. Water vapor is extremely important absorber of EM radiation within infrared part of the spectrum.

Platforms

In remote sensing, the sensor is mounted on a platform. In general, remote sensing sensors are attached to moving platforms such as aircraft and satellites. Static platforms are occasionally used in an experimental context

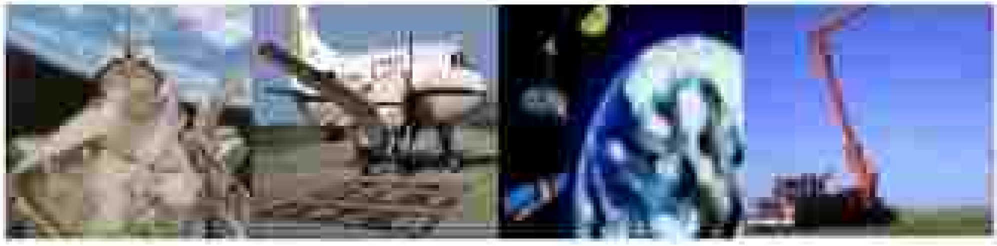


Figure 5-7 Remote sensing Platforms

In an earlier section, we learned some of the fundamental concepts required to understand the process that encompasses remote sensing. In this section, we will take a closer look at the remote sensing process by examining in greater detail, the characteristics of remote sensing platforms and sensors and the data they collect. We will also touch briefly, on how that data is processed once the sensor has recorded it. In order for a sensor to collect and record energy reflected or emitted from a target or surface, it must reside on a stable **platform** removed from the target or surface being observed. Platforms for remote sensors may be situated on the ground, on an aircraft or balloon (or some other platform within the Earth's atmosphere), or on a spacecraft or satellite outside of the Earth's atmosphere.

Ground-based sensors are often used to record detailed information about the surface, which is compared with information collected from aircraft or satellite sensors. In some cases, this can be used to better characterize the target, which is being imaged by these other sensors, making it possible to better understand the information in the imagery.

Sensors may be placed on a ladder, scaffolding, tall building, cherry picker, crane, etc. Aerial platforms are primarily stable wing **aircraft**, although helicopters are occasionally used. Aircraft are often used to collect very detailed images and facilitate the collection of data over virtually any portion of the Earth's surface at any time.

In space, remote sensing is sometimes conducted from the space shuttle or, more commonly, from satellites. Satellites are objects, which revolve around another object - in this case, the Earth. For example, the moon is a natural satellite, whereas man-made satellites include those platforms launched for remote sensing, communication, and telemetry (location and navigation) purposes. Because of their orbits, satellites permit repetitive coverage of the Earth's surface on a continuing basis. Cost is often a significant factor in choosing among the various platform options.

Image Data Characteristics:

Remote sensing image data are more than a picture: they are measurements of EM energy. Image data are stored in a regular grid format (rows and columns). A single image element is called a pixel, a contraction of 'picture element'. For each pixel, the measurements are stored as Digital Numbers, or DN-values. Typically, for each measured wavelength range a separate data set is stored, which is called a band or a channel and sometimes a layer.

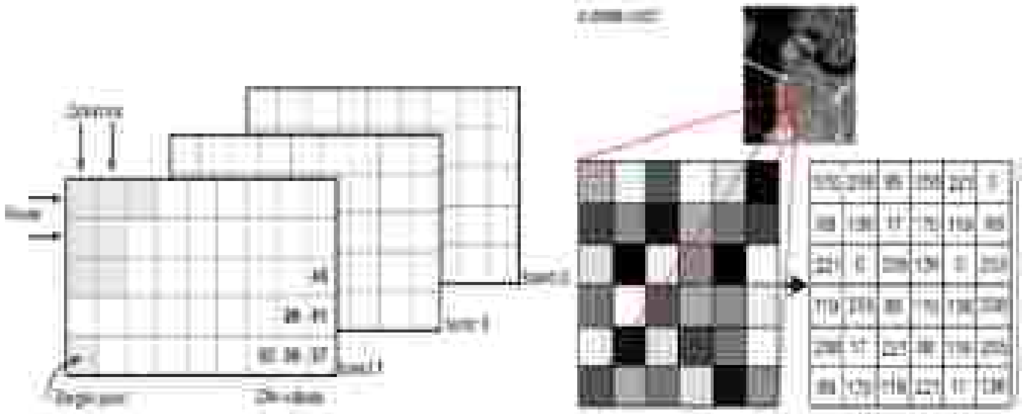
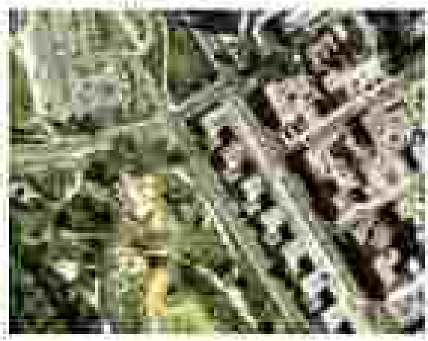


Figure 5-8 Remote Sensing Image

The image characteristics are usually referred to as:

- Spatial characteristics, which refer to the area measured.
- Spectral characteristics, which refer to the spectral wavelengths that the sensor is sensitive
- Radiometric characteristics, which refer to the energy levels that are measured by the sensor.
- Temporal characteristics, which refer to the time of the acquisition

Spatial resolution of the sensor and refers to the size of the smallest possible feature that can be detected.



Images where only large features are visible are said to have coarse or low resolution. In fine or high resolution images, small objects can be detected.

Spectral resolution: This is related to the widths of the spectral wavelength bands that the sensor is sensitive to. The finer the spectral resolution, the narrower the wavelength range for a particular channel or band

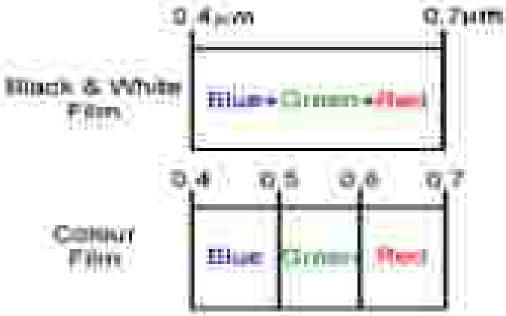


Figure 5.9 Spectral Resolution

Advanced multi-spectral sensors called hyperspectral sensors, detect hundreds of very narrow spectral bands throughout the visible, near-infrared, and mid-infrared portions of the electromagnetic spectrum. Their very high spectral resolution facilitates fine discrimination between different targets based on their spectral response in each of the narrow bands.

The radiometric resolution of an imaging system describes its ability to discriminate very slight differences in energy.

The finer the radiometric resolution of a sensor the more sensitive it is to detecting small differences in reflected or emitted energy.

Figure 5-10 Radiometric Resolution

Temporal resolution: The revisit period of a satellite.

Figure 5-11 Temporal Resolution

Application of remote sensing

Remote sensing affords a practical means for accurate and continuous monitoring of the earth's surface and other resources and of determining the impact of man's activities on air, water and land.

A summary of remote sensing application is given below, discipline wise.

Land use and soil

- Mapping of land use/cover
- Change detection
- Soil categorization

Urban land use

- Urban land use mapping
- Updating urban transport network

- Monitoring urban sprawl
- Identification unauthorized structure

Water Resources

- Monitoring surface water bodies frequently and estimation of their spatial extent

Watershed

- Delineation of watershed boundaries/partitioning of micro watershed
- Watershed characteristic at larger scale such as size, shape, drainage, land cover

Facilities management

- Locating underground pipes; cables
- Planning facility maintenance

Digital Elevation Model

- Contour (>10 m)
- Slope/aspect analysis
- Large scale thematic mapping

5.2 Introduction to Global Positioning System (GPS) Surveying

Over of GPS

Global Positioning System (GPS) is a satellite based navigation system that was developed by the U.S. Department of Defense (DoD) in the early 1970s.Initially, GPS was developed as a military system to fulfill U.S. military needs. However, it was later made available to civilians, and is now a dual-use system.

GPS provides continuous positioning and timing information, anywhere in the world under any weather conditions and it is a one-way- ranging system.

GPS consists of 24 operational satellites and some spares. GPS orbits are ellipses with maximum eccentricity is about 0.01. The six equally spaced orbital planes are inclined at 55° to the equator, resulting in five hours above the horizon. The system therefore guarantees that at least four satellites will always be in view. The semi major axis of a GPS orbit is about 26,560 Km .The corresponding GPS orbital period is about 11 hours: 58 minutes.

GPS segments:

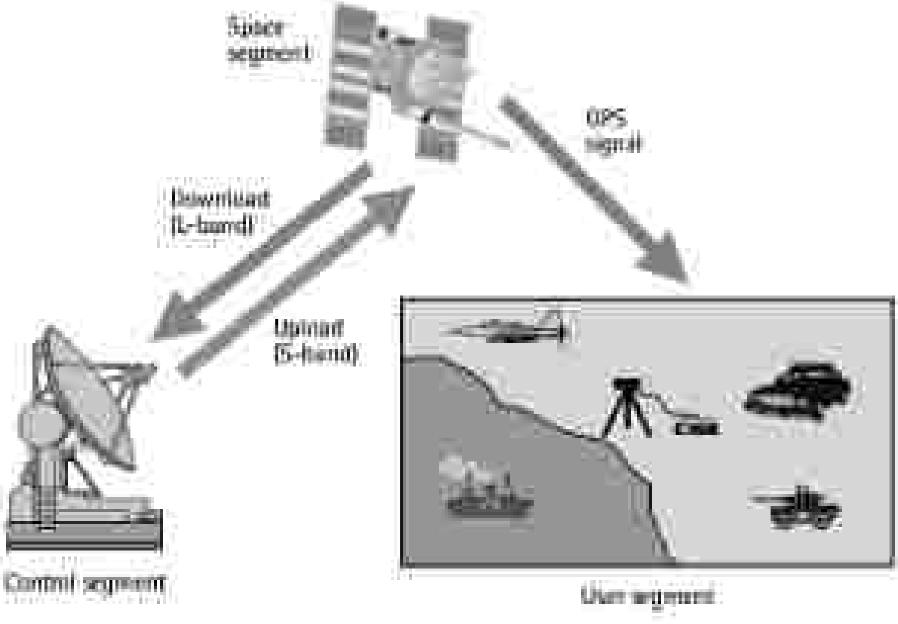
GPS consists of three segments-

1. **Space segment**- consists of 24 satellites, each transmits signal which has number of components. Each satellite has a fundamental frequency of 10.23 MHz and transmits two L-band radio signals. Signal L1 has a frequency of 1575.42 MHz (10.23 × 154) and L2 a frequency of 120). The signal also contains digital codes and 1227.60 MHz (10.23 X navigation messages. The carrier frequencies and the codes mainly used to determine the distance from the user's receiver to the GPS satellites. Navigation message contains along with other information, the coordinates of the satellites as a function of time.

2. **Control segment**-The *control* segment has the task of supervising the satellite timing system, the orbits and the mechanical condition of the individual satellites. Neither the timing system nor the orbits are

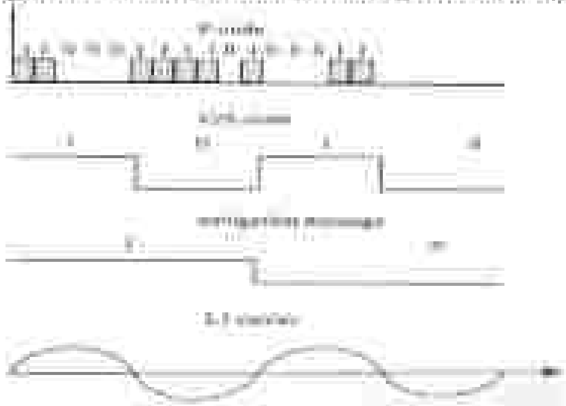
sufficiently stable to be left unchecked for any great period of time. The satellites are currently tracked by five monitor stations, situated in Kwajalein, Hawaii, Ascension and Diego Garcia, with the master control in Colorado Springs.

3. **User segment** – Includes all military and civilian users. With a GPS receiver connected to a GPS antenna, a user can receive the GPS signals, which can be used to determine his or her position anywhere in the world. GPS is currently available to all users worldwide at no direct charge.



GPS Signal

All GPS satellites have atomic clocks and transmit a microwave radio signal derived from such clock, composed of two carrier frequencies (L1 = 1575.42 MHz, L2 = 1227.6 MHz) and modulated by two digital codes and a navigation message. Even though all of the GPS satellites transmit the same L1 and L2 carrier frequencies, the code modulation is different for each satellite, which significantly minimizes the signal interference. Each part of the signal is different from every other, called pseudo-random code. This sequence is repeated continuously. All GPS receivers know this sequence and repeat it internally. The receiver picks up the satellite's transmission and compares the incoming signal to its own internal signal. By comparing how much the satellite signal is lagging, the travel time becomes known. There are three types of code on the carrier signals, C/A code, P code and Navigation Message.



The characteristics of these codes and navigation message are shown in the following table.

	C/A code	P- code	Navigation Message
Chipping rate	1.023MHz	10.23MHz	50 bit per second
Length per chip	293 m	29.3m	5950 Km
Repetition	1ms	1 week	N/A
Code type	Gold	PRN	N/A
Carriers on	L1	L1,L2	L1,L2
Code nature	Coarser code appropriate for initially locking onto the signal	10 times finer than C/A code	Very course
Included information	Time according to the satellite clock when the signal was transmitted	Time according to the satellite clock when the signal was transmitted.	Ephemeris, Satellite clock corrections, Almanac data, ionospheric information, and satellite health data.
Application	Moderate Accuracy, Benign Environment, P Code Acquisition	High Accuracy, ECM Environment, Secure	For all the cases

The GPS signal contains three different types of information, **pseudo random code, almanac data** and **ephemeris data.** The pseudo random code is simply an I.D. code that identifies which satellite is transmitting information. Almanac data is data that describes the orbital courses of the satellites. Every satellite will broadcast almanac data for every satellite. Your GPS receiver uses this data to determine which satellites it expects to see in the local sky. It can then determine which satellites it should track. Almanac data is not precise it can be valid for many months.

Ephemeris data is data that tells the GPS receiver where each GPS satellite should be at any time throughout the day. Each satellite will broadcast its own ephemeris data showing the orbital information for that satellite only.

Reference coordinate system for GPS

A reference system is the complete conceptual definition of how a coordinate system is formed. It defines the origin and the orientation of fundamental planes or axes of the system. It also includes the underlying fundamental mathematical and physical models. A conventional reference system is a reference system where all models, numerical constants and algorithms are explicitly specified. Time system is also included in reference system.

A number of time systems are used in the world. From all of them, UTC and GPS time are the most important time system for GPS users.

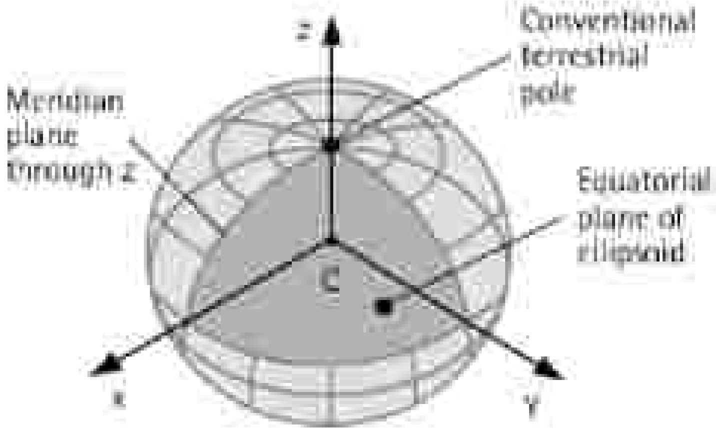
The fact that the topographic surface of the Earth is highly irregular makes it difficult for the geodetic calculations. To overcome these problem geodesists adopted a smooth mathematical surface, called the reference surface to approximate the irregular shape of the Earth. Which is **sphere** for low accuracy positioning, or **ellipsoid** for high accuracy positioning. The reference ellipsoid used of GPS work is the **WGS84** ellipsoid (geocentric). With semi major axis (a) = 6378137m and f = 1/298.257223563.

Geodetic datum is a mathematical surface, or a reference ellipsoid, with a well defined origin and orientation. **A geocentric datum** is geodetic datum with its origin coinciding with the center of the earth.

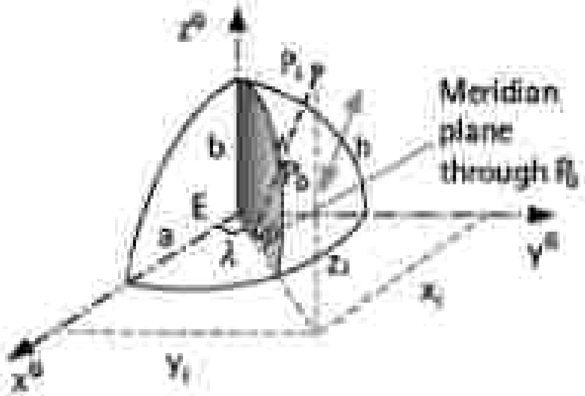
A geodetic datum is uniquely determined by specifying eight parameters: two parameters to define the dimension of the reference ellipsoid, three parameters to define the position of the origin, and three parameters to define the orientation of the three axes with respect to the earth. In addition to the geodetic datum, the so called vertical datum is used in practice as a reference surface to which the heights (elevations) of points are referred. The vertical datum is often selected to be the geoid; the surface that best approximates the mean sea level on a global basis.

Because the objective of GPS surveys is to locate points on the surface of the earth, it is necessary to have a so, called terrestrial frame of reference, which enables relating points physically to the Earth. **Geocentric coordinate system** is a coordinate system in which its the origin coincides with the center of the Earth. Its axis passes through the Greenwich meridian in the plane of equator, The Z axis coincides with the conventional Terrestrial pole and Y axis held right hand rule.

Conversion from Satellite reference system to geocentric system four angular parameters is required: *Inclination angle, the argument of perigee, right ascension and Greenwich hour angle.*



Of particular importance to GPS users is the 3-D geodetic coordinate system. In this system the coordinates of a point are identified by the geodetic latitude (φ), geodetic longitude (λ) and height above the reference surface.



The satellites coordinates as given in the broad cast ephemeris will refer to the WGS 84 reference system. Therefore, a GPS user who employs the broadcast ephemeris in the adjustment process will obtain his or her coordinates in WGS 84 system as well.

Ethiopia is using local **Adindan datum**, with Clark 1880 ellipsoid, so that the coordinates directly found from GPS have to be transformed into this local datum before using it.

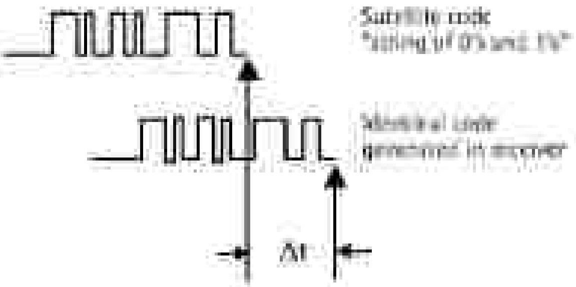
Fundamentals of GPS positioning

The fundamental technique of GPS is to measure the ranges (distances) between the receiver and a few simultaneously observed satellites. GPS positioning is based on trilateration, which is the method of determining position by measuring distances to points at known coordinates. At a minimum, trilateration requires 3 ranges to 3 known points. But in case of GPS, since there is an offset in clocks of Satellite and receiver, clock error will be treated as unknown in addition to the coordinates of a point. Such that one additional measurement is required, i.e. minimum 4 simultaneous "Pseudo ranges" to 4 satellites are necessary.

When a GPS receiver is switched on, it will pick up the GPS signal through the receiver antenna. Once the Receiver acquires the signal, Time that the signal is transmitted from the satellite is encoded on the signal, using the time according to an atomic clock onboard the satellite. Time of signal reception is recorded by receiver using an atomic clock. A receiver measures difference in these times and it will process it using its built-in software to calculate the range.

Distance = Travel time x Speed of light

The partial outcome of the signal processing consists of the distance to the GPS satellites through the digital codes (Known as the pseudo ranges) and the satellite coordinates through the navigation messages.
GPS receivers in determining distances to satellites employ two fundamental methods: code ranging or pseudorange measurement and carrier phase- shift measurements.
Pseudorange measurement is a measure of the range or distance between the GPS receiver and the GPS satellite which is needed for the position computation. Either the P-code or the C/A code can be used for measuring th pseudorange. When the PRN code is transmitted from the satellite, the receiver generates an exact replica of that code. After some time, equivalent to the signal travel time in space, the transmitted code will be picked up by the receiver. By comparing the transmitted code and its replica the receiver can compute the signal travel time. Multiplying the travel time by the speed of light gives the range between the satellite and the receiver.
The receiver and satellite clocks are not perfectly synchronized and the measured range is contaminated, along with other errors and biases, by the synchronization error between the satellite and receiver clocks. For this reason, this quantity is referred to as the pseudorange, not the range.

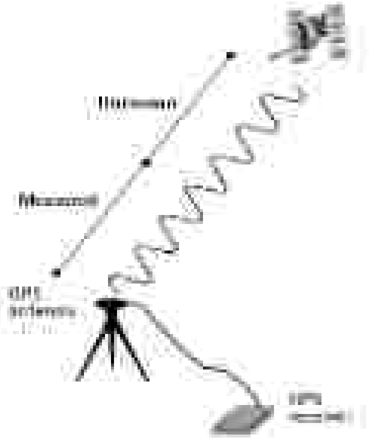


Carrier- Phase measurement is anotherway of measuring the ranges. The range would simply be the sum of the total number of full carrier cycles plus fractional cycles at the receiver and the satellite, multiplied by the carrier wavelength. The ranges determined with the carriers are far more accurate than those obtained with the codes. This is due to the fact that the wavelength of the carrier phase, 19 cm in the case of L1 frequency, is much smaller than those of the codes.

The carriers are just pure sinusoidal waves. This means that all cycles look the same. Therefore, a GPS receiver has no means to differentiate one cycle from another. It can only measure a fraction of a cycle very accurately, while the initial number of complete cycles remains unknown or ambiguous. This is therefore, commonly known as the initial cycle ambiguity, or the ambiguity bias. Fortunately, the receiver has the capability to keep track of the phase changes after being switched on. This means that the initial cycle ambiguity remains unchanged over time as long as no signal loss.

It is clear that if the initial cycle ambiguity parameters are resolved accurate range measurements can be obtained, which lead to accurate position determination.

Cycle Slips is a discontinuity or a jump in the GPS carrier- phase measurements, but an integer number of cycles, caused by temporary signal loss. This signal loss can be due to obstruction of the GPS satellite signal by building, bridges, trees, and other objects. This is mainly because the GPS signal is a weak and noisy signal. Cycle slips may occur briefly or remain for several minutes or even more. Cycle slips must be identified and corrected to avoid large errors into computed coordinates. This can be done using several methods. Examining the so called triple difference observable, which is formed by combining the GPS observables in a certain way, is the most popular in practice.



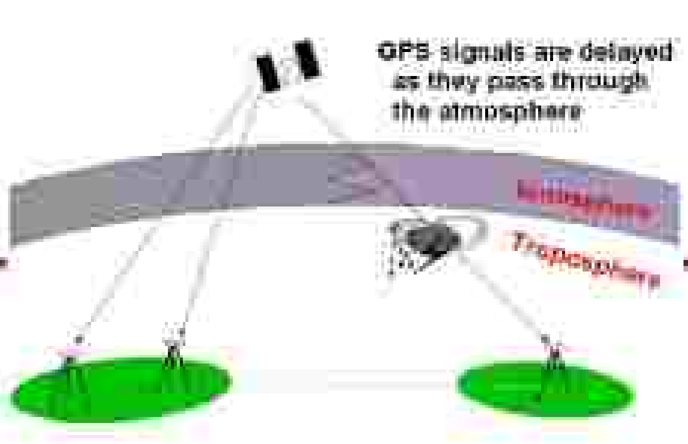
Errors in GPS observation

Even though GPS is the most accurate positioning method, it is not free from errors. GPS pseudo code and carrier phase measurements are subject to different types of errors. These errors can be Satellite dependent, propagation dependent or receiver dependent.

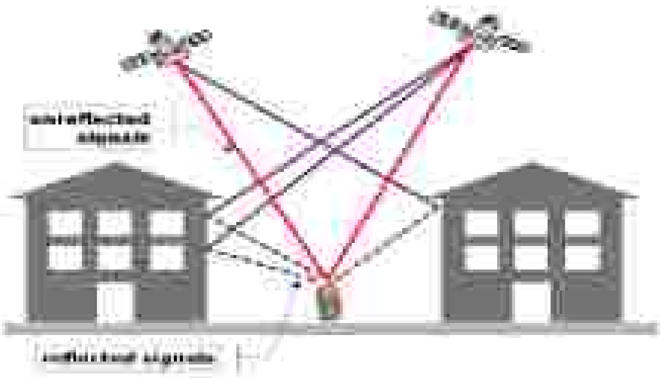
Satellite dependent errors
GPS carrier and pseudorange code measurements are subjected to errors due to satellite erroneous data transmition to the receiver.

a) **Satellite clock error**
b) **Satellite ephemeris errors**
c) **Selective Availability**
d) **Satellite Geometry**

Signal propagation dependent errors
Satellite signal propagates in the atmosphere and this travel media is composed of different layers, namely ionosphere and troposphere. Due to the presence of these layers the signal propagation is characterised by delay and bending of the light ray.



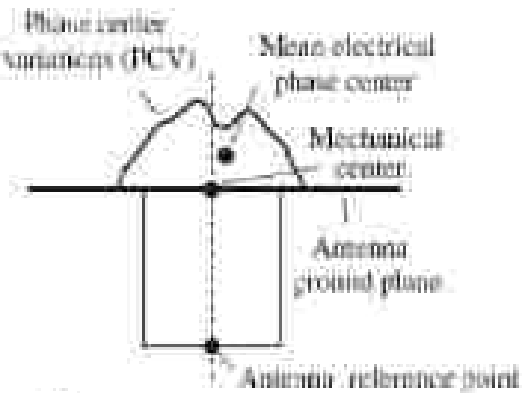
a. **Ionospheric delay**
b. **Tropospheric delay**
c. **Multipath error**



Receiver dependent errors

The space segment and the signal propagation media is not the only source of error and biases in GPS measurement, but also there are errors that depend up on the receiver clocks and the orientation of the antenna.

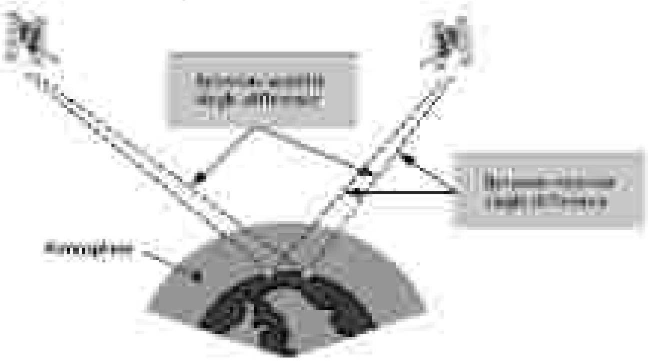
a. **Receiver clock error**
b. **Antenna- phase-center variation**



Linear combinations of GPS observables

GPS measurements are corrupted by a number of errors and biases as discussed in previous section, which are difficult to model fully. The unmodeled errors and biases limit the positioning accuracy of the stand alone GPS receiver. Fortunately, GPS receivers in close proximity will share to a high degree of similarity the same errors and biases. As such, for these receivers, a major part of the GPS errors budget can simply be removed by combining their GPS observables.

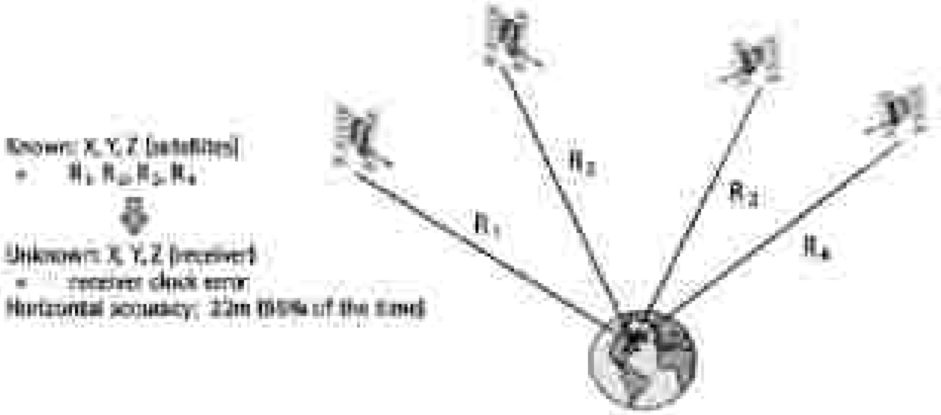
In principle, there are three groups of GPS errors and biases: satellite related errors, receiver related errors, and atmosphere related errors and biases. The measurements of two GPS receivers simultaneously tracking a particular satellite contain more or less the same satellite related errors and atmospheric errors. The shorter the separation between the two receivers, the more similar the errors and biases are. Therefore, if we take the difference between the measurements collected at the two receivers, the satellite related errors and the atmospheric related errors will be reduced significantly. In fact, as shown in the next chapter, the satellite clock error is effectively removed with this linear combination. This linear combination is known as **betweenreceiver single difference.**
Similarly, the two measurements of a single receiver tracking two satellites contain same clock errors. Therefore, taking the difference between these two measurements removes the receiver clock errors. This difference is known as **between satellite single difference**.



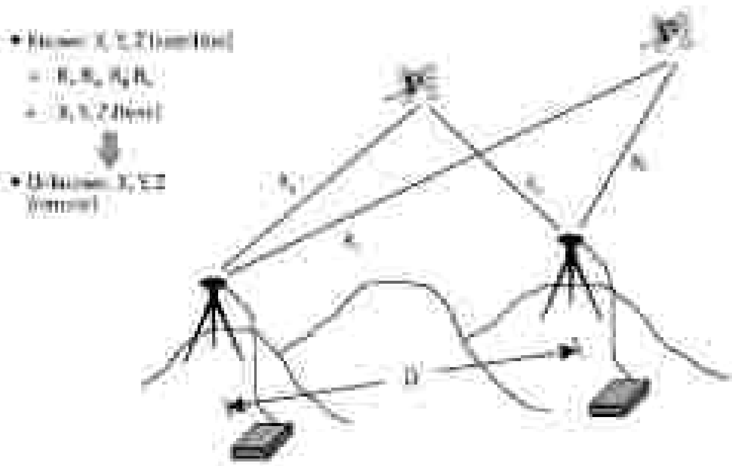
When two receivers track two satellites simultaneously, two between receiver single difference observables could be formed. Subtracting these two single difference observables from each other generates the so called **double differences**. This linear combination removes the satellite and receiver clock errors. The other errors are greatly reduced. In addition, this observable preserves the integer nature of the ambiguity parameters. It is therefore used for precise carrier phase based GPS positioning.
Another important linear combination is known as **the triple difference**, which results from differencing two double difference observables over two epochs of time. As explained in the previous sections, the ambiguity parameters remain constant over time, as long as there are no cycle slips as such, when forming the triple difference, the constant ambiguity parameters disappear. If, however, there is a cycle slips in the data, it will affect one triple difference observable only, and therefore will appear as a spike in the triple differences data series. It is for this reason that triple difference linear combination is used for detecting cycle slips.
All these linear combinations can be formed with a single frequency data, whether it is the carrier phase or the pseudorange observables. If dual frequency data is available, other useful linear combinations could be formed. One such linear combination is known as the ionosphere free linear combination. As shown in the next chapter, ionospheric delay is inversely proportional to the square of the carrier frequency. Based on this characteristic, the ionosphere-free observable combined to form the so called wide lane observable an artificial signal with an effective wavelength of about 86cm, this long wavelength helps in resolving the integer ambiguity parameters.

GPS Positioning Modes

Positioning with GPS can be performed by either of two ways: **point positioning** or **relative positioning.**
Point positioning employs one GPS receiver that measures the code pseudoranges to determine the user's position instantaneously as long as four or more satellites are visible at the receiver. This positioning method is used mainly when a relatively low accuracy is required. To determine the receiver's point positioning at any time, the satellite coordinate as well as a minimum of four ranges to four satellites are required.



Relative Positioning or differential positioning employs two GPS receivers simultaneously tracking the same satellites to determine their relative coordinates. Of the two receivers, one is selected as a reference, or base, which remains stationary at a site with precisely known coordinates. If both receivers track at least four common satellites, a position accuracy level of the order of a sub centimeter to a few meter can be obtained. Tracking more than four common satellites would improve the precision of the GPS position solution.



Static GPS surveying is a relative positioning technique that depends on the carrier phase measurements. It employs two or more receivers simultaneously tracking the same satellites. One receiver, the base receiver, is setup over a point with precisely known coordinates such as a survey monument. The other receiver, the remote receiver, is set up over a point whose coordinates are sought. The observation time varies from about 20 minutes to few hours depending on the distance between the base and the remote receivers, the number of visible satellites, and the satellite geometry.

Fast (Rapid) static surveying is similar to static surveying, but in this case only the base receiver remains stationary over the known point during the entire observation session. The rover receiver remains stationary over the unknown point for a short period of time only and then move to another point whose coordinates are sought. This method is suitable when the survey involves a number of unknown points located in the vicinity of known points.

Stop- and -Go GPS surveying is another carrier phase based relative positioning technique. In such case, a base receiver that remains stationary over the known point and one or more rover receivers. The rover receiver travels between the unknown points, and makes a brief stop in each point to collect the GPS data.

RTK GPS is a carrier phase based relative positioning technique that like previous methods employs two (or more) receivers simultaneously tracking the same satellites. This method is suitable when:

- The survey involves a large number unknown point located in the vicinity.
- The coordinates of the unknown points are required in real time.
- The line of sight, the propagation path, is relatively unobstructed.

Because of its ease of use as well as its capability to determine the coordinates in real time, this method is the preferred method by many users. The base receiver remains stationary over the known point and is attached to a radio transmitter. The rover is normally carried in a backpack and is attached to a radio receiver.
Real time differential GPS is a code based relative positioning technique that employs two or more receivers simultaneously tracking the same satellites. It is used when a real time meter level accuracy is enough. This method is based on the fact that the GPS errors in the measured pseudoranges are essentially the same at both the base and the rover as long as the baseline length is within a few hundred kilometers.
Other Satellite navigation systems

GPS has never been the only satellite positioning system: other systems such as Transit preceded it, and Russia has a system called Glonass, which has remained mainly a military system. To enhance the accuracy of GPS in certain areas and to provide a backup system in the event of GPS failing or being withdrawn, several countries have already developed regional augmentation to the GPS (and Glonass) signals, using geostationary satellites. These include WAAS in the United States, MSAS in Japan and EGNOS (European Geostationary Navigation Overlay System) in Europe. EGNOS is due to come online in 2004 and will deliver positional accuracy of better than 5 m from a single receiver, throughout Europe.

In addition, the European Union has planned an entirely independent satellite system called Galileo, which will further enhance navigational accuracy as well as provide a number of other services (e.g. for search and rescue). In particular, it is intended that there should be no common mode of failure between GPS and Galileo.

Galileo is planned to come online in 2008 and will consist of 30 satellites in circular orbits inclined at 56° to the equator; there will be three orbital planes, with ten satellites equally spaced on each plane. The orbital radius will be 30 Mm, giving an orbital period f about 14 h.

Galileo is conceptually quite similar to GPS and will also work by measuring the time taken for signals to travel from a satellite to a receiver. However, it will broadcast signals on three different frequency bands (1,164–1,215 MHz, 1,260–1,300 MHz and 1,559–1,591 MHz) which should significantly improve the calculation of atmospheric delays. In addition, some of the signals will incorporate an integrity check, intended to guard against false indications of position.

Galileo will bring two major benefits to the surveying community:

1. It will improve accuracy and reduce observation times by providing more satellites; GDOPs of greater than 6 will cease to occur.
2. It should perform noticeably better than GPS in built-up areas, due to the integrity checks in the signals.

On the other hand, Galileo is planned as a commercial venture, in which the users will pay for the deployment and maintenance of the system. In particular, it is intended that commercial users (i.e. surveyors) should pay for the enhanced positioning services that they will need, by means of access-protection keys on their receivers.

Application of GPS

GPS has been available for civil and military use for more than two decades. That period of time has witnessed the creation of numerous new GPS applications. Because it provides high-accuracy positioning in a cost effective manner, GPS has found its way into many industrial applications, replacing conventional methods in most cases.

GPS for civil engineering applications

Civil engineering works are often done in a complex and unfriendly environment,making it difficult for personnel to operate efficiently. The abilityof GPS to provide real-time submeter- and centimeter-level accuracy ina cost-effective manner has significantly changed the civil engineeringindustry. Construction firms are using GPS in many applications such asroad construction, Earth moving, and fleet management.In road construction and Earth moving, GPS, combined with wirelesscommunication and computer systems, is installed onboard the Earthmoving machine. Designed surface information, in a digital format, isuploaded into the system. With the help of the computer display and thereal-time GPS position information, the operator can view whether thecorrect grade has been reached. In situations in whichmillimeter-level elevation is needed, GPS can be integrated with rotatedbeam lasers.

The same technology (i.e., combined GPS, wireless communications, and computers) is also used for foundation works (e.g., pile positioning) and precise structural placement (e.g., prefabricated bridge sections and coastal structures). In these applications, the operators are guided through the onboard computer displays, eliminating the need for conventional methods. GPS is also used to track the location and usage of equipment at different sites. By sending this information to a central location, GPS enables contractors to deploy their equipment more efficiently. Moreover, vehicle operators can be efficiently guided to their destinations.

5.3 Introduction to Geographic Information System (GIS)

Introduction

Almost everything happens at somewhere. Largely we humans are confined in our activities to the surface and near surface of the Earth. We travel over it and in the lower levels of the atmosphere, and through tunnel dig just below the surface. We dig ditches and bury pipelines and cables, construct mines to get at mineral deposit, and drill wells to access oil and gas. Keeping track of all this activity is important, and knowing where it occurs can be the most convenient basis for tracking. Knowing where something happens is of critically important if we want to go there ourselves or send someone there, to find other information about the same place, or to inform people who live nearby. In addition, most (perhaps all) decisions have geographic consequences. Therefore, geographic location is an important attribute of activities, policies, strategies, and plans. Geographic information systems are a special class of information systems that keep the track not only of events, activities, and things, but also of where these events, activities, and things happen or exist. Because location is so important, it is an issue in many of the problems society must solve. Some of these are so usual that we almost fail to notice them- the daily questions of which route to take to and from work, for example. Others are quite extraordinary occurrence, and require rapid, concerted, and coordinated response by a wide range of individuals and organizations. Problems that involve an aspect of location, either in the information used to solve them, or in the solutions themselves, are termed as geographic problems.

Definitions of GIS

Different types of definition can be given to GIS depends on the profession and the purpose it is using. But common to all definition is the spatial location. For our purpose we can define GIS as "A GIS is a system, consisting of hardware,software, data, procedures and proper organizational context, which compiles, stores, manipulates, analyses, models and visualizes spatial data, to solve planning and management problems" (Christiansen, 1998). GIS as a system and science can be defined as follow: Geographic Information system (GIS) is a computer-based system used to store, analyze and display spatial information. Geographic Information Science (GISc) is a set of principles, procedures and technologies providing scientific foundation to spatial analysis or spatial science.

Key elements of GIS

GIS has three key elements: the Geographic, Information and the System. Geographic: It shows the real world or the spatial realities (Example: the location of a city, location of a school etc) Information: It is about the data and their meanings (Example: The name of a city, the area of a city, the population density of a city etc) System: The system is also about the computer technology (Example: Computer hardware &Software).

History of GIS

The history of GIS is split in to three eras: The era of innovation, era of commercialization, and era of exploitation.

1) Era of Innovation (1957- 1977): It is the era where GIS is introduced to the world. It was created by Harvard researchers in the Harvard Laboratory for computer graphics and spatial analysis. The most important events in the era of innovation were the foundation of ESRI (Environmental System and Research Institute) and the launch of Landsat I.

2) Era of Commercialization (1981-1999): It is the era where GIS is used to make a business. Hence, a number of Government and private organizations were established to make GIS a worldwide profit making industry. The main events of this era were the launch of ArcInfo, introduction of GPS operation (It is used for navigation, surveying and mapping), and Internet GIS products.

3) Era of Exploitation (1999 -): It is the era where we are now. It is distinct by a high number of GIS users. The prominent activities of this era are the availability of more than one million users, Launch of IKONOS and QUICKBIRD satellites, and the introduction of Google earth and Mobile mapping.

Components of GIS

The people, software, data, hardware and procedures are the five components of a GIS:

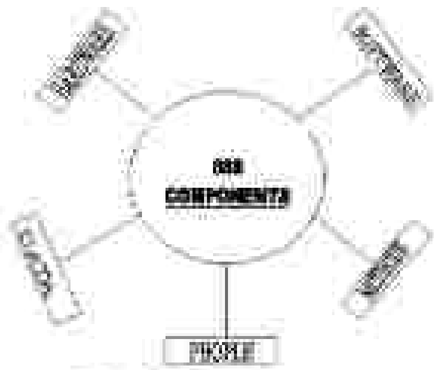


Fig.6.1 GIS Components

Hardware: It consists of the computer systems on which the GIS software will run.

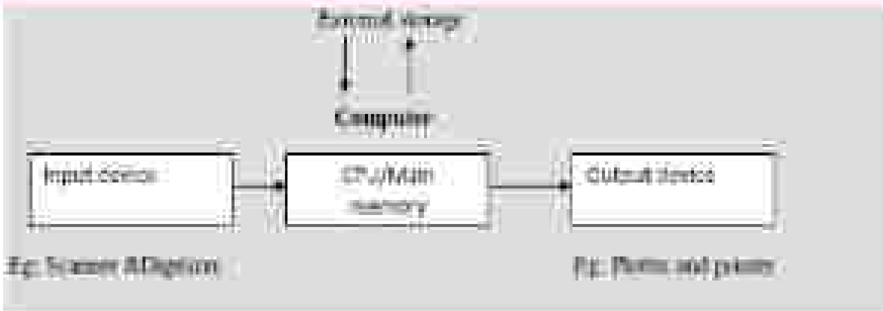


Fig6.2 Hardware component of GIS

Software: It provides tools to manage analyze and effectively display and disseminate spatial data and spatial information. The main functions of GIS software's are analytical functions that provide means for driving new geo-information from the existing spatial and attribute data. The key components of GIS softwares are:

- A database management system (DBMS)
- Tools for the input and manipulation of geographic information
- Tools that support geographic query, analysis, and visualization
- A graphical user interface (GUI) for easy access to tools

GIS software's can be divided as public domain and commercially available software's. Example: ArcView, ArcGIS, ENVI, ERDAS,IDRISI,ILWIS, and many more others.

Data: It is the most important component of a GIS. Geographic data and related tabular data can be collected in house, compiled to custom specifications and requirements, or purchased from a commercial data provider. A GIS can integrate spatial data with other existing data resources, often stored in a DBMS. The integration of spatial and tabular data stored in a DBMS is a key functionality afforded by GIS.

People: GIS technology has limited value without the people who manage and develop plans for applying it to real world problems. GIS user ranges from technical specialists, who design and maintain the system, to consumers who use it to help them perform their everyday work. The identification of GIS specialist's vs. end users is often critical to the proper implementation of GIS technology.

Method: Method include how the data will be retrieved, input into the system, stored, managed, transformed, analyzed, and finally presented in a final output. The procedures are the steps taken to answer the question needed to be resolved. The ability of a GIS to perform spatial analysis and answer these questions is what differentiates this type of system from any other information systems. A successful GIS operates according to a well designed implementation plan and business rules, which are the models and operating practices unique to each organization.

Data Model in GIS

The heart of any GIS is the data model, which is a set of constructs for description and representation of objects and processes in the digital environment of the computer. People (GIS users) interact with operational GIS in order to undertake tasks like making maps, querying databases, and performing analysis and presentation of out puts. Because the types of analyses that can be undertaken are strongly influenced by the way the real-world is modelled, decisions about the type of data model to be adopted are vital to the success of a GIS project.

Generally there are three types of data which are involved in GIS based analysis and mapping, namely **spatial data, attribute data** and **Meta data.**

Spatial datais also known as geospatial (Coordinate) data or geographic information/data. It is the data or information that identifies the geographic location of features and boundaries on earth, such as natural or constructed features, parcels, roads, buildings.

Attribute datais non geographic, tabular and descriptive data that GIS links to spatial data. Attribute data is collected and compiled for specific areas like states, cities, and so on and often comes packaged with map data.

Metadatais dataabout the data itself, which includes, the spatial reference system attached to the spatial data, accuracy, description about every attribute fields, and so on.

Spatial data models begin with a conceptualization and also there are rules which govern the view of real world phenomena or entities. GIS store information about the world as a collection of thematic layers that can be linked together by geography. This simple but extremely powerful and versatile concept has proven invaluable for solving many real-world problems from tracking delivery vehicles, to recording details of planning applications, to modeling global atmospheric circulation. The thematic layer approach allows us to organize the complexity of the real world into a simple representation to help facilitate our understanding of natural relationships.

Consider a road map suitable for use at a statewide or provincial level. This map is based on a conceptualization that defines roads as lines. These lines connect cities and towns that are shown as discrete points or polygons on the map. Road properties may include only the road type. Examples may include a limited access interstate, state highway, country road, or some other type of road. The roads have a width represented by the drawing symbol on the map. However this width, when scaled, may not represent the true road width. This conceptualization identifies each road as a linear feature that fits into a small number of categories. All state highways are represented by the same type of line, even though the state highways may vary. Some may be paved with concrete, others with bitumen. Some may have wide shoulders, others not, or dividing barriers of concrete, versus a broad vegetated median. We realize these differences can exist within this conceptualization.

Therefore, GIS represents real world objects (buildings, roads, land use, elevation) with digital data. Real world objects can be divided into two abstractions: discrete objects (a house) and continuous fields (elevation).
Two methods are used to reduce geographic phenomena to forms that can be coded in computer databases, and we call these **raster** and **vector.**

Fig.6.3 Thematic Layers

In principle, both can be used to code both fields and discrete objects, but in practice there is a strong association between raster and fields, and between vector and discrete objects.

Vector model

The first conceptualization defines discrete entities that may be represented by discrete objects, a vector data model. A farm field, road, wetland, cities and census tracts are examples of discrete entities that may be represented by discrete objects. A vector data model uses coordinates to store the shape of a spatial entity and associated attribute data to define discrete objects.

In vector data models, there are three vector object used to represent the geometry of real world entities, namely point, line and polygon. Groups of coordinates define the location and boundaries of these objects, and these coordinate data plus their associated attributes are used to represent the entities. In the vector world, the point is the building block from which all spatial entities are constructed. The smallest spatial entity, the point, is represented by a single (x, y) coordinate pair.

The vector representation using point, line or polygon depends up on the map scale and the extent of the work, since a city can be represented by a point in small scale map but polygon in large scale map which covers small area.

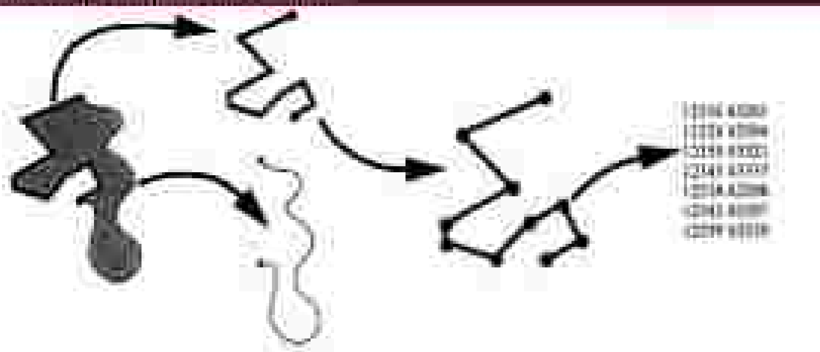


Fig.6.4 Relationships between points,lines and polygons

Raster model

Raster data models define the world as a regular set of cells in a grid pattern. The grid based square or rectangular cell is called pixel is organized in rows(from top to bottom) and columns(from left to right) where in each cell a single value of attribute which represents the phenomena or entities of interest is stored. These cells are usually evenly spaced in the x and y directions and the organizations are quite easy to code in an array structure in most computer languages.

Thus, individual cell is used as the building block for creating images of points, lines and areas in the raster world.

Theymight also represent photographic or scanned images. A raster cell stores a single value, it can be extended by using raster bands to represent RGB (Red, Green, Blue) colors, color maps (a mapping between a thematic code and RGB value), or an extended attribute table with one row for each unique cell value. A point is indicated with a single cell, a line by several cells with the same value forming a linear grouping, and an area by a clump of cells all having the same value.

Raster data models are the natural means to represent "continuous" spatial features or phenomena. Elevation, precipitation, slope, and pollutant concentration are examples of continuous spatial variables. These variables characteristically show significant changes in value over broad areas. The gradients can be quite steep (e.g., at cliffs), gentle (long, sloping ridges), or quite variable (rolling hills). Because raster data may be a dense sampling of points in two dimensions, they easily represent all variations in the changing surface. Raster data models depict these gradients by changes in the values associated with each cell.

<u>Raster data sets have a cell dimension, defining the size of the cell. The resolution of the raster dataset is its cell width in ground units. For example, in a Landsat TM raster image, each cell may be a pixel that represents an area of 30 meters by 30 meters.</u>

The figure below shows representation of the given reality both in raster and vector data models.

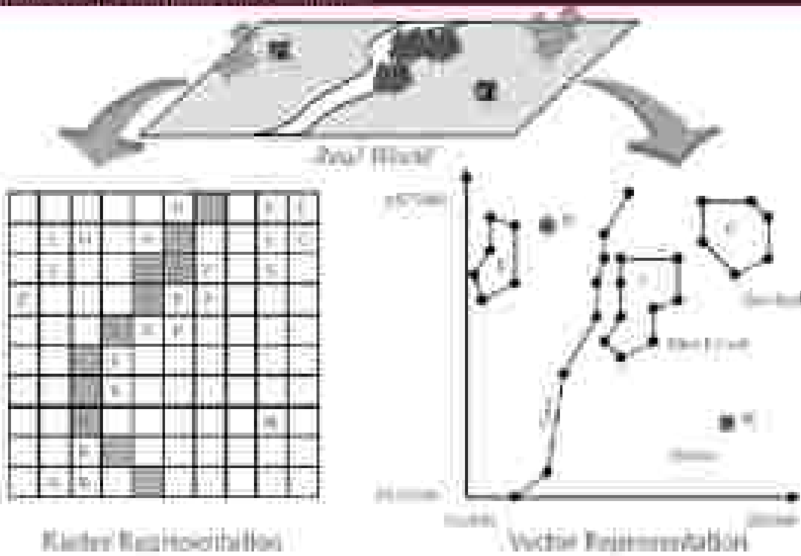


Fig.6.5 Raster and Vector representation of real world

Comparison of Vector and Raster

The question often arises, "which are better, raster or vector data models?" the answer is neither of both. Neither of the two classes of data models is better in all conditions or for all data. Both have advantages and disadvantages relative to each other and to additional, more complex data models. As an example, elevation may be represented as sets of contour lines in a vector data model or as a set of elevations in a raster grid. The choice often depends on a number of factors including the predominant type of data (discrete or continuous), the expected type of analysis, available storage, the main sources input data or the expertise of human operators.

Raster data models exhibit several advantages relative to vector data models. First raster data models are particularly suitable for representing themes or phenomena that change frequently in space. Each raster cell may contain a value different than its neighbors. Thus, trends as well as more rapid variability may be represented. Raster data structures are generally simpler, particularly when a fixed cell size is used.

Raster data models also facilitate easy overlays, at least relative to vector models. Each raster cell in a layer occupies a given position corresponding to a give location on the earth surface. Data in different locations align cell-to-cell over this position. Thus overlay involves locating the desired grid cell in each data layer comparing the values found for the given cell location. This operation is quite rapid in raster data structure.

Finally raster data structures are the most practical method for storing, displaying, and manipulating digital image data such as aerial photographs and satellite imagery.

Vector data models provide some advantages relative to raster data models. First, vector models generally lead to more compact data storage, particularly for discrete objects. Large homogenous regions are recorded by the coordinate boundaries in a vector data model, the same regions are recorded as a set of cells in case of raster. Vector data are more natural means for representing networks and other connected features. Vector data by their nature store information on intersections (nodes) and the linkage between them (lines).

Vector data models are easily presented in a preferred map format. Humans are familiar with continuous line and rounded curve representations in hand or machine drawn maps and vector based maps show these curves whereas raster data often show a "stair step" edge for curved boundaries, particularly when the cell resolution is large relative to the resolution at which the raster is displayed. Vector data models facilitate the calculation and storage of topological information. Topological information aids in performing adjacency, connectivity and other analysis in an efficient manner. Topological information also allows some forms of automated error and ambiguity detection, leading to improved data quality.

Triangulated Irregular Network (TIN)

TIN is a data model commonly used to represent terrain heights. It store GIS data for 3D surface model. Typically the x, y and z locations for measured points are entered into the TIN model. These points are distributed in space and the points may be connected in such a manner that the smallest triangle formed from any three points may be constructed. The TIN forms the connected network of triangles and therefore the basic unit is a triangle. Because a triangle consists of three lines connecting three nodes, each triangle will have three neighbours (except those on the side or periphery). The triangle is represented by a sequence of three nodes. Each triangle may have other associated attributes such as population density, crime rate, etc. in another table.

The TIN model usually uses some forms of indexing to connect neighbouring points. Each edge of a triangle connects to two points, which in turn each connect to other edges. These connections continue until the entire network is spanned. Thus, the TIN is a rather more complicated data model than the simplest raster grid when the objective is terrain representation

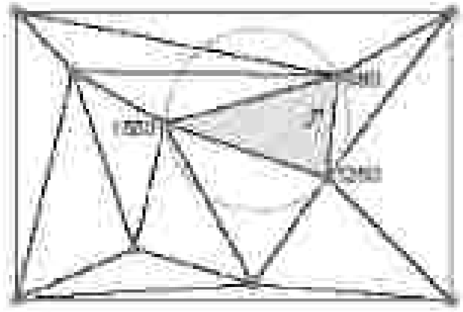


Fig.6.6. Triangulated irregular network data model

Data Capturing and Entry techniques in GIS

The first step of using a GIS is the provision of data. Much of the success of a GIS project depends on the quality of the data that is entered into the system, and thus this phase of a GIS project is critical and must be taken seriously.Data collection is one of the most time-consuming and expensive yet important, of GIS tasks.

Two main types of data capture are:

- **Primary data sources**are those collected in digital format specifically for use in a GIS project. Raster data capture includes remote sensing, aerial photograph, DEM. The advantage of such a GIS data capture is that there is a consistency in the data generated, and the whole process can be regularly and systematically assisted to get accuracy of the data at a very cost effective manner. Vector data capture refers ground surveying and GPS.Ground survey is a very time-consuming and expensive activity, but it is still the bestway to obtain highly accurate point locations.
- **Secondary data sources**are digital and analog datasets that were originally captured for another purpose and need to be converted into a suitable digital format for use in a GIS project. Paper maps, aerial photographs, and images are scanned (converting analogue map into a computer readable form) and georeferenced so that they provide geographic context for raster data source. Secondary vector data capture involves digitizing (converting raster data into vector data and the process called as Vectorization) vector objects from maps and other geographic data sources. Entry of coordinates using Coordinate geometry and conversion existing digital data are also another data capturing techniques

Data transfer: It involves importing digital data from other sources (E.g.: Internet, WAN, LAN and from physical media CD ROMS).

Attribute data can be entered by direct data loggers or manual keyboard entry.Data has to be entered and stored in a proper digital format in the computer to apply any GIS analysis.

Application areas of GIS

GIS can answer number of questions as long as it is based on spatial location. Condition can be accessed by answering what is....? Type of question, Where is....? Question is addressed by specifying the required event. Trends and patterns are also studied by analyzing temporal data.

Generally GIS is applicable in number of fields since everything is related to spatial location.

- ✓ Water Resource: - GIS provides calculations and methods for watershed characteristics, flow statistics, flood zoning and forecasting, flow direction, ground and surface water management, flow accumulation and facilitates the watershed delineation by using Digital Elevation Models (DEMs)
- ✓ Agriculture: - GIS is used in a variety of such as managing crop yields, monitoring crop rotation techniques, and projecting soil loss for individual farms or entire agricultural regions.
- ✓ Business: -GIS is a tool for managing business information by using the location. Business sites can be selected, sale territories can be optimized and customers location can be tracked
- ✓ Environmental and natural resource management: -GIS can be used to produce maps, inventory species, measure environmental impact, or trace pollutants, study geological features, analyze soil, access seismic information.
- ✓ **Engineering:** - GIS is a tool for traffic analysis and planning, topographic map preparation, location decision and suitability analysis, pavement management, route selection, landuse planning, urban planning, infrastructural mapping.
- ✓ **Risk management:** - GIS can help in risk management and analysis by showing which area will be subjected to natural and manmade disaster and prevention measures can be developed. e.g. soil erosion, flood, draught and diseases

  Thematic cartography: - GIS is used to show different even distributions in forms of maps, tables and charts.

- Tacheomertic is the branch of the surveying in which the horizontal distance between the instrument station to the staff station and also the vertical distance of a point are determine.
- Chaining operation is completely eliminated in this method.
- Less accurate as compare to chaining.

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Use of Tacheometry

- When obstacle (Step, broken ground, stretches of water)
- In rough country both horizontal and vertical measurement are tedious and chaining is inaccurate, difficult and slow.
- This method is used for find out the contour.

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Purposes of Tachometry

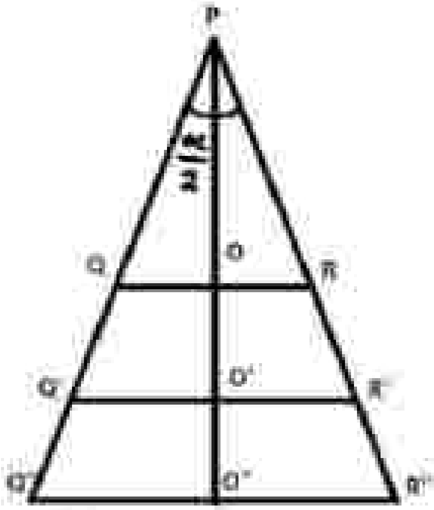
- Prepare contour map.
- Used in hydrographic survey.
- Location survey for road, railway, reservoir etc.
- Checking of the distance which measured with the help of the tap.
- To measure the horizontal distance at which the distance measured by the tap or chain is difficult.

Principle of Tacheometry

- The principle of techeometry is based on the property of isosceles triangle.

- Statement :-
- In isosceles triangle the ratio of the perpendiculars from the vertex on their bases

- Here; PQR, PQ'R', PQ''R'' are all isosceles triangle whose base are QR, Q'R' and Q''R'' and their vertex is at P. and here PO, PO' and PO'' are the perpendicular to their respective bases.



$$\frac{PO}{QR} = \frac{PO'}{Q'R'} = \frac{PO''}{Q''R''} = \textbf{constant K} = \mathbf{2\cot\frac{\alpha}{2}}$$

here constant K = $\frac{f}{i}$

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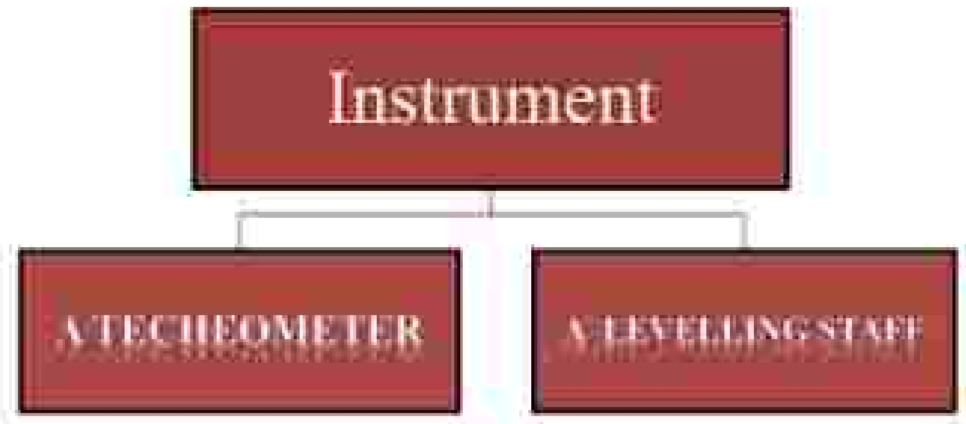
Instrument used



A Tacheometer

- A tacheomtry is usually transit theodolite having a stadia diaphragm.
- The diaphragm is equipped with two horizontal hairs called stadia hair in addition to regular cross hair.
- The additional hairs are equidistance from the central.
- The diaphragm commonly used in second slide.

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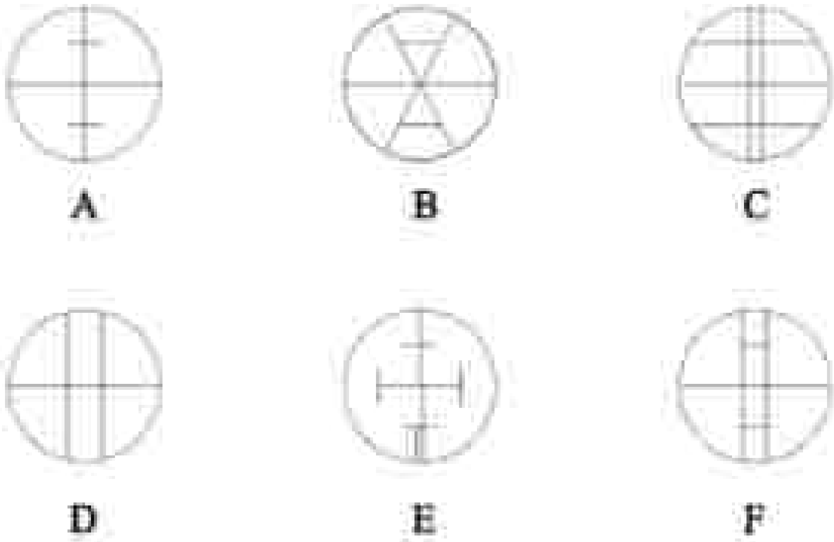

STADIA DIAPHRAGMS

Levelling Staff or Stadia Rod

- The stadia rod or staff used with tacheometry may be usual type of levelling staff having least count of 0.005m.
- Stadia rod is usually in one piece but for easy transport it may be folding.
- Width of the staff is 5cm to 15cm.
- Height may be 3m to 5m.
- It is graduated in meter, Centimeter.
- The graduation must be simple and clear.

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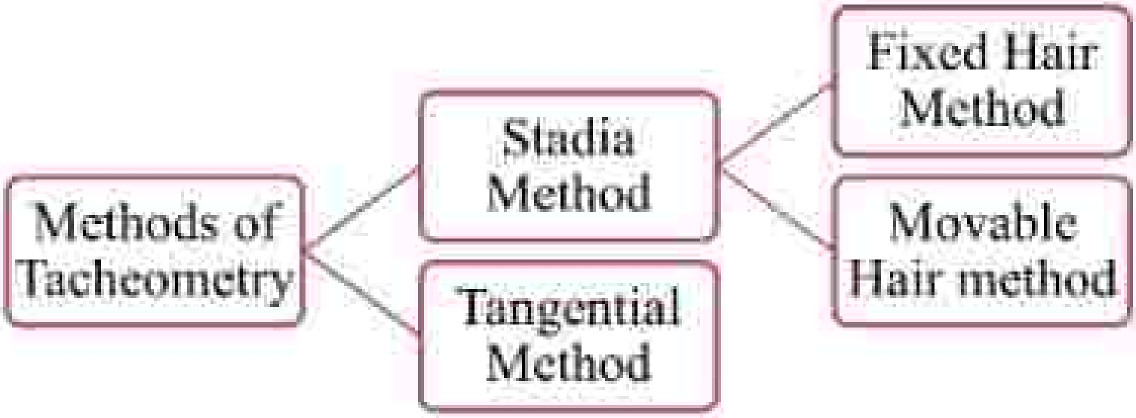
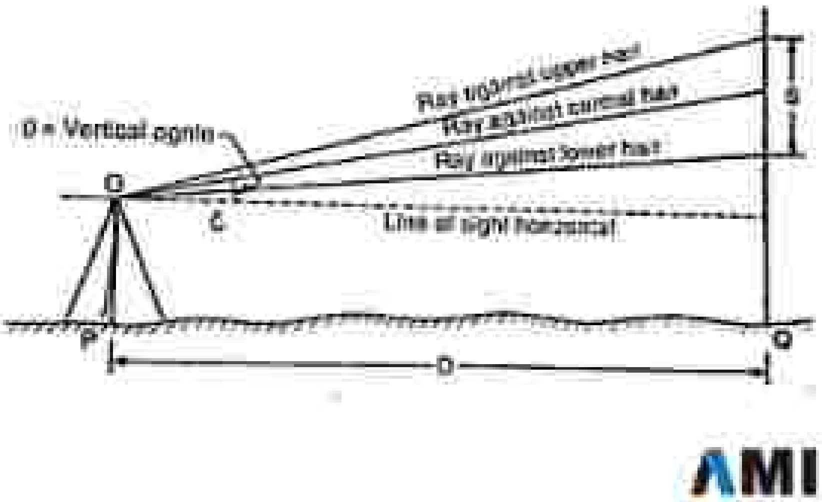
Methods of Tacheometry



Stadia Method

- In the stadia method, a tacheometry is setup at a station P and a staff is held at another station Q.



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- The staff intercept (S) between the upper stadia hair and the lower stadia hair is measured.
- The vertical angle (ϴ) is also measured.
- The horizontal distance D between P and Q, and the difference of elevation of P and Q is calculated from the staff intercept (S) and the vertical angle (ϴ) by using formula.

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Fixed hair method

- The upper and lower stadia hair is fixed. (stadia interval is fixed)
- The distance between the upper stadia hair and lower stadia hair, called stadia interval (i) is fixed.
- The value of the staff interval (S) varies with the distance.
- Generally stadia method means fixed hair method.

Movable hair method

- In this method the stadia hairs (i) is not fixed.
- Stadia hairs can be moved or adjusted by the micrometer screws.
- In this method the staff intercept (S) is fixed.
- The stadia interval measured corresponding to the staff intercept.

Difference

Fixed hair method

- Stadia interval (i) is fixed.
- Staff intercept (S) is not fixed.
- Fixed hair method is most commonly used to take staff reading speedy.
- Tacheometry and staff are used.

Movable hair method

- Stadia interval (i) is not fixed.
- Staff intercept (S) is fixed.
- This method is not generally used because unconvenient to measure the stadia interval accurately.
- Substance theodolite and staff are used.

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Tangential Method

- In this method diaphragm of the tacheometer is not provided with the stadia hair.
- Reading are taken by the central horizontal hair.
- Staff with two targets at a fixed distance (S) is used for taking reading.
- The vertical angles θ_1 & θ_2 are measured.
- The vertical angle and the fixed distance (S) are used to determine the horizontal distance (D).

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Difference

Stadia hair method	Tangential method
• Diaphragm of the tacheometer is provided with three stadia hair.	• Diaphragm of the tacheometer is not provided with stadia hair.
• Looking through the telescope the three stadia hair readings taken.	• The readings are taken by the single horizontal hair adjust upper and lower target respectively.
• One vertical angle is observed.	• Two vertical angle is observed.
• This method is most commonly used in practice.	• This method is not commonly used in practice

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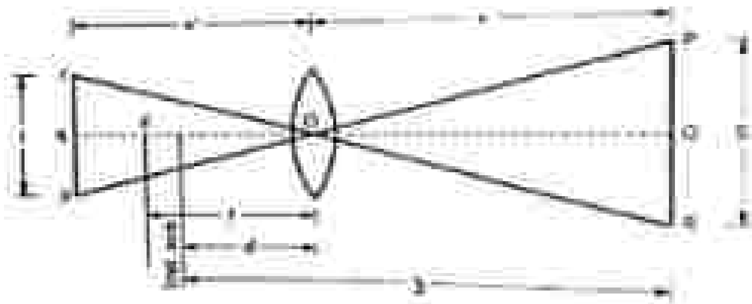
Fixed hair method

- There are main three cases for finding the distance and Elevation.
- Case : 1 When the line of sight is horizontal and staff is held Vertical.
- Case : 2 When the line of sight is inclined and staff is held Vertical. ((a) considering angle of elevation $+\theta$ (b) considering angle of depression $-\theta$)
- Case : 3 When the line of sight is inclined but staff is held normal to the line of sight.

Case : 1 When the line of sight is horizontal and staff is held Vertical.

Horizontal Distance Formula



- O = The optical center of the object glass.
- p,q,r = the top, axial, and bottom hair reading.
- pr = i = Length of the image.
- f = Focal length of the image glass.
- S = Staff in intercept on PQ.
- x = Horizontal distance from O to the staff.
- x' = Horizontal distance from O to the plane of the hairs.
- d = Horizontal distance from O to the vertical axis of the instrument.
- D = Horizontal distance from axis to the staff.

- The rays Pop and Qoq passing through O are the straight lines.
- Triangle POQ and poq are similar hence $\frac{x}{x'} = \frac{O}{I}$

But x and x' are conjugate focal length (distance)

$$\frac{1}{f} = \frac{1}{x'} + \frac{1}{x}$$

Multiplying both by fx

$$x = \frac{x}{x'} f + f$$

Substituting $\frac{x}{x'} = \frac{O}{I}$

$$x = \frac{O}{I} f + f$$

Add d on both the side

$$u + d = \frac{f}{i} s + f + d$$

<u>But u</u> $+ d = D$

$$D = \frac{f}{i} S + (f + d)$$

The constant $K = \frac{f}{i}$ is known as the multiplying constant or stadia interval factor and the constant $C = f + d$ is known as the additive stadia of the instrument.

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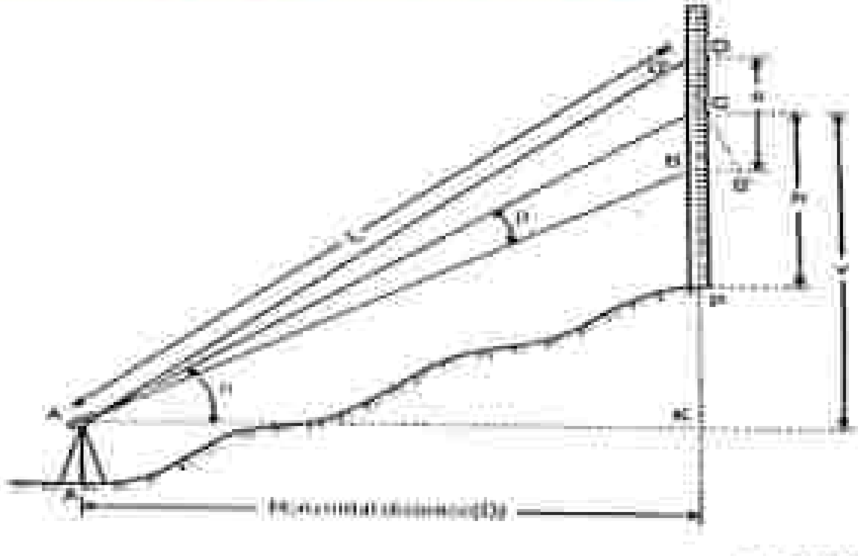
❑Vertical Distance formula

- When the line of sight is horizontal V = 0

Case : 2 When the line of sight is inclined and staff is held Vertical.

- Considering angle of elevation +θ



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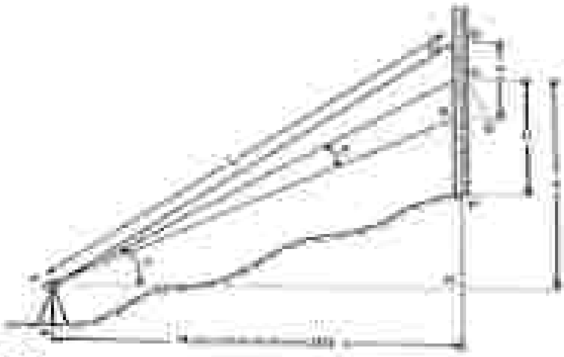
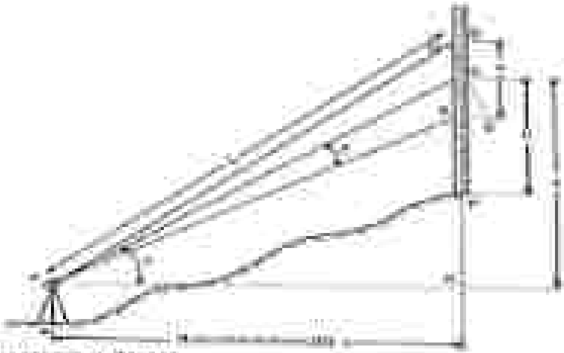


- Let A is the instrument station
- A' is the position of the instrument axis
- P is the staff station
- DBC are the points on the staff cut by the hair of the diaphragm.
- ∠CA'K = is an inclined of the line of sight A'C to the horizontal
- BD = S is the staff intercept (difference between the top and bottom hair reading)



- CP = h is the central hair or axial hair reading.
- A'C = L is the distance along the line of collimation from A' to C
- A'K = D is the horizontal distance from the instrumental to the staff station P
- CK = V is the vertical distance from the instrument axis to point C (Central hair reading)
- Draw a perpendicular line through C to the line of sight A'C so that it cuts A'D in D' and A'B in B' is the projection of DB perpendicular to A'C as shown in figure
- Line BD is perpendicular to the line A'K and B'D' is perpendicular to A'C

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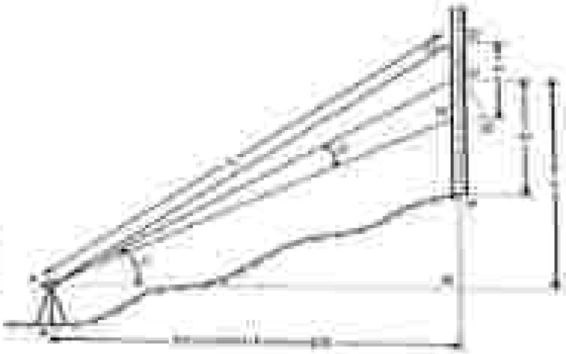
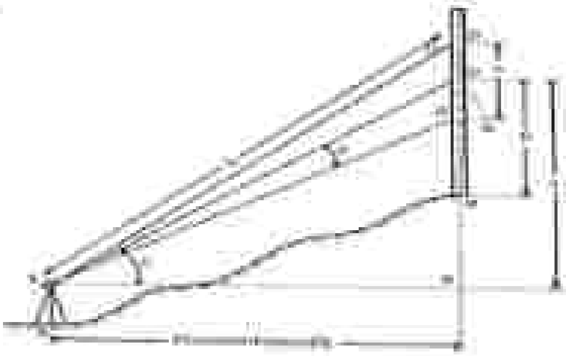


- $\angle DCD' = \angle BCB' =$ and $\angle DA'C = \angle BA'C = \beta$
- $\angle A'D'C = 90^0 - \beta$
- Angle $\angle DD'C = 180^0 - (90^0 - \beta)$
- $= 90^0 + \beta$
- Angle $\angle BB'C = 90^0 - \beta$

- From ΔS DD'C and BB'C
- D'C = DC Cosθ
- B'C = BC Cosθ
- D'C + B'C = DC Cosθ + BC Cosθ
- D'B' = (DC + BC) Cosθ
- D'B' = DB Cosθ
- D'B' = S Cosθ



Horizontal Distance D

Horizontal Distance D, When the line of sight is horizontal, then:

$$\mathbf{D = \frac{f}{i}(DB) + (f + d)}$$

Here DB = S

So,

$$\mathbf{D = \frac{f}{i}(S) + (f + d)}$$

Now inclined distance A'C = $\mathbf{L = \frac{f}{i}(D'B') + (f + d)}$

But here D'B' = S Cosθ

$$L = \frac{f}{i}(S\,Cos\theta) + (f + d)$$

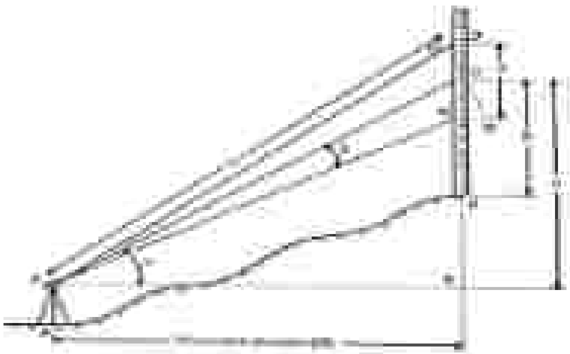
Horizontal distance $D = L\,Cos\theta$

$$D = L = \frac{f}{i}(S\,Cos\theta)(Cos\theta) + (f + d)(Cos\theta)$$

$$D = \frac{f}{i}S\,Cos^2\theta + (f + d)Cos\,\theta$$

Here $\frac{f}{i} = K$ and $(f + d) = C$

$$\mathbf{D = \frac{f}{i}S\,Cos^2\theta + (f + d)Cos\,\theta}$$



Vertical distance

From $\Delta A'CK$, $CK = V = L\ Sin\theta$

Put the value of $L = \frac{f}{i}(S\ Cos\theta) + (f + d)$

$V = \frac{f}{i}(S\ Cos\theta)(Sin\theta) + (f + d)(Sin\theta)$

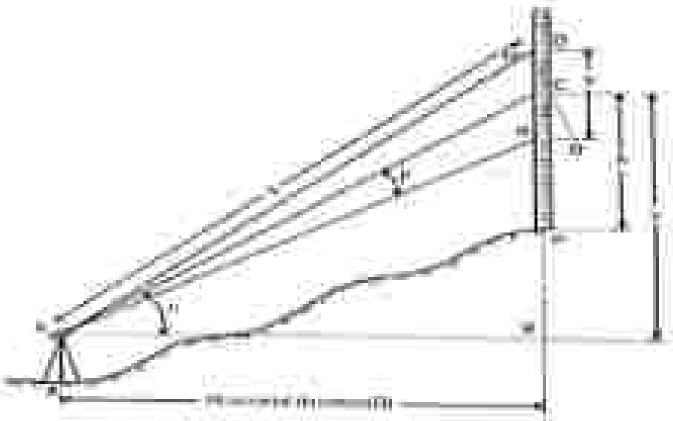
$V = \frac{f}{i}\frac{S\ Sin2\theta}{2} + (f + d)\ Sin\theta$

Here $\frac{f}{i} = K$ and $(f + d) = C$

So, $V = \frac{KS\ Sin2\theta}{2} + (C)\ Sin\theta$

* Elevation of the staff station for angle of elevation

- Elevation of staff station= Elevation of instrument + R.L. of B.M. + V- h



*** Elevation of the staff station for the angle of depression.**

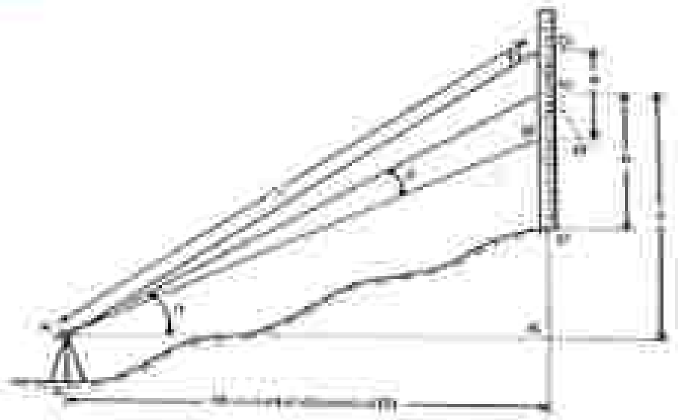
- Elevation of staff station= Elevation of instrument + R.L. of B.M. - V- h

- Horizontal Distance D:

$$D = \frac{f}{i} S \cos^2\theta + (f + d)\cos\theta$$

- Vertical Distance V

$$V = \frac{KS \sin 2\theta}{2} + (C)\sin\theta$$



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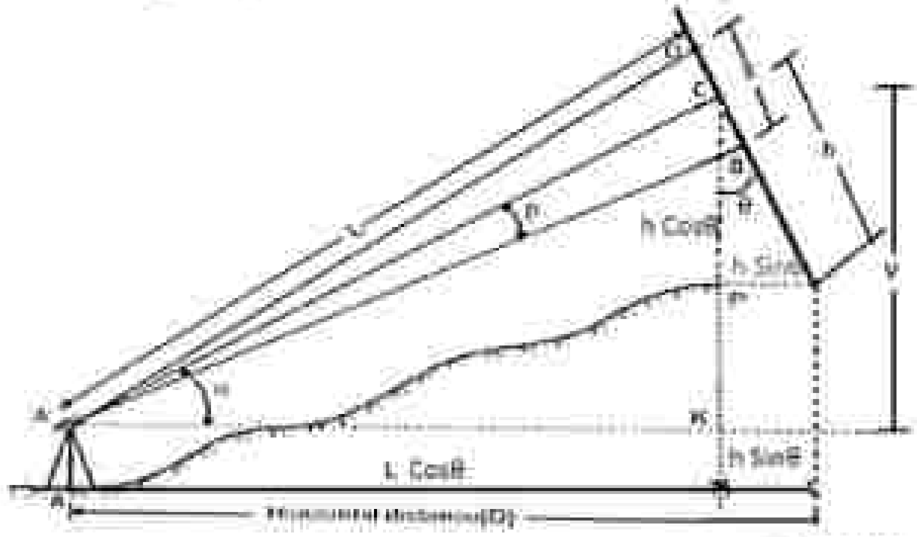
Case : 3 When the line of sight is inclined and staff is held normal to the line of sight.

Considering angle of Elevation +θ



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- Horizontal distance formula :-

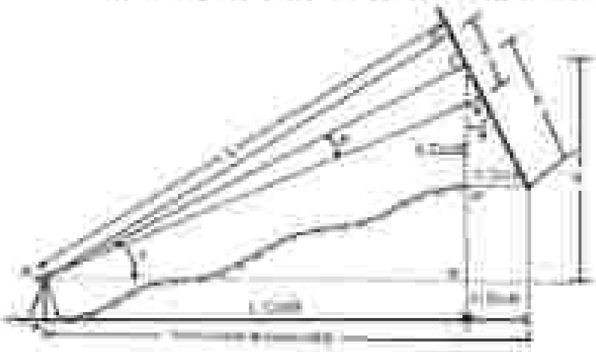
$$D = \frac{f}{i}(S) + (f + d)$$

From the figure the Horizontal distance D :-

$$D = L \cos\theta + h \sin\theta$$

$$= (KS + C) \cos\theta + h \sin\theta$$

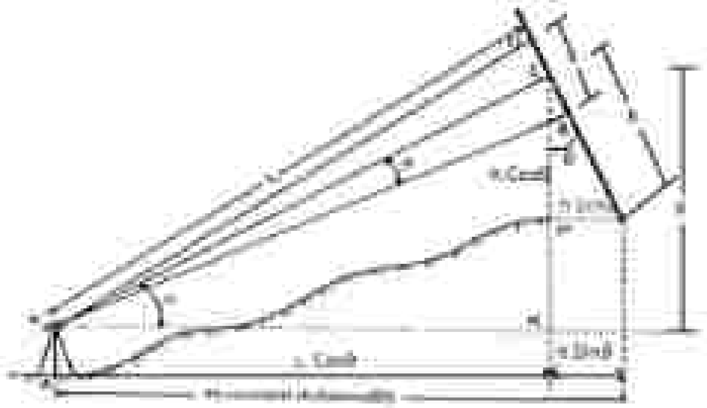
$$= KS \cos\theta + C \cos\theta + h \sin\theta$$



- Vertical distance formula :-
- Vertical distance $V = L \sin\theta$

$$= (KS + C)\sin\theta$$

$$= KS\sin\theta + C\sin\theta$$



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- Elevation of the staff station :-
- Elevation of staff station= Elevation of instrument + R.L. of B.M. + V- h Cosθ

- Considering angle of depression – θ
- Horizontal distance formula:-
- Horizontal

$$\text{distance } D = L\cos\theta - h\sin\theta$$
$$= (KS + C)\cos\theta - h\sin\theta$$
$$= KS\cos\theta + C\cos\theta - h\sin\theta$$

- Vertical distance formula :-

$$\text{Vertical distance } V = L \sin\theta$$
$$= (KS + C) \sin\theta$$
$$= KS \sin\theta + C \sin\theta$$

- Elevation of the staff station :-

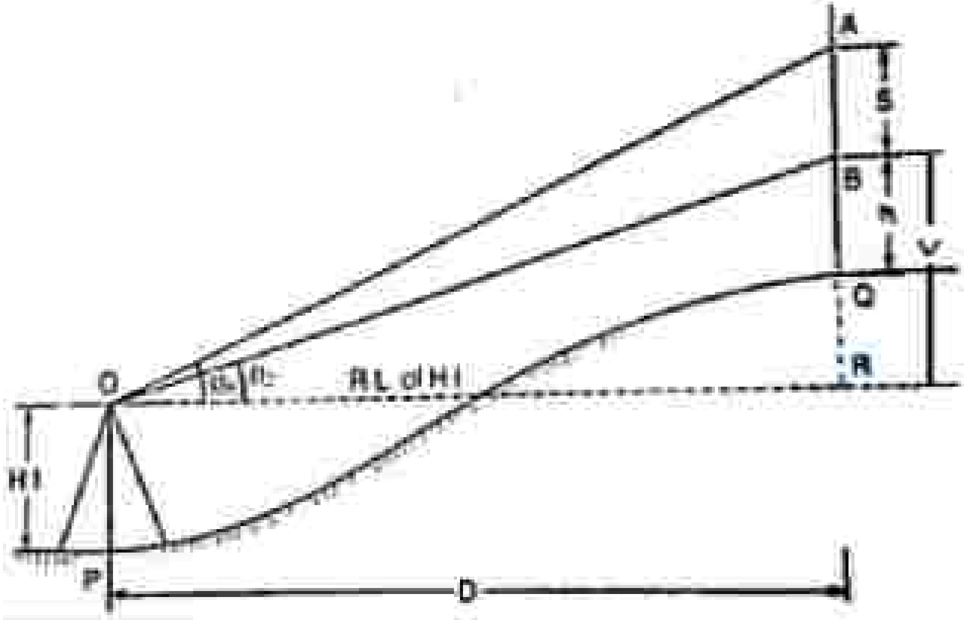
Elevation of staff station = Elevation of instrument + R.L. of B.M. - V- h $\cos\theta$

Tangential method

- This method is used only when the theodolite is simple and transit type.
- This method is also used when the staff is far away from the instrument.
- In this method the staff consist of two vanes or target (S) 2m to 3 m apart.
- The vertical angle θ_1 and θ_2 are measured in theodolite

- There are main three cases for finding the Distance and Elevation.
- Case : 1 Both the angle are angles of elevation in this case, staff is held vertically.
- Case : 2 Both the angle are angles of depression in this case, staff is held vertically.
- Case : 3 When the one angle is the angle of elevation and the another angle is the angle of depression and the staff held vertical.

Case : 1 Both the angle are angles of elevation in this case, staff is held vertically.



- From the fig.

$V + S = D \tan\theta_1$

$V = D \tan\theta_2$

$S = D \tan\theta_1 - V$

$S = D \tan\theta_1 - D \tan\theta_2$

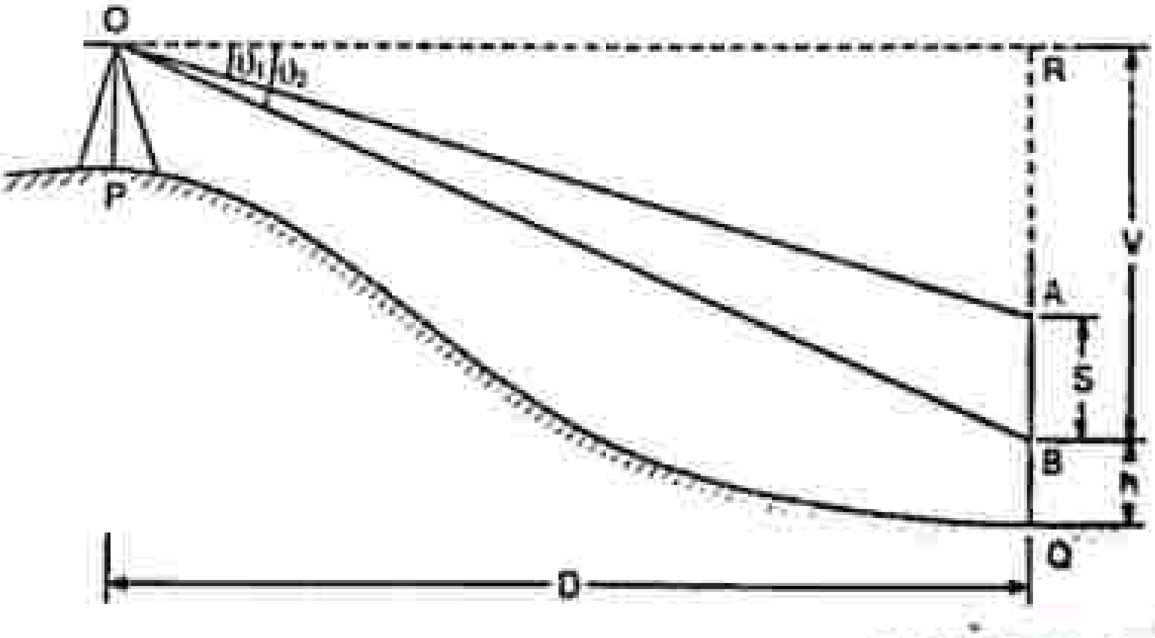
$S = D (\tan\theta_1 - \tan\theta_2)$

$$D = \frac{S}{(\tan\theta_1 - \tan\theta_2)}$$

$$V = \frac{S \tan\theta_2}{(\tan\theta_1 - \tan\theta_2)}$$

R.L of Q = R.L of H.I + V - h

Case : 2 Both the angle are angles of depression in this case, staff is held vertically.



- From the fig.

  $V - S = D \tan\theta_1$

  $V = D \tan\theta_2$

  $S = V - D \tan\theta_1$

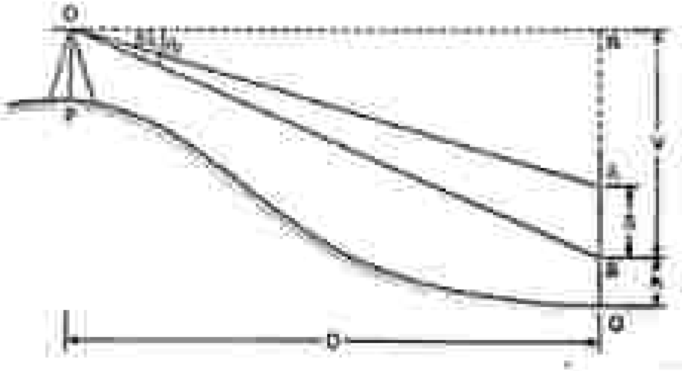
  $S = D \tan \theta_2 - D \tan \theta_1$

  $S = D (\tan\theta_2 - \tan\theta_1)$

  $$D = \frac{S}{(\tan\theta_2 - \tan\theta_1)}$$

  $$V = \frac{S \tan\theta_2}{(\tan\theta_2 - \tan\theta_1)}$$

R.L of Q = R.L of H.I - V - h



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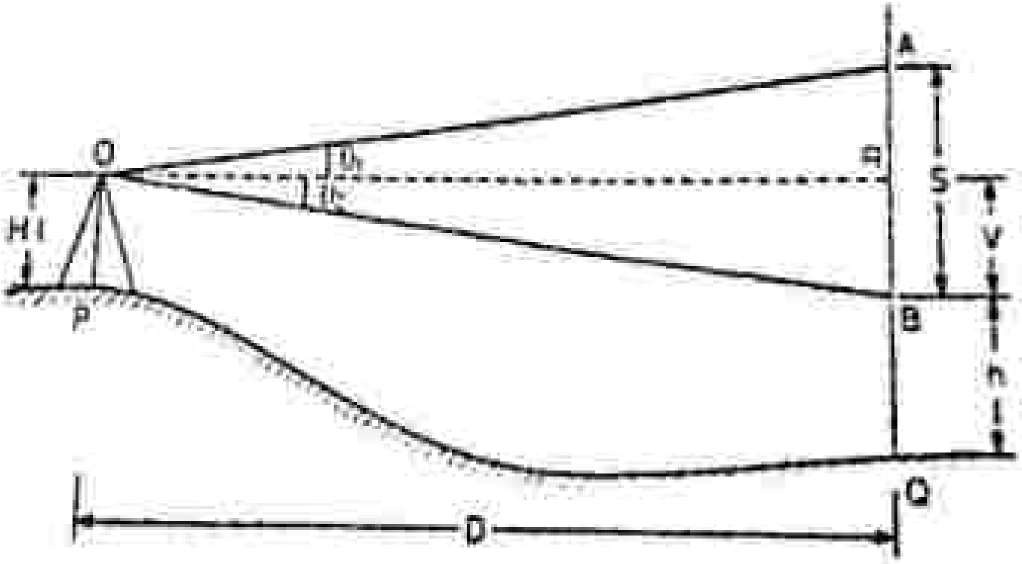
Case : 3 When the one angle is the angle of elevation and the another angle is the angle of depression and the staff held vertical.



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- From the fig.

$S - V = D \tan\theta_1$

$V = D \tan\theta_2$

$S = V + D \tan\theta_1$

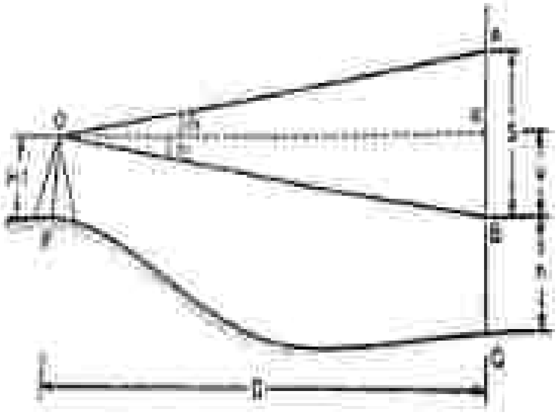
$S = D \tan \theta_2 + D \tan \theta_1$

$S = D (\tan\theta_2 + \tan\theta_1)$

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

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

R.L of Q = R.L of H.I - V - h



Disadvantages of the tangential method

- Two vertical angles are measured.
- It require comparatively more time.
- This method is very tedious.

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