KARPAGAM ACADEMY OF HIGHER EDUCATION

(Deemed to be University)

(Established Under Section 3 of UGC Act 1956)

Coimbatore- 641 021

(For the candidates admitted from 2016 onwards)

DEPARTMENT OF CIVIL ENGINEERING

SUBJECT CODE: 18BECE443 **SUBJECT: SURVEYING AND GEOMATICS**

SEMESTER:IV **CLASS: II Civil Engineering** **L T P C = 3 0 0 3**

Course Objectives

a) To describe the function of surveying in civil engineering construction, work with survey observations, and perform calculations,

b) Customary units of measure. Identify the sources of measurement errors and mistakes; understand the difference between accuracy and precision as it relates to distance, differential leveling, and angular measurements,

c) Identify and calculate the errors in measurements and to develop corrected values for differential level circuits, horizontal distances and angles for open or closed-loop traverses,

d) Operate an automatic level to perform differential and profile leveling; properly record notes; mathematically reduce and check leveling measurements,

e) Measure horizontal, vertical, and zenith angles with a transit, theodolite, total station instruments

f) Operate a total station to measure distance, angles, and to calculate differences in elevation. Reduce data for application in a geographic information system,

g) Calculate, design and layout horizontal and vertical curves, Understand, interpret, and prepare plan, profile, and cross-section drawings, Work with cross-sections and topographic maps to calculate areas, volumes, and earthwork quantities.

UNIT-I: *Introduction to Surveying (8 hours):* Principles, Linear, angular and graphical methods, Survey stations, Survey lines- ranging, Bearing of survey lines, Levelling: Plane table surveying, Principles of levelling- booking and reducing levels; differential, reciprocal leveling, profile levelling and cross sectioning. Digital and Auto Level, Errors in levelling; contouring: Characteristics, methods, uses; areas and volumes.

UNIT-II: *Triangulation and Trilateration*: Theodolite survey: Instruments, Measurement of horizontal and vertical angle; Horizontal and vertical control - methods -triangulation - network-Signals. Baseline - choices - instruments and accessories - extension of base lines - corrections - Satellite station - reduction to centre - Intervisibility of height and distances - Trigonometric leveling - Axis single corrections.

Curves-Elements of simple and compound curves – Method of setting out– Elements of Reverse curve - Transition curve – length of curve – Elements of transition curve - Vertical curves

UNIT-III: *Modern Field Survey Systems* : Principle of Electronic Distance Measurement, Modulation, Types of EDM instruments, Distomat, Total Station – Parts of a Total Station – Accessories –Advantages and Applications, Field Procedure for total station survey, Errors in Total Station Survey; Global Positioning Systems- Segments, GPS measurements, errors and biases, Surveying with GPS, Co-ordinate transformation, accuracy considerations.

UNIT-IV: *Photogrammetry Surveying*: Introduction, Basic concepts, perspective geometry of aerial photograph, relief and tilt displacements, terrestrial photogrammetry, flight planning; Stereoscopy, ground control extension for photographic mapping- aerial triangulation, radial triangulation, methods; photographic mapping- mapping using paper prints, mapping using stereoplotting instruments, mosaics, map substitutes.

UNIT-V: *Remote Sensing*: Introduction –Electromagnetic Spectrum, interaction of electromagnetic radiation with the atmosphere and earth surface, remote sensing data acquisition: platforms and sensors; visual image interpretation; digital image processing.

Text/Reference Books:

1. Madhu, N, Sathikumar, R and Satheesh Gobi, Advanced Surveying: Total Station, GIS and Remote Sensing, Pearson India, 2006.
2. Manoj, K. Arora and Badjatia, Geomatics Engineering, Nem Chand & Bros, 2011
3. Bhavikatti, S.S., Surveying and Levelling, Vol. I and II, I.K. International, 2010
4. Chandra, A.M., Higher Surveying, Third Edition, New Age International (P) Limited, 2002.
5. Anji Reddy, M., Remote sensing and Geographical information system, B.S. Publications, 2001.
6. Arora, K.R., Surveying, Vol-I, II and III, Standard Book House, 2015.

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DEPARTMENT OF CIVIL ENGINEERING

LECTURE PLAN

SURVEYING AND GEOMATICS (18BECE443)

LECTURER : Ms. S Solai Mathi
SEMESTER : IV (2019-2020) Even
NUMBER OF CREDITS : 3
COURSE TYPE : Regular Course

S.No	Hours	Topics to be Covered	Text Book	Page No.
UNIT I- INTRODUCTION TO SURVEYING				
1.	1	Principles, Linear, angular and graphical methods, Survey stations, Survey lines- ranging, Bearing of survey lines	T1	4, 85,46,49
2.	1	Plane table surveying	T1	195-195
3.	1	Principles of levelling- booking and reducing levels; differential, reciprocal leveling,	T1	216
4.	2	Profile levelling and cross sectioning	T1	233,237
5.	2	Digital and Auto Level, Errors in levelling; contouring: Characteristics, methods, uses, areas and volumes.	T1	257,259,260,267,315
6.	1	Theodolite survey: Instruments, Measurement of horizontal and vertical angle; Horizontal and vertical control	T1	137-150
7.	1	Baseline - choices - instruments and accessories - extension of base lines - corrections	T1	237,240,210,242
8.	1	Satellite station - reduction to centre	T1	263
9.	2	Intervisibility of height and distances	T1	225
10.	1	Trigonometric leveling - Axis single corrections	T2	131,150
				Total 13 Hours
UNIT II- CURVES				
11.	2	Elements of Simple curves	T2	4
12.	1	Elements of Compound curves	T2	47
13.	1	Methods of Setting out	T2	18
14.	1	Elements of reverse curve	T2	58
15.	2	Transition curve and elements of transition curve	T2	67,81
16.	1	Length of curve	T2	71
17.	1	Vertical curves	T2	111
				Total 09 Hours
UNIT-III- MODERN METHODS OF SURVEYING				
18.	1	Principle of Electronic Distance Measurement	T2	601
19.	1	Modulation, Types of EDM instruments	T2	606
20.	1	Distomat	T2	611
21.	1	Total Station – Parts of a Total Station – Accessories –Advantages and Applications	T2	618
22.	1	Field Procedure for total station survey	W5	W5
23.	1	Errors in Total Station Survey	W4	W4
24.	1	Global Positioning Systems- Segments	W1	W1
25.	1	GPS measurements, errors and biases	W1	W1
26.	1	Surveying with GPS, Co-ordinate transformation,	W6	W6

		accuracy considerations		
				Total 09 Hours
		UNIT-IV-PHOTOGRAMMETRIC SURVEYING		
27.	1	Introduction, Basic concepts, perspective geometry of aerial photograph	T2	523,524
28.	1	Relief and tilt displacements, terrestrial photogrammetry	T2	546,556
29.	1	Flight planning	T2	561
30.	1	Stereoscopy	T2	578
31.	1	Ground control extension for photographic mapping	T2	569
32.	1	Aerial triangulation, radial triangulation	T2	536
33.	1	Methods of radial triangulation	T2	570
34.	1	Mapping using paper prints, mapping using stereo plotting instruments	T2	597
35.	1	Mosaics, map substitutes	T2	597
				Total 09 Hours
		UNIT-V- REMOTE SENSING		
36.	1	Introduction	T2	623
37.	1	Electromagnetic Spectrum	T2	626
38.	2	Interaction of electromagnetic radiation with the atmosphere and earth surface	T2	631
39.	1	Remote sensing data acquisition	T4	70
40.	1	Platforms and sensors	T2	632,635
41.	2	Visual image interpretation	T3	124
42.	1	Digital image processing	T3	157
				Total 09 hours
				Total 49 hours

TEXT BOOKS:

Sl.No	**Title of Book**	**Author of Book**	**Publisher**	**Year of Publishing**
1	Surveying I	B.C Punmia, Ashok K Jain, Arun K Jain	Laxmi Publications	2005
2	Surveying II	B.C Punmia, Ashok K Jain, Arun K Jain	Laxmi Publications	2005
3.	Remote Sensing and Geographical Information System	Anji Reddy M	B.S Publications	2001
4.	Remote Sensing for the beginner	Pradip Kumar Guha	Affiliated East-West Press	2003

REFERENCES:

Sl. No	Title of Book	Author of Book	Publisher	Year of Publishing
1	Advanced Surveying: Total Station, GIS and Remote Sensing	Madhu, N, Sathikumar, R and Satheesh Gobi	Pearson	2006
2	Geomatics Engineering	Manoj, K. Arora and Badjatia	Nem Chand & Bros	2011
3	Surveying and Levelling, Vol. I and II	Bhavikatti, S.S	I.K. International	2010
4	Higher Surveying, Third Edition	Chandra, A.M	New Age International (P) Limited	2002
5	Surveying, Vol-I, II and III	Arora, K.R	Standard Book House,	2015

LIST OF WEBSITES:

1. http://nceg.uop.edu.pk/workshop-21-26-09/GIS_for_Beginners/Main_GIS_06.pdf
2. http://www.gisresources.com/total-station-and-its-applications-in-surveying/
3. https://www.state.nj.us/transportation/eng/documents/survey/Chapter7.shtm
4. http://www.gisresources.com/total-station-errors/
5. https://tmackinnon.com/2005/PDF/Total-Station-basics--Introduction-to-Using-the-Leica-Total-Station.pdf
6. https://www.civil.iitb.ac.in/~dhingra/ce152_files/ce152_MNK.pdf

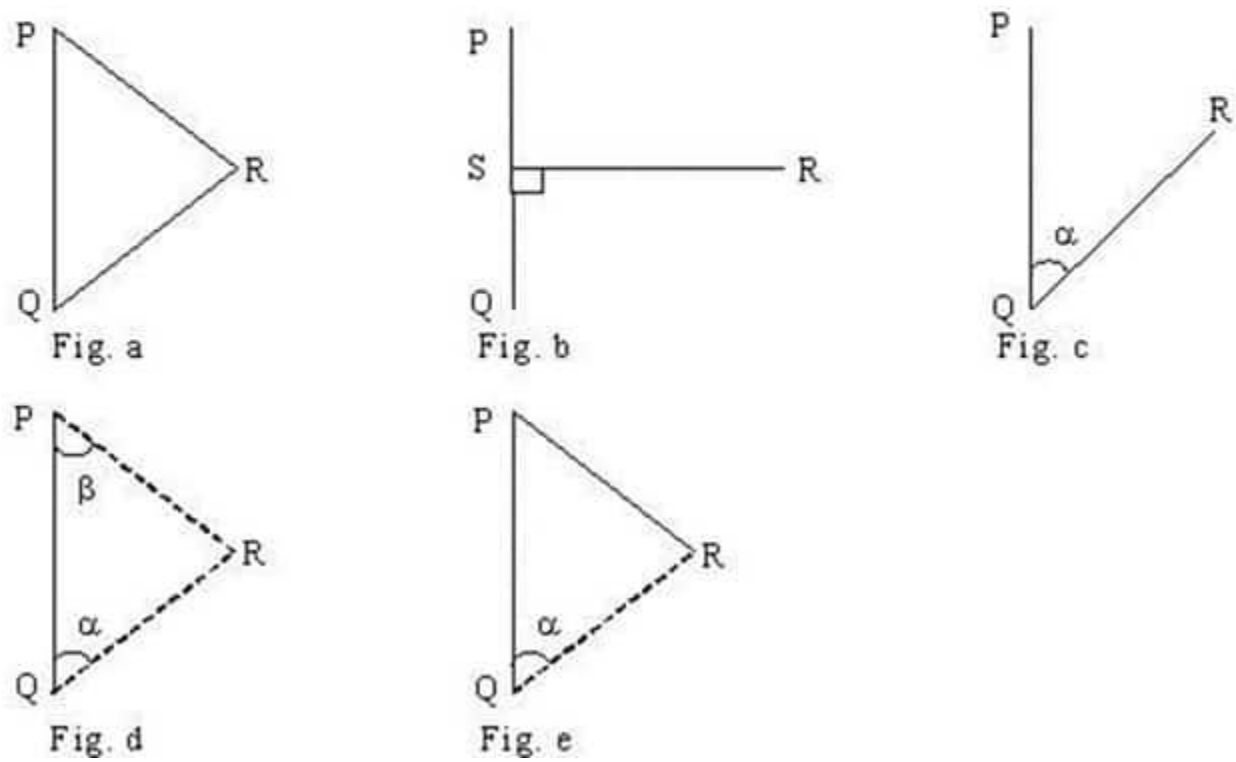
Unit 1
Introduction to Surveying

<u>Principles of Surveying</u>

The fundamental principle upon which the various methods of plane surveying are based can be stated under the following two aspects.

Location of a point by measurement from two points of reference

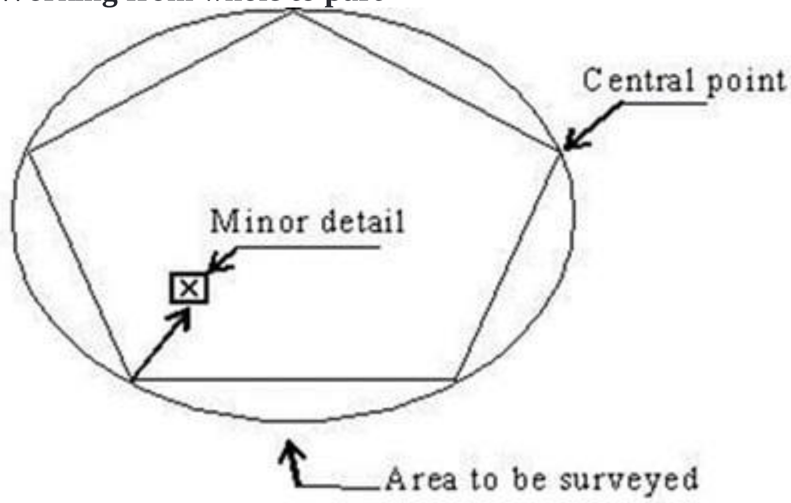
According to this principle, the relative position of a point to be surveyed should be located by measurement from at least two points of reference, the positions of which have already been fixed.



If P and Q are the two reference points on the ground, any other point, such as R, can be located by any of the direct methods shown in the above figures. But, although a single method is sufficient to locate the relative position of 'R' with respect to reference points P and Q, it is necessary to adopt at least any two methods to fix the position of point 'R'.

While the measurements made in the either of the first method or second method will be helpful in locating the point 'R', the measurements made in the other method will act as a check.

Working from whole to part



According to this principle, it is always desirable to carryout survey work from whole to part. This means, when an area is to be surveyed, first a system of control points is to be established covering the whole area with very high precision. Then minor details are located by less precise methods.

The idea of working this way is to prevent the accumulation of errors and to control and localize minor errors which, otherwise, would expand to greater magnitudes if the reverse process is followed, thus making the work uncontrolled at the end.

Method of Surveying in Civil Engineering
Primary types of Surveying are:

- Plane surveying
- Geodetic surveying

1. Plane surveying
Plane surveying is conducted by state agencies as well as private agencies. As we know earth is spherical in shape but its diameter is big enough to consider plane in small dimensions. It is that type of surveying in which the mean surface of the earth is considered as a plane and the spheroidal shape is neglected. All triangles formed by survey lines are considered as plane triangles. The level line is considered as straight and plumb lines are considered parallel. Plane surveying is done of the area of survey is less than 250 km^2.
2. Geodetic surveying
Geodetic survey is conducted by survey department of the country. It is that type of surveying in which the curved shape of the earth is taken in to account. The object of geodetic survey is to determine the precise position on the surface of the earth, of a system of widely distant points which form control stations in which surveys of less precision may be referred. . Line joining two points is considered as curved line and angles are assumed as spherical angles. It is carried out if the area exceeds over 250 km^2.
Secondary classification of Surveying

Surveys may be classified based on the nature of the field of survey, object of survey and instruments used.

1) Surveying based on Nature of Survey
a) Topographical Surveys
They are carried out determine the position of natural features of a region such as rivers, streams, hills etc. and artificial features such as roads and canals. The purpose of such surveys is to prepare maps and such maps of are called topo-sheets.

b) Hydrographic Survey
Hydro-graphic survey is carried out to determine M.S.L. (Mean Sea Level), water spread area, depth of water bodies, velocity of flow in streams, cross-section area of flow etc.

c) Astronomical Survey
The Astronomical Survey is carried out to determine the absolute location of any point on the surface of earth. The survey consists of making observations to heavenly bodies such as stars.

d) Engineering Survey
This type of survey is undertaken whenever sufficient data is to be collected for the purpose of planning and designing engineering works such as roads, bridges and reservoirs.

e) Archeological Survey
This type of survey is carried out to gather information about sites that are important from archeological considerations and for unearthing relics of antiquity.

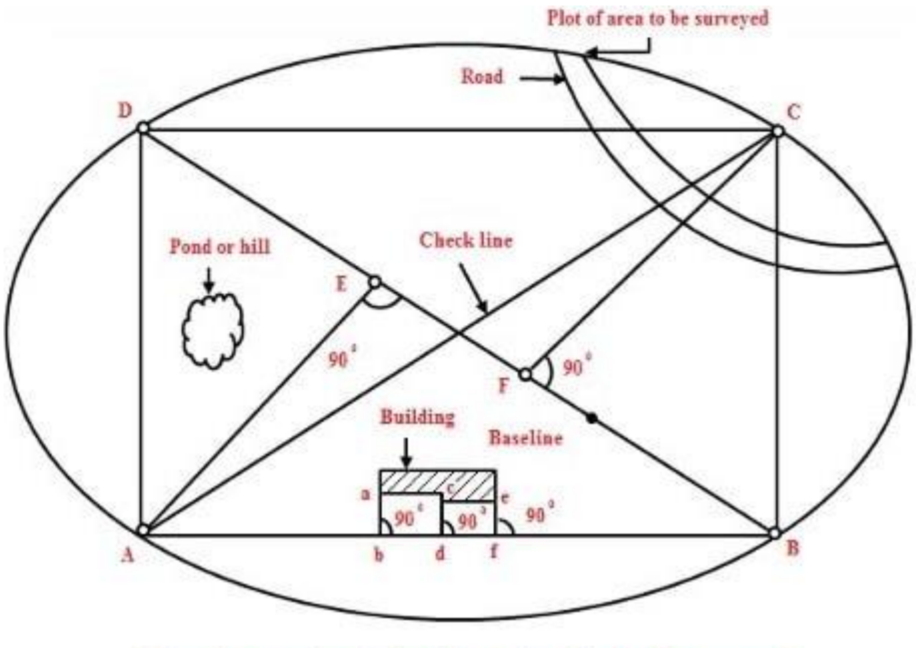
f) Photographic Survey
In this type of survey, information is collected by taking photographs from selected points using a camera.

g) Aerial Survey
In this type of survey data about large tracks of land is collected by taking photographs from an aero-plane.

h) Reconnaissance Survey
In this type of survey, data is collected by marking physical observation and some measurements using simple survey instruments.

2) Surveying based on Type of Instruments
a) Chain Surveying
Chain surveying is the basic and oldest type of surveying. The principle involved in chain survey is of triangulation. The area to be surveyed is divided into a number of small triangles. Angles of triangles must not be less than 30 degree and greater than 120 degree. Equilateral triangles are considered to be ideal triangles. No angular measurements are taken, tie line and check lines control accuracy of the work.

This method is suitable on level ground with little undulations and area to be survey is small.



A simple networks of triangle in a plot of land to be surveyed

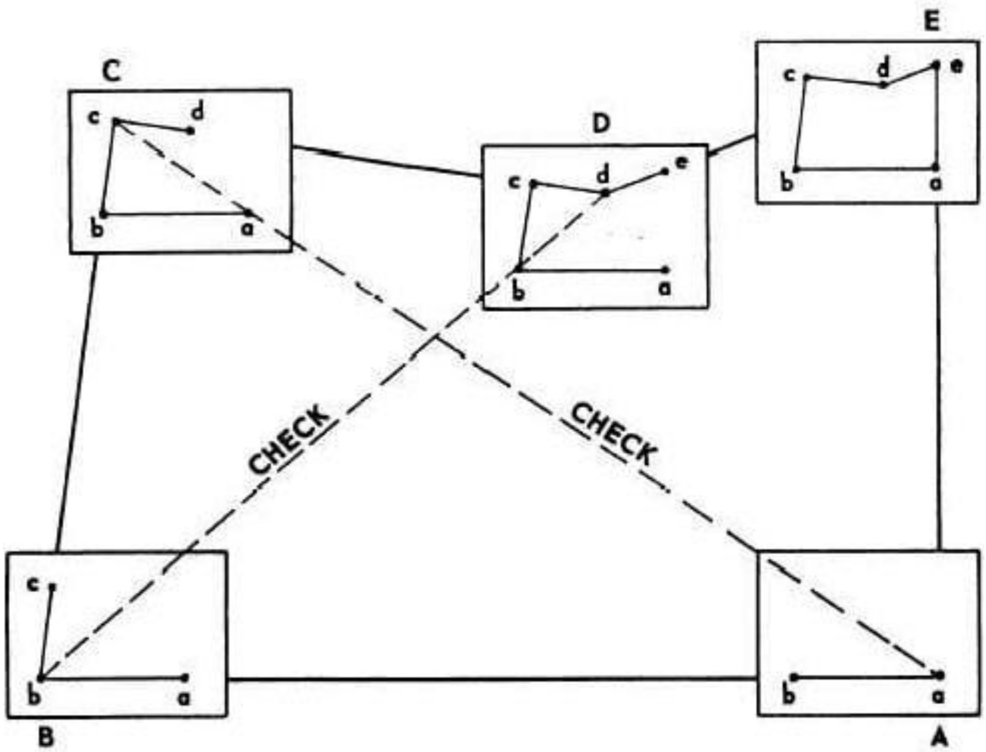
b) Compass Surveying

Compass survey uses the principle of traversing. This method does not requires the need to create triangles. It uses a prismatic compass for measuring magnetic bearing of line and the distance is measured by chain. A series of connecting lines is prepared using compass and measuring distances using chain. Interior details are located using offset from main survey lines.

They suitable for large area surveying crowded with many details. It can be used to survey a river course.

c) Plane Table Surveying
The principle of plane table survey is parallelism. They are plotted directly on paper with their relative position. The rays are drawn from station to object on ground. The table is placed at each of the successive station parallel to the position of the last station.

They are basically suitable for filling interior detailing and is recommended when great accuracy is not required.

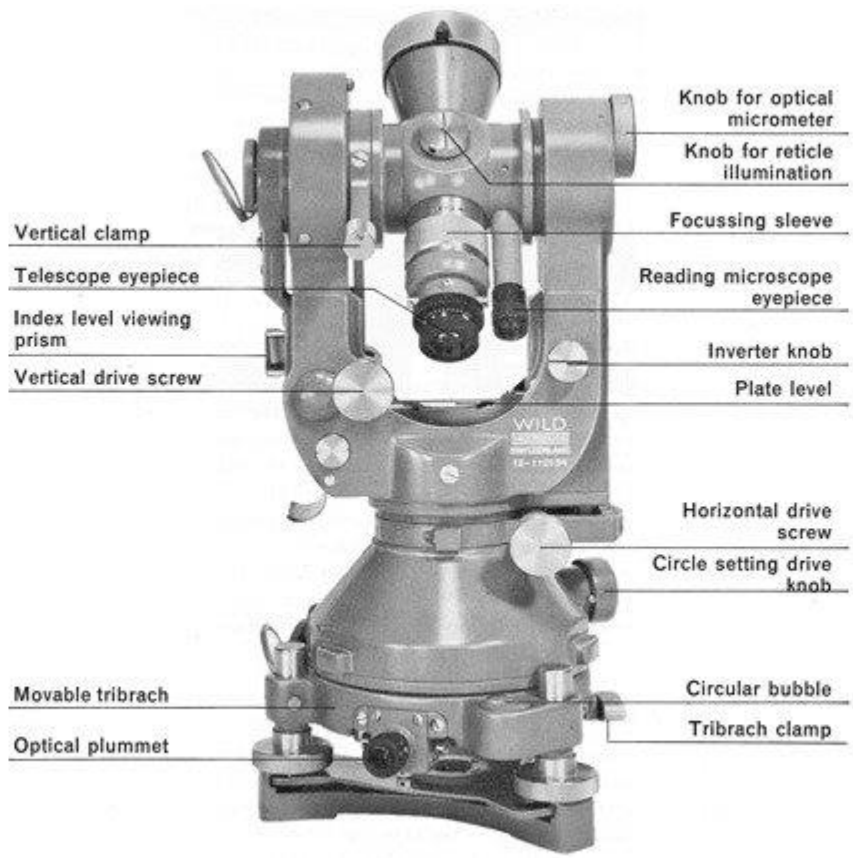


d) Theodolite Surveying
The theodolite is an instrument used mainly for accurate measurement of the horizontal and vertical angles. They are accurate to measure up to 10″ or 20″ angles.

Theodolite can be used to measure:

- Horizontal angles
- Vertical angles
- Deflection angle
- Magnetic bearing
- Horizontal distance between two points
- Vertical height between two points
- Difference in elevation

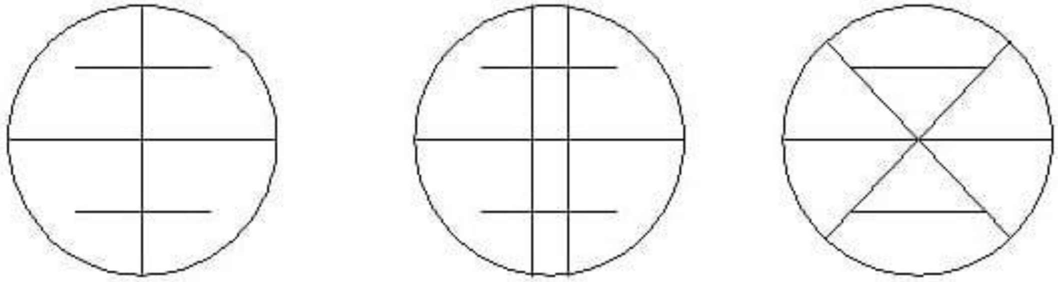
Nowadays theodolite is shadowed and replaced by the use of Total Station which can perform the same task with greater ease and accurate results



e) Tacheometric Surveying

Tacheometry is a branch of surveying in which horizontal and vertical distances are determined by taking angular observations with an instrument known as a tacheometer. Tacheometer is nothing but a transit theodolite fitted with a stadia diaphragm and an anallatic lens. There is no need for chaining in such survey. The principle of Tacheometer is based on property of isosceles triangle, where ratio of the distance of the base from the apex and the length of the base is always constant.

Different form of stadia diaphragm commonly used:



f) Photographic Surveying
Photographic survey is based on technique of taking photographs from different angle to prepare topographic details with relative high speed.

There are two type of photographic surveying

i). Terrestrial or ground photogrammetry
In terrestrial photogrammetry maps are prepared from ground photographs from different points on the earth surface for measurement purpose.

ii). Aerial photogrammetry
In aerial photogrammetry maps are produced from air from an airplane or helicopter.

Photogrammetry encompasses two major area of specialization.

- Metrical photogrammetry
- Interpretive photogrammetry

Metrical photogrammetry is of principal interest to surveyors since it is applied to determine distances, elevations, areas, volume, etc. to compile topographic maps made from measurements on photographs.

Intuitive photogrammetry involves objects from their photographic image and their significance. Critical factors considered in identifying object of shape, sizes, patterns, shadow.

Levelling:

Levelling or **leveling** is a branch of surveying, the object of which is to establish or verify or measure the height of specified points relative to a datum. It is widely used in cartography to measure geodetic height, and in construction to measure height differences of construction artifacts.

Principle of leveling

The principle of levelling is to obtain horizontal line of sight with respect to which vertical distances of the points above or below this line of sight are found.

What is Profile levelling?
Profile leveling is one of the most common applications of running levels and vertical distance measurement for the surveyor. The results are plotted in the form of a profile, which is a drawing that shows a vertical cross section. Profiles are required for the design and construction of roads, curbs, sidewalks, pipelines, etc. In short, profile leveling refers to the process of determining the elevation of points on the ground at mostly uniform intervals along a continuous line.

Equipment used for profile leveling

1. Dumpy level
2. Leveling staff
3. Tripod
4. Staff bubble
5. Chain or Tape

Procedure for profile leveling

1.Longitudinal levelling

Profile leveling is essentially the same as benchmark leveling, with one basic difference. At each instrument position, where an HI is determined by a back sight rod reading on a benchmark or turning point, several additional foresight readings may be taken on as many points as desired. These additional readings are called rod shots, and the elevation of all those points is determined by subtracting the rod shot from the HI at that instrument location. (See figure 1)

Plotting the Profile

The profile drawing is basically a graph of elevations, plotted on the vertical axis, as a function of stations, plotted on horizontal axis. A gridded sheet called profile paper is used to plot the profile data from the field book. All profile drawings must have a proper title block, and both axes must be fully labeled with stations and elevations.

The elevation or elevation scale is typically exaggerated; that is, it is 'stretched' in comparison to the horizontal scale. For example the vertical scale might be 10 times larger. The horizontal line at the bottom of the profile does not necessary have to start at zero elevation

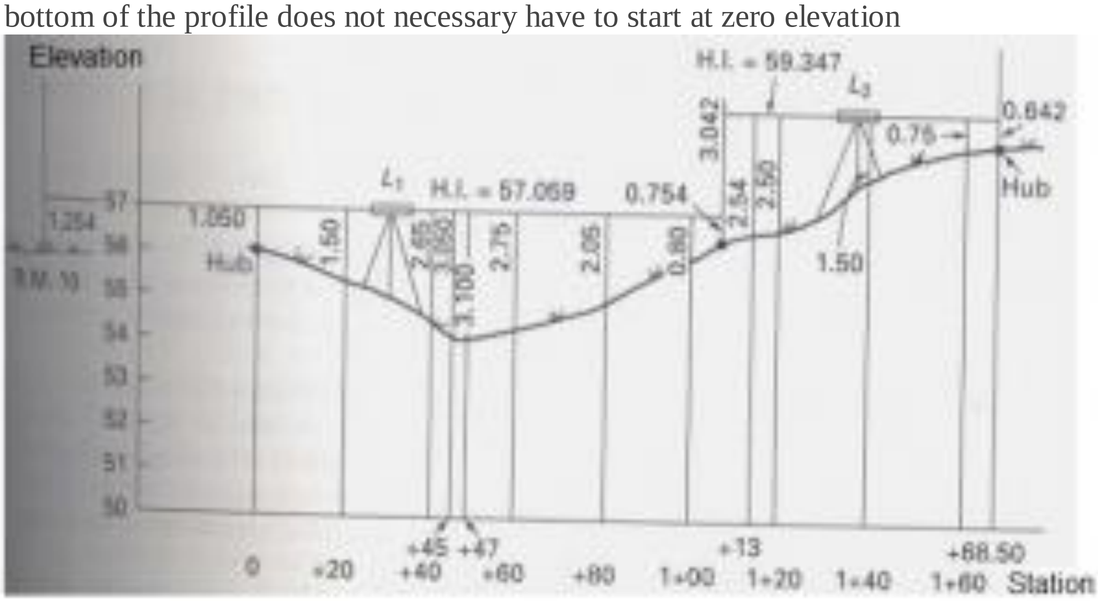


Figure 1: Profile leveling

2.Cross sectioning levelling

Cross sectioning levelling is another method in profile levelling. The term cross-section generally refers to a relatively short profile view of the ground, which is drawn perpendicular to the route centerline of a highway or other types of linear projects.

Cross-sectional drawings are particularly important for estimating the earthwork volumes needed to construct a roadway; they show the existing ground elevations, the proposed cut or fill side slopes, and the grade elevation for the road base.

There is really no difference in procedure between profile and cross-section leveling except for the form of the field notes. Cross-section rod shots are usually taken during the route profile survey from the same instrument positions used to take rod shots along the centerline.

Cross-section data are obtained at the same locations along the route that are used for the profile rod-shot stations. (See figure 2 a and b).

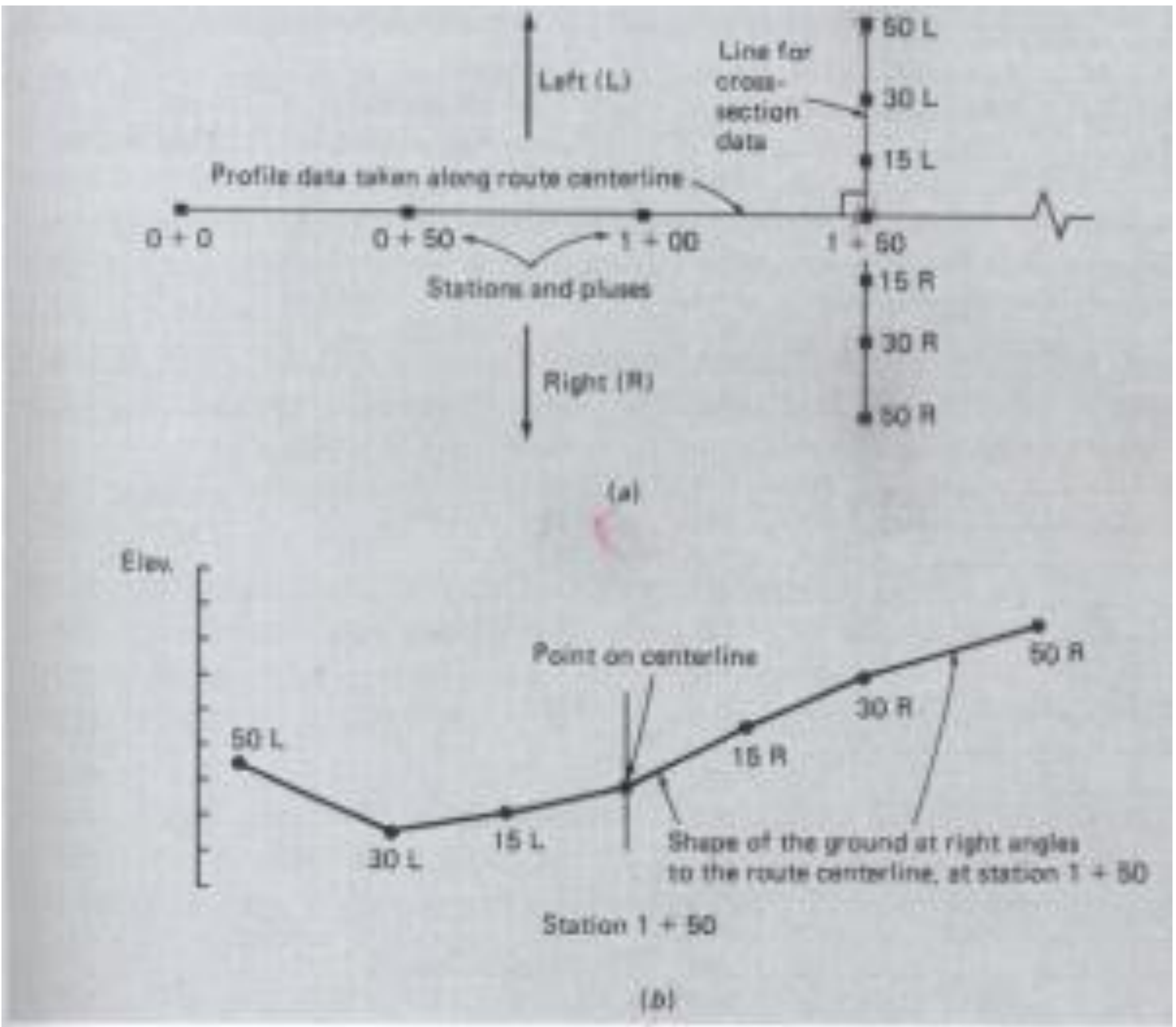


Figure 2

(a) Top view showing the route center line and the line for cross-section leveling at station 1+ 50.

(b) The cross-section showing ground elevations at points left and right of the centerline.

Hope you can understand this post, profile levelling (longitudinal and cross-section levelling)

Contouring:

Contouring in surveying is the determination of elevation of various points on the ground and fixing these points of same horizontal positions in the contour map.

To exercise vertical control leveling work is carried out and simultaneously to exercise horizontal control chain survey or compass survey or plane table survey is to be carried out.

If the theodolite is used, both horizontal and vertical controls can be achieved from the same instrument. Based on the instruments used one can classify the contouring in different groups.

Methods of Contour Surveying
There are two methods of contour surveying:

1. Direct method
2. Indirect method

Direct Method of Contouring
It consists in finding vertical and horizontal controls of the points which lie on the selected contour line.

For vertical control levelling instrument is commonly used. A level is set on a commanding position in the area after taking fly levels from the nearby bench mark. The plane of collimation/height of instrument is found and the required staff reading for a contour line is calculated.

The instrument man asks staff man to move up and down in the area till the required staff reading is found. A surveyor establishes the horizontal control of that point using his instruments.

After that instrument man directs the staff man to another point where the same staff reading can be found. It is followed by establishing horizontal control.

Thus, several points are established on a contour line on one or two contour lines and suitably noted down. Plane table survey is ideally suited for this work.

After required points are established from the instrument setting, the instrument is shifted to another point to cover more area. The level and survey instrument need not be shifted at the same time. It is better if both are nearby to communicate easily.

For getting speed in levelling some times hand level and Abney levels are also used. This method is slow, tedious but accurate. It is suitable for small areas.

Indirect Method of Contouring
In this method, levels are taken at some selected points and their levels are reduced. Thus in this method horizontal control is established first and then the levels of those points found.

After locating the points on the plan, reduced levels are marked and contour lines are interpolated between the selected points.

For selecting points any of the following methods can be used:

1. Method of squares
2. Method of cross-section
3. Radial line method

Method of Squares
In this method area is divided into a number of squares and all grid points are marked (Ref. Fig. 1).

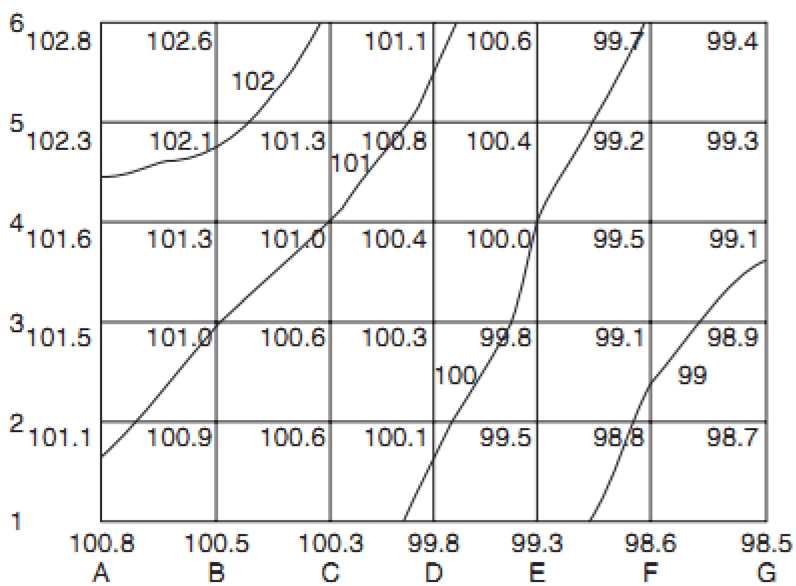


Fig. 1

Commonly used size of square varies from 5 m × 5 m to 20 m × 20 m. Levels of all grid points are established by levelling. Then grid square is plotted on the drawing sheet. Reduced levels of grid points marked and contour lines are drawn by interpolation [Ref. Fig. 1].

Method of Cross-Section
In this method cross-sectional points are taken at regular interval. By levelling the reduced level of all those points are established. The points are marked on the drawing sheets, their reduced levels (RL) are marked and contour lines interpolated.

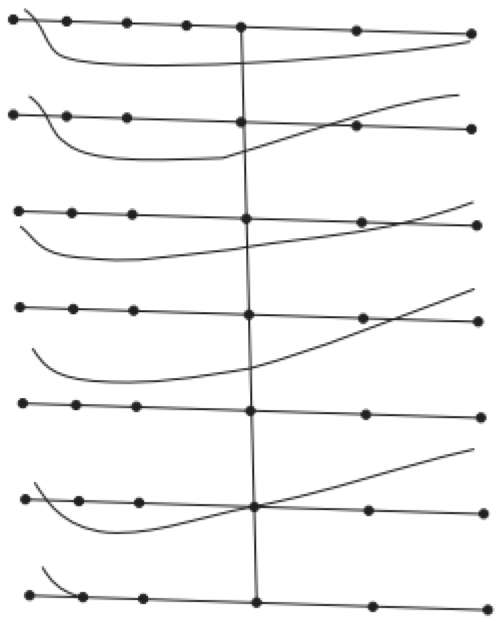


Fig. 2

Figure 2 shows a typical planning of this work. The spacing of cross-section depends upon the nature of the ground, scale of the map and the contour interval required. It varies from 20 m to 100 m. Closer intervals are required if ground level varies abruptly.

The cross- sectional line need not be always be at right angles to the main line. This method is ideally suited for road and railway projects.

Radial Line Method

[Fig. 3]. In this method several radial lines are taken from a point in the area. The direction of each line is noted. On these lines at selected distances points are marked and levels determined. This method is ideally suited for hilly areas. In this survey theodolite with tacheometry facility is commonly used.

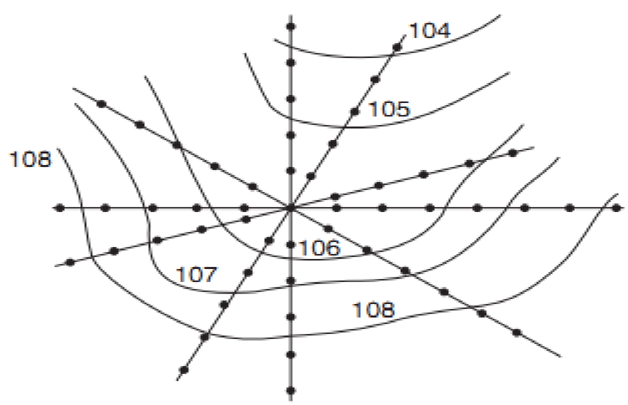


Fig. 3

For **interpolating contour points** between the two points any one of the following method may be used:
(a) Estimation
(b) Arithmetic calculation
(c) Mechanical or graphical method.

Mechanical or graphical method of interpolation consist in linearly interpolating contour points using tracing sheet:
On a tracing sheet several parallel lines are drawn at regular interval. Every 10th or 5th line is made darker for easy counting. If RL of A is 97.4 and that of B is 99.2 m. Assume the bottom most dark line represents 97 m RL and every parallel line is at 0.2 m intervals. Then hold the second parallel line on A.

Rotate the tracing sheet so that 100.2 the parallel line passes through point B. Then the intersection of dark lines on AB represents the points on 98 m and 99 m contours [Ref. Fig. 4].

Similarly the contour points along any line connecting two neighbouring points may be obtained and the points pricked. This method maintains the accuracy of arithmetic calculations at the same time it is fast.

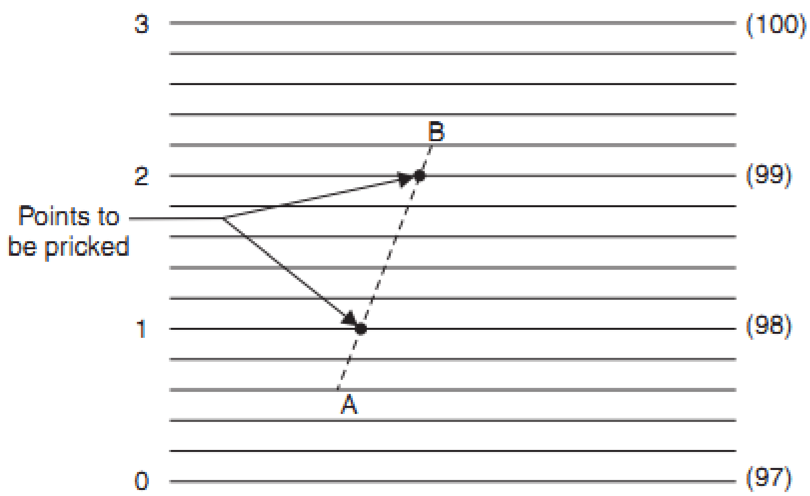


Fig. 4

Drawing Contours

After locating contour points smooth contour lines are drawn connecting corresponding points on a contour line. French curves may be used for drawing smooth lines. A surveyor should not lose the sight of the characteristic feature on the ground. Every fifth contour line is made thicker for easy readability. On every contour line its elevation is written. If the map size is large, it is written at the ends also.

Contour Maps and Its Uses
A contour maps consists of contour lines which are imaginary lines connecting points of equal elevation. Such lines are drawn on the plan of an area after establishing reduced levels of several points in the area.

The contour lines in an area are drawn keeping difference in elevation of between two consecutive lines constant. For example, the contour map in fig. 1 shows contours in an area with contour interval of 1 m. On contour lines the level of lines is also written.

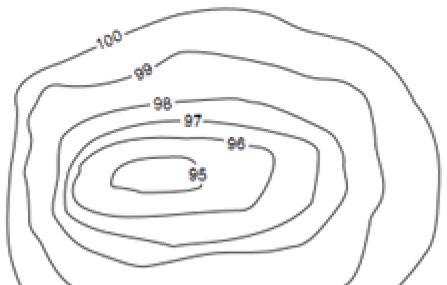


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Fig. 1: Contours

Characteristics of Contour Maps
The contours maps have the following characteristics:

1. Contour lines must close, not necessarily in the limits of the plan.
2. Widely spaced contour indicates flat surface.
3. Closely spaced contour indicates steep ground.
4. Equally spaced contour indicates uniform slope.
5. Irregular contours indicate uneven surface.
6. Approximately concentric closed contours with decreasing values towards centre (Fig. 1) indicate a pond.
7. Approximately concentric closed contours with increasing values towards centre indicate hills.
8. Contour lines with U-shape with convexity towards lower ground indicate ridge (Fig. 2).

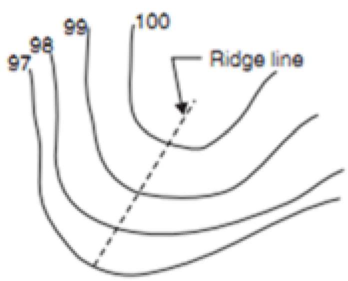


Fig. 2

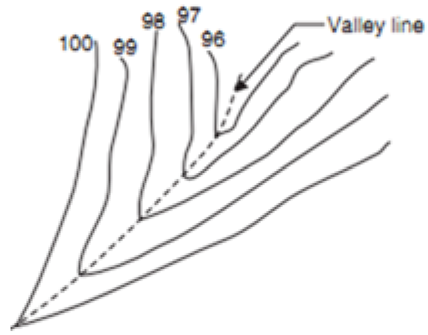


Fig. 3

9. Contour lines with V-shaped with convexity towards higher ground indicate valley (Fig.3).

10. Contour lines generally do not meet or intersect each other.

11. If contour lines are meeting in some portion, it shows existence of a vertical cliff (Fig. 4).

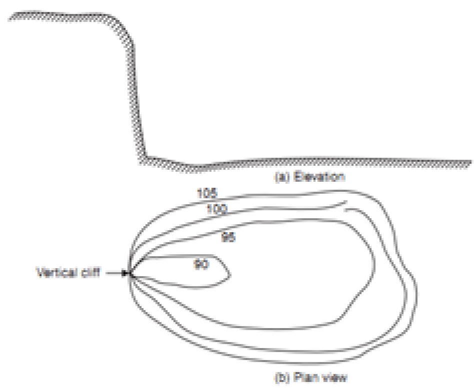


Fig. 4

12. If contour lines cross each other, it shows existence of overhanging cliffs or a cave (Fig. 5).

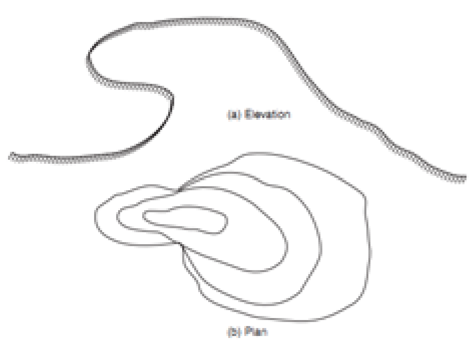


Fig. 5

Uses of Contour Maps

Contour maps are extremely useful for various engineering works:

1. A civil engineer studies the contours and finds out the nature of the ground to identify. Suitable site for the project works to be taken up.
2. By drawing the section in the plan, it is possible to find out profile of the ground along that line. It helps in finding out depth of cutting and filling, if formation level of road/railway is decided.
3. Intervisibility of any two points can be found by drawing profile of the ground along that line.
4. The routes of the railway, road, canal or sewer lines can be decided so as to minimize and balance earthworks.
5. Catchment area and hence quantity of water flow at any point of nalla or river can be found. This study is very important in locating bunds, dams and also to find out flood levels.
6. From the contours, it is possible to determine the capacity of a reservoir.

UNIT II

Curves

Definition of Curves:

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

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

Classification of Curves:

(i) Simple,
(ii) Compound
(iii) Reverse and
(iv) Deviation

(i) Simple Curve:

A simple curve consists of a single arc of a circle connecting two straights. It has radius of the same magnitude throughout. In fig. 11.1 T_1 D T_2 is the simple curve with T_1O as its radius.

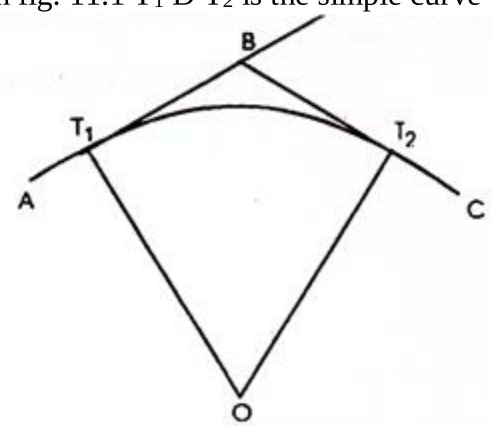


Fig 11.1

(ii) Compound Curve:

A compound curve consists of two or more simple curves having different radii bending in the same direction and lying on the same side of the common tangent. Their centres lie on the same side of the curve. In fig. 11.2, T_1 P T_2 is the compound curve with T_1O_1 and PO_2 as its radii.

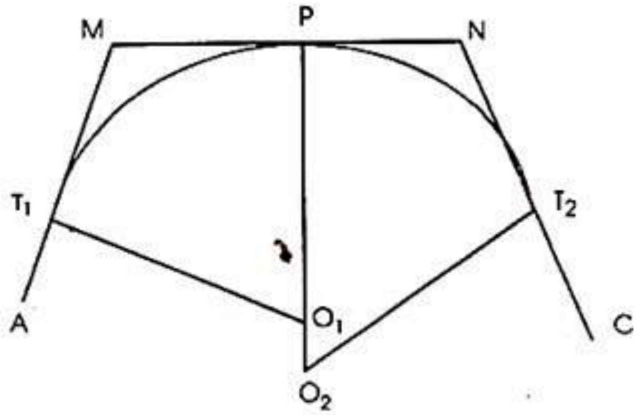


Fig 11.2

(iii) Reverse (or Serpentine) Curve:

A reverse or serpentine curve is made up of two arcs having equal or different radii bending in opposite directions with a common tangent at their junction. Their centres lie of opposite sides of the curve. In fig. 11.3 T_1 P T_2 is the reverse curve with T_1O_1 and PO_2 as its radii.

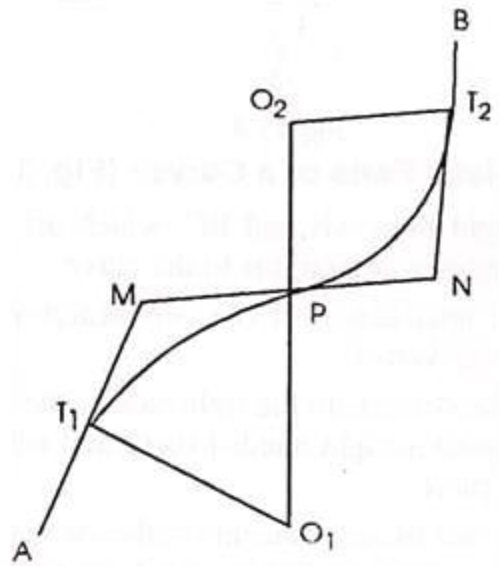


Fig 11 3

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

(iv) Deviation Curve:

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

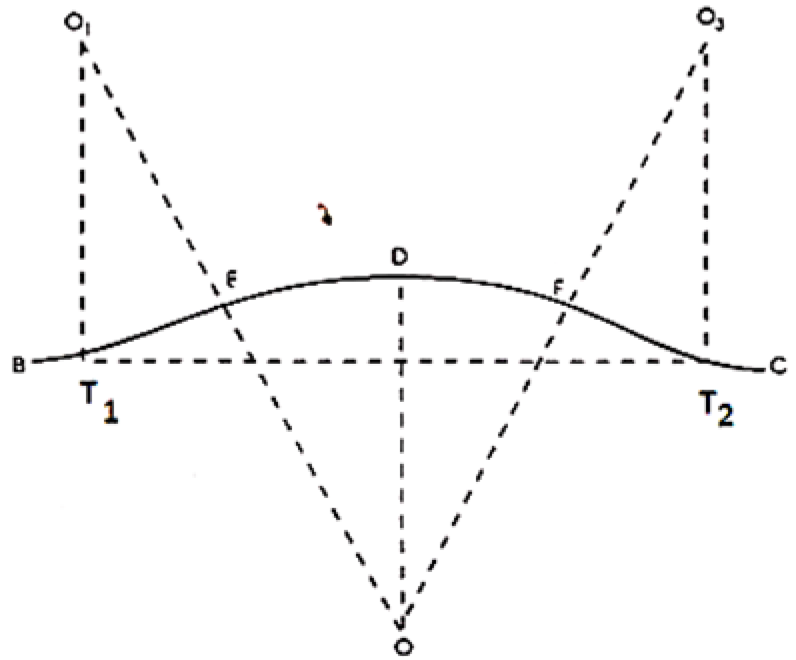


Fig 11.4

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

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

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

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

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

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

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

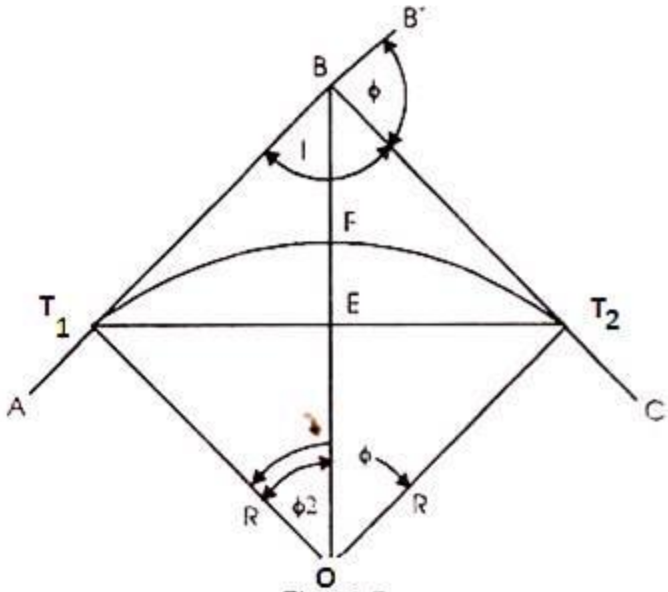


Fig 11.5

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

(viii) The distance the two tangent point of intersection to the tangent point is called the tangent length (BT_1 and BT_2).
(ix) The line joining the two tangent points (T_1 and T_2) is known as the long-chord
(x) The arc T_1FT_2 is called the length of the curve.
(xi) The mid-point (F) of the arc (T_1FT_2) in called summit or apex of the curve.
(xii) The distance from the point of intersection to the apex of the curve BF is called the apex distance.

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

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

Elements of a Curve (Fig. 11.5):

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

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

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

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

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

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

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

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

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

$= R\,\phi$ (in radians)

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

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

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

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

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

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

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

Designation of Curves:

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

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

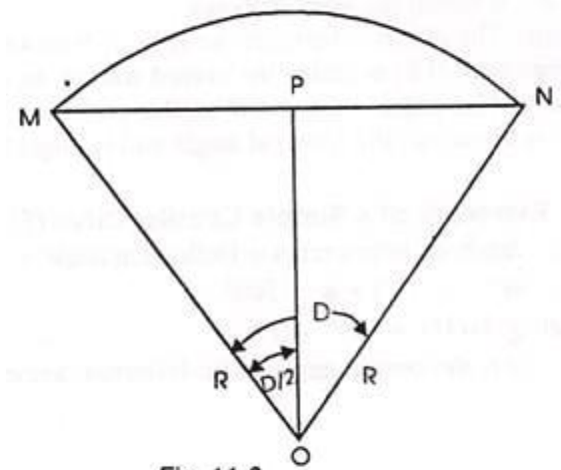


Fig. 11.6

Let R= The radius of the curves in meters

D= The degree of the curve

MN= The chord, 30m long

P= The mid-point of the chord

In Δ OMP, OM = R

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

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

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

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

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

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

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

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

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

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

Methods of Curve Ranging:

A curve may be set out:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Transition Curves:

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

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

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

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

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

(iv) Discomfort to passengers is eliminated.

Conditions to be fulfilled by the transition curve:

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

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

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

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

There are three types of transition curves in common use:
(1) A cubic parabola,

(2) A cubical spiral, and

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

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

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

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

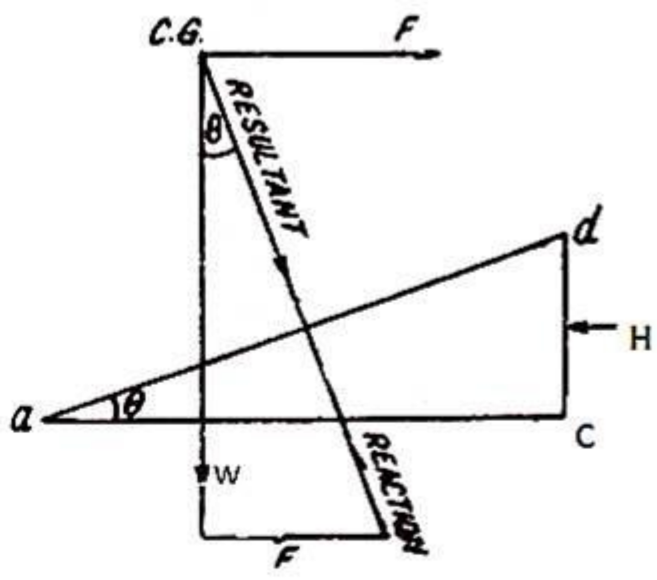


Fig. 11.24

Let:

W = the weight of vehicle acting vertically downwards.

F = the centrifugal force acting horizontally,

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

g = the acceleration due to gravity, 9.81 metres/sec^2.
R = the radius of the curve in metres,

h = the super-elevation in metres.

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

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

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

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

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

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

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

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

Characteristics of a Transition Curve (Fig 11.25):
Here two straights AB and BC make a deflection angle Δ, and a circular curve EE' of radius R, with two transition curves TE and E'T' at the two ends, has been inserted between the straights.

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

Fig. 11.25:

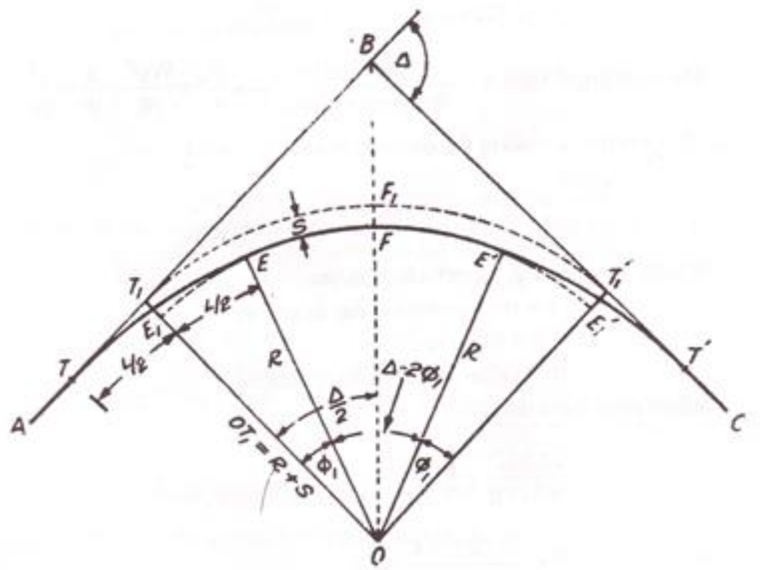


Fig 11.25

(ii) The tangent length for the combined curve

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

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

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

(iv) The central angle for the circular curve:

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

(v) Length of the circular curve EFE'

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

(vi) Length of the combined curve TEE'T"

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$$= 4S$$

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

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

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

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

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

(ii) Calculate the value of the shift by:

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

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

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

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

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

Check:

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

Note:

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

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

(viii) Set the vernier to a reading(360°-2/3 φ1) for a right-hand curve (or 2/3 φ1) for a left-hand curve and lake a back sight on T. Loosen the upper clamp and turn the telescope clockwise through an angle 2/3 φ1 the telescope is thus directed towards common tangent at E and the vernier reads 360°. Transit the telescope, now it points towards the forward direction of the common tangent at E i.e. towards the tangent for the circular curve.

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

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

Check:

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

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

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

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

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

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

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

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

II

Unit III
Modern Field Survey Systems

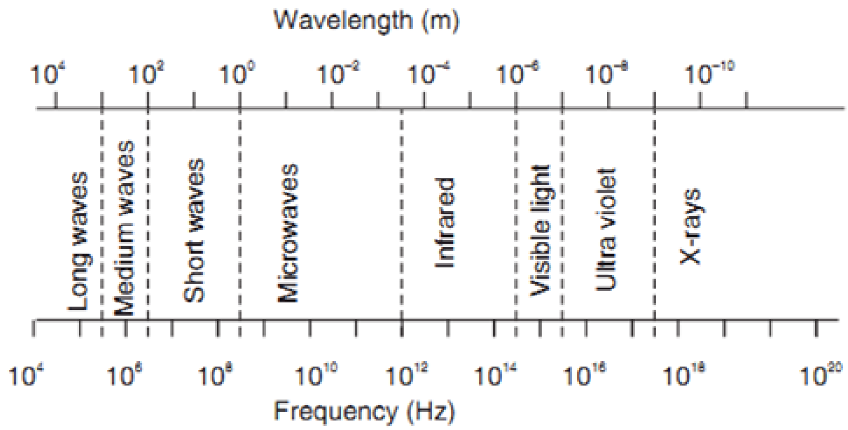
Electronic Distance Measurement:
Electronic distance measuring instrument is a surveying instrument for measuring distance electronically between two points through electromagnetic waves.
Electronic distance measurement (EDM) is a method of determining the length between two points, using phase changes, that occur as electromagnetic energy waves travels from one end of the line to the other end. As a background, there are three methods of measuring distance between two points:

DDM or Direct distance measurement – This is mainly done by chaining or taping.

ODM or Optical distance measurement – This measurement is conducted by tacheometry, horizontal subtense method or telemetric method. These are carried out with the help of optical wedge attachments.

EDM or Electromagnetic distance measurement – The method of direct distance measurement cannot be implemented in difficult terrains. When large amount of inconsistency in the terrain or large obstructions exist, this method is avoided.
As an alternative to this optical distance measurement method was developed. Still it gained a disadvantage of limited range of measurement. It is limited to 15 to 150m with an accuracy of 1 in 1000 to 1 in 10000. Above all we have EDM with an accuracy of 1 in 10^5, having a distance range of 100km.
Electronic distance measurement in general is a term used as a method for distance measurement by electronic means. In this method instruments are used to measure distance that rely on propagation, reflection and reception of electromagnetic waves like radio, visible light or infrared waves.

Sun light or artificially generated electromagnetic wave consists of waves of different lengths. The spectrum of an electromagnetic wave is as shown below:



Among these waves microwaves, infrared waves and visible light waves are useful for the distance measurement. In EDM instruments these waves are generated, modulated and then propagated. They are reflected at the point up to which distance is to be measured from the instrument station and again received by the instrument.
The time taken by the wave to travel this 2x distance may be measured and knowing the velocity of wave, the distance may be calculated. However time is too short, measuring the time taken is difficult.

The improved techniques use phase difference method in which the number of completed wave and incomplete wave is measured. Knowing the length of wave, distances are calculated.

Built up microprocessors provided in the instrument calculate the distances and display it by liquid crystal display (LCD).

Types of Electronic Distance Measurement Instrument
EDM instruments are classified based on the type of carrier wave as

1. Microwave instruments
2. Infrared wave instruments
3. Light wave instruments.

1. Microwave Instruments
These instruments make use of microwaves. Such instruments were invented as early as 1950 in South Africa by Dr. T.L. Wadley and named them as Tellurometers. The instrument needs only 12 to 24 V batteries. Hence they are light and highly portable. Tellurometers can be used in day as well as in night.
The range of these instruments is up to 100 km. It consists of two identical units. One unit is used as master unit and the other as remote unit. Just by pressing a button, a master unit can be converted into a remote unit and a remote unit into a master unit. It needs two skilled persons to operate. A speech facility is provided to each operator to interact during measurements.

2. Infrared Wave Instruments
In this instrument amplitude modulated infrared waves are used. Prism reflectors are used at the end of line to be measured. These instruments are light and economical and can be mounted on theodolite. With these instruments accuracy achieved is ± 10 mm. The range of these instruments is up to 3 km.
These instruments are useful for most of the civil engineering works. These instruments are available in the trade names DISTOMAT DI 1000 and DISTOMAT DI 55.

3. Visible Light Wave Instruments
These instruments rely on propagation of modulated light waves. This type of instrument was first developed in Sweden and was named as Geodimeter. During night its range is up to 2.5 km while in day its range is up to 3 km. Accuracy of these instruments varies from 0.5 mm to 5 mm/km distance. These instruments are also very useful for civil engineering projects.

Error in Electronic Distance Measurement Instruments:
Personal Errors

- Inaccuracy in initial setups of EDMs and the reflectors over the preferred stations
- Instrument and reflector measurements going wrong
- Atmospheric pressures and temperature determination errors

Instrumental Errors

- Calibration errors
- Chances of getting maladjusted time to time generating frequent errors
- Errors shown by the reflectors

Natural Errors

- Atmospheric variations in temperature, pressure as well as humidity. Micro wave EDM instruments are more susceptible to these.
- Multiple refraction of the signals.

The advantage of using EDM instruments is the speed and accuracy in measurement. Several obstacles to chaining are automatically overcome when these instruments are used.

Total Station:

Total station is a surveying equipment combination of **Electromagnetic Distance Measuring Instrument** and electronic theodolite. It is also integrated with microprocessor, electronic data collector and storage system. The instrument can be used to measure horizontal and vertical angles as well as sloping distance of object to the instrument.

Capability of a Total Station
Microprocessor unit in total station processes the data collected to compute:

1. Average of multiple angles measured.
2. Average of multiple distance measured.
3. Horizontal distance.
4. Distance between any two points.
5. Elevation of objects and
6. All the three coordinates of the observed points.

Data collected and processed in a Total Station can be downloaded to computers for further processing.

Total station is a compact instrument and weighs 50 to 55 N. A person can easily carry it to the field. Total stations with different accuracy, in angle measurement and different range of measurements are available in the market. Figure below shows one such instrument manufactured by SOKKIA Co. Ltd. Tokyo, Japan.

Important Operations of Total Station:

Distance Measurement
Electronic distance measuring (EDM) instrument is a major part of total station. Its range varies from 2.8 km to 4.2 km. The accuracy of measurement varies from 5 mm to 10 mm per km measurement. They are used with automatic target recognizer. The distance measured is always sloping distance from instrument to the object.

Angle Measurements
The electronic theodolite part of total station is used for measuring vertical and horizontal angle. For measurement of horizontal angles any convenient direction may be taken as reference direction. For vertical angle measurement vertical upward (zenith) direction is taken as reference direction. The accuracy of angle measurement varies from 2 to 6 seconds.

Data Processing
This instrument is provided with an inbuilt microprocessor. The microprocessor averages multiple observations. With the help of slope distance and vertical and horizontal angles measured, when height of axis of instrument and targets are supplied, the microprocessor computes the horizontal distance and X, Y, Z coordinates.

The processor is capable of applying temperature and pressure corrections to the measurements, if atmospheric temperature and pressures are supplied.

Display
Electronic display unit is capable of displaying various values when respective keys are pressed. The system is capable of displaying horizontal distance, vertical distance, horizontal and vertical angles, difference in elevations of two observed points and all the three coordinates of the observed points.

Electronic Book
Each point data can be stored in an electronic note book (like compact disc). The capacity of electronic note book varies from 2000 points to 4000 points data. Surveyor can unload the data stored in note book to computer and reuse the note book.

Uses of Total Station
The total station instrument is mounted on a tripod and is levelled by operating levelling screws. Within a small range instrument is capable of adjusting itself to the level position. Then vertical and horizontal reference directions are indexed using onboard keys.

It is possible to set required units for distance, temperature and pressure (FPS or SI). Surveyor can select measurement mode like fine, coarse, single or repeated.

When target is sighted, horizontal and vertical angles as well as sloping distances are measured and by pressing appropriate keys they are recorded along with point number. Heights of instrument and targets can be keyed in after measuring them with tapes. Then processor computes various information about the point and displays on screen.

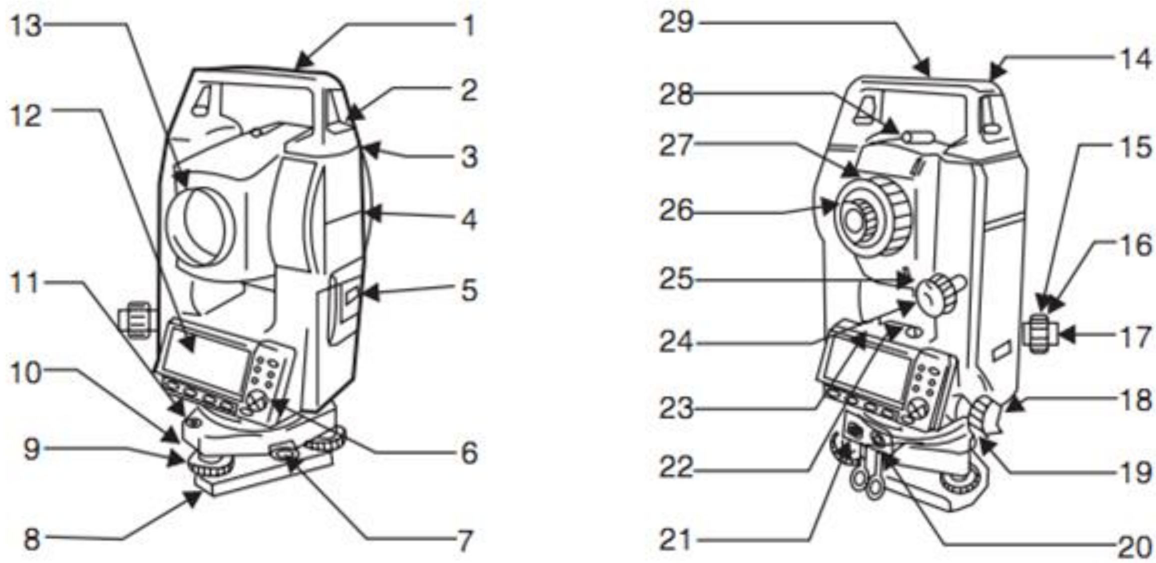
This information is also stored in the electronic notebook. At the end of the day or whenever electronic note book is full, the information stored is downloaded to computers.

The point data downloaded to the computer can be used for further processing. There are software like auto civil and auto plotter clubbed with AutoCad which can be used for plotting contours at any specified interval and for plotting cross-section along any specified line.

Advantages of Using Total Stations
The following are some of the **major advantages of using total station** over the conventional surveying instruments:

1. Field work is carried out very fast.
2. Accuracy of measurement is high.
3. Manual errors involved in reading and recording are eliminated.
4. Calculation of coordinates is very fast and accurate. Even corrections for temperature and pressure are automatically made.
5. Computers can be employed for map making and plotting contour and cross-sections. Contour intervals and scales can be changed in no time.

However, surveyor should check the working condition of the instruments before using. For this standard points may be located near survey office and before taking out instrument for field work, its working is checked by observing those standard points from the specified instrument station.



1. Handle
2. Handle securing screw
3. Data input/output terminal (Remove handle to view)
4. Instrument height mark
5. Battery cover
6. Operation panel
7. Tribrach clamp (SET300*S*/500*S*/600S: Shifting clamp)
8. Base plate
9. Levelling foot screw
10. Circular level adjusting screws
11. Circular level
12. Display
13. Objective lens
14. Tubular compass slot
15. Optical plummet focussing ring
16. Optical plummet reticle cover
17. Optical plummet eyepiece
18. Horizontal clamp
19. Horizontal fine motion screw
20. Data input/output connector (Besides the operation panel on SET600/600*S*)
21. External power source connector (Not included on SET600/600*S*)
22. Plate level
23. Plate level adjusting screw
24. Vertical clamp
25. Vertical fine motion screw
26. Telescope eyepiece
27. Telescope focussing ring
28. Peep sight
29. Instrument center mark

Parts of Total Station

Error Sources in Total Station in Surveying:

Circle Eccentricity

Circle eccentricity exists when the theoretical center of the mechanical axis of the theodolite does not coincide exactly with the center of the measuring circle.
The amount of error corresponds to the degree of eccentricity and the part of the circle being read. When represented graphically circle eccentricity appears as a sine wave
Circle eccentricity in the horizontal circle can always be compensated for by measuring in both faces (opposite sides of the circle) and using the mean as a result.

Vertical circle eccentricity cannot be compensated for in this manner since the circle moves with the telescope. More sophisticated techniques are required.
(1) Some theodolites are individually tested to determine the sine curve for the circle error in that particular instrument. Then a correction factor is stored in ROM that adds or subtracts from each angle reading so that a corrected measurement is displayed.
(2) Other instruments employ an angle-measuring system consisting of rotating glass circles that make a complete revolution for every angle measurement. These are scanned by fixed and moving light sensors. The glass circles are divided into equally spaced intervals which are diametrically scanned by the sensors.
The amount of time it takes to input a reading into the processor is equal to one interval, thus only every alternate graduation is scanned. As a result, measurements are made and averaged for each circle measurement. This eliminates scale graduation and circle eccentricity error.

Horizontal Collimation Error in Total Station

Horizontal collimation error exists when the optical axis of the theodolite is not exactly perpendicular to the telescope axis. To test for horizontal collimation error, point to a target in face one then point back to the same target in face two; the difference in horizontal circle readings should be 180 degrees.
Horizontal collimation error can always be corrected for by meaning the face one and face two pointings of the instrument.
(1) Most electronic theodolites have a method to provide a field adjustment for horizontal collimation error. Again, the manual for each instrument provides detailed instruction on the use of this correction.
(2) In some instruments, the correction stored for horizontal collimation error can affect only measurements on one side of the circle at a time. Therefore when the telescope is passed through zenith (the other side of the circle is being read), the horizontal circle reading will change by twice the collimation error. These instruments are functioning exactly as designed when this happens.
(3) When prolonging a line with an electronic theodolite, the instrument operator should either turn a 180-degree angle or plunge the telescope and turn the horizontal tangent so that the horizontal circle reading is the same as it was before plunging the telescope.

Height of Standards Error in Total Station

In order for the telescope to plunge through a truly vertical plane the telescope axis must be perpendicular to the standing axis. As stated before there is no such thing as perfection in the physical world.
All theodolites have a certain degree of error caused by imperfect positioning of the telescope axis. Generally, determination of this error should be accomplished by a qualified technician because horizontal collimation and height of standards errors interrelate and can magnify or offset one another.
Horizontal collimation error is usually eliminated before checking for height of standards. Height of standards error is checked by pointing to a scale the same zenith angle above a 90-degree zenith in “face-one” and “face-two.” The scales should read the same in face one as in face two.

Circle Graduation Error in Total Station

In the past, circle graduation error was considered a major problem. For precise measurements surveyors advanced their circle on each successive set of angles so that circle graduation errors were “meaned out”. Current technology eliminates the problem of graduation errors.

This is accomplished by photo-etching the graduations onto the glass circles and making a precise master circle and photographing it. An emulsion is then applied to the circle and a photo-reduced image of the master is projected onto the circle. The emulsion is removed and the glass circle has been etched with very precise graduations.

Vertical Circle Error in Total Station

It is important to check the vertical circle indexing adjustment on surveying instruments on a routine basis. When direct and indirect zenith angles are measured to the same point, the sum of the two angles should equal 360°.

Over time, the sum of these two angles may diverge from 360° and consequently cause errors in vertical angle measurements. While averaging the direct and indirect zenith angles easily eliminates this error, on many jobs it may not be cost effective to make two readings.

Acceptable accuracy may still be maintained for many applications with only a direct reading; however, as long as the index error is kept to a minimum by periodically performing a vertical adjustment, such as TOPCON's "Vertical Angle 0 Datum Adjustment".

Most total stations are provided with some type of electronic adjustment to correct the vertical circle indexing error. This adjustment takes just a few seconds and will insure that you get good vertical angle readings with just one measurement. Consult the manufacturer's manual for instructions on making this adjustment.

Pointing Errors in Total Station

Pointing errors are due to both human ability to point the instrument and environmental conditions limiting clear vision of the observed target. The best way to minimize pointing errors is to repeat the observation several times and use the average as the result.

Uneven Heating of the Instrument

Direct sunlight can heat one side of the instrument enough to cause small errors. For the highest accuracy, utilize an umbrella or pick a shaded spot for the instrument.

Vibrations

Avoid instrument locations that vibrate. Vibrations can cause the compensator to be unstable.

Collimation Errors

When sighting points a single time (e.g., direct position only) for elevations, check the instrument regularly for collimation errors.

Vertical Angles and Elevations

When using total stations to measure precise elevations, the adjustment of the electronic tilt sensor and the reticule of the telescope becomes very important. An easy way to check the adjustment of these components is to set a baseline. A line close to the office with a large difference in elevation will provide the best results.

The baseline should be as long as the longest distance that will be measured to determine elevations with intermediate points at 100- to 200-ft intervals. Precise elevations of the points along the baseline should be measured by differential leveling.

Set up the total station at one end of the baseline and measure the elevation of each point. Comparing the two sets of elevations provides a check on the accuracy and adjustment of the instrument.

Accuracy requirements may dictate that more than one set of angles and distances is measured to each point. Some examples are distances over 600 feet, adverse weather conditions, and steep observations.

Atmospheric Corrections in Total Station

Meteorological data corrections to observed EDM slope distances may be significant over longer distances. Usually for most topographic surveying over short distances, nominal (estimated) temperature and pressure data is acceptable for input into the data collector. Instruments used to measure atmospheric temperature and pressure must be periodically calibrated. This would include psychrometers and barometers.

Optical Plummet Errors

The optical plummet or tribrachs must be periodically checked for misalignment. This would include total stations with laser plummets.

Adjustment of Prism Poles

When using prism poles, precautions should be taken to ensure accurate measurements. A common problem encountered when using prism poles is the adjustment of the leveling bubble. Bubbles can be examined by establishing a check station under a doorway in the office.

First, mark a point on the top of the doorway. Using a plumb bob, establish a point under the point on the doorway. If possible, use a center punch to make a dent or hole in both the upper and lower marks. The prism pole can now be placed into the check station and easily adjusted.

Recording Errors

The two most common errors are reading an angle incorrectly and/or entering incorrect information into the field book. Another common (and potentially disastrous) error is an incorrect instrument or rod height.

Although electronic data collection has all but eliminated these errors, it is still possible for the surveyor to identify an object incorrectly, make a shot to the wrong spot, or input a bad target height (HR) or HI.

For example, if the surveyor normally shoots a fire hydrant at the ground level, but for some reason shoots it on top of the operating nut, erroneous contours would result if the program recognized the fire hydrant as a ground shot and was not notified of this change in field procedure.

Angles

As a rule, a surveyor will turn a doubled angle for move-ahead, traverse points, property corners, or other objects that require greater accuracy. On the other hand, single angles are all that are required for topographic shots. Refer to the total station operating instructions for repeating angle methods where required.

Slope to Grid and Sea Level EDM Corrections

Slope distances will be reduced to horizontal distances in the data collector, and then reduced to a grid distance if a grid scale factor (or combined scale sea level factor) is input into the data collector.

For most topographic survey applications involving short side shots, the grid scale factor is ignored (e.g., 1.000 is used). This would not be correct for control traverses covering larger distances. Scale factors can be obtained directly in CORPSCON.

EDM Calibration

All EDM instruments should be periodically (at least annually) checked over a NGS Calibration Baseline or a baseline established by local state surveying societies.

UNIT IV

PHOTOGRAMMETRY SURVEYING

Aerial photography refers to taking photograph of earth surface from space. Platform of aerial photography includes aircraft, helicopter, balloon, parachute etc. Aerial photography was first practiced by the French photographer and balloonist Gaspard-Félix Tournachon, known as "Nadar", in 1858 over Paris, France. It was the first means of remote sensing with immense application potentiality even uses now-a-days in the age of satellites with sophisticated electronic devices.

Types of Aerial Photos

Aerial photos can be distinguished depending on the position of camera axis with respect to the vertical and motion of the aircraft. Aerial photographs are divided into two major groups, vertical and oblique photos.

i) Vertical photos: The optical axis of the camera or camera axis is directed vertically as straight down as possible (Fig 7.1). The nadir and central point of the photograph are coincident. But in real a truly vertical aerial photograph is rarely obtainable because of unavoidable angular rotation or tilts of aircraft. The allowable tolerance is usually $+3^{o}$ from the perpendicular (plumb) line to the camera axis. Vertical photographs are taken for most common use in remote sensing and mapping purposes.

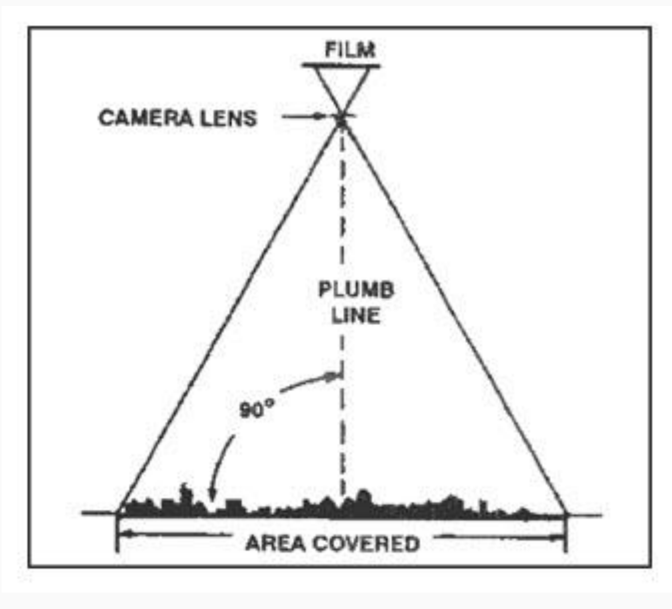


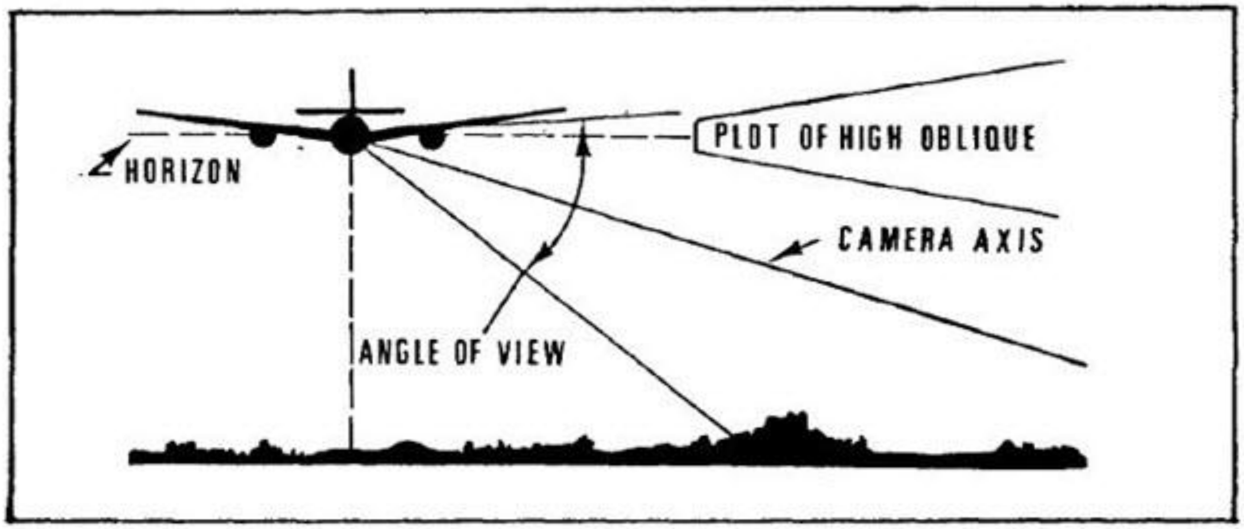
Fig. 7.1. Schematic diagram of taking a vertical photograph.

A vertical photograph has the following characteristics:
(1) The camera axis is perpendicular to the surface of the earth.
(2) It covers relatively small area than oblique photographs.
(3) The shape of the ground covered on a single vertical photo closely approximates a square or rectangle.
(4) Being a view from above, it gives an unfamiliar view of the ground.
(5) Distance and directions may approach the accuracy of maps if taken over flat terrain.
(6) Relief is not readily apparent.

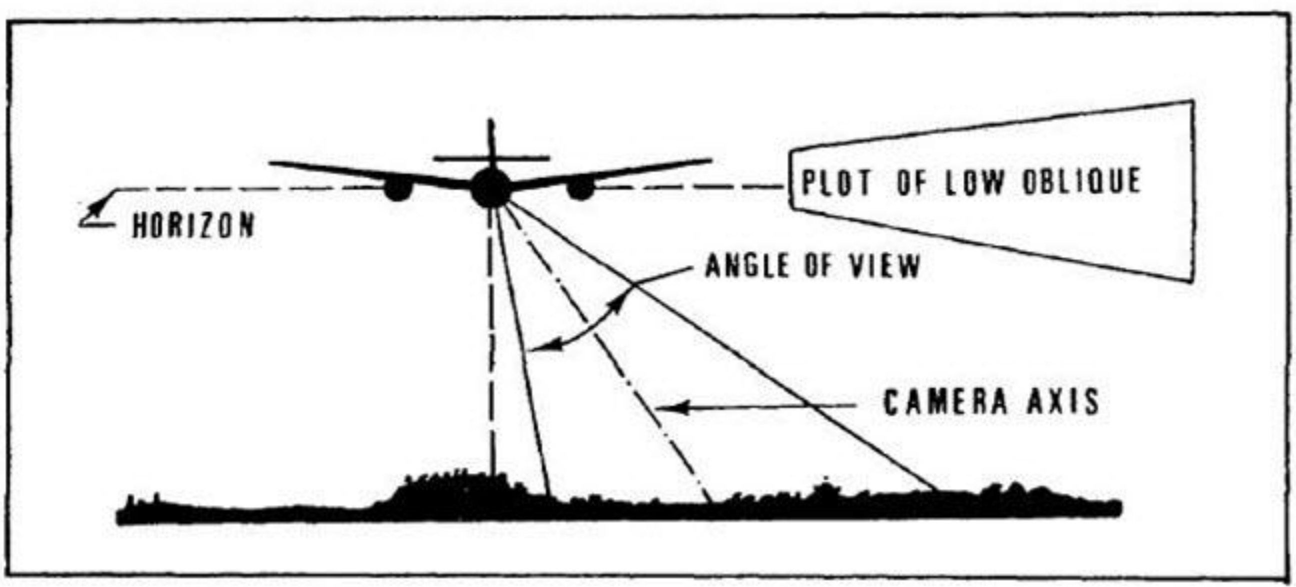
ii) Oblique photos: When the optical of the camera forms an angle of more than 5^0 with the vertical, oblique photographs are obtained (Fig. 7.2). The nadir and central point of the photograph are not coincident.

Fig. 7.2. Vertical and oblique photography.

There are two types of oblique aerial photography – high angle and low angle. In high angle oblique aerial photography the resulting images shows the apparent horizon and in low angle oblique photograph does not. Oblique photographs can be useful for covering very large areas in a single image and for depicting terrain relief and scale.



(a)



(b)

Fig. 7.3. (a) High oblique and (b) low oblique photographs.

A square outline on the ground appears as a trapezium in oblique aerial photo. These photographs can be distinguished as high oblique and low oblique (Fig.7.3). But these are not widely used for mapping as distortions in scale from foreground to the background preclude easy measurements of distance, area, and elevation.

An oblique photograph has the following characteristics:

1. Low oblique photograph covers relatively small area than high oblique photographs.

2. The ground area covered is trapezoid, but the photograph is square or rectangular. Hence scale is not applicable and direction (azimuth) also cannot be measured.

3. The relief is discernible but distorted.

Basic Geometric Characteristics of Aerial Photographs

Aerial photographs are taken using a camera fitted at the bottom of a aircraft along a line is termed as *flight line* or *flight strips* and the line traced on ground directly beneath the camera is called nadir line. The point on photograph where the nadir line meets the ground is termed as *principal point*. Lines drawn to connect marks located along opposite sides of the photo (*fiducial marks*) intersect precisely at the principal point. The point on the photo that falls on a line half- way between the principal point and the Nadir point is known as i*socenter*. The ground distance between the photo centers (principal points) is called *air base*.

In aerial photography, the aircraft acquires a series of exposures along each strip of multiple flight lines. Successive photographs are generally taken with some degree of overlap, which is known as endlap (Fig. 7.4). Standard endlap is 60%, which may be 80-90% in special cases such as in mountainous terrain. It ensures that each point of the ground appears in at least two successive photographs essential for stereoscopic coverage. Stereoscopic coverage consists of adjacent pairs of overlapping vertical photographs called stereopairs. Beside endlap the photograph is taken with some overlap of photographs of a strip with the adjacent strip, known as sidelap (Fig 7.5). It varies from 20% to 30% to ensure that no area of the ground is missing out to be photograph.

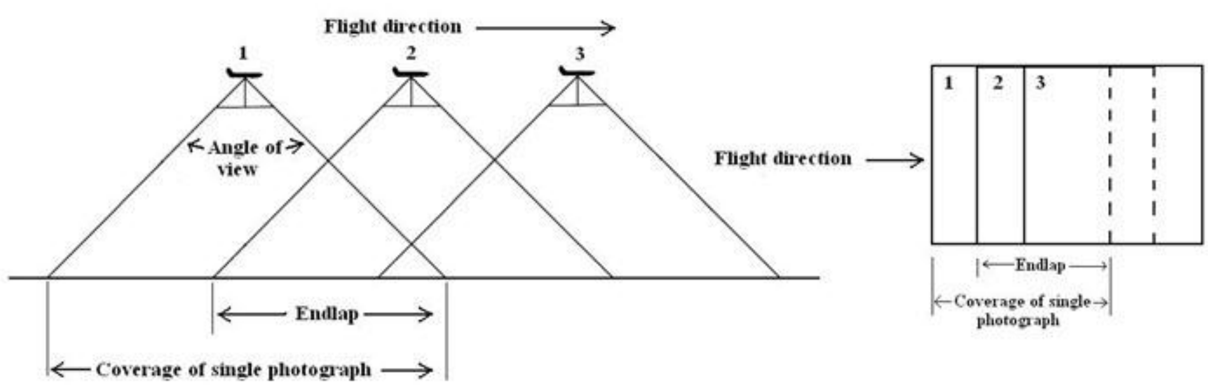


Fig. 7.4. Photographic coverage along flight line: endlap.

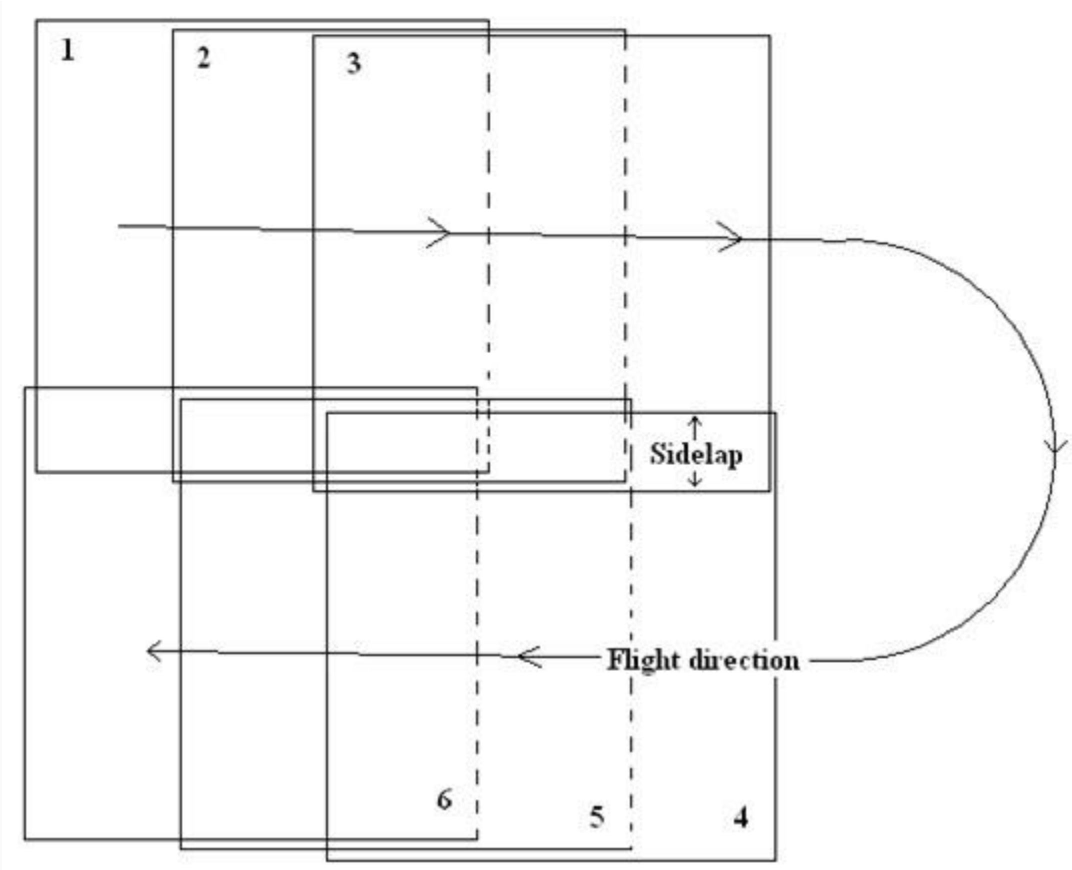


Fig. 7.5. Positions of aerial photos: sidelap.

A truly vertical photograph is rarely obtained because of unavoidable angular rotations or tilts, caused by the atmospheric conditions (air pockets or currents), human error of the pilot fails to maintain a steady flight and imperfections in the camera mounting. There are three kinds of tilts as it can be seen from Fig 7.6.

1. Tilting forward and backwards (pitch)
2. Tilting sideways (roll)
3. Tilting in the direction of flight (yaw)

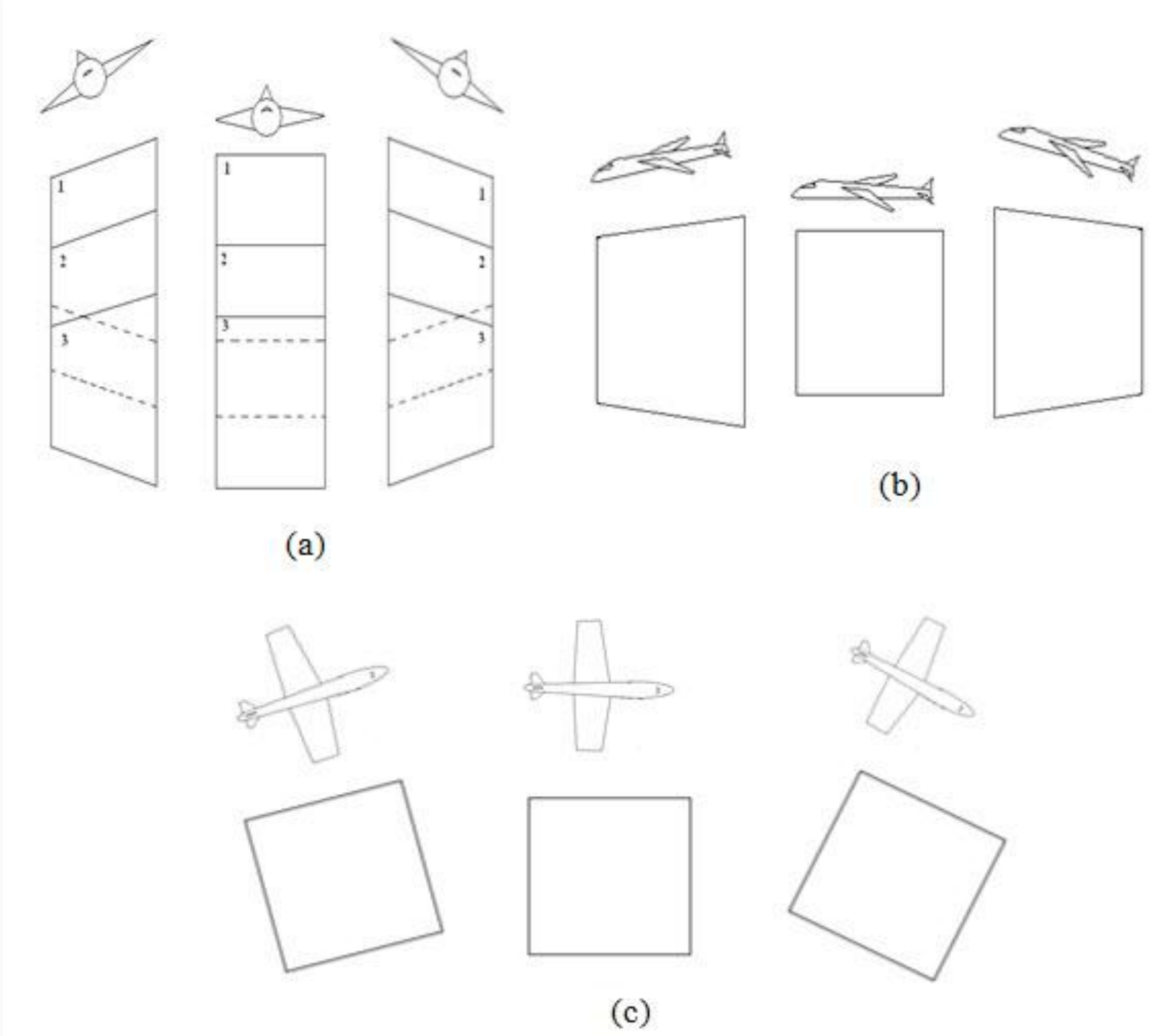


Fig. 7.6. (a) Roll, (b) Pitch, (c) Yow tilting of aircraft and corresponding ground coverage of aerial photograph.

In order to understand the geometric characteristics of an aerial photograph it is also necessary to understand the viewing perspective and projection. In case of viewing perspective on a map the objects and features are both planimetrically and geometrically accurate, because the objects and features is located in the same position relative to each other as they are on the ground or on the surface of earth. But there is a change in scale. On the other hand in aerial photography, central or perspective projection system is used as it can be seen from Fig 7.7. Therefore, there are not only changes in scale but also change in relative position and geometry between the objects depending on the location of the camera.

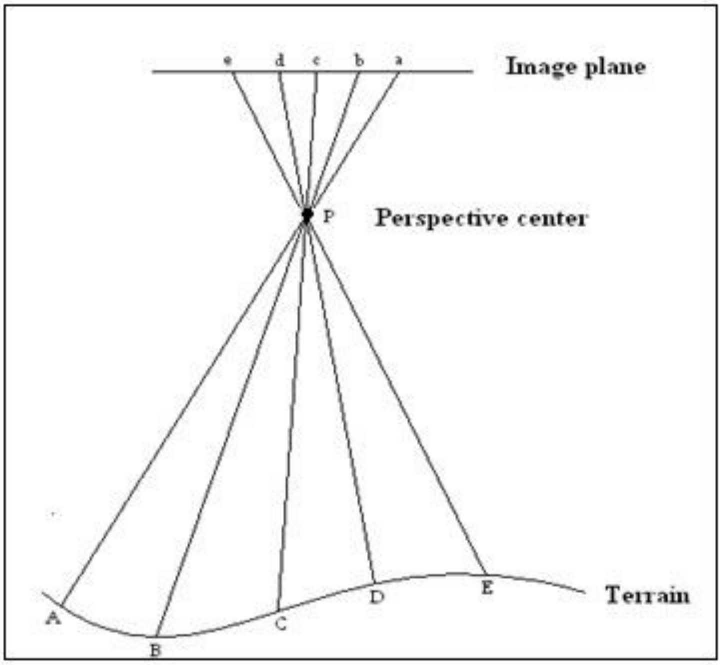


Fig. 7.7. Central or perspective projection.

Relief displacement: Relief is the difference in elevation between the high and low points of a feature or object. Due to central perspective projection system used in aerial photography, vertical objects standing above datum (average elevation) other than principal point lean outward and objects standing below the datum (average elevation) lean inward in an aerial photograph. This distortion is called relief displacement (Fig 7.8). An aerial photograph is a three-dimensional scene transferred onto a two-dimensional plane. Thus three-dimensional squashes literally due to lack of vertical dimension. Therefore image objects above and below mean ground level or datum are displaced from their true horizontal location or relief displacement takes place. Camera tilts, earth curvature, terrain relief, object height and object position in respect to principal point are the main causes of relief displacement. Relief displacement allows the measurement of objects height from a single photo or stereopair.

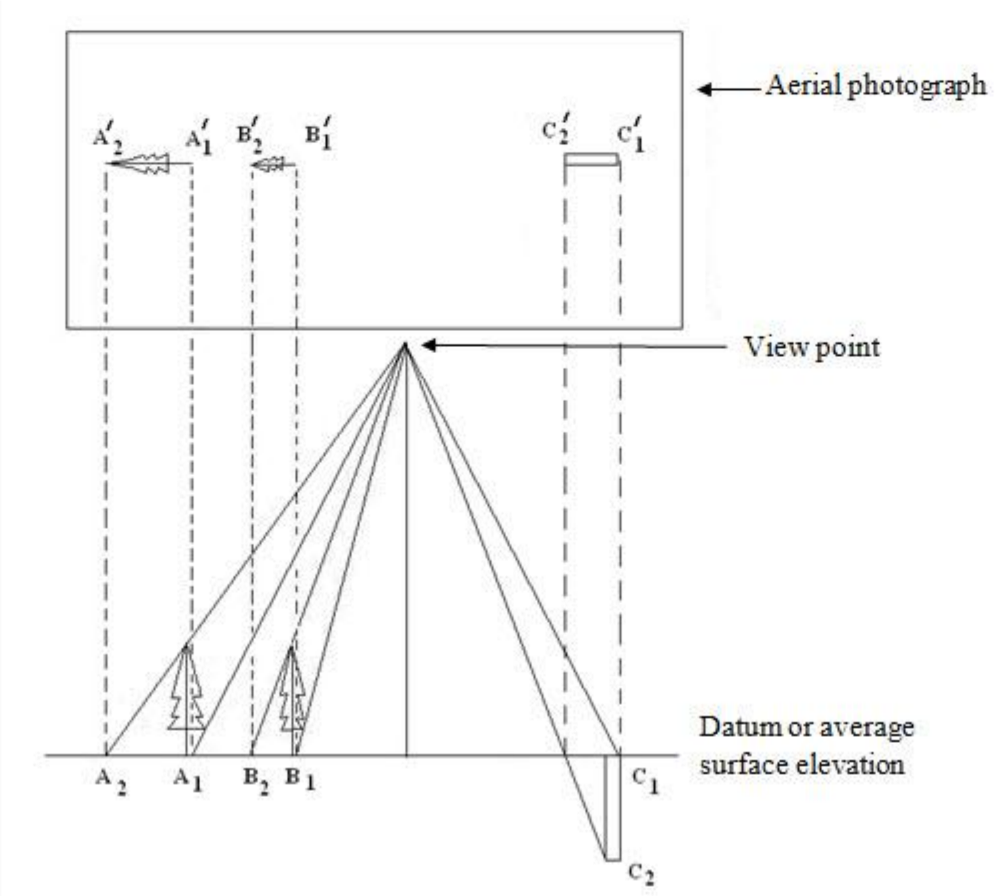


Fig. 7.8. Relief displacement in aerial photography.

Parallax: The term parallax refers to the apparent change in relative position due to change in viewing position. Objects near to the observer will move faster than the objects of far. When one views objects from the window of a moving train, objects nearer to the train will move faster than objects are at far. More details are available in lesson no. 8.

Orthophotos: (These are also referred as Orthoimages, which are the digital version of orthophotos) that has been orthorectified. They do not contain relief displacement, tilt, scale variation caused by different errors includes aircraft movement, camera tilt, curvature of earth, elevation changes in topography etc. Therefore like photos, orthophotos show actual detail and like maps they have only one scale.

7.4 Photographic Scale

The scale of a map or photograph is defined as the ratio of distance measured on the map to the same distance on the ground. The amount of detail in an aerial photograph depends on the scale of the photograph. Scales may be expressed as unit equivalents or dimensionless representative fractions and ratios. For example, if 1 mm on a photograph represents 25 m on the ground, the scale of the photograph can be expressed as 1mm =25m (Unit equivalents), or 1/25,000(representative fraction) or 1:25,000 (ratio).

A convenient way to clear confusion between large scale and small scale photograph is that the same objects are smaller on a small scale photograph than on a large scale photograph. For example, two photographs having scale 1:50,000 and 1:10,000. Aerial photo with scale 1:10,000 images shows ground features at a larger, more detailed size but less ground coverage than 1:50,000 scale photo. Hence, in spite of its smaller ground coverage, the 1:10,000 photos would be termed the large scale photo.

The most straightforward method for determining photo scale is to measure the corresponding photo and ground distances between any two points (Fig 7.9). The scale S is then computed as the ratio of the photo distance d to the ground distance D.

In the Figure 7.8 the triangle ΔLab and ΔLAB are similar.

Hence, ao/AO=Lo/LO

or, d/D= f/H

S = d/D= focal length/flying height

where,

S= scale of photograph

d= distance in photograph

D= distance in ground

f= focal length

H= flying height

Hence Scale of a photo α focal length of camera (f)

α 1/flying height (H)

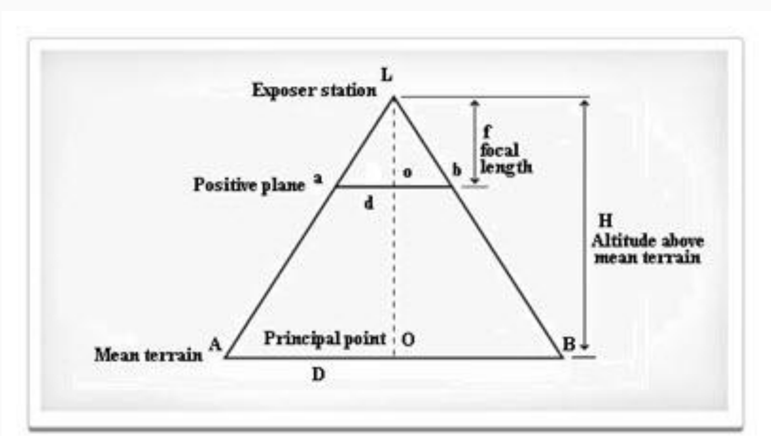


Fig. 7.9. Measurement of scale of an aerial photograph.

7.5 Ground Coverage of Aerial Photograph

The ground coverage of a photograph is, among other things, a function of camera format size. For example, an image taken with a camera having a 230×230 mm format (on 240 mm film) has about 17.5 times the ground area coverage of an image of equal scale taken with a camera having a 55×55-mm format (on 70-mm film) and about 61 times the ground area coverage of an image of equal scale taken with a camera having a 24×36-mm format (on 35-mm film). The ground coverage of a photo obtained with any given format is a function of focal length (f) and flying height above ground (H). The area covered in a photograph or ground coverage is inversely proportional to the scale of the photograph. Hence, for a constant flying height, the width of the ground area covered by a photo varies inversely with focal length and directly with flying height above terrain. Photos taken with shorter focal length lenses have larger areas of converge than taken with longer focal length lenses. On the other hand the ground coverage increases with increase in flying height.

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LECTURE NOTES

2016

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CONTENTS

1. SENSORS AND PLATFORMS FOR ACQUISITION OF AERIAL AND SATELLITE IMAGE DATA

This chapter will discuss the various sensors that are available for the acquisition of image and other data for applications in photogrammetry and remote sensing, as well as for data collection for GIS, and subsequent analysis. 'Photogrammetry and remote sensing' are often traditional terms used for the extraction of metric and semantic information respectively from aerial, including unmanned aerial systems (UAS, also referred to as UAV, Remote Piloted Aerial Systems RPAS and drones) and satellite images, as well as from images taken at close range to an object. The terms will be used often in these notes. A typical definition is:

Photogrammetry and Remote Sensing is the art, science, and technology of obtaining reliable information from non-contact imaging and other sensor systems about the Earth and its environment, and other physical objects and processes through recording, measuring, analyzing and representation.

1.1 Types of image sensors

Sensors used in photogrammetry and remote sensing can be separated into 'passive' and 'active'. Passive sensors simply record light, usually solar radiation that is projected by a lens system onto a digital sensor. In the case of close range applications (distances nominally less than 300 metres), the lighting may be from artificial sources. Active sensors emit their own illumination and then record the time and intensity of the signal that is reflected from the ground or other objects.

Passive Sensors

- Digital cameras and film cameras, which are referred to 'analogue photographic sensors', record visible and near infra-red electro-magnetic radiation. Film images can be digitised for use in digital photogrammetric systems although this process is being phased out.
- Pushbroom or Whiskbroom (sometimes referred to as electro-optical sensors), which scan the terrain surface, comprising special devices for detecting and then recording multispectral images of reflected electromagnetic radiation.

Active Sensors

- Radar sensors emit microwave radiation, and record the time of transmission to the ground and back to the sensor, as well as the intensity of the reflected radiation from the terrain surface. All current systems are based on Synthetic Aperture Radar (SAR). Interferometric SAR (InSAR) is a technique used to determine elevations from SAR images based on the phase differences between SAR images received by two separate antennas.
- Airborne Laser Scan (ALS) or lidar (**Li**ght **D**etection **a**nd **R**anging and also written as LiDAR) are techniques for determining elevations on the terrain surface or characteristics of vegetation canopy from aircraft or satellite. Terrestrial Laser Scanners (TLS) are ground based lidar systems. Only airborne lidar sensors will be discussed in this course.
- Other forms of active imaging may also be used for measurement, such as electron microscope and X-rays.

1.2 Platforms

The platforms on which the sensors are placed may be an aeroplane, UAS (unmanned aerial system), a satellite in space, or a suitable mount on or near the terrain surface, if close range imaging is undertaken. The principles of the geometry of sensors will not normally be affected by the platform on which they are placed. However, accuracies of extracted information from images will depend on the distance of the sensor to the object. For aerial photography, the positions of the sensor will be determined by Real-Time Kinematic (RTK) from GNSS receivers installed on the aircraft, while tilts of the camera may be determined by an inertial measuring unit (IMU), also referred to as inertial navigation system (INS). The three position and three tilt parameters are referred to as the parameters of *exterior orientation* of the sensor. These systems may be integrated to provide what is referred to as *direct orientation* of the sensor. The exterior orientation of an earth observation satellite in space may be determined by GNSS, satellite tracking systems and star trackers. In close range photogrammetry, the position of a camera can usually be determined on the ground by standard survey methods.

1.3 Digital Aerial Cameras

Digital aerial cameras have only become available in about the last 15 years and have now replaced film cameras in many parts of the world. Their design must satisfy the need to produce high quality images and also provide a wide coverage of the terrain surface. Modern digital aerial cameras continue to be improved as new digital imaging technologies are developed. The characteristics of modern digital aerial cameras are as follows:

- They acquire high spatial resolution images of more than 200 Mpixels, comprising Ground Sampling Distance (GSD) of 5 cm and larger with high geometric accuracy
- They have adequate angles of field of coverage for efficient for geospatial information extraction
- They take advantage of the particular characteristics of digital image acquisition, with quantization levels of 11 bit or 12 bits, that is, 2048 or 4096 grey scale values and improved image quality.
- They acquire 4 bands of multispectral images with the same or lower spatial resolution as the panchromatic images.

Currently digital aerial cameras used in photogrammetry are classified as '*high-resolution'* and '*medium resolution'*. High spatial resolution digital aerial cameras have the potential to collect hundreds of Mpixels while medium spatial resolution cameras usually are based on a smaller single area array chips with about100 Mpixel.

Advantages of using digital images are:

- Image processing for target location, automatic height measurement and semi-automatic information extraction can be carried out on the image data, using stereo observation on the screen
- Digital orthophotos can be automatically derived from the data, thus providing for much greater flexibility in correcting variations in density of neighbouring images
- Future developments based on machine vision techniques will enable the automatic extraction of semantic information from the images

Disadvantages of using digital images:

- An analogue photo is an extremely efficient way to store information about an object and can be stored for decades.
- Algorithms for the automatic extraction and identification of information from images have not been developed, which impacts on the type of products that can currently be extracted from digital aerial images.
- Storage of digital image data requires regular backups onto new storage technology and as the old storage devices become obsolete.

Since digital aerial cameras are undergoing continual development and new models and types are being released onto the market on a regular basis. In the development of the new digital high spatial resolution aerial cameras two approaches have been adopted as described below.

1.3.1 High Spatial Resolution Digital Aerial Cameras

(i) Systems based on CCD linear arrays - referred to as 'Pushbroom' sensors. Linear array scanners acquire data by scanning the terrain with one or more linear arrays oriented normal to the flight direction as the aircraft moves over the terrain, (likened to a broom sweeping a surface). The Leica system comprises three linear arrays, one pointing forward, one pointing vertically and one pointing backwards to acquire **three separate overlapping images** of the terrain surface. The acquisition of overlapping images is essential for determining 3D coordinates of objects, including elevations, as will be described later in these notes. The systems are usually designed for acquiring panchromatic (ie black and white) and also colour or multispectral images including in the short wavelength infrared region of the spectrum (CIR – colour and infrared), with wavelengths up to about 0.9 μm wavelength, at the same or reduced resolution as the panchromatic images. **An integrated GNSS/IMU (Inertial Measuring Unit) system is essential** for this camera configuration for determining camera position and tilts, because the image acquisition is a continuous process and not frame based. Images from pushbroom sensors cannot be processed by standard frame image software. An example of this type of digital camera is:

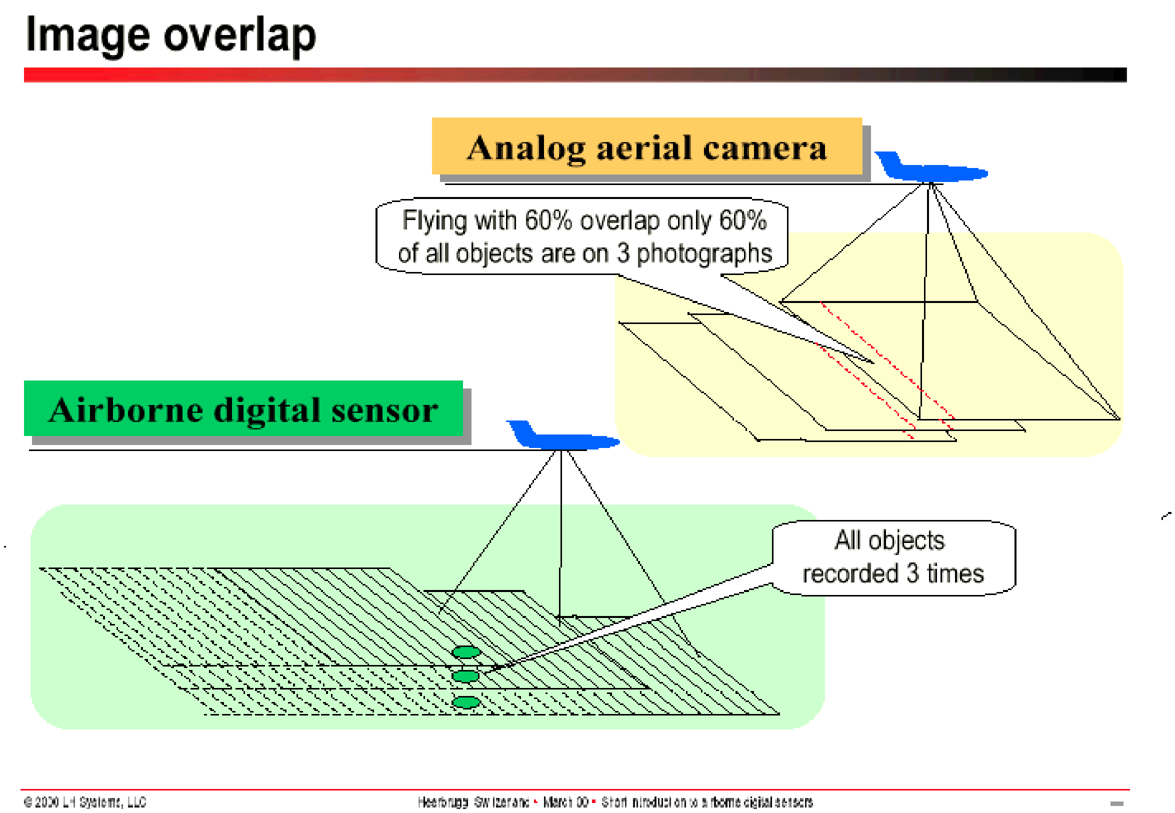


Figure 1.1 Comparison of linear array camera of Leica Geosystems (ADS100) and normal frame camera configurations

- **Leica ADS100** (Figure 1.1) acquires overlapping panchromatic and multi-spectral images, all with the same resolution, by 3 CCD linear arrays, looking forward, vertical and aft, by the SH100/SH120 camera heads.

(ii) Frame systems based on area arrays. These systems involve single large rectangular array or multiple small area arrays whose images are stitched together to form larger usually rectangular frame images. These images can be processed using standard digital photogrammetric software for frame images. GNSS/IMU systems are not essential for the operation of this type of camera, but some or all components of such a system may be included as an option. Cameras based on this configuration are:

- The DMC (Leica) cameras. The 1st generation was composed of 4 separately directed camera heads (7kx4k) for panchromatic images which were stitched together, and 4 camera heads (3kx2k) for 4 channels of multispectral images, each pointed so that they cover the required area of the terrain. The 2nd DMCII and 3rd generations DMCIII (various versions with different chip sizes are available) are based on a single monolithic area array. The multispectral images have a resolution 3.2 times less than the panchromatic images. The 3rd generation with 390 Mpixels is based on CMOS sensor (as opposed to CCD sensor for DMCII) with pixel sizes of 3.9 μm, 26113 pixels across the swath and 15,000 in the flight direction, together with mechanical forward motion compensation (FMC).
- The Vexcel UltraCam (Figure 1.2) is based on 'syntopic' imaging by 9 small format frame cameras for the panchromatic images, which are recorded sequentially during flight with 4 CCD arrays, so that the image acquisition for all arrays is based on the perspective centres of the cameras cones being in the same position. The separate images are stitched together to form a single area array. The multispectral images are acquired with 4 cameras cones each on a single CCD array at a lower resolution than the panchromatic images.

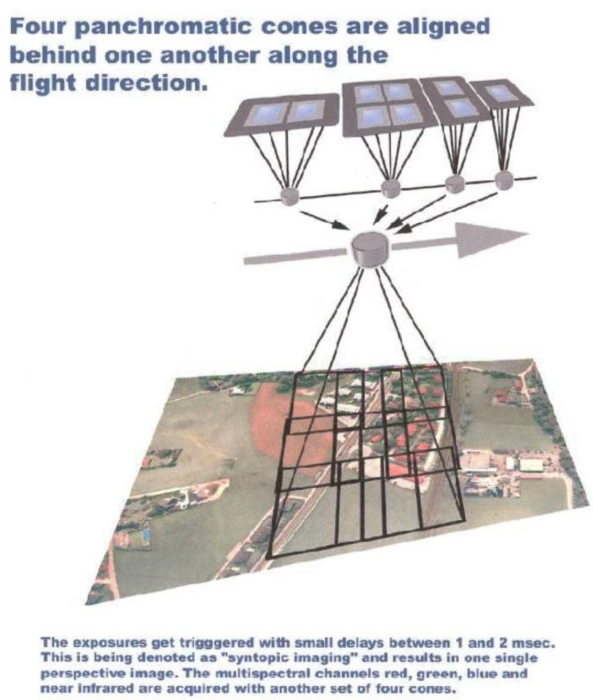


Figure 1.2 Digital frame camera (UltraCam) configuration by the Microsoft Vexcel Camera

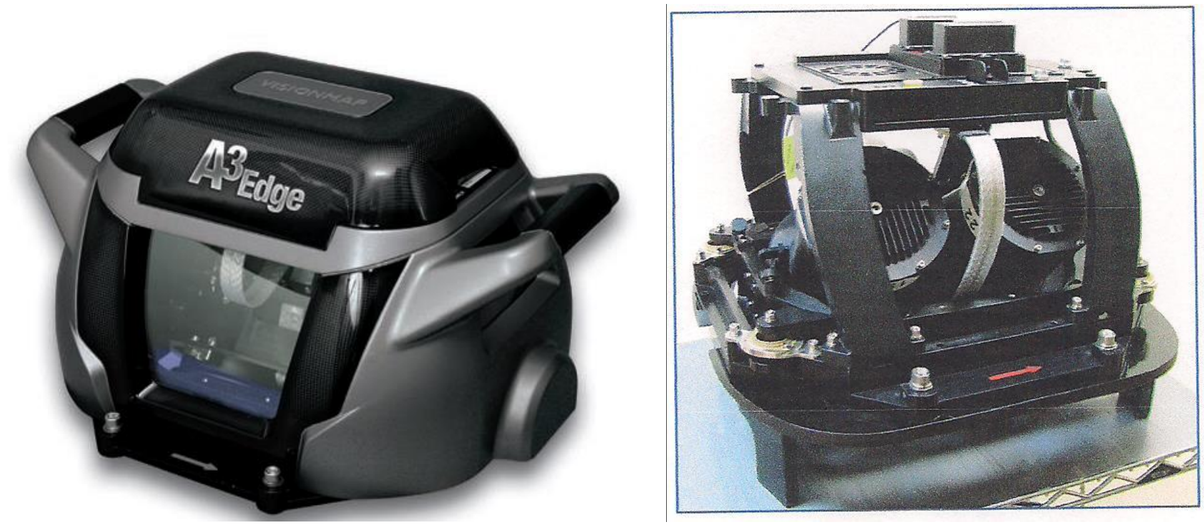


Figure 1.3 VisionMap A3 Edge camera and scanning system

- VisionMap System A3 Edge (Figure 1.3) from Israel is based on a double camera scanning system by which sequences of frames frames are imaged in the cross-track direction achieving very wide angle coverage of 100º FOV (field of view). The systems consist of dual CCD arrays with 300 mm lenses, a fast data compression system and a dual frequency GNSS system but no IMU is required. Given the unusual design, special fully automated software is required to process the data, which is also supplied by the company. The company claims that the advantages of this system are the higher productivity than other digital aerial cameras since it has much higher angle of field and very high scan rates.

1.3.2 Medium Resolution Digital Aerial Cameras
Medium resolution cameras, as the name suggests, are of lower resolution than the high resolution cameras, with most comprising a single area array of varying sizes for the capture of multi-spectral images. There are a number of these cameras on the market developed by different companies and they are undergoing considerable development with resolutions increasing as the development of chip technology improves. They are suitable for lower resolution aerial mapping or in combination with other forms of data, such as lidar.

1.3.3 General description of high spatial resolution near vertical aerial photogrammetric image acquisition
Most so-called ‘near vertical aerial photography’ is recorded with tilts from the vertical of less than 2˚. This has been traditional approach for film aerial photography and generally still applies for digital images. Such small tilts limit the distortions in the images which enable their processing to orthophotos to be less problematic. That is not to say that images with larger tilts cannot be processed to orthophotos, but the significant scale variations of photos with large tilts can lead to variations in the quality of the orthophoto, and large tilts can also cause important areas to be hidden behind buildings.

The theoretical assumptions made about the geometric processing of images are that all frame images are perspective projections. That is, all light rays forming an image pass through a single point, called a ‘perspective centre’. For linear array or pushbroom cameras the assumption of a perspective projection only applies along each scanline formed by each linear array. All aerial images are subject to certain distortions, due to tilts of the camera, elevation differences on the terrain for aerial images, deformations in the image formation process, particularly in the lens and atmospheric refraction which is a minor effect.

All photogrammetric measurements are based on overlapping images in order to obtain 3D object geometry. Typical overlaps on film images were 60% along track (forward overlap) and a minimum of 15% to 20% side overlap (sidelap) as shown in Figure 1.4. For the new era of digital imaging using area array cameras, the forward overlaps can be as large as 85%-90% and as high as 60% for sideways overlap between neighbouring strips of photography, also called ‘sidelap’. For linear array cameras the 3 overlapping strips are acquired simultaneously. The side overlaps between strips may be 15% or larger.

Close range photographs are recorded with attitudes, ie tilts, designed to suit the particular application, but usually the optical axis of the camera will be directed approximately horizontally. The procedures for close range photography are therefore significantly different from aerial photography. Tilts may be much larger than 2˚ and the overlaps between photos may be up to 100%.

The original images recorded for film and digital photography are in the so-called negative plane as shown in Figure 1.4. The reproduction of the equivalent positive for a digital image is a trivial task. Therefore, all reference to images in these notes will assume the positive image is used.

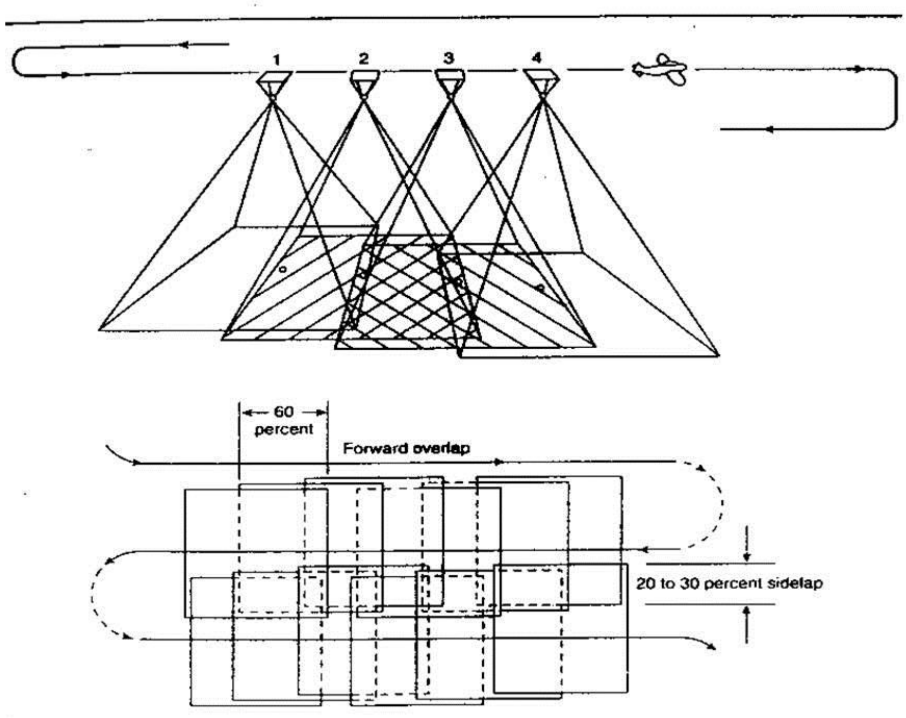


Figure 1.4 Configuration of aerial imaging with minimum overlap

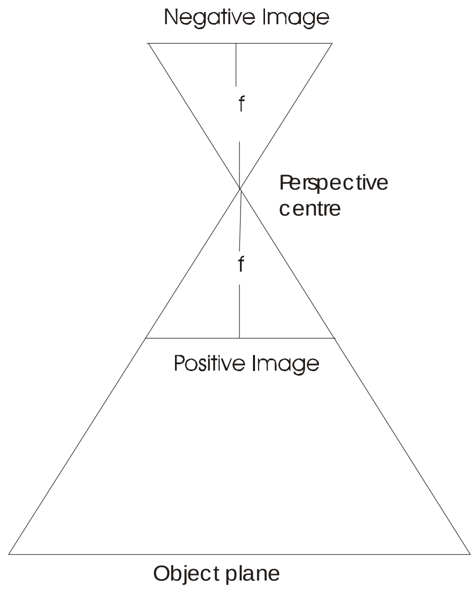


Figure 1.5 Geometric relationships between positive and negative image planes

The *principal distance* 'f', often called the *focal length*, defines the distance between the negative and positive image planes and the perspective centre (some textbooks use the symbol 'c' for the principal distance to avoid confusion with 'f' the focal length). As stated above the perspective centre defines the theoretical point in the camera, through which all light rays forming the image, are assumed to pass. The *principal point* is defined as the foot of the perpendicular from the perspective centre to the image plane. Hence the 3 components of what is known as the camera's '*interior or inner orientation'* comprise x_0 and y_0 the x and y coordinates of the principal point and the principal distance 'f'. (see Chapters 2, 3 and 6 for more details of their definitions) These components must be known before accurate measurement can be made from photographs.

1.3.4 Lens Distortion

All lenses will be subject to errors in image geometry due to design factors and manufacturing. The design of a lens is a compromise between the competing requirements of high image quality and near zero errors in the geometry of the image. Lens distortion is the deviation from a straight line path of the rays forming points in the images, which pass from the object through the perspective centre to the image plane. The largest component of lens distortion is radial to the principal point (Figure 1.6), while smaller tangential components may also occur in some lenses, but not those lenses specially designed for aerial photography. Aerial cameras will be subject to radial lens distortion of less than 5 µm, which will be symmetrical about the centre of the photograph. Recent research on digital aerial camera lenses has revealed distortions considerably less than 5 µm. Some 'non-metric' cameras used for close range photogrammetry may have lens distortions of the order of 100 µm. Lens distortions are determined as part of the process of camera calibration. It has been shown that radial-symmetric lens distortion in aerial lenses can be modelled by an odd-order polynomial of the form $dr=k_1r^3+k_2r^5+k_3r^7$, where dr is the radial distortion of a point on the image plane and r is the radial distance of the point measured from the principal point on the image plane, k_1, k_2 and k_3 are constants describing the characteristics of the distortion of a particular camera.

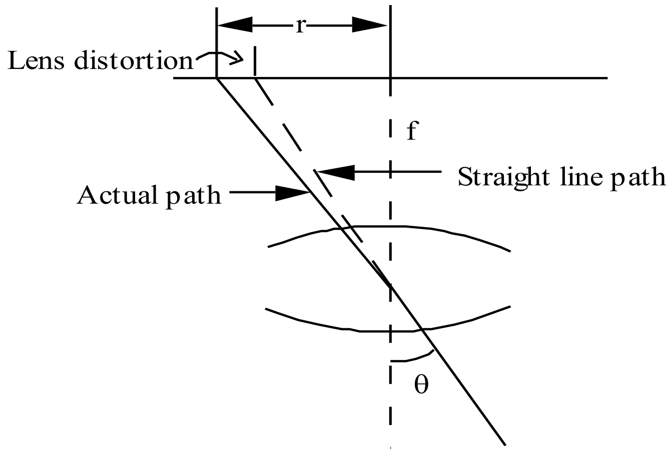
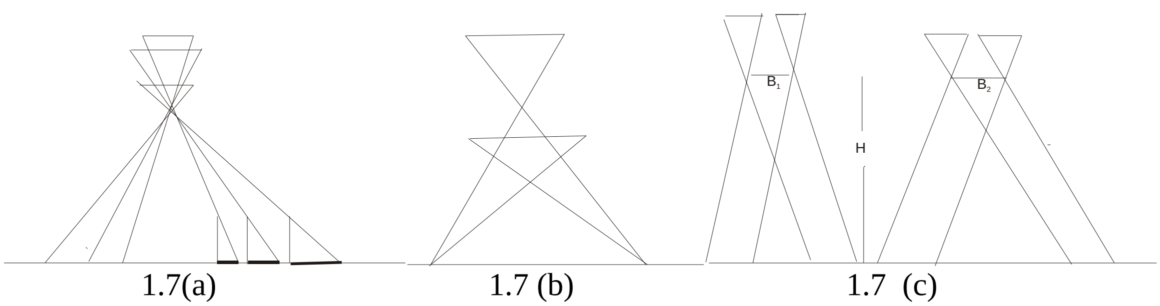


Figure 1.6 Radial lens distortion

1.3.5 Significance of the Angle of Field of a Camera

The angles of field of lens cones have a significant effect on the characteristics of photography acquired by cameras. The following general principles apply:

- The wider the angle of field, the greater the coverage for a given flying height - Figure 1.7a
- The same coverage, and hence the same scale of photography, can be obtained by varying the flying height for cameras with different angles of field. This means that for wider angle cameras, the flying height will be less - Figure 1.7b.



Figures 1.7a to 1.7c Examples of the relationships between the different camera lens cones

- The wider the angle of field, the greater will be the dead areas, ie obscured or occluded areas, hidden by buildings and steep terrain - Figure 1.7a
- The wider the angle of field, the larger the Base/Height (B/H) ratio - Figure 1.7c. In principle this will have an impact on the accuracy of height measurement.

1.3.6 Measurement of Exposure

Exposure is dependent on both the aperture in the lens and the exposure time. Doubling the aperture should enable halving the exposure time. The aperture is defined by an f/number, as a fraction of the focal length of the camera, as follows:

$$\text{Aperture f/number} = \frac{\text{Focal length (mm)}}{\text{Apertures stop diameter (mm)}} \tag{1.1}$$

Increasing the aperture number, that is, reducing the size of the aperture, will reduce the effects of aberrations in the lens, and also increase the depth of field of the lens, ie the range over which objects are in focus. However, reducing the aperture size will also require an increase in exposure time, which may result in large image movement if either the object or camera is in motion, as is the case for aerial photography. To reduce the effects of image motion all modern digital aerial CCD frame cameras incorporate, *forward motion compensation* (FMC) by TDI (Transfer Delay and Integration). TDI is not possible with pushbroom cameras or CMOS sensor cameras, and hence other procedures need to be developed such as mechanical movement of the image plane. For close range photogrammetry, apertures would be selected to suit the application and if an object is in motion synchronized multiple cameras should be used.

1.3.7 Multi-Camera Systems

While multi-camera systems have existed for about 80 years, they have only come into regular use in modern aerial surveys over the past few years. The first modern multi-camera system was introduced by Pictometry more than 10 years ago. The system was based on 5 low cost cameras, one looking vertically and the other 4 looking obliquely in 4 directions at right angles as shown in the Figure 1.8. Subsequently other companies have developed their own version of multi-camera systems with high quality metric camera lenses, as for example the Vexcel Osprey, the IGI DigiCAM, comprising various options from one to five cameras (Penta DigiCAM), the Leica camera based on the RCD30 medium format camera and with up to 5 cameras.

Figure 1.8 Configuration of Pictometry imaging

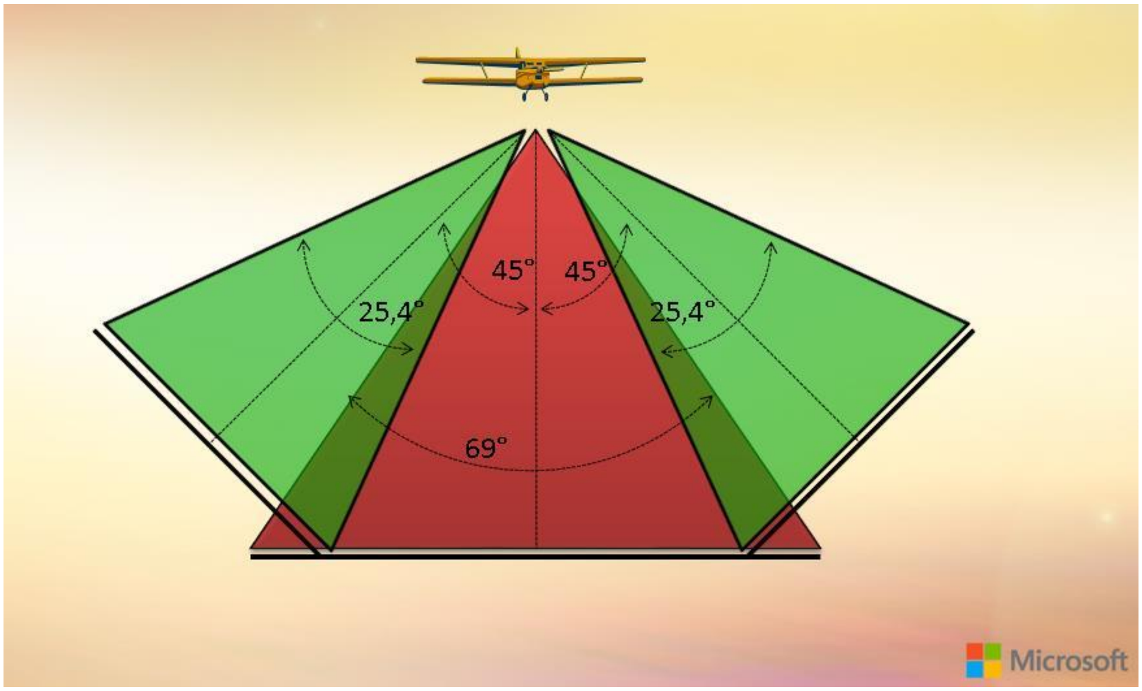


Figure 1.9. Plan view of overlaps between vertical and oblique images for Vexcel Osprey camera

The UltraCam Osprey camera for example, is designed with overlaps between the oblique and nadir images (Figure 1.9). All cameras are mounted rigidly on the platform with photogrammetric grade accuracy and are calibrated with respect to geometry and radiometry. These cameras are relatively new and therefore their applications are still being developed.

1.4 High Spatial Resolution Pushbroom Satellite Sensors

The design of satellite sensors is based on their specific purpose, but the recently developed so-called 'high spatial resolution' satellites are based on pushbroom sensors. They have resolutions or ground sampling distances (GSD) varying from about 2.5 m to about 0.3 m. Usually they detect both panchromatic images, which are black-and-white and cover the whole of the visible spectrum at the highest resolution, and up to 8 multispectral bands that have a GSD of 3 to 4 times larger than the panchromatic images. Stereo images are usually acquired by tilting the satellite forward and backwards during orbit. Most of these satellites are referred to as 'agile', since they can also acquire images by tilting sideways. Hence these satellites can acquire images within a day or so over anywhere on the earth. Typical currently available satellites with their respective ground sampling distances (GSD) are:

– IKONOS II (USA) launched in 1999 with 0.80 m panchromatic images and 3.2 m multispectral images
– Quickbird (USA) launched in 2002 with 0.60 m panchromatic images and 2.4m multispectral images; it is but now no longer operating but archive images are available
– WorldView-1 (USA) launched in 2007 with 0.50 m panchromatic images only
– CartoSat2B (India) launched in 2007 with 0.8 m panchromatic images
– GeoEye 1 (USA) launched in 2008, with 0.41 m panchromatic images and 1.65 m multispectral images

– WorldView-2 (USA) launched in 2009, with 0.4 m panchromatic images (originally supplied at 0.5 m resolution but currently available at 0.4 m resolution) and 1.85 m multi-spectral with 8 bands
– PLÉIADES (Europe) launched in 2011 with 50 cm Panchromatic and 2 m multi-spectral
– WorldView-3 (USA) launched in August 2014 with a GSD of 31 cm and 1.3 m multispectral.
– Terra Bella (formerly Skybox and now owned by Google) series of 21 satellites of which 2 are currently in orbit and all planned to be launched by end 2017. The satellites acquire panchromatic and multispectral images with <0.9 m spatial resolution and 1.1 m spatial resolution video images.
– Planet labs with over 100 microsatellites called 'Dove', with 3-5 m spatial resolution acquiring RGB using commercial grade CCD sensors. Their claim: 'In 2016 Planet Labs will have enough satellites in orbit to image the entire globe, every single day.'
– Urthecast has a sensor mounted on the International Space Station (ISS) can provide medium and high resolution imagery, with up to 75 cm pansharpened imagery available as well as video data.

The geometric processing of images from these satellites requires different procedures as will be described later.

1.5 MultiSpectral Sensing

Since the CCD technology used in pushbroom sensors is unable to detect electro-magnetic wavelengths longer than about 0.9 μm, other technologies and sensors are required to detect multiple bands with longer wavelengths for extraction of semantic information for remote sensing applications. These systems are referred to as either multispectral or hyperspectral sensors and usually described as 'whisk-broom' sensors although the designs of commercial satellites are 'commercial-in-confidence'. They are usually based on an optical scanning system, comprising a rotating mirror for aerial sensing and an oscillating mirror for satellite systems.

A typical configuration of these systems is based on either:

- **Across-track scanners** scan the Earth in a series of lines using a rotating mirror (A). The lines are oriented perpendicular to the direction of motion of the sensor platform. Successive scans build up a two-dimensional image of the Earth´s surface. The incoming reflected or emitted radiation is separated into spectral components that are detected independently and dispersed into their constituent wavelengths. A bank of internal detectors (B), each sensitive to a specific range of wavelengths between 0.4 µm at the blue end of the visible spectrum, to about 15µm in the thermal region of the spectrum, detects and measures the energy for each spectral band and then, as an electrical signal, they are converted to digital data and recorded for subsequent computer processing. The width of the bands varies according to the system design and wavelength being recorded. They may be of the order of several µm for thermal sensors to as little as 1-2 nm for hyperspectral systems, which may detect and record hundreds of very narrow bands. The IFOV (C) of the sensor and the altitude of the platform determine the ground resolution cell viewed (D), and thus the spatial resolution. The angular field of view (E) is the sweep of the mirror, measured in degrees, used to record a scan line, and determines the width of the imaged swath (F). Because the distance from the sensor to the target increases towards the edges of the swath, the ground resolution cells also become larger and introduce geometric distortions to the images. Also, the length of time the IFOV "sees" a ground resolution cell as the rotating mirror scans (called the dwell time), is generally short which influences the design of the spatial, spectral, and radiometric resolution of the sensor.

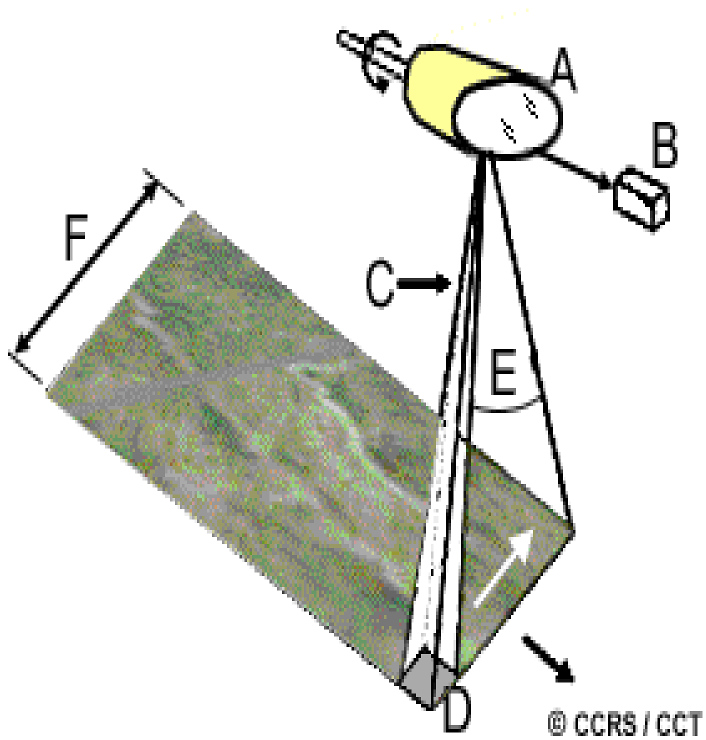


Figure 1.10 Multispectral Scanner Using a Scanner Mirror

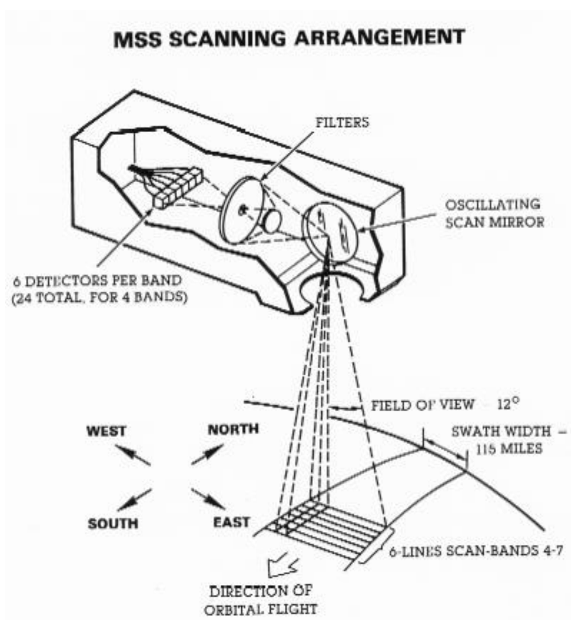


Figure 1.11 Oscillating mirror scanner used in Landsat satellites

- Satellite systems, such as a number of Landsat satellites, are usually based on an oscillating mirror as shown in Figure 1.11. The swath width of an oscillating mirror scanner can be limited to 100 km or so, based on small rotations of the mirror, and hence is preferred over a rotating mirror which would record a very large part of the globe from space.

1.7 Radar Sensors

Airborne or spaceborne imaging radar sensors are active microwave remote sensing systems which emit radiation from an antenna, and record the time of transmission for the radiation to be returned to the antenna. They primarily measure angles and distances based on the time of travel the reflected radiation, T. The radar beam is emitted sideways from the platform, approximately normal to the direction of flight, at an inclination to the horizontal, as shown in Figure 1.12.

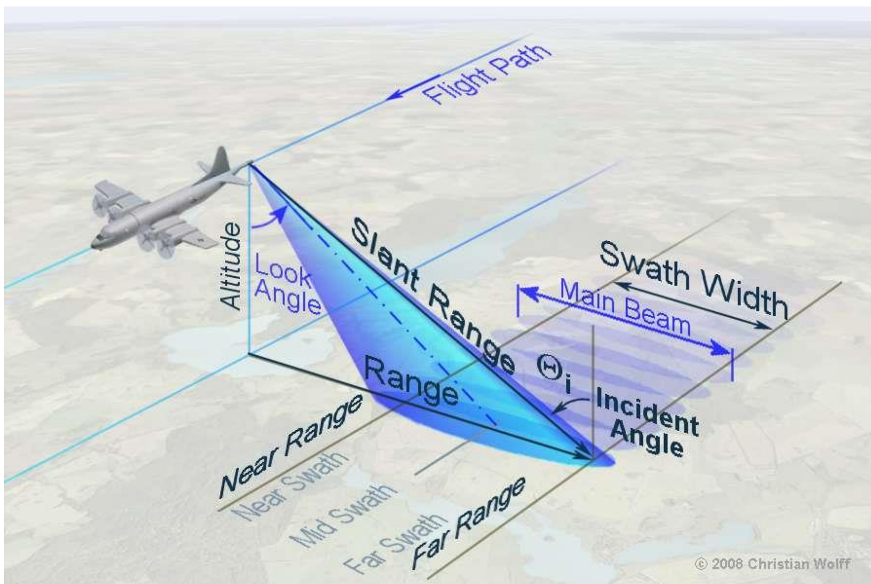


Figure 1.12 Data Acquisition by Radar System

The range, R, is therefore derived from the formula:

$$\mathbf{R=cT/2} \qquad (1.2)$$

where c is the velocity of the electro-magnetic radiation in the atmosphere. The range resolution, at right angles to the flight direction, is a function of the *pulse repetition frequency* (PRF) of the radar signal transmission. The azimuth resolution, in the direction of flight, is a function of the length of the antenna. To achieve a small azimuth resolution of the image, it is necessary to use an extremely long antenna (several km). A *synthetic aperture radar*, or SAR, overcomes this problem of achieving a suitable spatial resolution in the azimuth direction by synthesising a very long antenna, by processing the reflected signals that are acquired as the platform progresses. The azimuth resolution of SAR images can be shown to be approximately equal to d/2, where d is the actual length of the antenna on the platform. This means that the resolution of the images will decrease ie, improve as the antenna decreases in size. In addition, the azimuth resolution is independent of the elevation of the platform and the frequency of propagation. The limits of azimuth resolution are constrained, because the complexity of the radar system. Data storage and processing requirements all increase with increasing range and wavelength, while power requirements increase sharply as the antenna decreases in length.

Radar images are affected by geometric errors, which are functions of the combination of the elevation angle of the signal and variations in the elevations in the terrain. These errors are layover, foreshortening and shadow, as shown in Figure 1.13. Layover is caused by the imaging characteristics of radar, since ranges (distances) are measured from the antenna to the terrain. Elevated points are closer to the antenna than points below them and these points will therefore be imaged closer to the nadir point in the image. This effect will increase as the elevation or look angle of the emitted radiation decreases. Foreshortening will occur when slopes in the terrain facing the antenna are compressed in size due to the effects of layover. Shadows are image voids caused by certain areas being hidden from the radar beam, because of the intervening terrain features. In addition to errors caused by terrain elevations, the continuous process of data acquisition of SAR data will result in geometric distortions caused by variations in the platform attitude during flight.

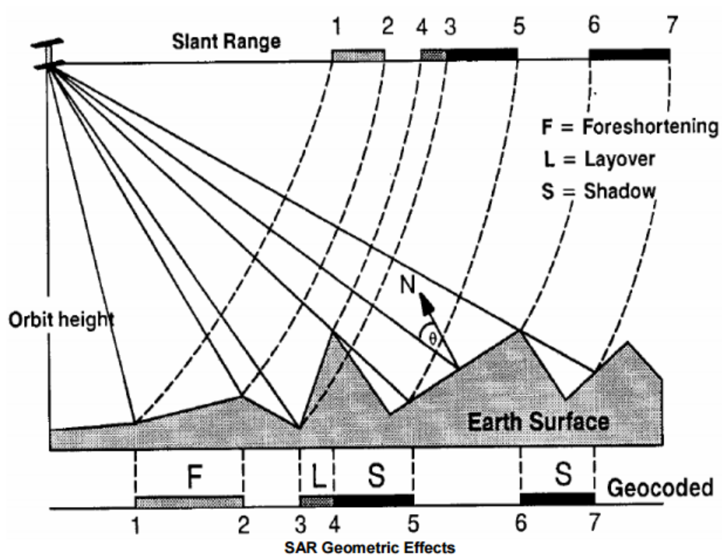


Figure 1.13 Geometric Distortions in SAR Data

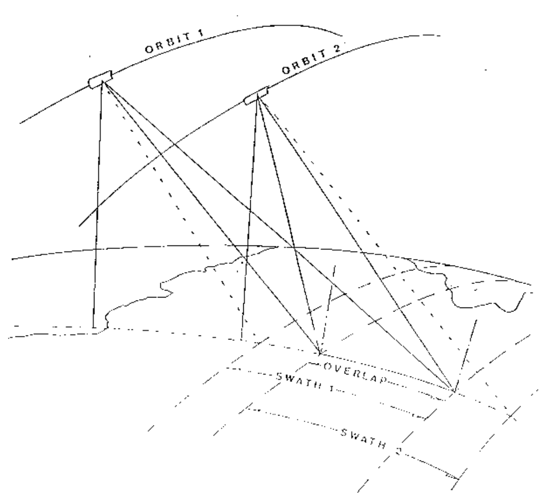


Figure 1.14 Same–side data acquisition for stereo observation of radar images

1.7.1 Overlapping Radar Images

Overlapping images for elevation computations are derived by the acquisition of two parallel passes as show in Figure 1.14, which demonstrates the so-called 'same-side' stereo configuration. That is, the antennas view the terrain from the same side of the platform. Opposite side configuration is also possible but there are limitations in the suitability of this configuration for computation of elevations because of the large geometric distortions as described above. The accuracy of elevation computation from overlapping radar images is usually poor. A more accurate method being developed is referred to as interferometric radar.

1.7.2 Interferometric SAR (InSAR and also referred to as IfSAR)

A SAR image of a scene comprises amplitude and phase information. In Figure 1.15, two antennas A_1 and A_2 with a baseline length B, record the echoes of the signal emitted by one of the antennas. The range distance from an illuminated point on the ground to antenna A_1 is r, while $r + \delta r$ is the distance to antenna A_2 from the same point. The difference in phases in wavelengths, between the signals received at the two antennas can be used to determine the difference in range δr and hence terrain elevations with high accuracy. After registering the two images, the phases are calculated and differenced on a pixel by pixel basis, resulting in a phase difference image or interferogram.

Accuracies of elevations determined by interferometric SAR depend on the parameters of the radar system and can be better than 0.5 metre for low wavelength airborne radar systems. Accuracies of the order of several metres are achievable with spaceborne systems. An approach called differential InSAR is based on the acquisition of SAR images in two or more epochs and can be used for monitoring small changes in elevation, such as those that occur due to earthquakes and mine subsidence. Accuracies of differential InSAR can be of the order of 1 cm, even when the images are recorded from space.

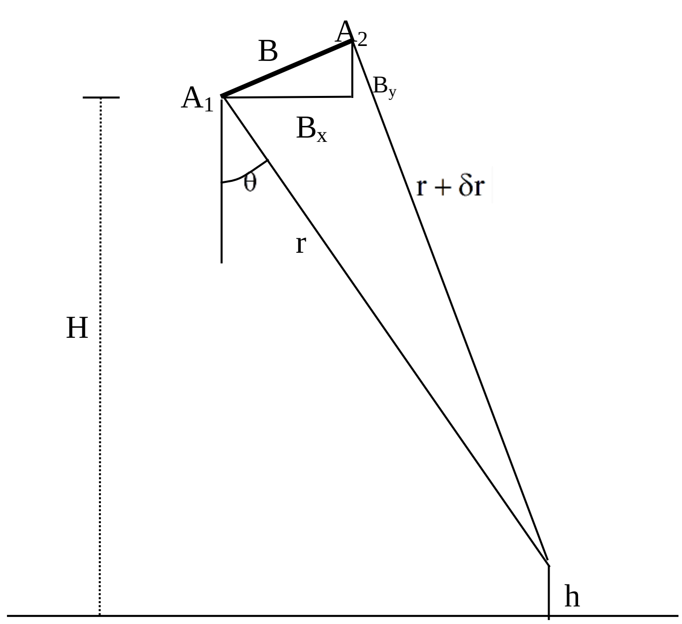


Figure 1.15 A schematic diagram of InSAR.

1.8 Airborne Lidar or Airborne Laser Scanning (ALS)

In airborne lidar (**Li**ght **D**etection **A**nd **R**anging also written as LiDAR) or airborne laser scanning (ALS), a laser scans the terrain surface normal to the flight direction of an aircraft AS SHOWN IN Figure 1.16. The measured distance from the aircraft to visible points on the terrain surface will enable the position and elevation of points to be determined. A lidar system includes the following equipment:

- The laser scanning normal to the flight direction for which the range to the object and rotation angle of the scanner are determined
- GNSS equipment to determine the location of the aircraft based on kinematic measurements
- IMU to continuously determine the tilts of the aircraft.

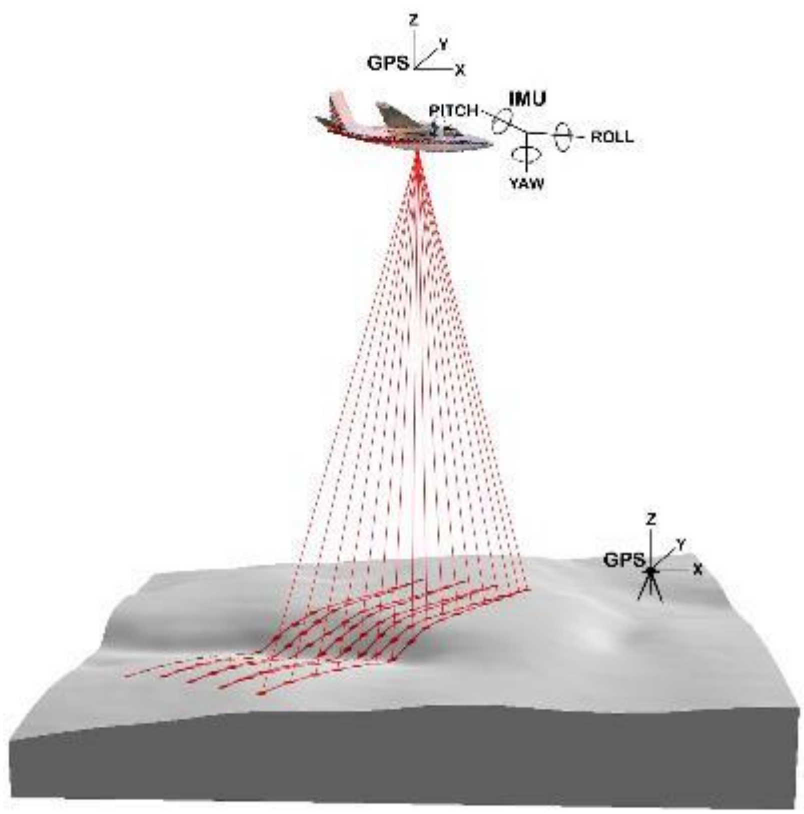


Figure 1.16 Airborne lidar system

Lidar systems acquire a dense set of elevation posts (XYZ coordinates), referred to as a *point cloud*, at a separation typically of 1 m or less for modern lidar systems, that represent a digital surface model (DSM) of the visible terrain, that is, objects such as buildings and trees, but also the terrain surface if the laser beam penetrates the vegetation. The accuracy of the elevation posts is of the order of 10-20 cm, although tests have shown that accuracies of interpolated hard surfaces better than 5 cm are achievable. The separation of the posts will depend on the configuration of the lidar equipment and the scanning frequency, which is increasing as the systems are developed further. This technology is also referred to as 'linear lidar' to differentiate them from single photon lidar (SPL) and Geiger systems as described below.

Some lidar systems can register multiple returns or echoes of the laser beam, but most systems will register, as a minimum the *first* and the *last pulses*. For example, if the laser beam hits a tree, a part of the laser beam will be reflected by the canopy, resulting in the returned signal being registered by the sensor as the *first pulse*. The rest of the beam may penetrate the canopy and, thus be reflected from further below the top of the tree, maybe even by the soil. The *last pulse* registered by the sensor corresponds to the lowest point from which the signal was reflected. In certain cases, the difference in elevation between the first and last pulses can be assumed to measure the heights of trees or buildings.

Along with the time of transmission of the signal from the sensor to the terrain and back to the sensor, the *intensity* of the returned laser beam is also registered by lidar systems. Lidar systems typically operate in the infrared part of the electromagnetic spectrum and therefore the intensity can be interpreted as an infrared (IR) image. However, this intensity image is usually under-sampled and thus very noisy, because the footprint of the lidar is about 0.3 m-0.5 m and the average sampling point distance is 0.3 m to 1 m apart. As well as the laser data, images of the terrain surface may also be recorded by a separate medium or high resolution digital camera. These images may be used to identify the location of points on the terrain surface. The combination of colour and the IR as multi-spectral images can provide valuable information for information extraction of the terrain surface. Multi-spectral lidar systems have now been developed incorporating 2 or 3 lidar beams with different wavelengths.

Technological advances have resulted in new systems being developed, referred to as single photon lidar (SPL) and Geiger systems, which enable the collection of data with much higher point densities. They offer better use of the photons generated by the laser source, resulting in a dense point cloud from the same or a less efficient laser source. These systems are not available on the market year, but they are likely to be available in a year or so.

Typical applications of lidar data may include:

- DEMs of the bare earth surface
- Beach erosion studies
- Infrastructure analysis
- Flood risk analysis, flood simulation, and drainage design
- Ground subsidence
- Visibility analysis
- Telecommunications planning
- Noise propagation studies
- Volume change monitoring
- Buildings extraction for 3D city models
- Forest analysis

The economics of lidar equipment require it to be used over large areas, and hence GBytes of data are likely to be acquired in a single mission (250,000 points may be recorded in a few seconds). Therefore, it is essential that automatic processes are developed that enable the extraction of information from the lidar data. There are described in more details later in these notes.