Surveying 2

By Dr. Khamis Naba Sayl

المحاضرة الحادية عشر

Hydrographic surveying

Fundamentals of Hydrographic Surveying

Hydrography is the branch of applied science which deals with the measurement and description of the physical features of oceans, seas, coastal areas, lakes and rivers, as well as with the prediction of their change over time, for the primary purpose of safety of navigation and in support of all other marine activities, including economic development, security and defense, scientific research, and environmental protection." International Hydrographic Organization – June 2009

1.1 Introduction

More than half of the world's population lives within 100 km of its shores. The effects of denser coastal population and accelerating climate change can be seen in degraded (and even disappearance of) ecosystems, coastal erosion, over-fishing, marine pollution, and higher vulnerability to marine disasters such as tsunami or volcanic activity.

Marine environments (oceans, lake, rivers, swamps, wetlands) cover more than two-thirds of the Earth's surface, and are not easily accessible to direct observations. In the past 20 to 30 years technological advances have allowed us to discover and map much more detailed coastal and ocean bathymetry and to delineate shore boundaries , mostly through acoustic remote sensing.

Hydrography is that branch of physical oceanography that deals with measurement and definition of the configuration of the bottoms and adjacent land area of oceans, lakes, harbors, and other water bodies on Earth. Hydrographic surveying, in the strictest sense, is defined merely as the surveying of a water area; however, in modern usage it may include a wide variety of other objectives such as measurements of tides, currents, gravity, and the determination of physical and chemical properties of water.

The principal objective of most hydrographic surveys that are conducted by large government agencies like the National Oceanic and Atmospheric Administration (NOAA) is to produce nautical charts and mapping. NOAA uses very large vessels to obtain basic data for the compilation of nautical charts with emphasis on features that affect safe navigation. Other objectives of NOAA include acquiring the information necessary to produce related marine navigational products for coastal zone management, engineering, and scientific investigations. Other government agencies such as the US Army Corp of Engineers (USACE), the Naval Oceanographic Office (NAVO), the US Geological Survey (USGS), are tasked with hydrographic surveys for a variety of purposes Some state and local agencies as well as the private sector also have hydrographic survey capabilities

The US Army Corps of Engineers (USACE) is responsible to collect, process, and map hydrographic survey data for federally authorized civil and military navigation channels and shore protection projects throughout the US including Puerto Rico and the Virgin Islands. The main purpose of collecting hydrographic survey data is to be used by engineers and scientists to monitor channel shoaling conditions. Survey results in the form of a bathymetric map become a decision making tool for channel maintenance operations, channel deepening contracts, planning studies, environmental monitoring, near shore engineering designs, location (and sometimes removal of obstructions such as sunken vessels, sediment transport modeling, and beach nourishment projects. Other objectives include volume computations for fair and equitable payment on dredging contracts. The overarching reason to perform hydrographic surveys is to ensure safe navigation conditions for all commercial and public users within the limits of the federal waterways. Hydrographic surveys are very complex in terms of (electronic) equipment integration, logistics on field operations and costs.

Hydrographic surveys support a variety of activities including:

- **nautical charting,**
- **port and harbor operations (maintenance & dredging),**
- **coastal engineering (beach erosion and replenishment studies)**
- **coastal zone management**
- **offshore resource mapping**

On smaller scale local marine environments, the survey operations can be far less complex. Surveys conducted in shallow waters, lakes and rivers may invoke conventional (manual) surveying procedures.

Nautical Charting: Periodic hydrographic surveys must be performed to determine shipping channel conditions. Minimum controlling depths along with location of shoals and other critical information regarding safe navigation gets documented. Reports of Channel Conditions are accessible to waterway users.

Port and Harbor Operations: Survey data are required for effective management of water resources and harbor estuaries. Operations include maintenance dredging, debris removal for clear passage of vessels, environmental restoration, marine structural design, and many others.

Coastal Geomorphology: Hydrographic surveys provide data for morphodynamic classification of coastal areas from sea state (breaking wave heights), bathymetry, tide regimes (F-factor computed from tide constituents).

Coastal Engineering: Coastal mapping data is required for civil works projects such as revetments, jetties, and beach nourishments. Hydrographic survey data is used to understand various processes that shape the coastlines and human interaction with these processes.

Coastal Zone Management: Hydrographic surveys provide data for coastal hazards and vulnerability assessment of coastal landscapes in relation to climate change, subsidence, glacial rebound, and others. Bathymetric data provide ancillary information on indicators that capture the biophysical conditions and morphodynamic classification.

Offshore Resource Mapping: Offshore energy resources include wind, wave, and geologic mineral (oil, natural gas etc) deposits. Surveys and Geographic Information Systems are invaluable tools to identify the exploitation of these energy resources.

1.2 Disciplines Associated with Hydrographic Surveying

Hydrography relies on a variety of scientific and engineering disciplines. Figure 1 illustrates the core disciplines like Geodesy, Photogrammetry, Cartography, Global Positioning System, Oceanography, Tides, Physics and Mathematics. These are the various disciplines that influence the science and products delivered by hydrographic survey.

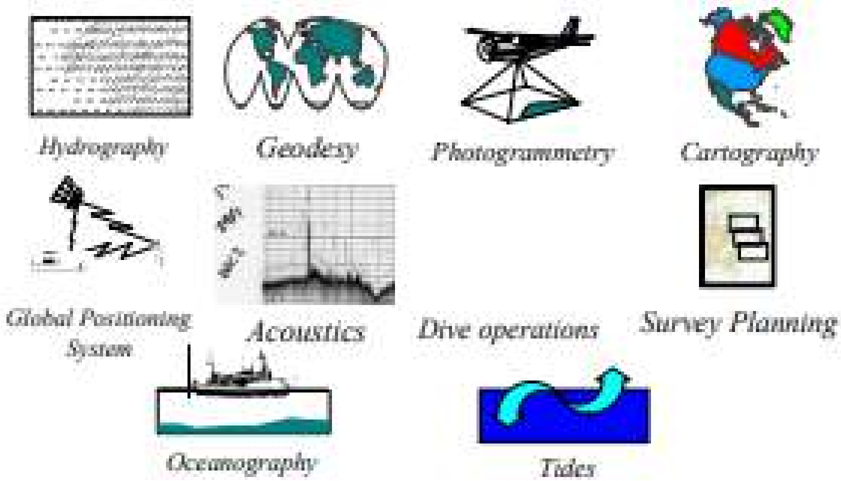

Figure 1. Disciplines that Influence the Science of Hydrography

Geodesy is an interdisciplinary science which uses space-borne and airborne remotely sensed, and ground-based measurements to study the shape and size of the Earth, the planets and their satellites, and their changes; to precisely determine position and velocity of points or objects at the surface or orbiting the planet, within a realized terrestrial reference system, and to apply these knowledge to a variety of scientific and engineering applications, using mathematics, physics, astronomy, and computer science.

Oceanograpby is the scientific discipline concerned with all aspects of the world's oceans and seas, including their physical and chemical properties, their origin and geologic framework, and the life forms that inhabit the marine environment. Traditionally, oceanography has been divided into four separate but related branches: physical oceanography, chemical oceanography, marine geology, and marine ecology. Physical oceanography deals with the properties of seawater (temperature, density, pressure, and so on), its movement (i.e., waves, currents, tides), and the interactions between the ocean waters and land surface waters (rivers and streams).

1.3.1 Marine Vessel

The size and payload of the marine vessel depends on the extent of the survey project requirements. Surveys can be classified by vessel size -small scale (from wading to small boats), medium scale (using medium size boats and acoustic methods), and regional scale surveys using deep sea research vessels with state-of-the art multi-disciplinary data collection systems. Essential equipment list for each survey is as follows;

A) Small Surveys:

1. Vessel: Oars, Life jackets, Gas tanks (minimum 2), extra oil, and 10 HP engine
2. Depth and Position: 50' leadline, range poles, and plans. Survey equipment may include Total Station Instrument (TSI), compensating level as required, prism pole with extension rods. Deeper water requires a fathometer and transducer installation.
3. Miscellaneous: Radio, 300 ft tape, Navigation chart, staff sheets, Batteries (2), repair kit, tool box

B) Medium Scale Surveys:

1. Vessel: 25-65 ft vessel, licensed operator.
2. Depth and Position: Echosounder with Transducer and aedquate power from batteries or generator, tool box, transducers , GPS or TSI positioning, motion reference units (MRU)
3. Miscellaneous: A small vessel for the near-shore shallow water survey system to perform as rover platform.

C) Regional Scale Surveys:

1. Vehicle: 65 ft and larger research vessels , with competent crew and equipment.
2. Depth and Position: Multi-beam transducer and GPS.
3. Other Equipment: Cameras for stereo imaging (require positioning of frames) Integrated multi-disciplinary data collection systems (e.g., gravity, magnetics), requires accurate in-ship surveys for sensor integration, calibration, and synchronization.
4. - ,

1.3.2 Positioning equipment

Offshore positioning equipment has been revolutionized due to dramatic evolution in sensor technology and computer science. Traditional offshore equipment includes a sextant, transit, stadia, and an electronic distance measuring (EDM) device. Nowadays, several methods for horizontal positioning include optical, land-based electronic ranging, and space-based positioning. A basic method of positioning is the resection. However, the positioning methodology employed on any project will be evaluated based on site-specific conditions and project specification.

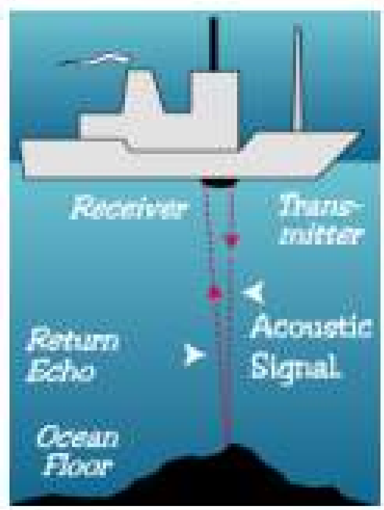

Figure 1.2: Acoustic depth measurement

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AREAS AND VOLUMES

In civil engineering works such as designing of long bridges, dams, reservoirs, etc., the area of catchments of rivers is required. The areas of fields are also required for planning and management of projects. The area is required for the title documents of land.
In many civil engineering projects, earthwork involves excavation and removal and dumping of earth, therefore it is required to make good estimates of volumes of earthwork. Volume computations are also needed to determine the capacity of bins, tanks, and reservoirs, and to check the stockpiles of coal, gravel, and other material.
Computing areas and volumes is an important part of the office work involved in surveying.

1. AREAS

The method of computation of area depends upon the shape of the boundary of the tract and accuracy required. The area of the tract of the land is computed from its plan which may be enclosed by straight, irregular or combination of straight and irregular boundaries.

1.1 Computation of Areas of Regular Figures

When the boundaries are straight the area is determined by subdividing the plan into simple geometrical figures such as triangles, rectangles, trapezoids, etc. Standard expressions as given below are available for the areas of straight figures.

(a) Triangle:

$$\text{Area of triangle} = \frac{1}{2}\, ab \sin C$$

in which C is the included angle between the sides a and b.
The area of a triangle whose lengths of sides are known can be computed by the
Formula:

$$\text{Area} = \sqrt{s(s-a)(s-b)(s-c)}$$

where a, b, and c are the lengths of sides of the triangle and $s = \frac{1}{2}(a + b + c)$.

(b) Rectangle:
If b and 'd' are the dimension of a rectangle,

$$A = bd$$

(c) Trapezium:

$$A = d\,\frac{h1 + h2}{2}$$

where d is the distance between two parallel sides and h1 and h2 lengths of parallel sides.

Units used for finding areas are square metres, hectare and square kilometre. Relation among them are:

Hectare = 100 m × 100 m = 1 × 10^4 m^2

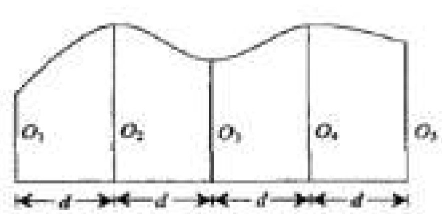
Square kilometer = 1000 m × 1000 m = 1 × 10^6 m^2
= 100 hectare

1.2 Areas of Irregular Shapes

For this purpose from a survey line offsets are taken at regular intervals and area is calculated from any one of the following methods:
(a) Area by Trapezoidal rule
(b) Area by Simpson's rule.

(a) Area by Trapezoidal Rule:

In trapezoidal rule, the area is divided into a number of trapezoids, boundaries being assumed to be straight between pairs of offsets.



The area of each trapezoid is determined and added together to derive the whole area. If there are n offsets at equal interval of d then the total area is

$$A = d\left(\frac{O_1 + O_n}{2} + O_2 + O_3 + \dots\dots\dots + O_{n-1}\right)$$

Example 1:Compute the area of the tract shown in Figure

Solution

By Equation above

$$area = 50\left(\frac{0+2.8}{2} + 5.2 + 8.7 + 9.2 + 4.9 + 10.4 + 5.2 + 12.2\right)$$

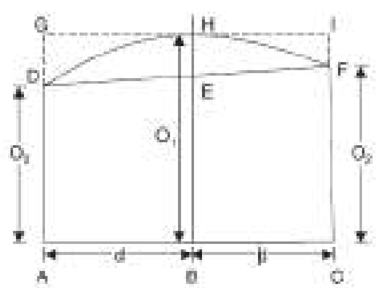
=2860 m^2

(b) Area by Simpson's Rule

In Simpson's rule it is assumed that the irregular boundary is made up of parabolic arcs. The areas of the successive pairs of intercepts are added together to get the total area.

$$A = \frac{d}{3}\left[(O_1 + O_n) + 4(O_2 + O_4 + ... + O_{n-2}) + 2(O_3 + O_5 + ... + O_{n-1})\right]$$

To derive the equation of Simpson's Rule



In this method, the boundary line between two segment is assumed parabolic.

The first two segments of figure above in which boundary between the ordinates is assumed parabolic.

∴ Area of the first two segments

$$= \text{Area of trapezium ACFD} + \text{Area of parabola DEFH}$$

$$= \frac{O_0 + O_2}{2} 2d + \frac{2}{3} \times 2d \times EH$$

$$= (O_0 + O_2)\, d + \frac{4}{3} d \left(O_1 - \frac{O_0 + O_2}{2} \right)$$

$$= \frac{d}{3}\left[3O_0 + 3O_2 + 4O_1 - 2O_0 - 2O_2\right]$$

$$= \frac{d}{3}\left[O_0 + 4O_1 + O_2\right]$$

Area of next two segments

$$= \frac{d}{3}\left[O_2 + 4O_3 + O_4\right]$$

Area of last two segments

$$= \frac{d}{3}\left[O_{n-2} + 4O_{n-1} + O_n\right]$$

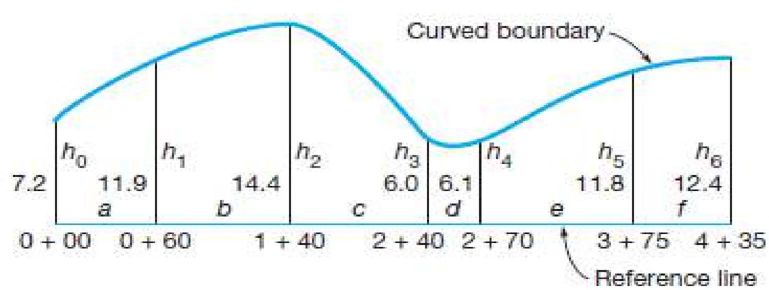
Area of Irregularly Spaced Offsets

For irregularly curved boundaries like that in Figure 12.3, the spacing of offsets along the reference line varies. Spacing should be selected so that the curved boundary is accurately defined when adjacent offset points on it are connected by straight lines. A formula for calculating area for this case is:

$$\text{area} = \frac{1}{2}[a(h_0 + h_1) + b(h_1 + h_2) + c(h_2 + h_3) + \cdots]$$

Where a, b, c are the varying offset spaces, and h_0, h_1, h_2 are the observed offsets.

Example: compute the area of the tract shown in figure



Solution

$$\text{area} = \frac{1}{2}[60(7.2 + 11.9) + 80(11.9 + 14.4) + 100(14.4 + 6.0) + 30(6.0 + 6.1) + 105(6.1 + 11.8) + 60(11.8 + 12.4)]$$

$$= 4490 \text{ m}^2$$

Now to calculate the area of the irregular figure, use of trapezoidal rule or Simpson's rule can be made. The Simpson's rule require even number of increments, whereas the trapezoidal rule can be used for odd as well as even number of increments. In the present case since the number of increments is even, the area can be determined with either trapezoidal rule or Simpson's rule.

Area by trapezoidal rule

$$A = d\left(\frac{O_1 + O_7}{2} + O_2 + O_3 + O_4 + O_5 + O_6\right)$$

In this case O_1 and O_7 are the end offsets, and therefore $O_1 = O_7 = 0$ m.

Thus
$$A_3 = 30 \times \left(\frac{0+0}{2} + 3.6 + 2.8 + 4.2 + 4.9 + 3.7\right)$$
$$= 30 \times 19.2 = 576.00 \text{ m}^2.$$

Hence the total area of the tract

$$= A_1 + A_2 + A_3$$
$$= 11604.42 + 11608.76 + 576.00$$
$$= 23789.18 \text{ m}^2 = \textbf{2.4 hectares}.$$

Problem 1 The following offsets were taken from a chain line to an irregular boundary line at an interval of 10 m:

0, 2.50, 3.50, 5.00, 4.60, 3.20, 0 m

compute the area between the chain line, the irregular boundary line and the end offsets by:

a- Mid-ordinate rule
b- The average coordinate rule
c- The trapezoidal rule
d- Simpson's rule

Solution

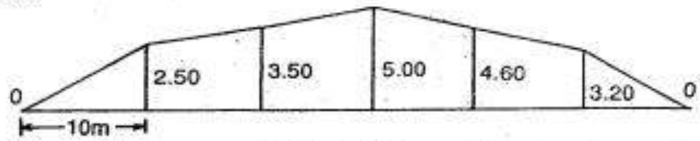


Fig. P.7.1

(a) *By mid-ordinate rule*: The mid-ordinates are

$$h_1 = \frac{0 + 2.50}{2} = 1.25 \text{ m}$$

$$h_2 = \frac{2.50 + 3.50}{2} = 3.00 \text{ m}$$

$$h_3 = \frac{3.50 + 5.00}{2} = 4.25 \text{ m}$$

$$h_4 = \frac{5.00 + 4.60}{2} = 4.80 \text{ m}$$

$$h_5 = \frac{4.60 + 3.20}{2} = 3.90 \text{ m}$$

$$h_6 = \frac{3.20 + 0}{2} = 1.60 \text{ m}$$

Required area = 10 (1.25 + 3.00 + 4.25 + 4.80 + 3.90 + 1.60)
= 10 × 18.80 = 188 m^2

(b) *By average-ordinate rule*:

Here $d = 10$ m and $n = 6$ (no. of divs)

Base length = 10 × 6 = 60 m
Number of ordinates = 7

$$\text{Required area} = 60 \times \left\{\frac{0 + 2.50 + 3.50 + 5.00 + 4.60 + 3.20 + 0}{7}\right\}$$

$$= 60 \times \frac{18.80}{7} = 161.14 \text{ m}^2$$

(c) *By trapezoidal rule*:
Here $d = 10$

$$\text{Required area} = \frac{10}{2}\{0 + 0 + 2(2.50 + 3.50 + 5.00 + 4.60 + 3.20)\}$$

$$= 5 \times 37.60 = 188 \text{ m}^2$$

(d) *By Simpson's rule*:

$d = 10$

$$\text{Required area} = \frac{10}{3}\{0 + 0 + 4(2.50 + 5.00 + 3.20) + 2(3.50 + 4.60)\}$$

$$= \frac{10}{3}\{42.80 + 16.20\} = \frac{10}{3} \times 59.00$$

$$= \frac{10}{3} \times 59.00 = 196.66 \text{ m}^2$$

H.W

Problem 2 The following offsets were taken at 15 m intervals from a survey line to an irregular boundary line:

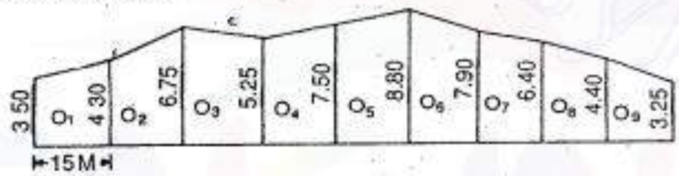
3.50, 4.30, 6.75, 5.25, 7.50, 8.80, 7.90, 6.40, 4.40, 3.25 m

Calculate the area enclosed between the survey line, the irregular boundary line, and the first and last offsets, by:

(a) The trapezoidal rule

(b) Simpson's rule

Solution (Fig. P-7.2)



Example: A tract of land has three straight boundaries AB, BC, and CD. The fourth Boundary DA is irregular. The measured length are as under

AB = 135 m, BC = 191 m, CD = 126 m, BD = 255 m.

The offsets measured outside the boundary DA to the irregular boundary at a regular interval of 30 m from D, are as below:

Distance from D (m)	0.0	30	60	90	120	150	180
Offsets (m)	0.0	3.7	4.9	4.2	2.8	3.6	0.0

Determine the area of the tract.

Solution (Fig. 8.8):

Let us first calculate the areas of triangles ABD and BCD.

The area of a triangle is given by

$$A = \sqrt{S(S-a)(S-b)(S-c)}$$

in which a, b, and c are the lengths of the sides, and $S = \dfrac{a+b+c}{2}$

For ΔABD

$$S = \frac{135+255+180}{2} = 285\,\text{m}$$

$$A_1 = \sqrt{285 \times (285-135) \times (285-255) \times (285-180)}$$

$= 11604.42\ \text{m}^2$.

For ΔBCD

$$S = \frac{191+126+255}{2} = 286\,\text{m}$$

$$A_2 = \sqrt{286 \times (286-191) \times (286-126) \times (286-255)}$$

$= 11608.76\ \text{m}^2$.

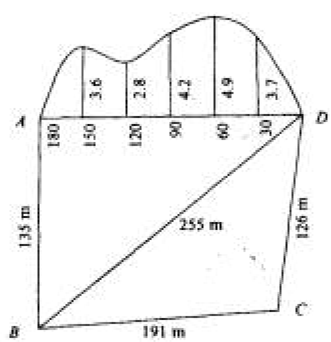


Fig. 8.8

Problem 3 The following offsets are taken from a survey line to a curved boundary line:

Distance (m)	0	5	10	15	20	30	40	60	80
Offset (m)	2.50	3.80	4.60	5.20	6.10	4.70	5.80	3.90	2.20

Find the area between the survey line, the curved boundary line, and the first and the last offsets by:

(i) The trapezoidal rule, and (ii) Simpson's rule.

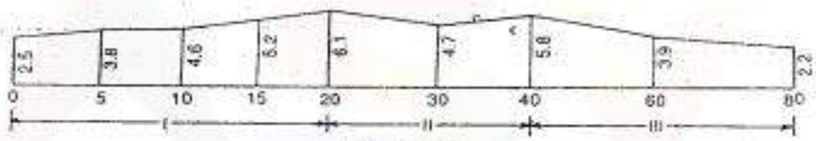


Fig. P.7.3

Solution Here, the intervals between the offsets are not regular throughout the length.

So, the section is divided into three compartments.

Let Δ_I = area of 1st section

Δ_{II} = area of 2nd section

Δ_{III} = area of 3rd section

Here, d_1 = 5 m

d_2 = 10 m

d_3 = 20 m

(a) *By Trapezoidal rule*:

$$\Delta_I = \frac{5}{2}\{2.50 + 6.10 + 2(3.80 + 4.60 + 5.20)\} = 89.50 \text{ m}^2$$

$$\Delta_{II} = \frac{10}{2}\{6.10 + 5.80 + 2(4.70)\} = 106.50 \text{ m}^2$$

$$\Delta_{III} = \frac{20}{2}\{5.80 + 2.20 + 2(3.90)\} = 158.00 \text{ m}^2$$

Total area = 89.50 + 106.50 + 158.00 = 354.00 m^2

(b) *By Simpson's rule*:

$$\Delta_I = \frac{5}{3}\{2.50 + 6.10 + 4(3.80 + 5.20) + 2(4.60)\} = 89.66 \text{ m}^2$$

$$\Delta_{II} = \frac{10}{3}\{6.10 + 5.80 + 4(4.70)\} = 102.33 \text{ m}^2$$

$$\Delta_{III} = \frac{20}{3}\{5.80 + 2.20 + 4(3.90)\} = 157.33 \text{ sq m}$$

Total area = 89.66 + 102.33 + 157.33 = 349.32 m^2.

Surveying 2

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المحاضرة التاسعة

Global position system (GPS)

FUNDAMENTALS OF SATELLITE POSITIONING

The precise travel time of the signal is necessary to determine the distance, or so-called range, to the satellite. Since the satellite is in an orbit approximately 20,200 km above the Earth, the travel time of the signal will be roughly 0.07 sec after the receiver generates the same signal. If this time delay between the two signals is multiplied by the signal velocity (speed of light in a vacuum) c, the range to the satellite can be determined from

$$r = c \times t \qquad (13.11)$$

where r is the range to the satellite and t the elapsed time for the wave to travel from the satellite to the receiver.

Satellite receivers in determining distances to satellites employ two fundamental methods: code ranging and carrier phase-shift measurements. Those that employ the former method are often called mapping grade receivers; those using the latter procedure are called survey-grade receivers. From distance observations made to multiple satellites, receiver positions can be calculated. Descriptions of the two methods and their mathematical models are presented in the subsections that follow. These mathematical models are presented to help students better understand the underlying principles of GPS operation. Computers that employ software provided by manufacturers of the equipment perform solutions of the equations.

Code Ranging

The code ranging (also called code matching) method of determining the time it takes the signals to travel from satellites to receivers was the procedure briefly described in Section 13.3. With the travel times known, the corresponding distances to the satellites can then be calculated by applying Equation (13.11). With one range known, the receiver would lie on a sphere. If the range were determined from two satellites, the results would be two intersecting spheres. As shown in Figure 13.8(a), the intersection of two spheres is a circle. Thus, two ranges from two satellites would place the receiver somewhere on this circle. Now if the range for a third satellite is added, this range would add an additional sphere, which when intersected with one of the other two spheres would produce another circle of intersection. As shown in Figure 13.8(b), the intersection of two circles would leave only two possible locations for the position of the receiver. A "seed position" is used to quickly eliminate one of these two intersections.

For observations taken on three satellites, the system of equations that could be used to determine the position of a receiver at station A is

$$
\begin{aligned}
\rho_A^1 &= \sqrt{(X^1 - X_A)^2 + (Y^1 - Y_A)^2 + (Z^1 - Z_A)^2} \\
\rho_A^2 &= \sqrt{(X^2 - X_A)^2 + (Y^2 - Y_A)^2 + (Z^2 - Z_A)^2} \qquad (13.12) \\
\rho_A^3 &= \sqrt{(X^3 - X_A)^2 + (Y^3 - Y_A)^2 + (Z^3 - Z_A)^2}
\end{aligned}
$$

where ρ_A^n are the *geometric ranges* for the three satellites to the receiver at station A, (X^n, Y^n, Z^n) are the geocentric coordinates of the satellites at the time of the signal transmission, and (X_A, Y_A, Z_A) are the geocentric coordinates of the receiver at transmission time. Note that the variable n pertains to superscripts and takes on values of 1, 2, or 3.

However, in order to obtain a valid time observation, the systematic error (known as *bias*) in the clocks, and the refraction of the wave as it passes through the Earth's atmosphere, must also be considered. In this example, the receiver clock bias is the same for all three ranges since the same receiver is observing

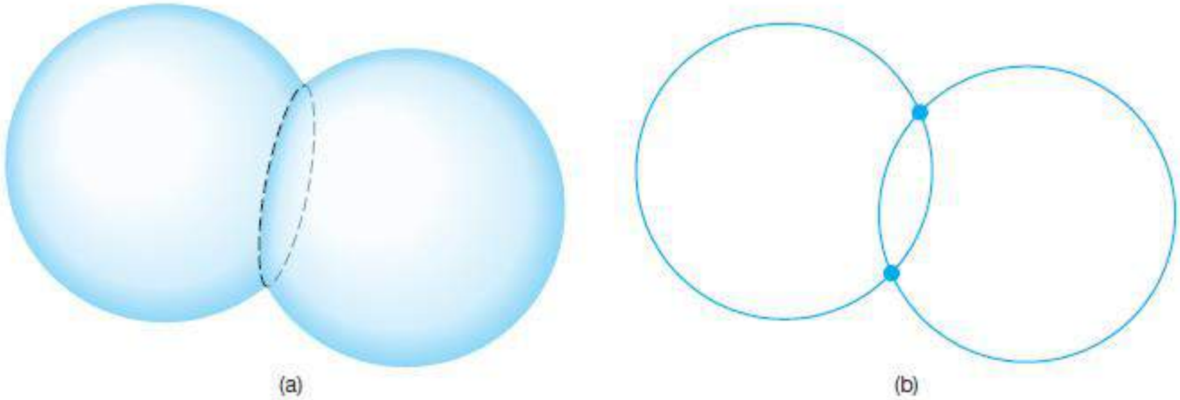

Figure 13.8 (a) The intersection of two spheres and (b) the intersection of two circles.

each range. With the introduction of a fourth satellite range, the receiver clock bias can be mathematically determined. This solution procedure allows the receiver to have a less accurate (and less expensive) clock.Algebraically, the system of equations used to solve for the position of the receiver and clock bias are:

$$\begin{aligned} R_A^1(t) &= \rho_A^1(t) + c(\delta^1(t) - \delta_A(t)) \\ R_A^2(t) &= \rho_A^2(t) + c(\delta^2(t) - \delta_A(t)) \\ R_A^3(t) &= \rho_A^3(t) + c(\delta^3(t) - \delta_A(t)) \\ R_A^4(t) &= \rho_A^4(t) + c(\delta^4(t) - \delta_A(t)) \end{aligned} \qquad (13.13)$$

where $R_A^n(t)$ is the observed *range* (also called *pseudorange*) from receiver A to satellites 1 through 4 at epoch (time) t, $\rho_A^n(t)$ the geometric range as defined in Equation (13.12), c the speed of light in a vacuum, $\delta_A(t)$ the receiver clock bias, and $\delta^n(t)$ the satellite clock bias, which can be modeled using the coefficients supplied in the broadcast message. These four equations can be simultaneously solved yielding the position of the receiver (X_A, Y_A, Z_A), and the receiver clock bias $\delta_A(t)$. Equations (13.13) are known as the *point positioning equations* and as noted earlier they apply to code-based receivers.

THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION

The Universal Transverse Mercator Projection (UTM) is a worldwide system of transverse Mercator projections. It comprises 60 zones, each 6◦ wide in longitude, with central meridians at 3◦, 9◦, etc. The zones are numbered from 1 to 60, starting with 180◦ to 174◦Was zone 1 and proceeding eastwards to zone 60. Therefore the central meridian (CM) of zone n is given by CM = 6n◦ – 183◦. In latitude, the UTM system extends from 84◦ N to 80◦ S, with the polar caps covered by a polar stereographic projection.

The scale factor at each central meridian is 0.9996 to counteract the enlargement ratio at the edges of the strips. The false origin of northings is zero at the equator for the northern hemisphere and 106 m at the equator for the southern hemisphere. The false origin for eastings is 5 × 105 m west of the zone central meridian.

2- **Carrier Phase-Shift Measurements**

Better accuracy in measuring ranges to satellites can be obtained by observing phase-shifts of the satellite signals. In this approach, the phase-shift in the signal that occurs from the instant it is transmitted by the satellite until it is received at the ground station, is observed. This procedure, which is similar to that used by EDM instruments, yields the fractional cycle of the signal from satellite to receiver. However, it does not account for the number of full wavelengths or cycles that occurred as the signal traveled between the satellite and receiver. This number is called the integer ambiguity or simply ambiguity. Unlike EDM instruments, the satellites utilize one-way communication, but because the satellites are moving and thus their ranges are constantly changing, the ambiguity cannot be determined by simply transmitting additional frequencies. There are different techniques used to determine the ambiguity. All of these techniques require that additional observations be obtained. Once the ambiguity is determined, the mathematical model for carrier phase-shift, corrected for clock biases, is

$$\Phi_i^j(t) = \frac{1}{\lambda}\rho_i^j(t) + N_i^j + f^j[\delta^j(t) - \delta_i(t)] \qquad (13.14)$$

where for any particular epoch in time, t, $\Phi_i^j(t)$ is the carrier phase-shift measurement between satellite j and receiver i, f^j the frequency of the broadcast signal generated by satellite j, $\delta^j(t)$ the clock bias for satellite j, λ the wavelength of the signal, $\rho_i^j(t)$ the range as defined in Equations (13.12) between receiver i and satellite j, N_i^j the integer ambiguity of the signal from satellite j to receiver i, and $\delta_i(t)$ the receiver clock bias.

ERRORS IN OBSERVATIONS

Electromagnetic waves can be affected by several sources of error during their transmission. Some of the larger errors include (1) satellite and receiver clock biases and (2) ionospheric and tropospheric refraction. Other errors in satellite surveying work stem from (a) satellite ephemeris errors, (b) multipathing, (c) instrument miscentering, (d) antenna height measurements, (e) satellite geometry, and (f) before May 1, 2000, selective availability. All of these errors contribute to the total error of satellite-derived coordinates in the ground stations.

THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION

The Universal Transverse Mercator Projection (UTM) is a worldwide system of transverse Mercator projections. It comprises 60 zones, each 6◦ wide in longitude, with central meridians at 3◦, 9◦, etc. The zones are numbered from 1 to 60, starting with 180◦ to 174◦Was zone 1 and proceeding eastwards to zone 60. Therefore the central meridian (CM) of zone n is given by CM = 6n◦ − 183◦. In latitude, the UTM system extends from 84◦ N to 80◦ S, with the polar caps covered by a polar stereographic projection.

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Surveying 2

By Dr. Khamis Naba Sayl

المحاضرة الثالثة

METHODS OF VOLUME MEASUREMENT

Direct measurement of volumes is rarely made in surveying, since it is difficult to actually apply a unit of measure to the material involved. Instead, indirect measurements are obtained by measuring lines and areas that have a relationship to the volume desired.

Three principal systems are used: (1) the cross-section method, (2) the unit-area (or borrow-pit) method, and (3) the contour-area method.

2.1 THE CROSS-SECTION METHOD

The cross-section method is employed almost exclusively for computing volumes on linear construction projects such as highways, railroads, and canals. In this procedure, after the centerline has been staked, ground profiles called cross sections are taken (at right angles to the centerline), usually at intervals of full or half stations if the English system of units is being used, or at perhaps 10, 20, 30, or 40 m if the metric system is being employed. Cross-sectioning consists of observing ground elevations and their corresponding distances left and right perpendicular to the centerline. Readings must be taken at the centerline, at high and low points, and at locations where slope changes occur to determine the ground profile accurately. This can be done in the field using a level, level rod, and tape.

Much of the fieldwork formerly involved in running preliminary centerline, getting cross-section data, and making slope-stake and other measurements on long route surveys is now being done more efficiently by photogrammetry.

After cross sections have been taken and plotted, design templates (outlines of base widths and side slopes of the planned excavation or embankment) are superimposed on each plot to define the excavation or embankment to be constructed at each cross-section location. Areas of these sections, called end areas, are obtained by computation or by planimeter. Nowadays, using computers, end areas are calculated directly from field cross-section data and design information. From the end areas, volumes are determined by the average-end-area, or prismoidal formula.

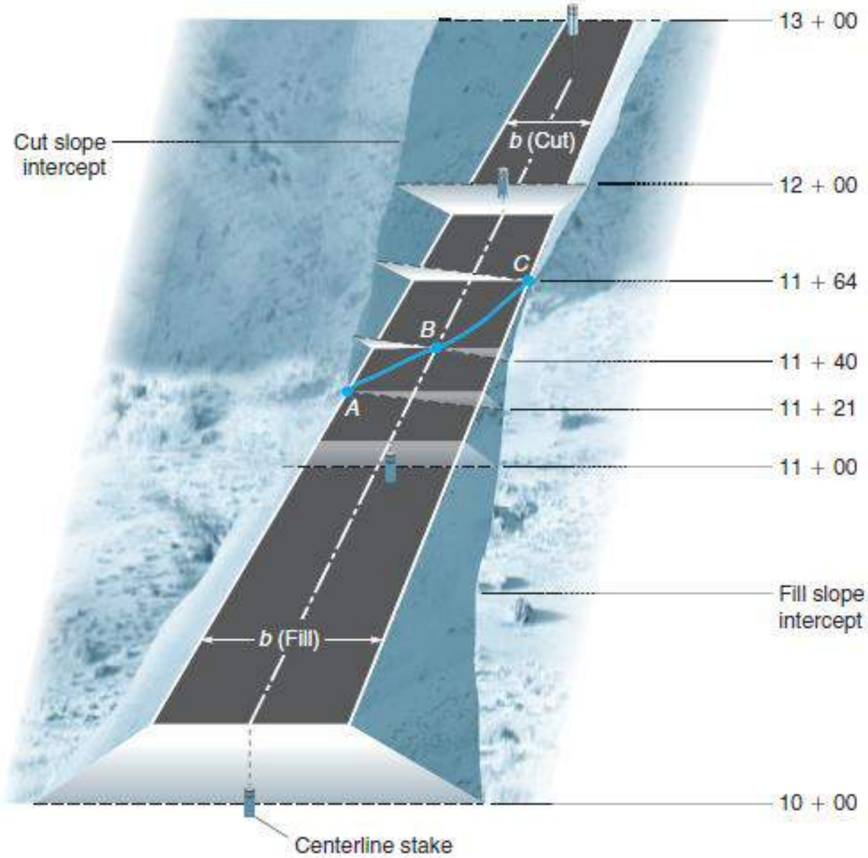

Figure 26.1 Section of roadway illustrating excavation (cut) and embankment (fill).

Figure 26.1 portrays a section of planned highway construction and illustrates some of the points just discussed. Centerline stakes are shown in place, with their stationing given in the English system of units. They mark locations where cross sections are taken, in this instance at full stations. End areas, based on the planned grade line, size of roadway, and selected embankment and excavation slopes, are superimposed at each station and are shown shaded. Areas of these shaded sections are determined, whereupon volumes are computed using formulas given in the next Section. Note in the figure that embankment, or fill, is planned from stations 10+00 through 11+21 a transition from fill to excavation, or cut, occurs from station11+21 to 11+64 and cut is required from stations11+64 to 13+00.

Type of cross sections

The types of cross sections commonly used on route surveys are shown in Figure 26.2. In flat terrain the level section (a) is suitable. The three-level section (b) is generally used where ordinary ground conditions prevail. Rough topography may require a five-level section (c), or more practically an irregular section (d). A transition section (e) and a side-hill section (f) occur when passing from cut to fill and on side-hill locations. In Figure 26.1, transition sections occur at stations and while a side-hill section exists at 11+21 and 11+64, while a side – hill section exists at 11+40.

The width of base b, the finished roadway, is fixed by project requirements. As shown in Figure 26.1, it is usually wider in cuts than on fills to provide for drainage ditches. The side slope s [the horizontal dimension required for a unit vertical rise and illustrated in Figure 26.2(a)] depends on the type of soil encountered. Side slopes in fills usually are flatter than those in cuts where the soil remains in its natural state.

Cut slopes of 1:1 (1 horizontal to 1 vertical) and fill slopes of 1-1/2:1 might be satisfactory for ordinary loam soils, but 1-1/2:1 in excavation and 2:1 in embankment are common. Even flatter proportions may be required—one cut in other factors. Formulas for areas of sections are readily derived and listed with some of the sketches in Figure 26.2.

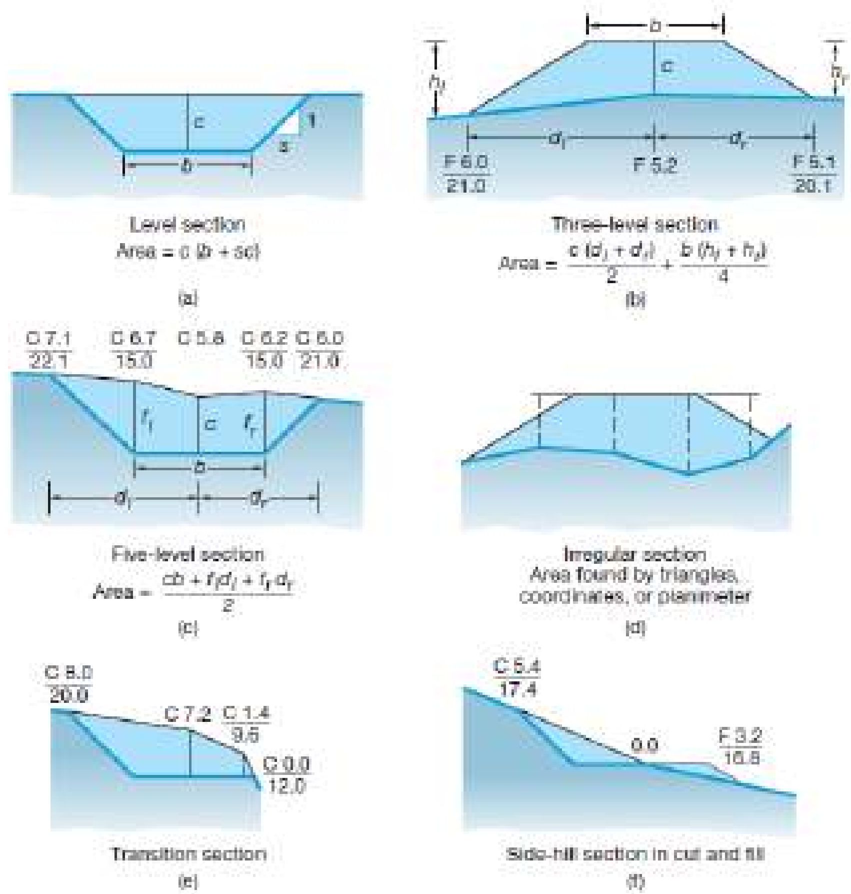

Figure 26.2
Earthwork sections.

Example 8.6. Calculate the area of cross-section that has breadth of formation as 10 m, center height as 3.2 m and side slopes as 1 vertical to 2 horizontal.

Solution (Fig. 8.10):

A cross-section having no cross-fall, i.e., the ground transverse to the center line of the road is level, is called as a *level- section.* The area of a level-section is given by

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

where

h = the depth at the center line in case of cutting , and the height in case of embankment,

b = the formation width, and

1 in s = the side slope.

The widths w are given by

$$w = \frac{b}{2} + sh$$

It is given that

$$b = 10 \text{ m}$$
$$h = 3.2 \text{ m}$$
$$s = 2.$$

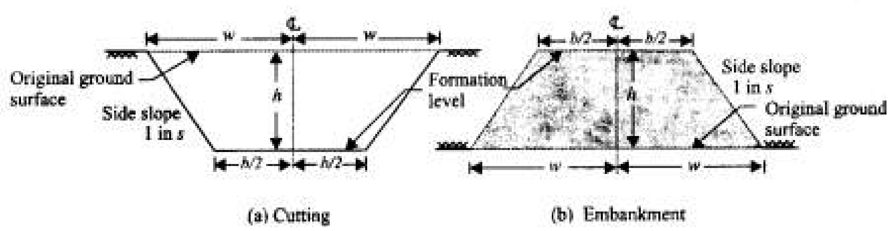

Fig. 8.10

Hence the area

$$A = 3.2 \times (10 + 2 \times 3.2) = \mathbf{52.48\ m^2}.$$

B. Two-Level Section

When the ground surface has a transverse slope:

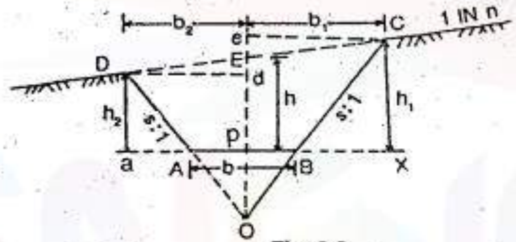

Fig. 8.3

$$PB = \frac{b}{2}$$

$$Bx = sh_1$$

$$b_1 = \frac{b}{2} + sh_1 \tag{a}$$

$$Ee = (h_1 - h)$$

and $b_1 = n \times Ee = n(h_1 - h)$ (b)

From (a) and (b), $\frac{b}{2} + sh_1 = n(h_1 - h)$

or $$h_1(n - s) = n\left(h + \frac{b}{2n}\right)$$

or $$h_1 = \frac{n}{(n-s)} \times \left(h + \frac{b}{2n}\right) \tag{2}$$

From (2), and (a)

$$b_1 = \frac{b}{2} + \frac{ns}{n-s}\left(h + \frac{b}{2n}\right) \tag{3}$$

Similarly,

$$h_2 = \frac{n}{n+s}\left(h - \frac{b}{2n}\right) \tag{4}$$

$$b_2 = \frac{b}{2} + \frac{ns}{n+s} \times \left(h - \frac{b}{2n}\right) \tag{5}$$

$$\text{Area ABCED} = \Delta\text{DOE} + \Delta\text{COE} - \Delta\text{AOB}$$

$$= \frac{1}{2}\,\text{OE} \times \text{Dd} + \frac{1}{2}\,\text{OE} \times \text{Ce} - \frac{1}{2}\,\text{AB} \times \text{OP}$$

Here, $\text{OE} = \text{OP} + \text{PE} = \frac{b}{2s} + h$

$\text{Dd} = b_2$ $\quad \text{Ce} = b_1$

$\text{AB} = b$ $\quad \text{OP} = \frac{b}{2s}$

$$\text{Area} = \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)b_2 + \frac{1}{2}\left(\frac{b}{2s} + h\right)b_1 - \frac{1}{2}\,b \times \frac{b}{2s}\right\}$$

$$= \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)(b_1 + b_2) - \frac{b^2}{2s}\right\} \tag{6}$$

Example The width at the formation level of a certain cutting is 10 m and side slope 1 : 1. The surface of the ground has a uniform slope of 1 in 6 in the transverse direction. Find the cross-sectional area when the depth of cutting at the centre is 3 m.

Solution Here, $b = 10$ m $\quad s = 1$
$n = 6 \quad h = 3$ m

From Eq. (3),

$$b_1 = \frac{b}{2} + \frac{ns}{n-s}\left(h + \frac{h}{2n}\right)$$

$$= \frac{10}{2} + \frac{6 \times 1}{6-1} \times \left(3 + \frac{10}{2 \times 6}\right) = 9.6 \text{ m}$$

From Eq. (5),

$$b_2 = \frac{b}{2} + \frac{ns}{n+s} \times \left(h - \frac{b}{2n}\right)$$

$$= \frac{10}{2} + \frac{6 \times 1}{6+1} \times \left(3 - \frac{10}{2 \times 6}\right) = 6.85 \text{ m}$$

From Eq. (6),

$$\text{Area} = \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)(b_1 + b_2) - \frac{b^2}{2s}\right\}$$

$$= \frac{1}{2}\left\{\left(\frac{10}{2 \times 1} + 3\right)(9.6 + 6.85) - \frac{10^2}{2 \times 1}\right\}$$

$$= \frac{1}{2}\{8 \times 16.45 - 50\} = 40.8 \text{ m}^2$$

C. Three-level Section

When the transverse slope is not uniform:

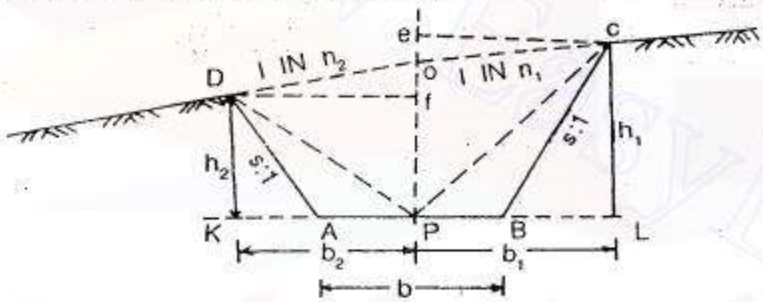

Fig. 8.4

Area ABCOD = ΔDOP + ΔCOP + ΔDAP + ΔBCP

$$= \frac{1}{2} \times h \times b_2 + \frac{1}{2} \times h \times b_1 + \frac{1}{2} \times \frac{b}{2} \times h_2 + \frac{1}{2} \times \frac{b}{2} \times h_1$$

i.e. $\text{Area} = \left\{\frac{h}{2}(b_1 + b_2) + \frac{b}{4}(h_1 + h_2)\right\}$ (9)

Here $h_1 = \text{OP} + \text{Oe} = h + \frac{b_1}{n_1}$ (10)

$h_2 = \text{OP} - \text{ef} = h - \frac{b_2}{n_2}$ (11)

Deduction of formula for b_2 and b_1

$b_2 = \text{AP} + \text{AK} = \frac{b}{2} + sh_2$ or $h_2 = \frac{\{b_2 - (b/2)\}}{s}$ (a)

Also

$b_2 = \text{ef} \times n_2 = (h - h_2)\, n_2$ or $h_2 = \frac{hn_2 - b_2}{n_2}$ (b)

From **(a)** and (b),

$$\frac{\{b_2 - (b/2)\}}{s} = \frac{hn_2 - b_2}{n_2}$$

or

$$b_2 n_2 - \frac{bn_2}{2} = hn_2 s - b_2 s$$

$$b_2 (n_2 + s) = n_2\left(sh + \frac{b}{2}\right) = n_2 s\left(h + \frac{b}{2s}\right)$$

$$b_2 = \frac{n_2 s}{n_2 + s} \times \left(h + \frac{b}{2s}\right) \quad (10)$$

Similarly,

$$b_1 = \frac{n_1 s}{n_1 - s}\left(h + \frac{b}{2s}\right) \quad (11)$$

Example The following notes refer to a three-level section:

Station	Cross-section		
1	$\frac{+0.95}{4.55}$	$\frac{+1.50}{0}$	$\frac{+2.90}{6.50}$
2	$\frac{+1.75}{5.50}$	$\frac{+2.00}{0}$	$\frac{+3.20}{8.30}$

Find the sectional area at stations 1 and 2, asuming a formation width of 8 m.

Solution From Eq. (7), we know that

$$\text{Area} = \left\{\frac{h}{2}(b_1 + b_2) + \frac{b}{4}(h_1 + h_2)\right\}$$

Data for cross-section at station 1:

$h = 1.50$ m $b = 8$ m
$h_1 = 2.90$ m $b_1 = 6.50$ m
$h_2 = 0.95$ m $b_2 = 4.55$ m

Cross-sectional area at station 1:

$$\Delta_1 = \left\{\frac{1.50}{2}(6.50 + 4.55) + \frac{8}{4}(2.90 + 0.95)\right\}$$

$$= (0.75 \times 11.05 + 2 \times 3.85) = 15.99 \text{ m}^2$$

Data for cross-section at station 2:

$h = 2.00$ m
$h_1 = 3.20$ m
$h_2 = 1.75$ m

$b = 8$ m
$b_1 = 8.30$ m
$b_2 = 5.50$ m

Cross-sectional area at station 2:

$$\Delta_2 = \left\{\frac{2.00}{2}(8.30 + 5.50) + \frac{8}{4}(3.20 + 1.75)\right\}$$

$$= (1.00 \times 13.80 + 2 \times 4.95) = 23.70 \text{ m}^2$$

Surveying 2

By Dr. Khamis Naba Sayl

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Global position system (GPS)

Global position system (GPS)

1-INTRODUCTION

During the 1970s, a new and unique approach to surveying, the global positioning system (GPS), emerged. This system, which grew out of the space program, relies upon signals transmitted from satellites for its operation. It has resulted from research and development paid for by the military to produce a system for global navigation and guidance. More recently other countries are developing their own systems. Thus, the entire scope of satellite systems used in positioning is now referred to as global navigation satellite systems (GNSS). Receivers that use GPS satellites and another system such as GLONASS. These systems provide precise timing and positioning information anywhere on the Earth with high reliability and low cost. The systems can be operated day or night, rain or shine, and do not require cleared lines of sight between survey stations. This represents a revolutionary departure from conventional surveying procedures, which rely on observed angles and distances for determining point positions. Since these systems all share similar features, the global positioning system will be discussed in further detail herein.

2- OVERVIEW OF GPS

Precise distances from the satellites to the receivers are determined from timing and signal information, enabling receiver positions to be computed. In satellite surveying, the satellites become the reference or control stations, and the ranges (distances) to these satellites are used to compute the positions of the receiver. Conceptually, this is equivalent to resection in traditional ground surveying work, where distances and/or angles are observed from an unknown ground station to control points of known position.

The global positioning system can be arbitrarily broken into three parts: (a) the space segment, (b) the control segment, and (c) the user segment. The space segment consists nominally of 24 satellites operating in six orbital planes spaced at 60° intervals around the equator. Four additional satellites are held in reserve as spares. The orbital planes are inclined to the equator at 55°.

This configuration provides 24-h satellite coverage between the latitudes of 80°N and 80°S. The satellites travel in near-circular orbits that have a mean altitude of 20,200 km above the Earth and an orbital period of 12 sidereal hours.1 The individual satellites are normally identified by their Pseudo Random Noise (PRN) number, (described below), but can also be identified by their satellite vehicle number (SVN) or orbital position.

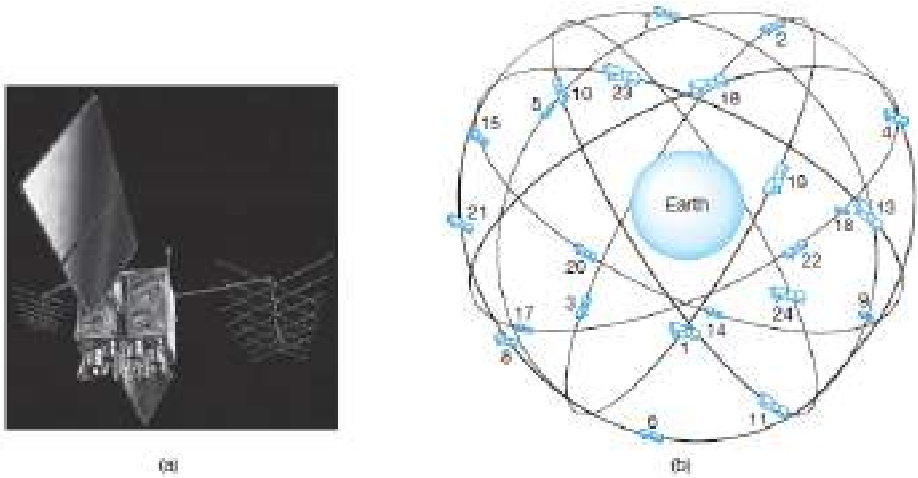


Figure 13.2 (a) A GPS satellite and (b) the GPS constellation.

The control segment consists of monitoring stations which monitor the signals and track the positions of the satellites over time. The initial GPS monitoring stations are at Colorado Springs, and on the islands of Hawaii, Ascension, Diego Garcia, and Kwajalein. The tracking information is relayed to the master control station in the Consolidated Space Operations Center (CSOC) located at Schriever Air Force base in Colorado Springs. The master control station uses this data to make precise, near-future predictions of the satellite orbits, and their clock correction parameters. This information is uploaded to the satellites, and in turn, transmitted by them as part of their broadcast message to be used by receivers to predict satellite positions and their clock biases (systematic errors).

The user segment in GPS consists of two categories of receivers that are classified by their access to two services that the system provides. These services are referred to as the Standard Position Service (SPS) and the Precise Positioning Service (PPS). The SPS is provided on the L1 broadcast frequency and more recently the L2 at no cost to the user. This service was initially intended to provide accuracies of 100 m in horizontal positions, and 156 m in vertical positions at the 95% error level. However, improvements in the system and the processing software have substantially reduced these error estimates. The PPS is broadcast on both the L1 and L2 frequencies, and is only available to receivers having valid cryptographic keys, which are reserved almost entirely for DoD use. This message provides a published accuracy of 18 m in the horizontal, and 28 m in the vertical at the 95% error level.

3-BASIC PRINCIPLE OF POSITION FIXING

Position fixing in three dimensions may involve the measurement of distance (or range) to at least three satellites whose X, Y and Z position is known, in order to define the user's Xp, Yp and Zp position. In its simplest form, the satellite transmits a signal on which the time of its departure (t_D) from the satellite is modulated. The receiver in turn notes the time of arrival (t_A) of this time mark. Then the time which it took the signal to go from satellite to receiver is

$(t_A - t_D) = t$ called the delay time. The measured range R is obtained from

$R_1 = (t_A - t_D)c = \Delta tc$

where c = the velocity of light.

Whilst the above describes the basic principle of range measurement, to achieve it one would require the receiver to have a clock as accurate as the satellite's and perfectly synchronized with it. As this would render the receiver impossibly expensive, a correlation procedure, using the pseudo-random binary codes (P or C/A), usually 'C/A', is adopted. The signal from the satellite arrives at the receiver and triggers the receiver to commence generating its own internal copy of the C/A code. The receiver-generated code is cross-correlated with the satellite code (Figure 9.10). The ground receiver is then able to determine the time delay (t) since it generated the same portion of the code received from the satellite. However, whilst this eliminates the problem of the need for an expensive receiver clock, it does not eliminate the problem of exact synchronization of the two clocks. Thus, the time difference between the two clocks, termed clock bias, results in an incorrect assessment of t. The distances computed are therefore called 'pseudo-ranges'.

The use of four satellites rather than three, however, can eliminate the effect of clock bias. A line in space is defined by its difference in coordinates in an X, Y and Z system:

$$R = (\Delta X^2 + \Delta Y^2 + \Delta Z^2)^{\frac{1}{2}}$$

If the error in R, due to clock bias, is δR and is constant throughout, then:

$$R_1 + \delta R = [(X_1 - X_p)^2 + (Y_1 - Y_p)^2 + (Z_1 - Z_p)^2]^{\frac{1}{2}}$$

$$R_2 + \delta R = [(X_2 - X_p)^2 + (Y_2 - Y_p)^2 + (Z_2 - Z_p)^2]^{\frac{1}{2}}$$

$$R_3 + \delta R = [(X_3 - X_p)^2 + (Y_3 - Y_p)^2 + (Z_3 - Z_p)^2]^{\frac{1}{2}}$$

$$R_4 + \delta R = [(X_4 - X_p)^2 + (Y_4 - Y_p)^2 + (Z_4 - Z_p)^2]^{\frac{1}{2}}$$

where X_n, Y_n, Z_n = the coordinates of satellites 1, 2, 3 and 4 (n = 1 to 4)
X_p, Y_p, Z_p = the coordinates required for point P
R_n = the measured ranges to the satellites

Solving the four equations for the four unknowns X_p, Y_p, Z_p and δR also solves for the error due to clock bias.

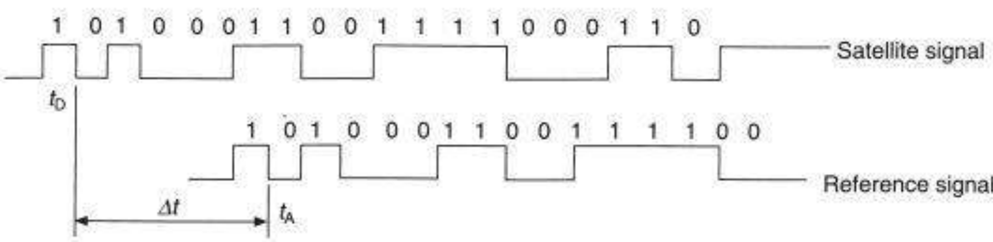


Fig. 9.10 *Correlation of the pseudo-binary codes*

REFERENCE COORDINATE SYSTEMS

In determining the positions of points on Earth from satellite observations, three different reference coordinate systems are important. First of all, satellite positions at the instant they are observed are specified in the “space-related” satellite reference coordinate systems. These are three-dimensional rectangular systems defined by the satellite orbits. Satellite positions are then transformed into a three-dimensional rectangular geocentric coordinate system, which is physically related to the Earth. As a result of satellite positioning observations, the positions of new points on Earth are determined in this coordinate system. Finally, the geocentric coordinates are transformed into the more commonly used and locally oriented geodetic coordinate system. The following subsections describe these three coordinate systems.

The Geodetic Coordinate System

Although the positions of points in a satellite survey are computed in the geocentric coordinate system, in that form they are inconvenient for use by surveyors (geomatics engineers). This is the case for three reasons: (1) with their origin at the Earth’s center, geocentric coordinates are typically extremely large values, (2) with the X-Y plane in the plane of the equator, the axes are unrelated to the conventional directions of north-south or east-west on the surface of the Earth, and (3) geocentric coordinates give no indication about relative elevations between points. For these reasons, the geocentric coordinates are converted to geodetic coordinates of latitude ϕ longitude λ and height (h) so that reported point positions become more meaningful and convenient for users.

Figure 13.6 also illustrates a quadrant of the reference ellipsoid, and shows both the geocentric coordinate system (X,Y,Z), and the geodetic coordinate system (ϕ, λ, h). Conversions from geocentric to geodetic coordinates, and vice versa are readily made. From the figure it can be shown that geocentric coordinates of point P can be computed from its geodetic coordinates using the following equations:

$$X_P = (R_{N_P} + h_P)\cos\phi_P \cos\lambda_P$$
$$Y_P = (R_{N_P} + h_P)\cos\phi_P \sin\lambda_P \qquad (13.1)$$
$$Z_P = [R_{N_P}(1 - e^2) + h_P]\sin\phi_P$$

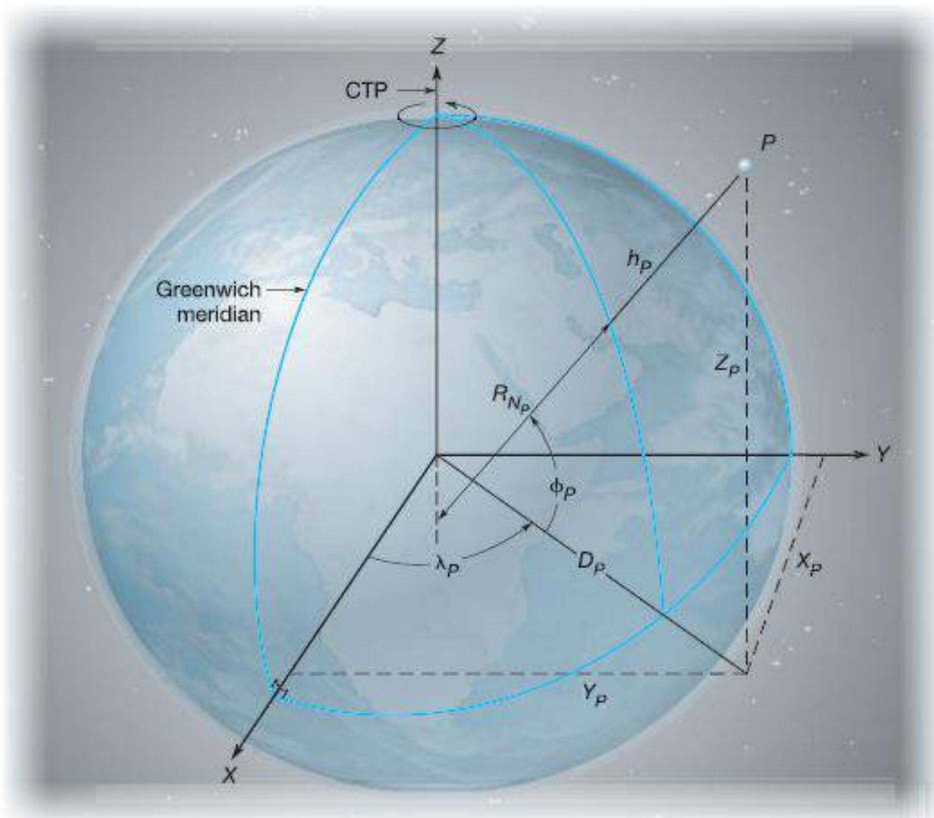


Figure 13.6 The geodetic and geocentric coordinate systems.

where

$$R_{N_P} = \frac{a}{\sqrt{1 - e^2 \sin^2 \phi_P}} \qquad (13.2)$$

In Equations (13.1), X_p , Y_p and Z_p are the geocentric coordinates of any point P, and the term e, which appears in both Equations (13.1) and (13.2), is the eccentricity of the WGS84 reference ellipsoid. Its value is 0.08181919084. In Equation (13.2), R_{NP} is the radius in the prime vertical of the ellipsoid at point P, and a, as noted earlier, is the semimajor axis of the ellipsoid. In Equations (13.1) and (13.2), north latitudes are considered positive and south latitudes negative.

Similarly, east longitudes are considered positive and west longitudes negative. Additionally, the programming for the conversion of geodetic coordinates to geocentric coordinates and vice versa is demonstrated.

Example 13.1

The geodetic latitude, longitude, and height of a point *A* are 41°15′18.2106″N, 75°00′58.6127″ W, and 312.391 m, respectively. Using WGS84 values, what are the geocentric coordinates of the point?

Solution

Substituting the appropriate values into Equations (13.1) and (13.2) yields

$$R_{N_A} = \frac{6{,}378{,}137}{\sqrt{1 - 0.0066943799 \sin^2(41°15'18.2106'')}} = 6{,}387{,}440.3113 \text{ m}$$

$$X_A = (6{,}387{,}440.3113 + 312.391) \cos 41°15'18.2106'' \cos(-75°00'58.6127'')$$

$$= 1{,}241{,}581.343 \text{ m}$$

$$Y_A = (6{,}387{,}440.3113 + 312.391) \cos 41°15'18.2106'' \sin(-75°00'58.6127'')$$

$$= -4{,}638{,}917.074 \text{ m}$$

$$Z_A = [6{,}387{,}440.3113(1 - 0.00669437999) + 312.391)] \sin(41°15'18.2106'')$$

$$= 4{,}183{,}965.568 \text{ m}$$

Conversion of geocentric coordinates of any point P to its geodetic values is accomplished using the following steps (refer again to Figure 13.6).

Step 1: Compute D_P as

$$D_P = \sqrt{X_P^2 + Y_P^2} \tag{13.3}$$

Step 2: Compute the longitude as[5]

$$\lambda_P = 2\tan^{-1}\left(\frac{D_P - X_P}{Y_P}\right) \tag{13.4}$$

Step 3: Calculate approximate latitude, ϕ_0[6]

$$\phi_0 = \tan^{-1}\left[\frac{Z_P}{D_P(1 - e^2)}\right] \tag{13.5}$$

Step 4: Calculate the approximate radius of the prime vertical, R_N, using ϕ_0 from step 3, and Equation (13.2).

Step 5: Calculate an improved value for the latitude from

$$\phi = \tan^{-1}\left(\frac{Z_P + e^2 R_{N_P}\sin(\phi_0)}{D_P}\right) \tag{13.6}$$

Step 6: Repeat the computations of steps 4 and 5 until the change in ϕ between iterations becomes negligible. This final value, ϕ_P, is the latitude of the station.

Step 7: Use the following formulas to compute the geodetic height of the station. For latitudes less than 45°, use

$$h_P = \frac{D_P}{\cos(\phi_P)} - R_{N_P} \tag{13.7a}$$

For latitudes greater than 45° use the formula

$$h_P = \left[\frac{Z_P}{\sin(\phi_P)}\right] - R_{N_P}(1 - e^2) \tag{13.7b}$$

Example 13.2

What are the geodetic coordinates of a point that has X, Y, Z geocentric coordinates of 1,241,581.343, –4,638,917.074, and 4,183,965.568, respectively? (Note: Units are meters.)

Solution

To visualize the solution, refer to Figure 13.6. Since the X coordinate value is positive, the longitude of the point is between 0° and 90°. Also, since the Y coordinate value is negative, the point is in the western hemisphere. Similarly since the Z coordinate value is positive, the point is in the northern hemisphere.

Substituting the appropriate values into Equations (13.3) through (13.7) yields

Step 1:

$$D = \sqrt{(1{,}241{,}581.343)^2 + (-4{,}638{,}917.074)^2} = 4{,}802{,}194.8993$$

Step 2:

$$\lambda = 2\tan^{-1}\left(\frac{4{,}802{,}194.8993 - 1{,}241{,}581.343}{-4{,}638{,}917.074}\right) = -75°00'58.6127'' \text{ (West)}$$

Step 3:

$$\phi_0 = \tan^{-1}\left[\frac{4{,}183{,}965.568}{4{,}802{,}194.8993(1 - 0.00669437999)}\right] = 41°15'18.2443''$$

Step 4:

$$R_N = \frac{6{,}378{,}137}{\sqrt{1 - 0.00669437999\sin^2(41°15'18.2443'')}} = 6{,}387{,}440.3148$$

Step 5:

$$\phi_0 = \tan^{-1}\left[\frac{4{,}183{,}965.568 + e^2 6{,}387{,}440.3148 \sin 41°15'18.2443''}{4{,}802{,}194.8993}\right]$$

$$= 41°15'18.2107''$$

Step 6: Repeat steps 4 and 5 until the latitude converges. The values for the next iteration are

$$R_N = 6{,}387{,}440.3113$$
$$\phi_0 = 41°15'18.2106''$$

Repeating with the above values results in the same value for latitude to four decimal places, so the latitude of the station is 41°15′18.2106″ N.

Step 7: Compute the geodetic height using Equation (13.7a) as

$$h = \frac{4{,}802{,}194.8993}{\cos 41°15'18.2106''} - 6{,}387{,}440.3113 = 312.391$$

The geodetic coordinates of the station are latitude = 41°15′18.2106″ N, longitude = 75°00′58.6127″ W, and height = 312.391 m. Note that this example was the reverse computations of Example 13.1, and it reproduced the starting geodetic coordinate values for that example.

It is important to note that geodetic heights obtained with satellite surveys are measured with respect to the ellipsoid. That is, the geodetic height of a point is the vertical distance between the ellipsoid and the point as illustrated in Figure 13.7. As shown, these are not equivalent to elevations (also called orthometric heights) given with respect to the geoid. To convert geodetic heights to elevations, the geoid height (vertical distance between ellipsoid and geoid) must be known. Then elevations can be expressed as:

$$H = h - N \qquad (13.8)$$

where H is elevation above the geoid (orthometric height), h the geodetic height (determined from satellite surveys), and N the geoidal height. Figure 13.7 shows the correct relationship of the geoid and the WGS84 ellipsoid in the continental United States. Here the ellipsoid is above the geoid, and geoid height (measured from the ellipsoid) is negative. The geoid height at any point can be estimated with mathematical models developed by combining gravimetric data with distributed networks of points where geoidal height has been observed.

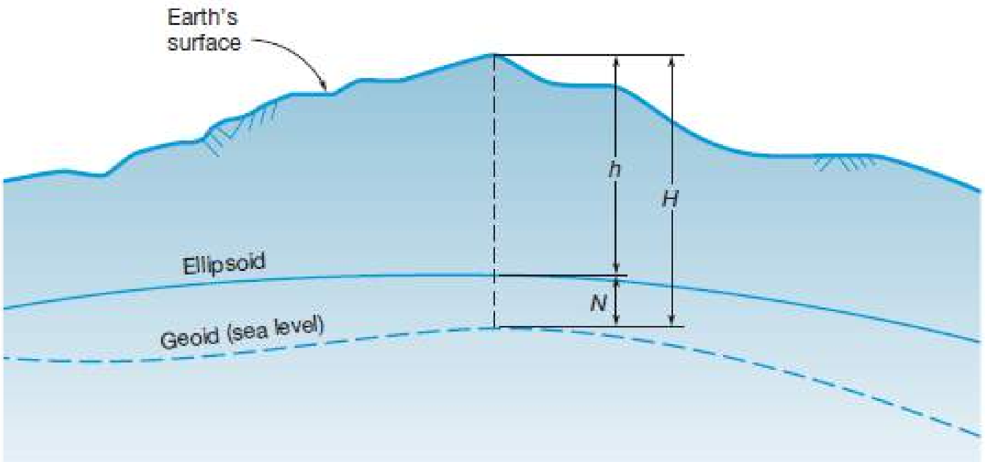


Figure 13.7 Relationships between elevation *H*, geodetic height *h*, and geoid undulation *N*.

Surveying 2

By Dr. Khamis Naba Sayl

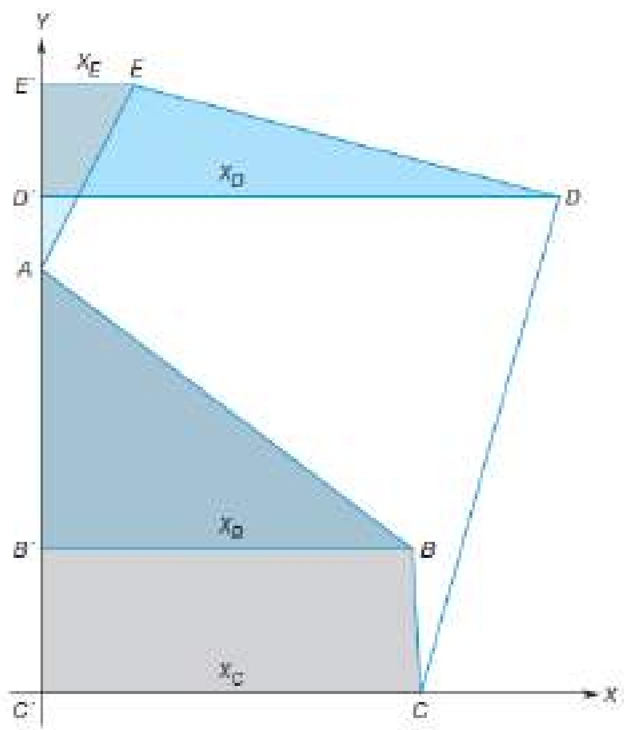
المحاضرة الثانية

Area by coordinates

Computation of area within a closed polygon is most frequently done by the coordinate method. In this procedure, coordinates of each angle point in the figure must be known. They are normally obtained by traversing, although any method that yields the coordinates of these points is appropriate.

The coordinate method is easily visualized; it reduces to one simple equation that applies to all geometric configurations of closed polygons and is readily programmed for computer solution.

The procedure for computing areas by coordinates can be developed with reference to Figure as shown:



As shown in that figure, it is convenient (but not necessary) to adopt a reference coordinate system with the X and Y axes passing through the most southerly and the most westerly traverse stations, respectively.

Lines BB′, CC′, DD′ and EE′ in the figure are constructed perpendicular to the Y axis. These lines create a series of trapezoids and triangles (shown by different color shadings). The area enclosed with traverse ABCDEA can be expressed in terms of the areas of these individual trapezoids and triangles as:

$$\text{area}_{ABCDEA} = E'EDD' + D'DCC' - AE'EA - CC'B'B - ABB'A \quad (12.5)$$

The area of each trapezoid, for example $E'EDD'E'$ can be expressed in terms of lengths as

$$\text{area}_{E'EDD'} = \frac{E'E + DD'}{2} \times E'D'$$

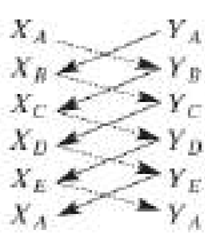
In terms of coordinate values, this same area $E'EDD'E'$ is

$$\text{area}_{E'EDD'} = \frac{X_E + X_D}{2}(Y_E - Y_D)$$

Each of the trapezoids and triangles of Equation (12.5) can be expressed by coordinates in a similar manner. Substituting these coordinate expressions into Equation (12.5), multiplying by 2 to clear fractions, and rearranging

$$\begin{aligned} 2(\text{area}) = &+X_AY_B + X_BY_C + X_CY_D + X_DY_E + X_EY_A \\ &-X_BY_A - X_CY_B - X_DY_C - X_EY_D - X_AY_E \end{aligned} \quad (12.6)$$

Equation (12.6) can be reduced to an easily remembered form by listing the X and Y coordinates of each point in succession in two columns, as shown in Equation (12.7), with coordinates of the starting point repeated at the end. The products noted by diagonal arrows are ascertained with dashed arrows considered plus and solid ones minus. The algebraic summation of all products is computed and its absolute value divided by 2 to get the area.



(12.7)

The procedure indicated in Equation (12.7) is applicable to calculating any size traverse. The following formula, easily derived from Equation (12.6), is a variation that can also be used,

$$\begin{aligned} \text{area} = \frac{1}{2}\big[&X_A(Y_E - Y_B) + X_B(Y_A - Y_C) + X_C(Y_B - Y_D) \\ &+ X_D(Y_C - Y_E) + X_E(Y_D - Y_A)\big] \end{aligned} \quad (12.8)$$

It was noted earlier that for convenience, an axis system can be adopted in which for the most westerly traverse point, and for the most southerly station. Magnitudes of coordinates and products are thereby reduced, and the amount of work lessened, since four products will be zero. However, selection of a special origin like that just described is of little consequence if the problem has been programmed for computer solution. Then the coordinates obtained from traverse adjustment can be used directly in the solution. However, a word of caution applies, if coordinate values are extremely large as they would normally be; for example, if state plane values are used. In those cases, to ensure sufficient precision and prevent serious round-off errors, double precision should be used. Or, as an alternative, the decimal place in each coordinate can arbitrarily be moved n places to the left, the area calculated and then multiplied by 10^{2n}.

Example : Find the area as shown in the figure

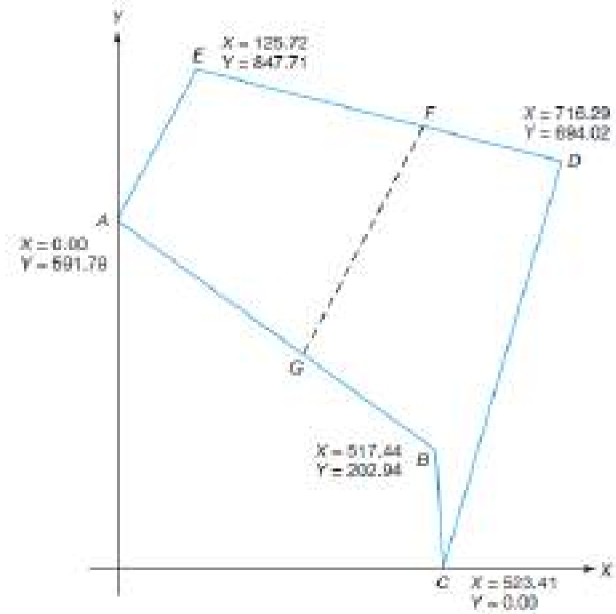


Figure 12.5 Traverse for computation of area by coordinates.

Point	X	Y	Plus (XY)	Minus (YX)
A	0.00	591.78		
B	517.44	202.94	0	306,211
C	523.41	0.00	0	106,221
D	716.29	694.02	363,257	0
E	125.72	847.71	607,206	87,252
A	0.00	591.78	74,398	0
			Σ = 1,044,861	Σ = 499,684
			−499,684	
			545,177	

2 area= 545177

Area= 272588 m^2

Area by DOUBLE-MERIDIAN DISTANCE METHOD

The area within a closed figure can also be computed by the double-meridian distance (DMD) method. This procedure requires balanced departures and latitudes of the tract's boundary lines, which are normally obtained in traverse computations. The DMD method is not as commonly used as the coordinate method because it is not as convenient, but given the data from an adjusted traverse, it will yield the same answer. The DMD method is useful for checking answers obtained by the coordinate method when performing hand computations.

By definition, ***the meridian distance of a traverse course is the perpendicular distance from the midpoint of the course to the reference meridian*****.** To ease the problem of signs, a reference meridian usually is placed through the most westerly traverse station.

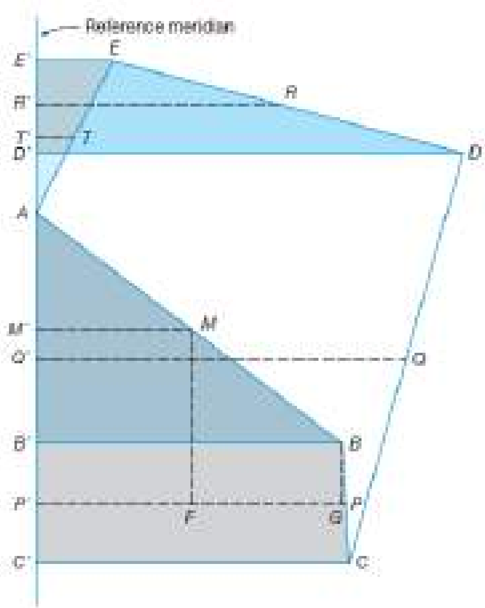


Figure 12.6 Meridian distances and traverse area computation by DMD method.

In Figure 12.6, the meridian distances of courses AB, BC, CD, DE, and EA are MM', PP', QQ', RR', and TT', respectively. To express PP' in terms of convenient distances, MF and BG are drawn perpendicular to PP'. Then

$$PP' = P'F + FG + GP$$
$$= \text{meridian distance of } AB + \frac{1}{2}\text{departure of } AB + \frac{1}{2}\text{departure of } BC$$

Thus, the meridian distance for any course of a traverse equals the meridian distance of the preceding course plus one half the departure of the preceding course plus half the departure of the course itself. It is simpler to employ full departures of courses. Therefore, DMDs equal to twice the meridian distances that are used, and a single division by 2 is made at the end of the computation.

Based on the considerations described, the following general rule can be applied in calculating DMDs: The DMD for any traverse course is equal to the DMD of the preceding course, plus the departure of the preceding course, plus the departure of the course itself. Signs of the departures must be considered. When the reference meridian is taken through the most westerly station of a closed traverse and calculations of the DMDs are started with a course through that station, the DMD of the first course is its departure. Applying these rules, for the traverse in Figure 12.6.

$$\text{DMD of } AB = \text{departure of } AB$$
$$\text{DMD of } BC = \text{DMD of } AB + \text{departure of } AB + \text{departure of } BC$$

A check on all computations is obtained if the DMD of the last course, after computing around the traverse, is also equal to its departure but has the opposite sign. If there is a difference, the departures were not correctly adjusted before starting, or a mistake was made in the computations. With reference to Figure 12.6, the area enclosed by traverse ABCDEA may be expressed in terms of trapezoid areas (shown by different color shadings) as:

$$\text{area} = E'EDD'E' + C'CDD'C' - (AB'BA + BB'C'CB + AEE'A) \qquad (12.10)$$

The area of each figure equals the meridian distance of a course times its balanced latitude. For example, the area of trapezoid C′CDD′C′ = Q′Q * C′D′, where Q′Q and C′D′ are the meridian distance and latitude, respectively, of line CD. The DMD of a course multiplied by its latitude equals double the area. Thus, the algebraic summation of all double areas gives twice the area inside the entire traverse. Signs of the products of DMDs and latitudes must be considered. If the reference line is passed through the most westerly station, all DMDs are positive. The products of DMDs and north latitudes are therefore plus and those of DMDs and south latitudes are minus.

Example: Find the area for the figure as shown below using DMD method

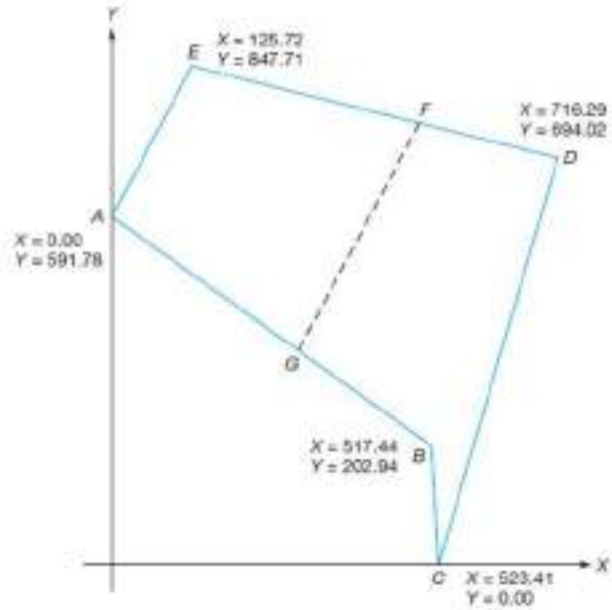


Figure 12.5
Traverse for computation of area by coordinates.

Solution

TABLE 12.3 COMPUTATION OF DMDs

Departure of *AB* =	+517.444	= DMD of *AB*
Departure of *AB* =	+517.444	
Departure of *BC* =	+5.964	
	+1040.852	= DMD of *BC*
Departure of *BC* =	+5.964	
Departure of *CD* =	+192.881	
	+1239.697	= DMD of *CD*
Departure of *CD* =	+192.881	
Departure of *DE* =	−590.571	
	+842.007	= DMD of *DE*
Departure of *DE* =	−590.571	
Departure of *EA* =	−125.718	
	+125.718	= DMD of *EA* ✓

TABLE 12.4 COMPUTATION OF AREA BY DMDs

Course	Balanced Departure	Balanced Latitude	DMD	Double Areas Plus	Double Areas Minus
AB	517.44	−388.84	517.44		201,201
BC	5.96	−202.95	1040.85		211,240
CD	192.88	694.02	1239.70	860,376	
DE	−590.57	153.69	842.01	129,408	
EA	−125.72	−255.93	125.72	———	32,176
Total	0.00	0.00		989,784	444,617
				−444,617	
				545,167	

2 area= 545177

Area= 272588 m^2

Example for second method Find the area of a closed traverse considering the following data, by the latitude and DMD method.

Side	Latitude	Departure
AB	+ 225.5	+ 120.5
BC	− 245.0	+ 210.0
CD	− 150.5	− 110.5
DA	+ 170.0	− 220.0

Solution

Calculation of DMD

DMD of AB = + 120.5

DMD of BC = + 120.5 + 120.5 + 210.0 = 451.0

DMD of CD = + 451.0 + 210.0 − 110.5 = + 550.5

DMD of DA = + 550.5 − 110.5 − 220.0 = + 220.0

The result is tabulated as follows:

Side	Latitude	Departure	DMD	Double area = (col. 2 × col. 4)	
1	2	3	4	5 (+)	(–) 6
AB	+ 225.5	+ 120.5	+ 120.5	27,172.75	—
BC	– 245.0	+ 210.0	+ 451.0	—	110,495.00
CD	– 150.5	– 110.5	+ 550.5	—	82,850.25
DA	+ 170.0	– 220.0	+ 220.0	37,400.00	—
			Total =	+ 64,572.75	– 193,345.25
			Algebraic sum =		– 128,772.5

(Negative sign neglected)

Twice area = algebraic sum

∴ Required area of traverse $= \frac{1}{2} \times 128{,}772.5$

$= 64{,}386.25 \text{ m}^2$

Surveying 2

By Dr. Khamis Naba Sayl

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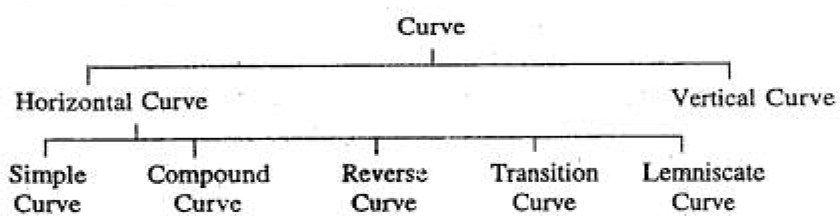
CURVES

In highways, railways, or canals the curve are provided for smooth or gradual change in direction due the nature of terrain, cultural features, or other unavoidable reasons. In highway practice, it is recommended to provide curves deliberately on straight route to break the monotony in driving on long straight route to avoid accidents.

The horizontal curve may be a simple circular curve or a compound curve. For a smooth transition between straight and a curve, a transition or easement curve is provided. The vertical curves are used to provide a smooth change in direction taking place in the vertical plane due to change of grade.



10.2 TYPES OF HORIZONTAL CURVES

The following are the different types of horizontal curves:

1. Simple circular curve When a curve consists of a single arc with a constant radius connecting the two tangents, it is said to be a circular curve (Fig. 10.5).

2. Compound curve When a curve consists of two or more arcs with different radii, it is called a compound curve. Such a curve lies on the same side of a common tangent and the centres of the different arcs lie on the same side of their respective tangents (Fig. 10.6).

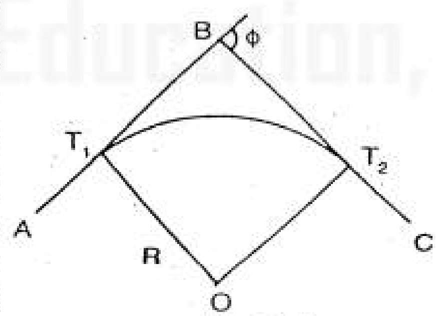


Fig. 10.5

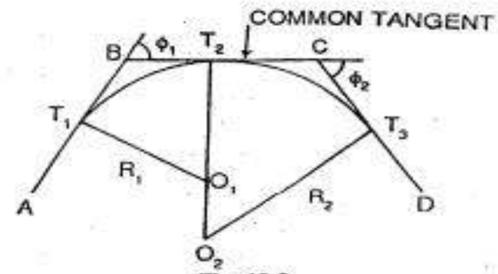


Fig. 10.6

3. Reverse curve A reverse curve consists of two arc bending in opposite directions. Their centres lie on opposite sides of the curve. Their radii may be either equal or different, and they have one common tangent (Fig. 10.7).

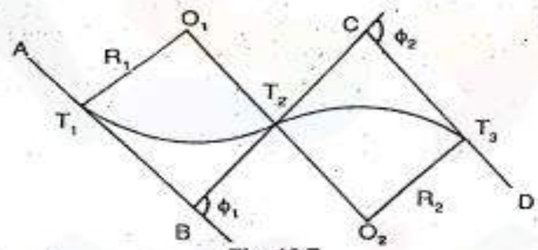


Fig. 10.7

4. Transition curve A curve of variable radius is known as a transition curve. It is also called a spiral curve or easement curve. In railways, such a curve is provided on both sides of a circular curve to minimise superelevation. Excessive superelevation may cause wear and tear of the rail section and discomfort to passengers (Fig. 10.8).

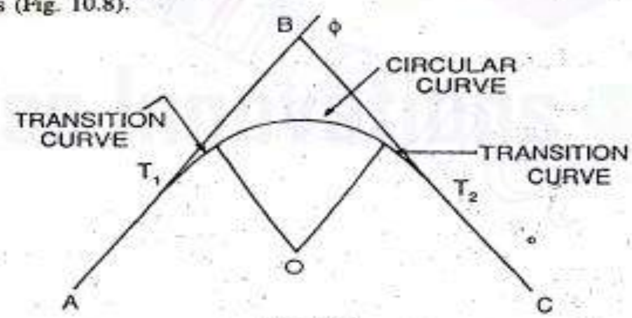


Fig. 10.8

Elements of CIRCULAR CURVES

A simple circular curve shown in Fig. 7.1, consists of simple arc of a circle of radius R connecting two straights AI and IB at tangent points T1 called the point of commencement (P.C.) and T2 called the point of tangency (P.T.), intersecting at I, called the point of intersection (P.I.), having a deflection angle Δ or angle of intersection φ. The distance E of the midpoint of the curve from I is called the external distance. The arc length from T1 to T2 is the length of curve, and the chord T1T2 is called the long chord. The distance M between the midpoints of the curve and the long chord, is called the mid-ordinate. The distance T1I which is equal to the distance IT2, is called the tangent length. The tangent AI is called the back tangent and the tangent IB is the forward tangent.

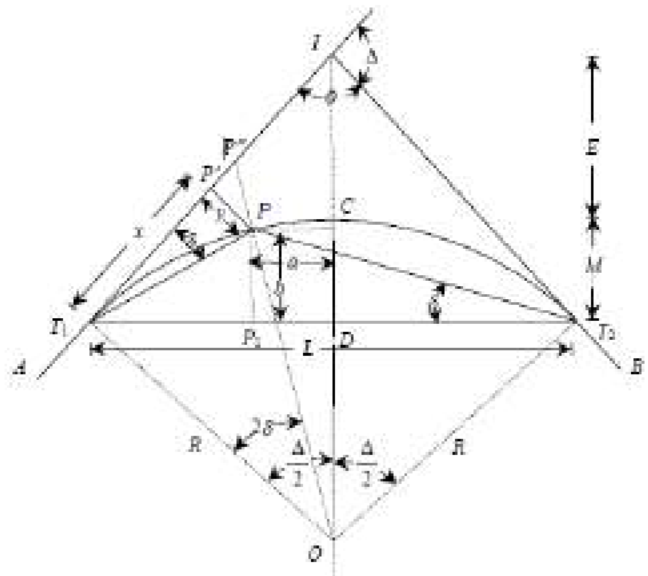


Fig. 7.1

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

$$\text{Tangent length } (T) = R\tan\frac{\Delta}{2} \qquad \text{...(7.1)}$$

$$\text{Length of curve } (l) = \frac{\pi R\Delta}{180} \qquad \text{...(7.2)}$$

$$\text{Long chord } (L) = 2R\sin\frac{\Delta}{2} \qquad \text{...(7.3)}$$

$$\text{External distance } (E) = R\left(\sec\frac{\Delta}{2} - 1\right) \qquad \text{...(7.4)}$$

$$\text{Mid-ordinate } (M) = R\left(1 - \cos\frac{\Delta}{2}\right) \qquad \text{...(7.5)}$$

$$\text{Chainage of } T_1 = \text{Chainage of P.I.} - T \qquad \text{...(7.6)}$$

$$\text{Chainage of } T_2 = \text{Chainage of } T_1 + l. \qquad \text{...(7.7)}$$

Setting of circular curve

There are various methods for setting out circular curves. Some of them are:

1-Perpendicular Offsets from Tangent (Fig. 7.1) using the equation:

Offsets from tangents may be:

1. Radial
2. Perpendicular

1. Radial Offsets

(a) In Fig. 10.16, AB and BC are two tangents intersecting at B, and that the tangent points are T_1 and T_2.

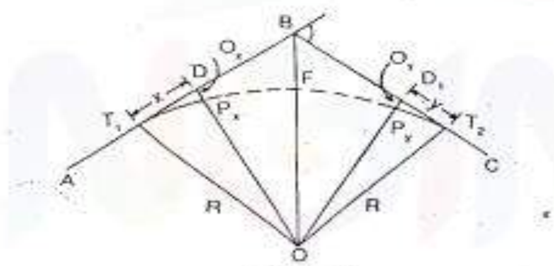


Fig. 10.16

Let us take a point D on the rear tangent AB such that

$$T_1D = x$$

Let O_x be the radial offset at D.
The point D is joined with the centre O. So, OD is the radial line.
Now, from ΔT_1OD

$$OT_1^2 + T_1D^2 = OD^2$$

where

$$OT_1 = R$$
$$OD = R + O_x$$
$$T_1D = x$$

$$\therefore \quad R^2 + x^2 = (R + O_x)^2$$

$$\text{or} \quad R + O_x = \sqrt{R^2 + x^2}$$

$$O_x = \sqrt{R^2 + x^2} - R$$

(b) The calculated distance O_x is cut off from the radial line OD to get the first point of the curve P_x.

(c) By increasing the value of x by a regular amount, a number of offsets are obtained. These are set off along the respective radial lines.

From the tangent point T_1, one half of the curve can be set out. In this case, the left half of the curve can be set out from T_1 up to the apex point F.

(d) The other half of the curve can be set out from the second tangent point T_2. Let a point D_1 be taken at a distance y from T_2. The offset O_y is then calculated as $O_y = \sqrt{R^2 + y^2} - R$

The calculated distance O_y is set off along the radial line OD_1 to get the point P_y on the curve. Thus by increasing the value of Y, the required offsets are calculated and set off along their respective radial lines to get the points on the curve for the right half.

2. By perpendicular offsets

(a) In Fig. 10.17, AB and BC are two tangents meeting at a point B. The tangent length is calculated and the tangent points T_1 and T_2 are marked on the ground.

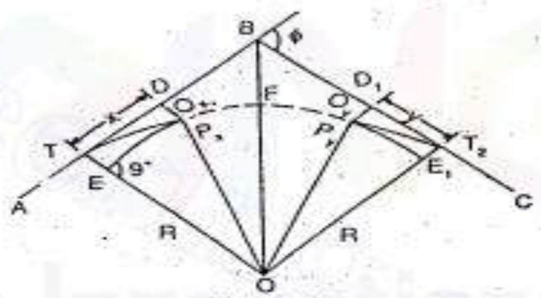


Fig. 10.17

(b) A point D is taken along the rear tangent AB at a distance x from T_1. Let O_x be the perpendicular offset at D. The line EP_x is drawn parallel to T_1D.

Here $\quad OE = R - O_x, \quad OP_x = R, \quad EP_x = x$

From qOEP$_x$, $\quad OP_x^2 = EP_x^2 + OE^2$

$$\text{or} \quad R^2 = x^2 + (R - O_x)^2$$

$$\text{or} \quad R - O_x = \sqrt{R^2 - x^2}$$

$$\text{or} \quad O_x = R - \sqrt{R^2 - x^2}$$

(c) This calculated distance O_x is set out along the perpendicular drawn at D to get the point P_x on the curve.

Similarly, by progressively increasing the value of x by a regular amount, a series of offsets are obtained. These are set out along the perpendicular drawn through the respective points.

Thus the left half of the curve is set out from T_1 up to the apex F.

(d) The other half of the curve is set out from T_2, by calculating the offset by the relation

$$O_y = R - \sqrt{R^2 - y^2}$$

The calculated distance O_y is set out along the perpendicular drawn at D_1 to get the point P_y on the curve. This process is continued until the apex F is reached.

2-Offsets from Long Chord using the equation:

Let AB and BC be two tangents meeting at a point B, with a deflection angle ϕ. The following data are calculated for setting out the curve (Fig. 10.11).

1. The tangent length is calculated according to the formula; TL = $R \tan \phi/2$
2. Tangent points T_1 and T_2 are marked.
3. The length of the curve is calculated according to the formula:

$$CL = \frac{\pi R \phi^{\circ}}{180^{\circ}}$$

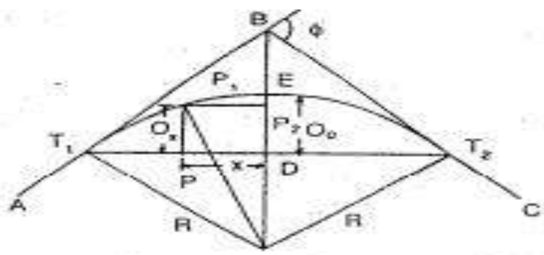


Fig. 10.11

4. The chainages of T_1 and T_2 are found out.
5. The length of the long chord (L) is calculated from:

$$L = 2R \sin \phi/2$$

6. The long chord is divided into two equal halves the left half and the right half). Here the curve is symmetrical in both the halves.
7. The mid-ordinate O_0 is calculated as follows:

(a) O_0 = DE = versed sine of curve = $R(1 - \cos \phi/2)$ (1)

(b) Again OF = R and OD = $R - O_0$

From triangle OT_1D, $OT_1^2 = OD^2 + T_1D^2$

or $$R^2 = (R - O_0)^2 + \left(\frac{L}{2}\right)^2$$

or $$R - O_0 = \sqrt{R^2 - (L/2)^2}$$

or $$O_0 = R - \sqrt{R^2 - (L/2)^2} \qquad (2)$$

Thus, the mid-ordinate O_0 can be calculated from Eq. (1) or (2).

8. Considering the left half of the long chord, the ordinates O_1, O_2, ... are calculated at distances X_1, X_2, ... taken from D towards the tangent point T_1.

The formula for the calculation of ordinates is deduced as follows.

Let P be a point at a distance x from D. Then PP_1 (O_x) is the required ordinate. A line P_1P_2 is drawn parallel to T_1T_2. From triangle OP_1P_2,

$$OP_1^2 = OP_2^2 + P_1P_2^2$$

or $R^2 = \{(R - O_0) + O_x\}^2 + x^2$ [where, $OP_2 = (R - O_0) + O_x$]

or $$R - O_0 + O_x = \sqrt{R^2 - x^2}$$

or $$O_x = \sqrt{R^2 - x^2} - (R - O_0) \qquad (3)$$

9. The ordinates for the right half are similar to these obtained for the left half.

Example Two tangents AB and BC intersect at a point B at chainage 150.5 m. Calculate all the necessary data for setting out a circular curve of radius 100 m and deflection angle 30° by the method of offsets from the long chord.

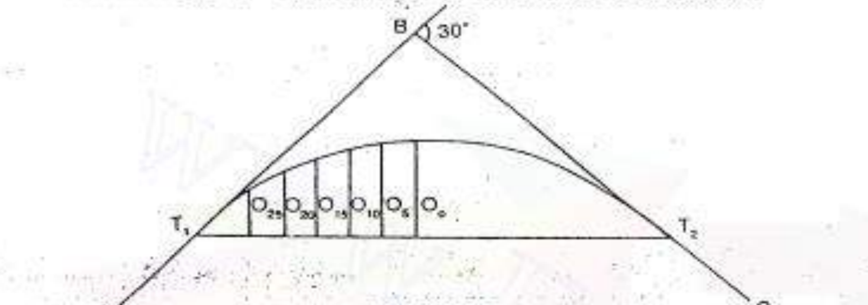


Fig. E.10.1

Solution

1. Tangent length $= R \tan \frac{\phi}{2}$

$$= 100 \times \tan 15° = 26.79 \text{ m}$$

2. Chainage of $T_1 = 150.50 - 26.79 = 123.71$ m

3. Curve length $= \frac{\pi R \phi°}{180°} = \frac{3.14 \times 100 \times 30°}{180°} = 52.36$ m

4. Chainage of $T_2 = 123.71 + 52.36 = 176.07$ m

5. Length of long chord $(L) = 2R \sin \phi/2$

$$= 2 \times 100 \times \sin 15° = 51.76 \text{ m}$$

6. The long chord is divided into two equal halves.

$$\text{Each half} = 1/2 \times 51.76 = 25.88 \text{ m}$$

7. Mid-ordinate, $O_0 = R - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$

$$= 100 - \sqrt{100^2 - 25.88^2} = 3.41 \text{ m}$$

8. The ordinates are calculated at 5 m intervals starting from the centre towards T_1 for the left half.

$$O_5 = \sqrt{R^2 - x^2}\,(R - O_0)$$
$$= \sqrt{(100^2 - 5^2)} - (100 - 3.41)$$
$$= 99.87 - 96.59 = 3.28 \text{ m}$$
$$O_{10} = \sqrt{(100^2 - 10^2)} - 96.59$$
$$= 99.50 - 96.59 = 2.91 \text{ m}$$

$$O_{15} = \sqrt{([illegible] - 15^2)} - 96.59$$
$$= [illegible] - 96.59 = 2.28 \text{ m}$$
$$O_{20} = \sqrt{([illegible] - 20^2)} - 96.59$$
$$= 97.97 - [illegible] = 1.38 \text{ m}$$
$$O_{25} = \sqrt{(100^2 - 25^2)} - 96.59$$
$$= 96.82 - [illegible] = 0.2[illegible]$$
$$O_{25.88} = \sqrt{(100^2 - 25.88^2)} - 96.59 = 0 \quad \text{(checked)}$$

9. The ordinates for the right half are similar to those for the left half.

Surveying 2

By Dr. Khamis Naba Sayl

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AVERAGE-END-AREA FORMULA

Figure 26.3 illustrates the concept of computing volumes by the average-end area method. In the figure A_1 and A_2, and are end areas at two stations separated by a horizontal distance L. The volume between the two stations is equal to the average of the end areas multiplied by the horizontal distance L between them.

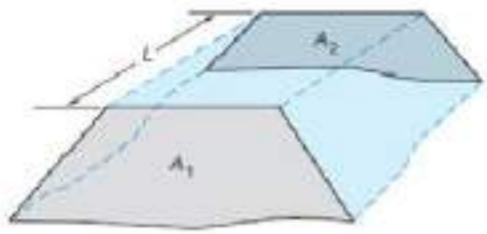

Figure 26.3
Volume by average-end-area-method.

Thus:

$$v = \frac{A_1 + A_2}{2} * L \qquad (26.1)$$

In Equation (26.1), $A_1$and A_2 are in m^2 L is in m, and V_C is in m^3. Equations (26.1) is approximate and give answers that generally are slightly larger than the true prismoidal volumes. They are used in practice because of their simplicity, and contractors are satisfied because pay quantities are generally slightly greater than true values. Increased accuracy is obtained by decreasing the distance L between sections. When the ground is irregular, cross sections must be taken closer together.

Example: Compute the volume of excavation between station 24+00 with an end area of 711 m^2 and 25+00 station with an end area of 515 m^2.

Solution

$V_C = (\frac{V_1+V_2}{2}) * L$

$V_C = (\frac{711+515}{2})100 = 61300\ m^3$

End Areas by Coordinates

The coordinate method for computing end areas can be used for any type of section, and has many engineering applications. The procedure was described to determine the area contained within a closed polygon traverse.

For example

Station	*H*	*L*	*C*	*D*	*E*	*R*	*G*
24 + 00	$\frac{0}{15}$	$\frac{C12.5}{33.8}$	$\frac{C15.8}{20}$	$\frac{C18.0}{0}$	$\frac{C10.1}{12}$	$\frac{C12.2}{33.3}$	$\frac{0}{15}$

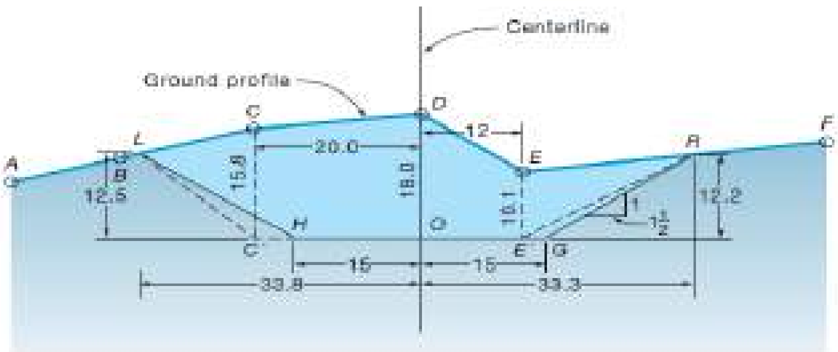

Figure 26.4
End-area computation.

$$\frac{0}{15} \quad \frac{12.5}{33.8} \quad \frac{15.8}{20} \quad \frac{18}{0} \quad \frac{10.1}{-12} \quad \frac{12.2}{-33.3} \quad \frac{0}{-15} \quad \frac{0}{15}$$

Answer = 710 m^2

PRISMOIDAL FORMULA

The prismoidal formula applies to volumes of all geometric solids that can be considered prismoids. A prismoid, illustrated in Figure 26.6, is a solid having ends that are parallel but not similar and trapezoidal sides that are also not congruent. Most earthwork solids obtained from cross-section data fit this classification.

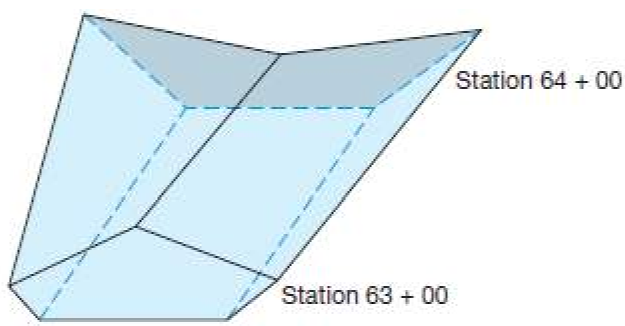

Figure 26.6
Sections for which the prismoidal correction is added to the end-area volume.

However, from a practical standpoint, the differences in volumes computed by the average-end-area method and the prismoidal formula are usually so small as to be negligible. Where extreme accuracy is needed, such as in expensive rock cuts, the prismoidal method can be used. One arrangement of the prismoidal formula is:

$$V_p = L(A_1 + 4A_m + A_2)/6$$

Where V_P is the prismoidal volume in cubic meter, A_1 and A_2 are areas of successive cross sections taken in the field, A_m is the area of a "computed" section midway between A_1 and A_2, and L is the horizontal distance between A_1 and A_2.

To use the prismoidal formula, it is necessary to know area A_m of the section halfway between the stations of A_1 and A_2. This is found by the usual computation after averaging the heights and widths of the two end sections. Obviously, the middle area is not the average of the end areas, since there would then be no difference between the results of the end-area formula and the prismoidal formula.

The prismoidal formula generally gives a volume smaller than that found by the average-end-area formula. For example, the volume of a pyramid by the prismoidal formula is Ah/3 (the exact value), whereas by the average-end-area method it is Ah/2. An exception occurs when the center height is great but the width narrow at one station, and the center height small but the width large at the adjacent station.

Example: Compute the volume using the prismoidal formula and by average end areas for the following three-level sections of a roadbed having a base of 24 m and side slopes of 1.5/1.

Solution

Station	L	C	R	Area
12 + 00	$\frac{C7.8}{23.7}$	$\frac{C5.3}{0}$	$\frac{C7.4}{23.0}$	$\frac{5.3(23.7 + 23.0)}{2} + \frac{24(7.8 + 7.4)}{4} = 215.0$
12 + 50	$\frac{C6.5}{21.8}$	$\frac{C6.0}{0}$	$\frac{C7.5}{23.2}$	$\frac{6.0(21.8 + 23.2)}{2} + \frac{24(6.5 + 7.5)}{4} = 219.0$
13 + 00	$\frac{C5.8}{24.8}$	$\frac{C6.6}{0}$	$\frac{C7.0}{23.5}$	$\frac{6.6(24.8 + 23.5)}{2} + \frac{24(5.8 + 7.0)}{4} = 236.2$

$$V_P = \frac{100(215.0 + 4(219.0) + 236.2)}{6} = 22120\ m^3$$

$$V_c = \frac{100\ (215.0+236.2)}{2} = 22560\ m^3$$

Volume computations

Volume calculations for route construction projects are usually done and arranged in tabular form. To illustrate this procedure, assume that end areas listed in columns (2) and (3) of Table 26.3 apply to the section of roadway illustrated in Figure 26.1. By using Equation (26.1), cut and fill volumes are computed and tabulated in columns (4) and (5).

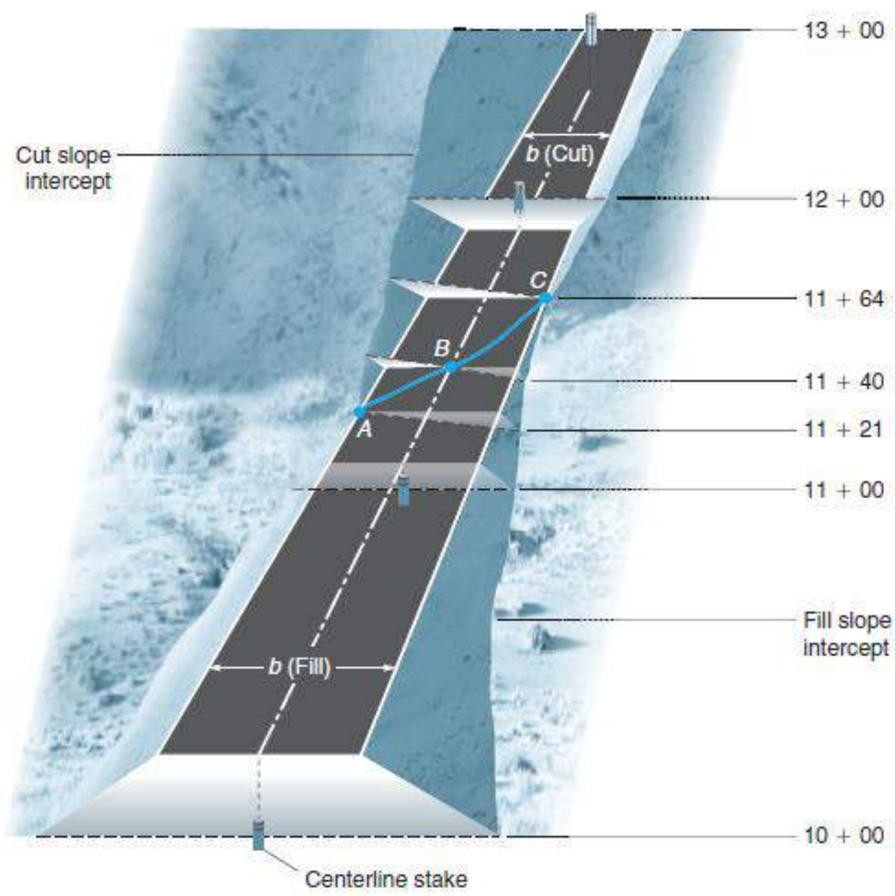

Figure 26.1 Section of roadway illustrating excavation (cut) and embankment (fill).

The volume computations illustrated in Table 26.3 include the transition sections of Figure 26.1. This is normally not done when preliminary earthwork volumes are being estimated (during design and prior to construction) because the exact locations of the transition sections and their configurations are usually unknown until slope staking occurs. Thus, for calculating preliminary earthwork quantities, an end area of zero would be used at the station of the centerline grade point (station 11+40 of Figure 26.1), and transition sections (stations 11+21and 11+64 of Figure 26.1) would not appear in the computations. After slope staking the locations and end areas of transition sections are known, and they should be included in final volume computations, especially if they significantly affect the quantities for which payment is made.

Table 26.3

Stations	Area Cut(m^2)	Area Fill(m^2)	Volume Cut(m^3)	Volume Fill (m^3)
10+00		992		
				70650
11+00		421		
				5134.5
11+21	0	68		
			215.3	940.5
11+40	34	31		
			2136	248
11+64	144	0		
			14940	
12+00	686			
			80200	
13+00	918			

Problem 1 An embankment of width 10 m and side slopes $1\frac{1}{2} : 1$ is required to be made on a ground which is level in a direction transverse to the centre line. The central heights at 40 m intervals are as follows:

0.90, 1.25, 2.15, 2.50, 1.85, 1.35, and 0.85

Calculate the volume of earth work according to (i) the trapezoidal formula, and (ii) the prismoidal formula.

Solution The cross-sectional areas are calculated by Eq. (1):

Area, $\Delta = (b + Sh) \times h$

$\Delta_1 = (10 + 1.5 \times 0.90) \times 0.90 = 10.22\ m^2$
$\Delta_2 = (10 + 1.5 \times 1.25) \times 1.25 = 14.84\ m^2$
$\Delta_3 = (10 + 1.5 \times 2.15) \times 2.15 = 28.43\ m^2$
$\Delta_4 = (10 + 1.5 \times 2.50) \times 2.50 = 34.38\ m^2$
$\Delta_5 = (10 + 1.5 \times 1.85) \times 1.85 = 23.63\ m^2$
$\Delta_6 = (10 + 1.5 \times 1.35) \times 1.35 = 16.23\ m^2$
$\Delta_7 = (10 + 1.5 \times 0.85) \times 0.85 = 9.58\ m^2$

(a) Volume according to trapezoidal formula:

$$V = \frac{40}{2}\{10.22 + 9.58 + 2(14.84 + 28.43 + 34.38 + 23.63 + 16.23)\}$$
$$= 20\{19.80 + 235.02\} = 5{,}096.4\ m^3$$

(b) Volume calculated in prismoidal formula:

$$V = \frac{40}{3}\{10.22 + 9.58 + 4(14.84 + 34.38 + 16.23) + 2(28.43 + 23.63)\}$$
$$= \frac{40}{3}(19.80 + 261.80 + 104.12) = 5{,}142.9\ m^3$$

VOLUME USING SPOT HEIGHT METHOD

This method is generally used for calculating the volumes of excavations for basements or tanks, i.e. any volume where the sides and base are planes, whilst the surface is broken naturally (Figure 11.22(a)). Figure 11.22(b) shows the limits of the excavation with surface levels in metres at A, B, C and D. The sides are vertical to a formation level of 20 m. If the area ABCD was a plane, then the volume of excavation would be:

V = plan area ABCD × mean height

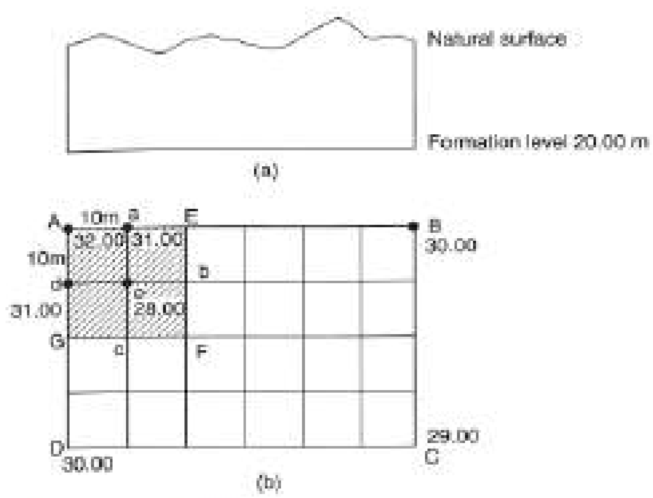

Fig. 11.22 *(a) Section, and (b) plan*

However, as the illustration shows, the surface is very broken and so must be covered with a grid such that the area within each 10-m grid square is approximately a plane. It is therefore the ruggedness of the ground that controls the grid size. If, for instance, the surface Aaed was not a plane, it could be split into two triangles by a diagonal (Ae) if this would produce better surface planes. Considering square Aaed only:

V = plan area × mean height

$$V = 100 * \frac{1}{4}(12 + 11 + 8 + 11) = 1050\ m^3$$

If the grid squares are all equal in area, then the data is easily tabulated and worked as follows: Considering AEFG only, instead of taking each grid square separately, one can treat it as a whole.

$$V = \frac{100}{4}(h_A + h_E + h_F + h_G) + 2(h_a + h_b + h_c + h_d) + 4h_e$$

If one took each grid separately it would be seen that the heights of AEFG occur only once, whilst the heights of abcd occur twice and he occurs four times; one still divides by four to get the mean

VOLUME USING CONTOUR AREA METHOD

Volumes based on contours can be obtained from contour maps by using a planimeter to determine the area enclosed by each contour. Alternatively, CAD software can be used to determine these areas. Then the average area of the adjacent contours is obtained using Equation (26.1) and the volume obtained by multiplying by the contour spacing (i.e., contour interval). Use of the prismoidal formula is seldom, if ever, justified in this type of computation. This procedure is the basis for volume computations in CAD software.

The contour-area method is suitable for determining volumes over large areas, for example, computing the amounts and locations of cut and fill in the grading for a proposed airport runway to be constructed at a given elevation. Another useful application of the contour-area method is in determining the volume of water that will be impounded in the reservoir created by a proposed dam.

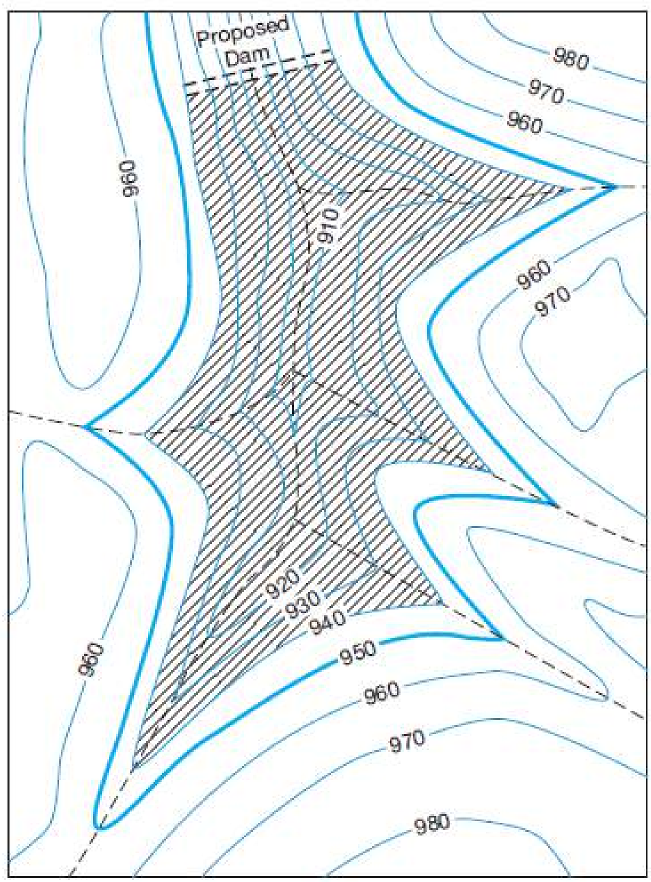

Figure 26.8 Determining the volume of water impounded in a reservoir by the contour-area method.

Example 2

A reservoir is to be formed in a river valley by building a dam across it. The entire area that will be covered by the reservoir has been contoured and contours drawn at 1.5-m intervals. The lowest point in the reservoir is at a reduced level of 249 m above datum, whilst the top water level will not be above a reduced level of 264.5 m. The area enclosed by each contour and the upstream face of the dam is shown in the table below.

Contour (m)	Area enclosed (m^2)
250.0	1 874
251.5	6 355
253.0	11 070
254.5	14 152
256.0	19 310
257.5	22 605
259.0	24 781
260.5	26 349
262.0	29 830
263.5	33 728
265.0	37 800

Estimate by the use of the trapezoidal rule the capacity of the reservoir when full. What will be the reduced level of the water surface if, in a time of drought, this volume is reduced by 25%?

SOLUTION

$$V_1 = \frac{1}{3}\,(1874)(1) = 624.6\,m^3$$

$$V_2 = \left(\frac{1874 + 6355}{2}\right)(1.5) = 6171.75\,m^3$$

$$V_3 = \left(\frac{6355+11070}{2}\right)(1.5) = 13068.75\,m^3$$

$$V_4 = \left(\frac{11070+14152}{2}\right)(1.5) = 18916.5\,m^3$$

$$V_5 = \left(\frac{14152 + 19310}{2}\right)(1.5) = 25096.5\,m^3$$

$$V_6 = \left(\frac{19310 + 22605}{2}\right)(1.5) = 31436.25m^3$$

$$V_{11} = \left(\frac{33728 + 37800}{2}\right)(0.5) = 17882m^3$$

V total =Σ v_i

Problem 4 The areas enclosed by the contours in a lake are as follows:

Contour (m)	270	275	280	285	290
Area (m^2)	2,050	8,400	16,300	24,600	31,500

Calculate the volume of water between the contours 270 m and 290 m by: (i) the trapezoidal formula, and (ii) the prismoidal formula.

Solution (a) Volume according to trapezoidal formula

$$= \frac{5}{2}\{2,050 + 31,500 + 2(8,400 + 16,300 + 24,600)\}$$

$$= 330,375 \text{ m}^3$$

(b) Volume by prismoidal formula:

$$= \frac{5}{3}\{2,050 + 31,500 + 4(8,400 + 24,600) + 2(16,300)\}$$

$$= 330,250 \text{ m}^3$$

Problem 5 An excavation is to be made for a reservoir 40 m long and 30 m wide at the bottom. The side slope of the excavation has to be 2 : 1. Calculate the volume of earth work if the depth of excavation is 5 m. Assume level ground at the site.

Solution

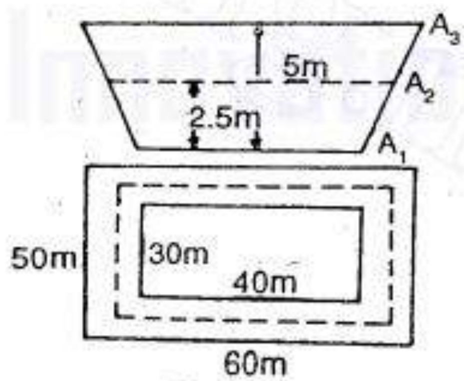

Fig. P.8.2

Bottom section: $L = 40$ m $B = 30$ m

$\therefore$ Area $A_1 = 40 \times 30 = 1{,}200$ m^2

Mid-section: $L = b + 2sh = 40 + 2 \times 2 \times 2.5 = 50$ m

$B = 30 + 2 \times 2 \times 2.5 = 40$ m

$\therefore$ Area $A_2 = 50 \times 40 = 2{,}000$ m^2

Top section: $L = 40 + 2 \times 5 = 60$ m

$B = 30 \times 2 \times 2 \times 5 = 50$ m

$\therefore$ Area $A_3 = 60 \times 50 = 3{,}000$ m

Volume according to prismoidal formula $= \frac{2.5}{3}\{1{,}200 + 3{,}000 + 4\,(2{,}000)\}$

$= 10{,}166.66$ m^3

H.W

Problem 6 The formation width of a certain cutting is 8 m and the side slope is 1 : 1. The surface of the ground has a uniform slope of 1 in 10. If the depths of cutting at the centres of three sections 40 m apart are 2, 3 and 4 m respectively, find the volume of earth work.

Problem 7 Calculate the volume of the earth work for a road having the following data:

Formation width = 10 m

Side lope = 1 : 1

Chainage (m)	Depth of cutting	Transverse slope
0	1.00	1 in 10
50	2.00	1 in 5
100	1.50	1 in 8

Problem 8 Data for the three-level section of a road are as follows:

Station	Left	Centre	Right
1	$\frac{+0.95}{5.25}$	$\frac{+1.00}{0}$	$\frac{+2.55}{7.50}$
2	$\frac{+1.35}{4.75}$	$\frac{+1.50}{0}$	$\frac{+2.80}{8.10}$

The width of cutting at formation level is 9 m, and the side slope is 1 : 1. The stations are 50 m apart. Calculate the volume of cutting.

Problem 10 The following notes are given for a multilevel section of a road of formation width 6 m and side slope 1 : 1. The stations are taken at 50 m intervals.

Station	Left		Centre	Right	
1	$\frac{+2.20}{5.50}$	$\frac{+1.75}{3.00}$	$\frac{+1.50}{0}$	$\frac{+4.75}{5.25}$	$\frac{+6.40}{7.30}$
2	$\frac{+3.10}{5.25}$	$\frac{+2.20}{3.00}$	$\frac{+2.00}{0}$	$\frac{+5.25}{6.00}$	$\frac{+7.40}{8.50}$

Calculate the volume of earth work.

Surveying 2

By Dr. Khamis Naba Sayl

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CURVES

Example 2: The straight lines ABI and CDI are tangents to a proposed circular curve of radius 1600 m. The lengths AB and CD are each 1200 m. The intersection point is inaccessible so that it is not possible directly to measure the deflection angle; but the angles at B and D are measured as:

ABD = $123^{\circ}\ 48'$, BD C= $126^{\circ}\ 12'$, and the length BD is 1485 m.

Calculate the distances from A and C of the tangent points on their respective straights and calculate the deflection angles for setting out 30-m chords from one of the tangent points.

Referring to *Figure 10.19*:

$$\Delta_1 = 180^{\circ} - 123^{\circ}\,48' = 56^{\circ}\,12', \quad \Delta_2 = 180^{\circ} - 126^{\circ}\,12' = 53^{\circ}\,48'$$

$$\therefore \Delta = \Delta_1 + \Delta_2 = 110^{\circ}$$

$$\phi = 180^{\circ} - \Delta = 70^{\circ}$$

Tangent lengths IT_1 and $IT_2 = R \tan \Delta/2 = 1600 \tan 55^{\circ} = 2285$ m

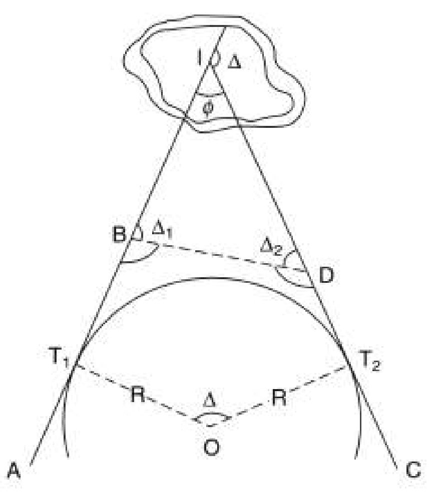

Fig. 10.19 *Inaccessible intersection point*

By sine rule in triangle *BID*:

$$BI = \frac{BD \sin \Delta_2}{\sin \phi} = \frac{1484 \sin 53^\circ 48'}{\sin 70^\circ} = 1275.2 \text{ m}$$

$$ID = \frac{BD \sin \Delta_1}{\sin \phi} = \frac{1485 \sin 56^\circ 15'}{\sin 70^\circ} = 1314 \text{ m}$$

Thus: $AI = AB + BI = 1200 + 1275.2 = 2475.2$ m

$CI = CD + ID = 1200 + 1314 = 2514$ m

$\therefore AT_1 = AI - IT_1 = 2475.2 - 2285 = 190.2$ m

$CT_2 = CI - IT_2 = 2514 - 2285 = 229$ m

Deflection angle for 30-m chord $= 28.6479 \times 30/1600 = 0.537148^\circ$

$= 0^\circ 32' 14''$

Example 10.3 A circular curve of 800 m radius has been set out connecting two straights with a deflection angle of 42°. It is decided, for construction reasons, that the mid-point of the curve must be moved 4 m towards the centre, i.e. away from the intersection point. The alignment of the straights is to remain unaltered.

Calculate:

(1) The radius of the new curve.
(2) The distances from the intersection point to the new tangent points.
(3) The deflection angles required for setting out 30-m chords of the new curve.
(4) The length of the final sub-chord. (LU)

Referring to *Figure 10.20*:

$$IA = R_1(\sec \Delta/2 - 1) = 800(\sec 21^\circ - 1) = 56.92 \text{ m}$$

$$\therefore IB = IA + 4 \text{ m} = 60.92 \text{ m}$$

(1) Thus, $60.92 = R_2(\sec 21^\circ - 1)$, from which $R_2 = 856$ m
(2) Tangent length $= IT_1 = R_2 \tan \Delta/2 = 856 \tan 21^\circ = 328.6$ m

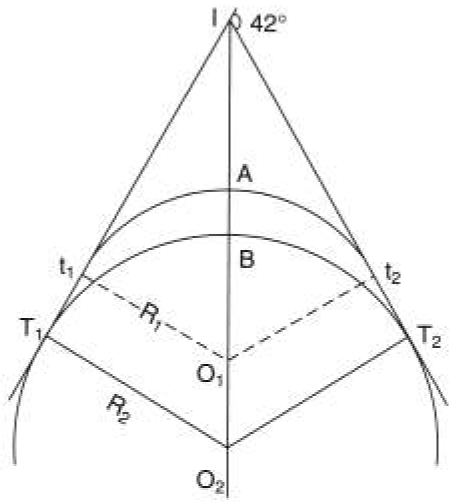

Fig. 10.20 *A realigned road*

(3) Deflection angle for 30-m chord $= 28.6479 \cdot C/R = 28.6479 \cdot \dfrac{30}{856} = 1^\circ\,00'\,14''$

(4) Curve length $= R\Delta$ rad $= \dfrac{856 \times 42^\circ \times 3600}{206\,265} = 627.5$ m

$\therefore$ Length of final sub-chord = 27.5 m

10.11 VERTICAL CURVES

1. Definition When two different gradients meet at a point along a road surface, they form a sharp point at the apex. Unless this apex point is rounded off to form a smooth curve, no vehicle can move along that portion of the road. So, for the smooth and safe running of vehicles, the meeting point of the gradients is rounded off to form a smooth curve in a vertical plane. This curve is known as a vertical curve.

Generally, the parabolic curves are preferred as it is easy to work out the minimum sight distance in their case, and the minimum sight distance is an important factor to be considered while calculating the length of the vertical curve.

2. Gradient The gradient is expressed in two ways:

(a) As a percentage, e.g. 1%, 1.5%, etc.

(b) As 1 in *n*, where *n* is the horizontal distance and 1 represents vertical distance, e.g. 1 in 100, 1 in 200, etc.

Again, the gradient may be 'rise' or 'fall'. An up gradient is known as 'rise' and is denoted by a positive sign. A down gradient is known as 'fall' and is indicated by a negative sign.

3. Rate of change of grade The characteristic of a parabolic curve is that the gradient changes from point to point but the rate of change of grade remains constant. Hence, for finding the length of the vertical curve, the rate of change of grade should be an important consideration as this factor remains constant throughout the length of the vertical curve.

Generally, the recommended rate of change of grade is 0.1% per 30 m at summits and 0.05% per 30 m at sags.

4. Length of vertical curve The length of the vertical curve is calculated by considering the sight distance. To provide minimum sight distance, a certain permissible rate of change of grade is determined and the length of the vertical curve is calculated as follows:

$$\text{Length of vertical curve} = \frac{\text{change of grade}}{\text{rate of change of grade}}$$

$$= \frac{\text{algebraic difference of grades}}{\text{rate of change of grade}} = \frac{g_1 - g_2}{r}$$

where, g_1 and g_2 = percentage of grade and r = rate of change of grade

Example Find the length of vertical curve connecting two grades + 0.5% and − 0.4% where rate of change of grade is 0.1%.

Solution

$$\text{Length of vertical curve} = \frac{0.5 - (-0.4) \times 30}{0.1}$$

$$= \frac{(0.5 + 0.4) \times 30 \times 10}{1}$$

$$= 0.9 \times 30 \times 10 = 270 \text{ m}$$

5. Types of vertical curves The following are the different types of vertical curves that may occur.

(a) Summit Curve Figure 10.36(a) shows a summit curve where an up gradient is followed by a down gradient.

Figure 10.36(b) shows a summit curve where a down gradient is followed by another down gradient.

(b) Sag Curve Figure 10.36(c) shows a sag curve where a down gradient is followed by an up gradient.

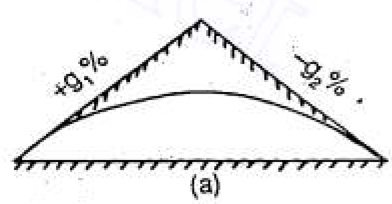

Fig. 10.36(a)

Thus, $$y_1 = \frac{(g_1 - g_2)}{400 \times l} \times x_1^2$$

$$y_2 = \frac{(g_1 - g_2)}{400 \times l} \times x_2^2 \qquad \text{and so on}$$

where, $x_1, x_2 \ldots$ = distances taken along the slope measured from tangent point

l = half-length of vertical curve

g_1 and g_2 = percentages of grade

Figure 10.36(d) shows a sag curve where an up gradient is followed by another up gradient.

Fig. 10.36(b)

6. Setting out vertical curve The vertical curve may be set out by the following two methods:

(a) The tangent correction method
(b) The chord gradient method

The tangent correction method is preferred in practical situations, as it involves simple calculations and curve setting.

Fig. 10.36(c)

(a) Tangent Correction Method In Fig. 10.37, the tangent correction or tangent offset is the difference of elevation between points P and P_1, P being a point on the curve, P_1 a point on the gradient.

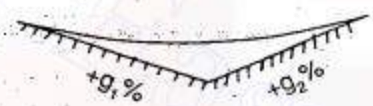

Fig. 10.36(d)

Then

y = RL = P_1 − RL of P = tangent correction

Let x be the horizontal distance of point P from the origin. x_1 is the sloping distance along the gradient of the point P_1. Here, x is taken to be approximately equal to x_1.

The equation of the curve is

$$y = Cx^2$$

where, C = constant = $\dfrac{g_1 - g_2}{400 \times l}$

l = half-length of vertical curve

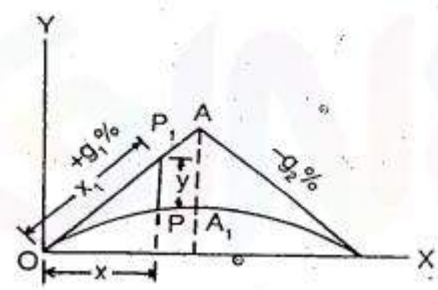

Fig. 10.37

Tangent correction at any point,

$$y = \frac{(g_1 - g_2) \times x_1^2}{400 \times l} \qquad (x = x_1)$$

7. Points to be remembered while calculating data required for setting out vertical curve

(a) The length of the vertical curve is assumed equal to the length of two tangents.

That is, $BT_1 + BT_2 = T_1B_1 + B_1T_2$
$= 2l$ (l = half length of vertical curve)

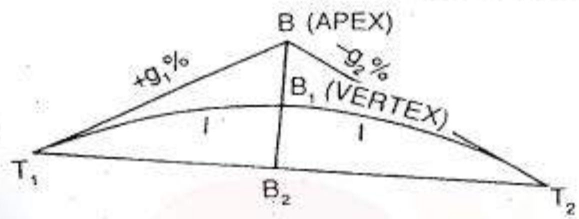

Fig. 10.38

(b) The curve is assumed to be equally long on either side of the apex point.

That is, $T_1B_1 = B_1T_2 = l$ so, $BT_1 = BT_2 = l$

(c) The length of the vertical curve is given by the formula:

$$L = \frac{g_1 - g_2}{r} \quad r \text{ being the rate of change of grade.}$$

(d) Chainage of T_1 = chainage of B − BT_1

(e) Chainage of T_2 = chainage of B + BT_2

(f) RL of T_1 = RL of B $\pm\, l \times \frac{g_1}{100}$

(g) RL of T_2 = RL of B $\pm\, l \times \frac{g_2}{100}$

(h) RL of B_2 $= \frac{1}{2}$ (RL of T_1 + RL of T_2)

(i) RL of B_1 $= \frac{1}{2}$ (RL of B + RL of B_2)

(j) Tangent correction at distance x,

$$y_x = \frac{g_1 - g_2}{400 \times l} \times x^2$$

(k) The tangent correction is deducted from the RL of a point on the grade to get the corresponding point on the curve.

(l) A setting out table is prepared.

(m) Since the curve is symmetrical, tangent corrections are calculated for one side of the point of intersection. The tangent corrections for the other side will be exactly the same.

Example Calculate the RL of the various station pegs on a vertical curve connecting two grades of + 0.6% and − 0.6%. The chainage and the RL of intersection point are 550 and 325.50 m respectively. The rate of change of grade is 0.1% per 30 m.

l being half the curve length.

Tangent correction at point 1,

$$y_1 = \frac{0.6 - (-0.6)}{400 \times 180} \times (30)^2 = \frac{1.2 \times (30)^2}{400 \times 180} = 0.015 \text{ m}$$

Tangent correction at point 2, $\quad y_2 = \frac{1.2 \times (60)^2}{400 \times 180} = 0.060 \text{ m}$

Tangent correction at point 3, $\quad y_3 = \frac{1.2 \times (90)^2}{400 \times 180} = 0.135 \text{ m}$

Tangent correction at point 4, $\quad y_4 = \frac{1.2 \times (120)^2}{400 \times 180} = 0.240 \text{ m}$

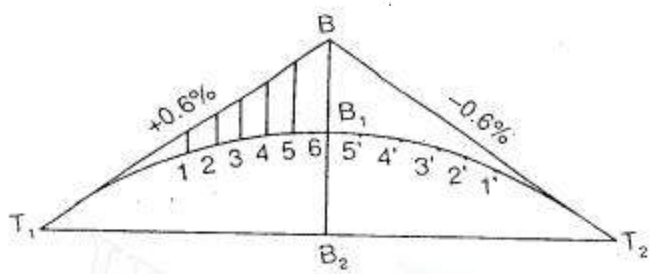

Fig. E. 10.5

Solution

(a) Length of vertical curve,

$$L = \frac{+0.6 - (0.6)}{0.1} \times 30 = \frac{1.2}{0.1} \times 30 = 360 \text{ m}$$

length of curve on either side of apex is taken as 180 m.

(b) Chainage of $T_1 = 550 - 180 = 370$ m

(c) Chainage of $T_2 = 550 + 180 = 730$ m

(d) RL of $T_1 = 325.50 - \frac{0.6 \times 180}{100} = 324.42$ m

(e) RL of $T_2 = 325.50 - \frac{0.6 \times 180}{100} = 324.42$ m

(f) RL of $B_2 = \frac{1}{2}(324.42 + 324.42) = 324.42$ m

(g) RL of $B_1 = \frac{1}{2}(325.50 + 324.42) = 324.96$ m (vertex)

(h) Tangent correction at the centre = 325.50 − 324.96 = 0.54 m

(i) Tangent corrections are found out at 30 m intervals from the relation:

$$y = \frac{g_1 - g_2}{400 \times l} \times x^2$$

l being half the curve length.

Tangent correction at point 1,

$$y_1 = \frac{0.6 - (-0.6)}{400 \times 180} \times (30)^2 = \frac{1.2 \times (30)^2}{400 \times 180} = 0.015 \text{ m}$$

Tangent correction at point 2, $y_2 = \frac{1.2 \times (60)^2}{400 \times 180} = 0.060$ m

Tangent correction at point 3, $y_3 = \frac{1.2 \times (90)^2}{400 \times 180} = 0.135$ m

Tangent correction at point 4, $y_4 = \frac{1.2 \times (120)^2}{400 \times 180} = 0.240$ m

Tangent correction at point 5, $y_5 = \frac{1.2 \times (150)^2}{400 \times 180} = 0.375$ **m**

Check: **Tangent correction at point 6,** $y_6 = \frac{1.2 \times (180)^2}{400 \times 180} = 0.540$ **m** (checked)

(j) **Reduced levels on grade:**

$$\text{Rise per 30 m} = \frac{0.6 \times 30}{100} = 0.18 \text{ m}$$

RL of point 1 = RL of T_1 + 0.18 m
= 324.42 + 0.18 = 324.60 m

RL of point 2 = 324.60 + 0.18 = 324.78 m
RL of point 3 = 324.78 + 0.18 = 324.96 m
RL of point 4 = 324.96 + 0.18 = 325.14 m
RL of point 5 = 325.14 + 0.18 = 325.32 m
RL of point 6 = 325.32 + 0.18 = 325.50 m (RL of B) (checked)

(k) Reduced level on the curve:

RL of point 1 = 324.60 − 0.015 = 324.585
RL of point 2 = 324.78 − 0.060 = 324.720
RL of point 3 = 324.96 − 0.135 = 324.825
RL of point 4 = 325.14 − 0.240 = 324.900
RL of point 5 = 325.32 − 0.375 = 324.945
RL of point 6 = 325.50 − 0.540 = 324.960 (RL of B_1) (checked)

Setting out table

Setting out table

Point	Chainage	Grade RL	Tangent correction (−ve)	Curve RL	Remark
T_1	370	324.42	0	324.42	Starting of curve
1	400	324.60	0.015	324.585	
2	430	324.78	0.060	324.720	
3	460	324.96	0.135	324.825	
4	490	325.14	0.240	324.900	
5	520	325.32	0.375	324.945	
6	550	325.50	0.540	324.960	Vertex of curve
5′	580	325.32	0.375	324.945	
4′	610	325.14	0.240	324.900	
3′	640	324.96	0.135	324.825	
2′	670	324.76	0.060	324.720	
1′	700	324.60	0.015	324.585	
T_2	730	324.42	0	324.42	Finishing point of curve

Surveying 2

By Dr. Khamis Naba Sayl

المحاضرة السادسة

3- Rankin's Method or Deflection Angle Method

Tangential angle $\delta_n = 1718.9\frac{c_n}{R}$ minutes

Deflection angles $\Delta_n = \Delta_{n-1} + \delta\_n$

Example

The centre-line of two straights is projected forward to meet at I, the deflection angle being 30◦. If the straights are to be connected by a circular curve of radius 200 m, tabulate all the setting-out data, assuming 20-m chords on a through chainage basis, the chainage of I being 2259.59 m.

Solution

$$\text{Tangent length } T = R\tan\frac{\Delta}{2}$$

=200 tan 15= 53.59 m

Chainge of T1 = 2259.59 − 53.59 = 2206 m

∴ 1st sub-chord = 14m

Length of circular arc = RΔ = 200(30◦ · π/180) m = 104.72 m

From which the number of chords may now be deduced i.e.,

1st sub-chord = 14 m

2nd, 3rd, 4th, 5th chords = 20 m each

Final sub-chord = 10.72 m

Total = 104.72 m (Check)

∴ Chainage of T2 = 2206 m + 104.72 m = 2310.72 m

Deflection angles:

δ_1=1718.9 (14/200) minutes= 2˚ 00′ 19″

Standard chord = 1718.9 (20/200) minutes = 2˚ 51′ 53″

Final sub-chord= 1718.9 (10.72/200) minutes = 1˚ 32′ 08″

Check: The sum of the deflection angles =
$\Delta/2 = 14^\circ\ 59'\ 59'' \approx 15^\circ$

Chord number	*Chord length*	*Chainage*	*Deflection angle*			*Setting-out angle*			*Remarks*
	(m)	(m)	°	′	″	°	′	″	
1	14	2220.00	2	00	19	2	00	19	peg 1
2	20	2240.00	2	51	53	4	52	12	peg 2
3	20	2260.00	2	51	53	7	44	05	peg 3
4	20	2280.00	2	51	53	10	35	58	peg 4
5	20	2300.00	2	51	53	13	27	51	peg 5
6	10.72	2310.72	1	32	08	14	59	59	peg 6

The error of 1″ is, in this case, due to the rounding-off of the angles to the nearest second and is negligible.

Example Two tangents intersect at chainage 1,250 m. The angle of intersection is 150°. Calculate all data necessary for setting out a curve of radius 250 m by the deflection angle method. The peg intervals may be taken as 20 m. Prepare a setting out table when the least count of the vernier is 20″. Calculate the data for field checking.

Solution Given data:

Radius = 250 m

Deflection angle $\phi = 180° - 150° = 30°$

Chainage of intersection point = 1,250 m

Peg interval = 20 m

LC of vernier = 20″

1. Tangent length = $R \tan \phi/2$
$= 250 \times \tan 15° = 67.0$ m
2. Curve length $= \dfrac{\pi R \phi°}{180°} = \dfrac{\pi \times 250 \times 30°}{180°} = 130.89$ m
3. Chainage of first TP, $T_1 = 1{,}250.0 - 67.0 = 1{,}183.0$ m
4. Chainage of second TP, $T_2 = 1{,}183.0 + 130.89 = 1{,}313.89$ m
5. Length of initial sub-chord = 1,190.0 − 1,183.0 = 7.0 m
6. No. of full chords (20 m) = 6

   Chainage covered = 1,190.0 + (6 × 20) = 1,310.00 m
7. Length of final sub-chord = 1,313.89 − 1,310.00 = 3.89 m
8. Deflection angle for initial sub-chord,
$$\delta_1 = \frac{1{,}718.9 \times 7.0}{250} \text{ mins} = 0°48'8''$$
9. Deflection angle for full chord,
$$\delta = \frac{1{,}718.9 \times 20}{250} \text{ mins} = 2°17'31''$$
10. Deflection angle for final sub-chord,
$$\delta_n = \frac{1{,}718.9 \times 3.89}{250} = 0°26'45''$$
11. Arithmetical check:
$$\text{Total deflection angle } (\Delta_n) = \delta_1 + 6 \times \delta + \delta_n$$
$$\phi/2 = \frac{30°}{2} = 15°$$
Here,
$$\Delta_n = 0°48'8'' + 6 \times 2°17'31'' + 0°26'45'' = 14°59'59''$$
$$= 15° \text{ (approximately)}$$
So, the calculated deflection angles are correct.
12. Data for field check:

   (a) Apex distance = $R (\sec \phi/2 - 1)$
   $= 250 (\sec 15° - 1) = 8.82$ m

   (b) Versed sine of curve = $R (1 - \cos \phi/2)$
   $= 250 (1 - \cos 15°) = 8.52$ m

Point	Chainage	Chord length	Deflection angle for chord	Total deflection angle (Δ)	Angle to be set	Remark
T_1	1,183.0	—	—	—	—	Starting point of curve
P_1	1,190.0	7.0	0°48′8″	0°48′8″	0°48′0″	LC of vernier = 20″
P_2	1,210.0	20.0	2°17′31″	3°5′39″	3°5′40″	
P_3	1,230.0	20.0	2°17′31″	5°23′10″	5°23′0″	
P_4	1,250.0	20.0	2°17′31″	7°40′41″	7°40′40″	
P_5	1,270.0	20.0	2°17′31″	9°58′12″	9°58′0″	
P_6	1,290.0	20.0	2°17′31″	12°15′43″	12°15′40″	
P_7	1,310.0	20.0	2°17′31″	14°33′14″	14°33′20″	
T_2	1,313.89	3.89	0°26′45″	14°59′59″	15°0′0″	Finishing point of curve.

H.W

Assume that $\Delta = 8^\circ\ 24'$ the station of the PI is 64 + 27.46 and terrain conditions require the minimum radius permitted by the specifications of, say, 2864.79 m (arc definition). Calculate the PC and PT stationing and the external and middle ordinate distances for this curve.

H.W

Assume that a metric curve will be used at a PI where$\Delta = 8^\circ\ 24'$ Assume also that the station of the PI is 6+427.464 and that terrain conditions require a minimum radius of 900 m. Calculate the PC and PT stationing, and other defining elements of the curve. Also compute notes for staking the curve using 20-m increments.

Field procedure of setting out curve (by deflection angles) by one-theodolite method

1. In Fig. 10.19, AB and BC are two tangents intersecting at B. The tangent length and curve lengths are calculated, and the points T_1 and T_2 are fixed.

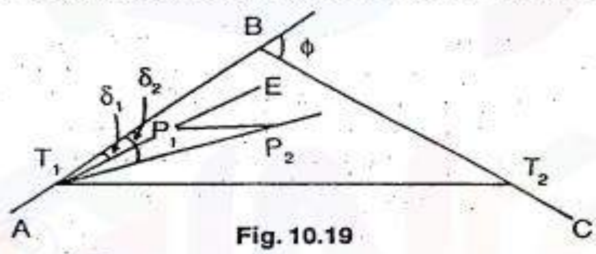

Fig. 10.19

2. The lengths of the initial and final sub-chords, and the number of full chords are ascertained.

3. The deflection angles for the chords are calculated and verified by arithmetical check.

4. A setting out table is prepared, depending on the least count of the theodolite. For setting the curve, only the angles from the "angles to be set" column should be taken.

5. The theodolite is centred over T_1 and properly levelled. Then vernier A is set to 0° of the main scale. The upper clamp is fixed.

6. The lower clamp is released and the ranging rod at the intersection point B is perfectly bisected with the help of the lower tangent screw. The lower clamp is now tightened.

7. The upper clamp is released and the first deflection angle (δ_1) is set on vernier A. The telescope is directed along the line T_1E.

8. Now, the zero end of the tape is held at T_1 and the distance T_1P_1 is measured equal to the length of the initial sub-chord in such a way that the ranging rod at P_1 is also bisected by the telescope. Then the telescope is lowered to mark the

base of ranging rod perfectly. So, P_1 is a point on the curve which is marked by a nail or arrow.

9. The next deflection angle (δ_2) is set on vernier A and the point P_2 is so marked that P_1P_2 is equal to the length of a full chord, and the ranging rod at P_2 is perfectly bisected by the telescope. So, P_2 is the next point on the curve.

10. This process is continued until all the deflection angles are set out and all the points on the curve are marked. Finally, the last point should coincide with T_2.

If it does not, the amount of error is found out. If this error is small, it is distributed among the last few pegs.

If the error is large, the entire operation should be repeated. Finally, all the points p_1, p_2, p_3, ... are marked by stout pegs.

Procedure for Setting Deflection Angles 1. The theodolite is centred and levelled at the first tangent point and the lower clamp is fixed. The upper clamp is loosened and vernier A is set approximately to the zero of the main scale. After that, the upper clamp is tightened and by turning the upper tangent screw the arrow of vernier A is brought into exact coincidence with the zero of the main scale.

2. Now, the lower clamp is loosened and the ranging rod at the intersection point is perfectly bisected with the help of the lower tangent screw. Then both the clamps are tightened.

3. Suppose the deflection angle 0°48′20″ is to be set. By turning the upper tangent screw very slowly, the arrow of vernier A is made to cross two small divisions (i.e. 40′) of the main scale. Then, looking through the divisions of the vernier scale carefully, the first small division after eight big divisions (i.e. 8′20″) of the vernier scale is made to coincide with any division of the main scale.

Thus, Deflection angle = 0°40′0″ + 0°8′20″
= 0°48′20″

4. Similarly, by turning the upper tangent screw very slowly, subsequent deflection angles are set out one by one according to the entries in the "angles to be set" column of the setting out table.

10.8 COMPOUND CURVE—CALCULATION OF DATA AND SETTING OUT

When it is not possible to connect the two tangents by one circular curve, it becomes necessary to take a suitable common tangent, and set out two curves of different radii to connect the rear and forward tangents. This curve is known as a compound curve (Fig. 10.21(a)).

Notation

AB = rear tangent
BC = forward tangent
DE = common tangent
ϕ = deflection angle between rear and forward tangent
ϕ_1 = deflection angle between rear and common tangent
ϕ_2 = deflection angle between common and forward tangent.
O_1 = centre of short curve
O_2 = centre of long curve
R_s = radius of short curve
R_L = radius of long curve
T_1 and T_2 = tangent points for short curve
T_2 and T_3 = tangent points for long curve

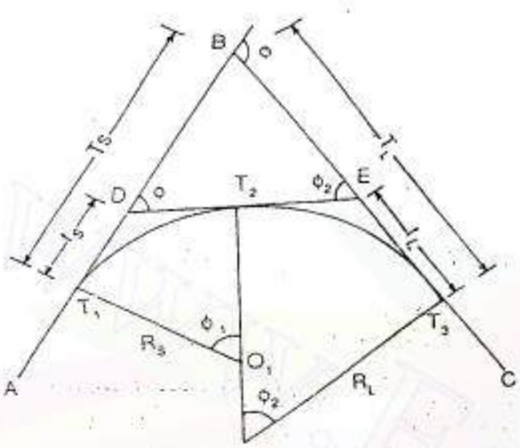

Fig. 10.21(a)

T_s = total tangent length of shortest side ($\mathbf{BT_1}$)
T_L = total tangent length of longer side ($\mathbf{BT_3}$)
t_s = tangent length of short curve
t_L = tangent length of long curve

Calculation of data

1. $\phi = \phi_1 + \phi_2$
2. $T_s = BD + DT_1 = BD + t_s$

$$= DE \times \frac{\sin \phi_2}{\sin \phi} + R_s \tan \frac{\phi_1}{2}$$

3. $T_L = BE + ET_3 = BE + t_L$

$$= DE \times \frac{\sin \phi_1}{\sin \phi} + R_L \frac{\phi_2}{2}$$

4. Common tangent, $DE = t_s + t_L = R_s \tan\frac{\phi_1}{2} + R_L \tan\frac{\phi_2}{2}$

where $t_s = R_s \tan\frac{\phi_1}{2}$ $\quad t_L = R_L \tan\frac{\phi_2}{2}$

From ΔBDE,

$$\frac{BD}{\sin\phi_2} = \frac{BE}{\sin\phi_1}$$

$$= \frac{DE}{\sin\{180^\circ - (\phi_1 + \phi_2)\}} = \frac{DE}{\sin(180^\circ - \phi)}$$

$\therefore$ $$BD = DE \times \frac{\sin\phi_2}{\sin\phi}$$

and $$BE = DE \times \frac{\sin\phi_1}{\sin\phi}$$

5. Curve length (short curve) $= \frac{\pi R_s \phi_1}{180^\circ}$

Curve length (long curve) $= \frac{\pi R_L \phi_2}{180^\circ}$

6. Deflection angle (short curve), $\delta_s = \frac{1718.9 \times C_s}{R_s}$ mins

where C_s = chord of short curve

Deflection angle (long curve) $\delta_L = \frac{1{,}718.9 \times C_L}{R_L}$ mins

where C_L = chord of long curve

7. Chainage of T_1 = chainage of B – T_s
8. Chainage of T_2 = chainage of T_1 + short curve length
9. Chainage of T_3 = chainage of T_2 + long curve length.

Example Two tangents AB and BC intersect at B. Another line DE intersects AB and BC at D and E such that ∠ADE = 150° and ∠DEC = 140°. The radius of the first curve is 200 m and that of the second is 300 m. The chainage of B is 950 m.

Calculate all data necessary for setting out the compound curve.

Solution Consider Fig. 10.21(b).

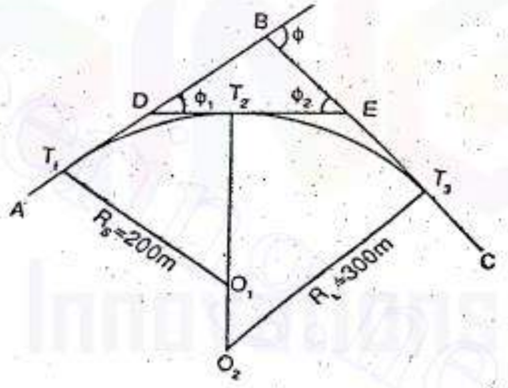

Fig. 10.21 (b)

Given data: $\phi_1 = 180° - 150° = 30°$, $\phi = 30° + 40° = 70°$
$\phi_2 = 180° - 140° = 40°$

1. $T_1D = DT_2 = R_s \tan \frac{\phi_1}{2} = 200 \times \tan 15° = 53.58 \text{ m}$

2. $T_3E = ET_2 = R_L \tan \frac{\phi_2}{2} = 300 \times \tan 20° = 109.19 \text{ m}$

3. $DE = DT_2 + T_2E = 53.58 + 109.19 = 162.77 \text{ m}$

4. From ΔBDE,

$$\frac{DB}{\sin 40°} = \frac{BE}{\sin 30°} = \frac{DE}{\sin 110°}$$

$$DB = DE \times \frac{\sin 40°}{\sin 110°} = 162.77 \times \frac{0.6427}{0.9396} = 111.34 \text{ m}$$

$$BE = DE \times \frac{\sin 30°}{\sin 110°} = 162.77 \times \frac{.0.5}{0.9396} = 86.61 \text{ m}$$

$$BT_1 = BD + DT_1 = 111.34 + 53.58 = 164.92 \text{ m}$$

$$BT_3 = BE + ET_3 = 86.61 + 109.19 = 195.8 \text{ m}$$

5. Chainage of $T_1 = 950 - 164.92 = 785.08$ m

6. Short curve length $= \frac{\pi \times 200 \times 30°}{180°} = 104.72$ m

7. Chainage of $T_2 = 785.08 + 104.72 = 889.80$ m

8. Long curve length $= \frac{\pi \times 300 \times 40°}{180°} = 209.44$ m

9. Chainage of $T_3 = 889.80 + 209.44 = 1{,}099.24$ m

Deflection angle for short curve:
Taking a full chord of 20 m,

$$\text{Number of full chords} = 5\ (5 \times 20 = 100\text{ m})$$

$$\text{Length of final sub-chord} = 104.72 - 100 = 4.72\text{ m}$$

$$\delta \text{ for full chord} = \frac{1{,}718.9 \times 20}{200} = 2°51'53''$$

$$\delta \text{ for final sub-chord} = \frac{1{,}718.9 \times 4.72}{200} = 0°40'34''$$

Check:

$$\text{Total deflection angle} = \frac{\phi_1}{2} = \frac{30°}{2} = 15°$$

$$\text{Calculated angles} = 5 \times 2°51'53'' + 0°40'34''$$

$$= 14°59'59'' = 15° \text{ (say)}$$

Deflection angle for long curve:
Taking a full chord of 30 m,

$$\text{Number of full chords} = 6\ (6 \times 30 = 180\text{ m})$$

$$\text{Length of final sub-chord} = 209.44 - 180.00 = 29.44\text{ m}$$

$$\delta \text{ for full chord} = \frac{1{,}718.9 \times 30}{300} = 2°51'53''$$

$$\delta \text{ for final sub-chord} = \frac{1{,}718.9 \times 29.44}{300} = 2°48'41''$$

Check:

$$\text{Total deflection} = \frac{\phi_2}{2} = \frac{40°}{2} = 20°$$

$$\text{Calculated angles} = 6 \times 2°51'53'' + 2°48'41''$$

$$= 19°59'59'' = 20° \text{ (say)}$$

10.9 REVERSE CURVE—CALCULATION OF DATA AND SETTING OUT

A reverse curve consists of two circular arcs of equal or different radii turning in opposite directions with a common tangent at the junction of the arcs. The junction point is said to have reverse curvature. The reverse curve is also known as a serpentine curve.

Reverse curves are generally used to connect two parallel roads or railway lines, or when two lines intersect at a very small angle.

These curves are suitable for railway sidings, city roads, etc. But they should be avoided as far as possible for important tracks or highways for the following reasons:

1. Superelevation cannot be provided at the point of reverse curvature.
2. A sudden change of direction would be dangerous for a vehicle.
3. A sudden change of cant causes discomfort to passengers.
4. Carelessness of the driver may cause the vehicle to overturn over a reverse curve.

Reverse curves are generally short, and hence they are set out by the chain and tape method.

Notation

1. In Fig. 10.23, AB and EF the straight lines, BE is the common tangent and C is the point of reverse curvature.

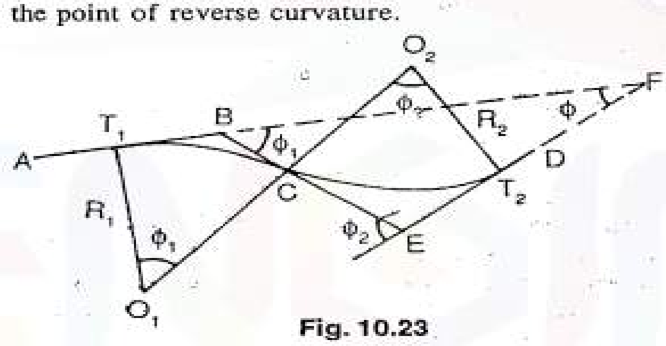

Fig. 10.23

2. T_1 and T_2 are the tangent points.
3. ϕ is the angle of intersection between the straight lines.
4. ϕ_1 and ϕ_2 are the deflection angles of the common tangent.
5. R_1 and R_2 are the radii of the arcs.

Reverse curves may involve various cases. Here we shall illustrate two of them.

Case I—When the straights are non-parallel Suppose AB, BC and CD are lines of an open traverse along the alignment of a road (Fig. 10.24). AB and CD when produced meet at a point E, where ϕ is the angle of intersection. It is required to connect the lines AB and CD by a reverse curve with BC as the common tangent.

Let ϕ_1 = angle of deflection for the first arc

ϕ_2 = angle of deflection for the second arc

ϕ = angle of intersection between AB and CD

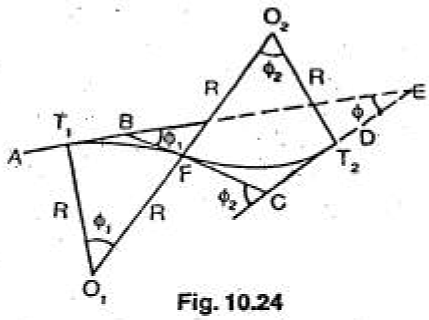

Fig. 10.24

T_1 and T_2 = tangent points
F = point of reverse curve
R = common radius for the arcs

The following data have to be calculated for setting out the curve.

1. Tangent length of first arc, $T_1B = BF = R \tan \frac{\phi_1}{2}$
2. Tangent length of second arc, $T_2C = CF = R \tan \frac{\phi_2}{2}$
3. Length of common tangent, $BC = BF + CF$
$$= R \tan \frac{\phi_1}{2} + R \tan \frac{\phi_2}{2}$$
4. Length of first curve, $T_1F = \frac{\pi R \phi_1}{180°}$
5. Length of second curve, $T_2F = \frac{\pi R \phi_2}{180°}$
6. Chainage of T_1 = chainage of B − T_1 B.
7. Chainage of F = chainage of T_1 + 1st curve length
8. Chainage of T_2 = chainage of F + 2nd curve length

The length of the reverse curve is normally small. So, the curve may be set out by taking offsets from (i) the long chord, or (ii) the chord produced. (Both these methods have already been described.

If the length of the curve becomes large and chaining along it difficult, the curve may be set out by the deflection-angle method (Rankine's method). (This method has also been described previously.)

Surveying 2

By Dr. Khamis Naba Sayl

المحاضرة الثانية عشر

REMOTE SENSING

REMOTE SENSING

Remote sensing (RS), also called earth observation, refers to obtaining information about objects or areas at the Earth's surface without being in direct contact with the object or area. Humans accomplish this task with aid of eyes or by the sense of smell or hearing; so, remote sensing is day-today business for people. Reading the newspaper, watching cars driving in front of you are all remote sensing activities. Most sensing devices record information about an object by measuring an object's transmission of electromagnetic energy from reflecting and radiating surfaces.

Remote sensing techniques allow taking images of the earth surface in various wavelength region of the electromagnetic spectrum (EMS). One of the major characteristics of a remotely sensed image is the wavelength region it represents in the EMS. Some of the images represent reflected solar radiation in the visible and the near infrared regions of the electromagnetic spectrum, others are the measurements of the energy emitted by the earth surface itself i.e. in the thermal infrared wavelength region. The energy measured in the microwave region is the measure of relative return from the earth's surface, where the energy is transmitted from the vehicle itself. This is known as active remote sensing, since the energy source is provided by the remote sensing platform. Whereas the systems where the remote sensing measurements depend upon the external energy source, such as sun are referred to as passive remote sensing systems.

PRINCIPLES OF REMOTE SENSING

Detection and discrimination of objects or surface features means detecting and recording of radiant energy reflected or emitted by objects or surface material (Fig. 1). Different objects return different amount of energy in different bands of the electromagnetic spectrum, incident upon it. This depends on the property of material (structural, chemical, and physical), surface roughness, angle of incidence, intensity, and wavelength of radiant energy.

The Remote Sensing is basically a multi-disciplinary science which includes a combination of various disciplines such as optics, spectroscopy, photography, computer, electronics and telecommunication, satellite launching etc. All these technologies are integrated to act as one complete system in itself, known as Remote Sensing System. There are a number of stages in a Remote Sensing process, and each of them is important for successful operation.

Stages in Remote Sensing

• Emission of electromagnetic radiation, or EMR (sun/self- emission)

• Transmission of energy from the source to the surface of the earth, as well as absorption and scattering

• Interaction of EMR with the earth's surface: reflection and emission

• Transmission of energy from the surface to the remote sensor

• Sensor data output

• Data transmission, processing and analysis

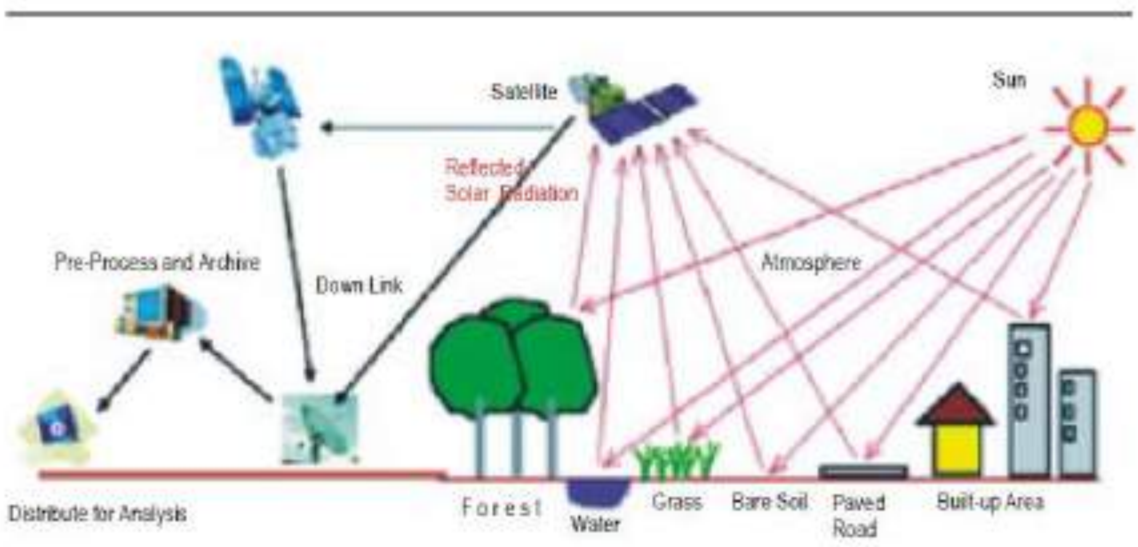

Figure 1: Remote Sensing process

At temperature above absolute zero, all objects radiate electromagnetic energy by virtue of their atomic and molecular oscillations. The total amount of emitted radiation increases with the body's absolute temperature and peaks at progressively shorter wavelengths. The sun, being a major source of energy, radiation and illumination, allows capturing reflected light with conventional (and some not-so-conventional) cameras and films.

The basic strategy for sensing electromagnetic radiation is clear. Everything in nature has its own unique distribution of reflected, emitted and absorbed radiation. These spectral characteristics, if ingeniously exploited, can be used to distinguish one thing from another or to obtain information about shape, size and other physical and chemical properties.

Modern Remote Sensing Technology versus Conventional Aerial Photography

The use of different and extended portions of the electromagnetic spectrum, development in sensor technology, different platforms for remote sensing (spacecraft, in addition to aircraft), emphasize on the use of spectral information as compared to spatial information, advancement in image processing and enhancement techniques, and automated image analysis in addition to manual interpretation are points for comparison of conventional aerial photography with modern remote sensing system.

During early half of twentieth century, aerial photos were used in military surveys and topographical mapping. Main advantage of aerial photos has been the high spatial resolution with fine details and therefore they are still used for mapping at large scale such as in route surveys, town planning, construction project surveying, cadastral mapping etc. Modern remote sensing system provide satellite images suitable for medium scale mapping used in natural resources surveys and monitoring such as forestry, geology, watershed management etc. However the future generation satellites are going to provide much high-resolution images for more versatile applications.

ELECTROMAGNETIC RADIATION AND THE ELECTROMAGNETIC SPECTRUM

EMR is a dynamic form of energy that propagates as wave motion at a velocity of c = 3 x 1010 cm/sec. The parameters that characterize a wave motion are wavelength (λ), frequency (ν) and velocity (c) (Fig. 2). The relationship between the above is

$$c = \nu\lambda.$$

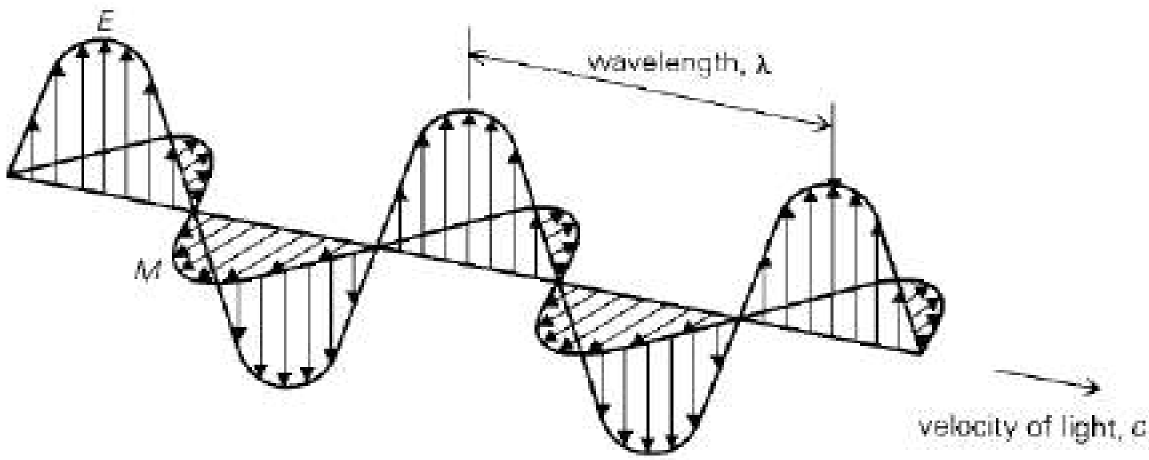

Figure 2: Electromagnetic wave. It has two components, Electric field E and Magnetic field M, both perpendicular to the direction of propagation

Electromagnetic energy radiates in accordance with the basic wave theory. This theory describes the EM energy as travelling in a harmonic sinusoidal fashion at the velocity of light. Although many characteristics of EM energy are easily described by wave theory, another theory known as particle theory offers insight into how electromagnetic energy interacts with matter. It suggests that EMR is composed of many discrete units called photons/quanta. The energy of photon is

$$Q = hc / \lambda = h\,\nu$$

Where

Q is the energy of quantum,

h = Planck's constant

Table 2: Principal Divisions of the Electromagnetic Spectrum

Wavelength	Description
Gamma rays	Gamma rays
X-rays	X-rays
Ultraviolet (UV) region 0.30 µm - 0.38 µm ($1\mu m = 10^{-5}m$)	This region is beyond the violet portion of the visible wavelength, and hence its name. Some earth's surface material primarily rocks and minerals emit visible UV radiation. However UV radiation is largely scattered by earth's atmosphere and hence not used in field of remote sensing.
Visible Spectrum 0.4 µm - 0.7 µm Violet 0.4 µm -0.446 µm Blue 0.446 µm -0.5 µm Green 0.5 µm - 0.578 µm Yellow 0.578 µm - 0.592 µm Orange 0.592 µm - 0.62 µm Red 0.62 µm -0.7 µm	This is the light, which our eyes can detect. This is the only portion of the spectrum that can be associated with the concept of color. Blue Green and Red are the three primary colors of the visible spectrum. They are defined as such because no single primary color can be created from the other two, but all other colors can be formed by combining the three in various proportions. The color of an object is defined by the color of the light it reflects.
Infrared (IR) Spectrum 0.7 µm – 100 µm	Wavelengths longer than the red portion of the visible spectrum are designated as the infrared spectrum. British Astronomer William Herschel discovered this in 1800. The infrared region can be divided into two categories based on their radiation properties. Reflected IR (.7 µm - 3.0 µm) is used for remote sensing. Thermal IR (3 µm - 35 µm) is the radiation emitted from earth's surface in the form of heat and used for remote sensing.
Microwave Region 1 mm - 1 m	This is the longest wavelength used in remote sensing. The shortest wavelengths in this range have properties similar to thermal infrared region. The main advantage of this spectrum is its ability to penetrate through clouds.
Radio Waves (>1 m)	This is the longest portion of the spectrum mostly used for commercial broadcast and meteorology.

Types of Remote Sensing

Remote sensing can be either passive or active. ACTIVE systems have their own source of energy (such as RADAR) whereas the PASSIVE systems depend upon external source of illumination (such as SUN) or self-emission for remote sensing.

INTERACTION OF EMR WITH THE EARTH'S SURFACE

Radiation from the sun, when incident upon the earth's surface, is either reflected by the surface, transmitted into the surface or absorbed and emitted by the surface (Fig. 3). The EMR, on interaction, experiences a number of changes in magnitude, direction, wavelength, polarization and phase. These changes are detected by the remote sensor and enable the interpreter to obtain useful information about the object of interest. The remotely sensed data contain both spatial information (size, shape and orientation) and spectral information (tone, colour and spectral signature).

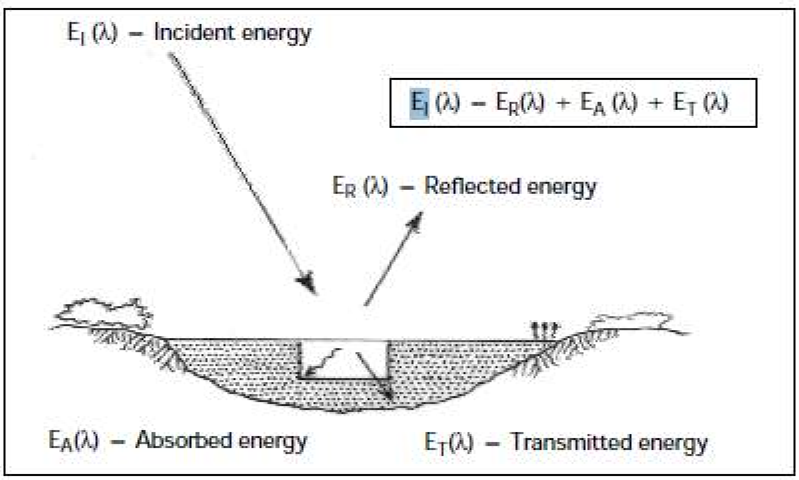

Figure 3: Interaction of Energy with the earth's surface. (source: Lillesand & Kiefer, 1993)

From the viewpoint of interaction mechanisms, with the object-visible and infrared wavelengths from 0.3 μm to 16 μm can be divided into three regions. The spectral band from 0.3 μm to 3 μm is known as the reflective region. In this band, the radiation sensed by the sensor is that due to the sun, reflectedby the earth's surface. The band corresponding to the atmospheric window between 8 μm and 14 μm is known as the thermal infrared band. The energy available in this band for remotesensing is due to thermal emission from the earth's surface. Both reflection and self-emission are important in the intermediate band from 3 µm to 5.5 µm. In the microwave region of the spectrum, the sensor is radar, which is an active sensor, as it provides its own source of EMR. The EMR produced by the radar is transmitted to the earth's surface and the EMR reflected (back scattered) from the surface is recorded and analyzed. The microwave region can also be monitored with passive sensors, called microwave radiometers, which record the radiation emitted by the terrain in the microwave region.

Reflection

Of all the interactions in the reflective region, surface reflections are the most useful and revealing in remote sensing applications. Reflection occurs when a ray of light is redirected as it strikes a non-transparent surface. The reflection intensity depends on the surface refractive index, absorption coefficient and the angles of incidence and reflection (Fig. 4).

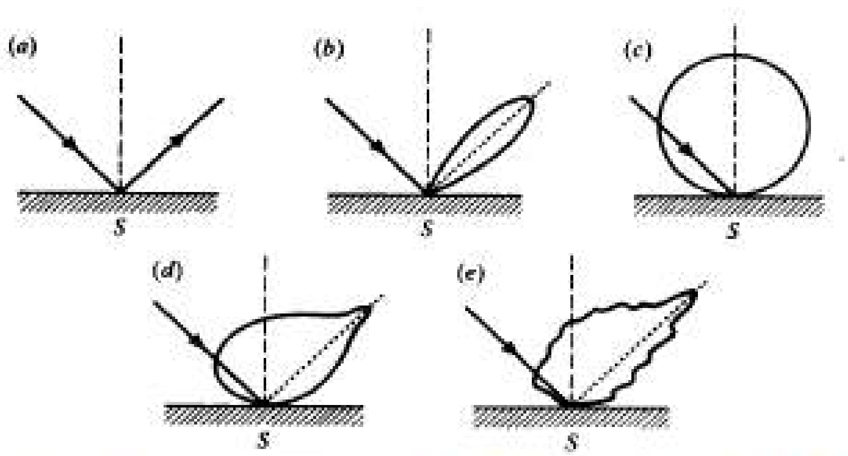

Figure 4. Different types of scattering surfaces (a) Perfect specular reflector (b) Near perfect specular reflector (c) Lambertain (d) Quasi-Lambertian (e) Complex.

Transmission

Transmission of radiation occurs when radiation passes through a substance without significant attenuation. For a given thickness, or depth of a substance, the ability of a medium to transmit energy is measured as transmittance (τ).

$$\tau = \frac{\text{Transmitted radiation}}{\text{Incident radiation}}$$

Spectral Signature

Spectral reflectance, [ρ(λ)], is the ratio of reflected energy to incident energy as a function of wavelength. Various materials of the earth's surface have different spectral reflectance characteristics. Spectral reflectance is responsible for the color or tone in a photographic image of an object. Trees appear green because they reflect more of the green wavelength. The values of the spectral reflectance of objects averaged over different, well-defined wavelength intervals comprise the spectral signature of the objects or features by which they can be distinguished. To obtain the necessary ground truth for the interpretation of multispectral imagery, the spectral characteristics of various natural objects have been extensively measured and recorded. The spectral reflectance is dependent on wavelength, it has different values at different wavelengths for a given terrain feature. The reflectance characteristics of the earth's surface features are expressed by spectral reflectance, which is given by:

$$\rho(\lambda) = [ER(\lambda) / EI(\lambda)] \times 100$$

ρ(λ) = Spectral reflectance (reflectivity) at a particular wavelength.

ER(λ) = Energy of wavelength reflected from object

EI(λ) = Energy of wavelength incident upon the object

The plot between ρ(λ) and λ is called a spectral reflectance curve. This varies with the variation in the chemical composition and physical conditions of the feature, which results in a range of values. The spectral response patterns are averaged to get a generalized form, which is called generalized spectral response pattern for the object concerned. Spectral signature is a term used for unique spectral response pattern, which is characteristic of a terrain feature. Figure 5 shows a typical reflectance curves for three basic types of earth surface features, healthy vegetation, dry bare soil (grey-brown and loamy) and clear lake water.

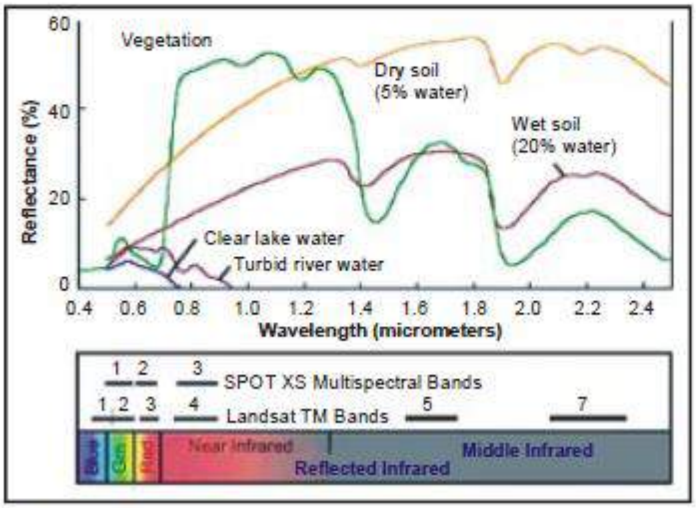

Figure 5. Typical Spectral Reflectance curves for vegetation, soil and water

Reflectance Characteristics of Earth's Cover types

The spectral characteristics of the three main earth surface features are discussed below :

Vegetation: The spectral characteristics of vegetation vary with wavelength. Plant pigment in leaves called chlorophyll strongly absorbs radiation in the red and blue wavelengths but reflects green wavelength. The internal structure of healthy leaves acts as diffuse reflector of near infrared wavelengths. Measuring and monitoring the near infrared reflectance is one way that scientists determine how healthy particular vegetation may be.

Water: Majority of the radiation incident upon water is not reflected but is either absorbed or transmitted. Longer visible wavelengths and near infrared radiation is absorbed more by water than by the visible wavelengths. Thus water looks blue or blue green due to stronger reflectance at these shorter wavelengths and darker if viewed at red or near infrared wavelengths. The factors that affect the variability in reflectance of a water body are depth of water, materials within water and surface roughness of water.

Soil: The majority of radiation incident on a soil surface is either reflected or absorbed and little is transmitted. The characteristics of soil that determineits reflectance properties are its moisture content, organic matter content, texture, structure and iron oxide content. The soil curve shows less peak and valley variations. The presence of moisture in soil decreases its reflectance.

By measuring the energy that is reflected by targets on earth's surface over a variety of different wavelengths, we can build up a spectral signature for that object. And by comparing the response pattern of different features we may be able to distinguish between them, which we may not be able to do if we only compare them at one wavelength. For example, Water and Vegetation reflect somewhat similarly in the visible wavelength but not in the infrared.

INTERACTIONS WITH THE ATMOSPHERE

The sun is the source of radiation, and electromagnetic radiation (EMR) from the sun that is reflected by the earth and detected by the satellite or aircraft-borne sensor must pass through the atmosphere twice, once on its journey from the sun to the earth and second after being reflected by the surface of the earth back to the sensor. Interactions of the direct solar radiation and reflected radiation from the target with the atmospheric constituents interfere with the process of remote sensing and are called as "Atmospheric Effects".

The interaction of EMR with the atmosphere is important to remote sensing for two main reasons. First, information carried by EMR reflected/ emitted by the earth's surface is modified while traversing through the atmosphere. Second, the interaction of EMR with the atmosphere can be used to obtain useful information about the atmosphere itself.

The atmospheric constituents scatter and absorb the radiation modulating the radiation reflected from the target by attenuating it, changing its spatial distribution and introducing into field of view radiation from sunlight scattered in the atmosphere and some of the energy reflected from nearby ground area. Both scattering and absorption vary in their effect from one part of the spectrum to the other.

The solar energy is subjected to modification by several physical process esas it passes the atmosphere, viz.

1) Scattering; 2) Absorption, and 3) Refraction

Atmospheric Scattering

Scattering is the redirection of EMR by particles suspended in the atmosphere or by large molecules of atmospheric gases. Scattering not only reduces the image contrast but also changes the spectral signature of ground objects as seen by the sensor. The amount of scattering depends upon the size of the particles, their abundance, the wavelength of radiation, depth of the atmosphere through which the energy is traveling and the concentration of the particles. The concentration of particulate matter varies both in time and over season. Thus the effects of scattering will be uneven spatially and will vary from time to time. Theoretically scattering can be divided into three categories depending upon the wavelength of radiation being scattered and the size of the particles causing the scattering. The three different types of scattering from particles of different sizes are summarized below:

Scattering process	Wavelength	Approximate dependence particle size	Kinds of particles
Selective			
• Rayleigh	λ^{-4}	< 1 µm	Air molecules
• Mie	λ^{o} to λ^{-4}	0.1 to 10 µm	Smoke, haze
• Non-selective	λ^{o}	> 10 µm	Dust, fog, clouds

Rayleigh Scattering

Rayleigh scattering predominates where electromagnetic radiation interacts with particles that are smaller than the wavelength of the incoming light. The effect of the Rayleigh scattering is inversely proportional to the fourth power of the wavelength. Shorter wavelengths are scattered more than longer wavelengths. In the absence of these particles and scattering the sky would appear black. In the context of remote sensing, the Rayleigh scattering is the most important type of scattering. It causes a distortion of spectral characteristics of the reflected light when compared to measurements taken on the ground.

Mie Scattering

Mie scattering occurs when the wavelength of the incoming radiation is similar in size to the atmospheric particles. These are caused by aerosols: a mixture of gases, water vapor and dust. It is generally restricted to the lower atmosphere where the larger particles are abundant and dominates under overcast cloud conditions. It influences the entire spectral region from ultra violet to near infrared regions.

Non-selective Scattering

This type of scattering occurs when the particle size is much larger than the wavelength of the incoming radiation. Particles responsible for this effect are water droplets and larger dust particles. The scattering is independent of the wavelength, all the wavelength are scattered equally. The most common example of non-selective scattering is the appearance of clouds as white. As cloud consist of water droplet particles and the wavelengths are scattered in equal amount, the cloud appears as white. Occurrence of this scattering mechanism gives a clue to the existence of large particulate matter in the atmosphere above the scene of interest which itself is a useful data. Using minus blue filters can eliminate the effects of the Rayleigh component of scattering. However, the effect of heavy haze i.e. when all the wavelengths are scattered uniformly, cannot be eliminated using haze filters. The effects of haze are less pronounced in the thermal infrared region. Microwave radiation is completely immune to haze and can even penetrate clouds.

Atmospheric Absorption

The gas molecules present in the atmosphere strongly absorb the EMR passing through the atmosphere in certain spectral bands. Mainly three gases are responsible for most of absorption of solar radiation, viz. ozone, carbon dioxide and water vapour. Ozone absorbs the high energy, short wavelength portions of the ultraviolet spectrum ($\lambda < 0.24$ μm) thereby preventing the transmission of this radiation to the lower atmosphere. Carbon dioxide is important in remote sensing as it effectively absorbs the radiation in mid and far infrared regions of the spectrum. It strongly absorbs in the region from about 13-17.5 μm, whereas two most important regions of water vapour absorption are in bands 5.5 - 7.0 μm and above 27 μm. Absorption relatively reduces the amount of light that reaches our eye making the scene look relatively duller.

Atmospheric Windows

The general atmospheric transmittance across the whole spectrum of wavelengths is shown in Figure 6. The atmosphere selectively transmits energy of certain wavelengths. The spectral bands for which the atmosphere is relatively transparent are known as atmospheric windows. Atmospheric windows are present in the visible part (.4 μm - .76 μm) and the infrared regions of the EM spectrum. In the visible part transmission is mainly effected by ozone absorption and by molecular scattering. The atmosphere is transparent again beyond about λ= 1mm, the region used for microwave remote sensing

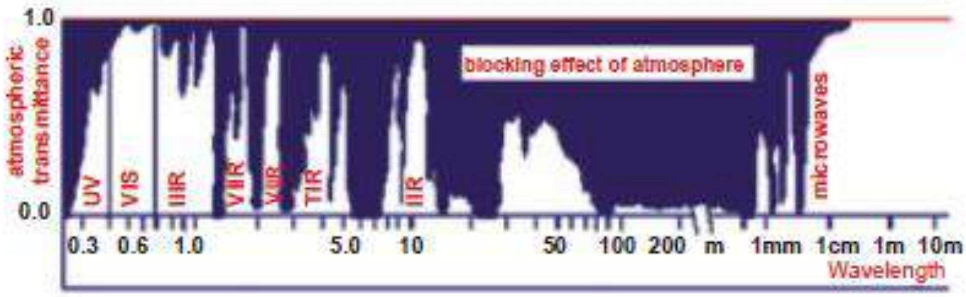

Figure 6 : Atmospheric windows

Refraction

The phenomenon of refraction, that is bending of light at the contact between two media, also occurs in the atmosphere as the light passes through the atmospheric layers of varied clarity, humidity and temperature. These variations influence the density of atmospheric layers, which in turn, causes the bending of light rays as they pass from one layer to another. The most common phenomena are the mirage like apparitions sometimes visible in the distance on hot summer days.