N5 Building and Structural Surveying

Gateways to Engineering Studies - Chris Brink

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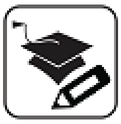
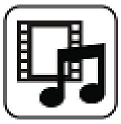
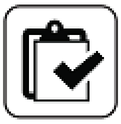
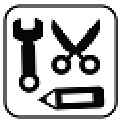
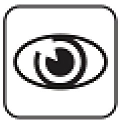
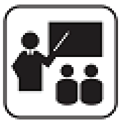
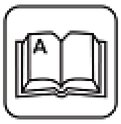
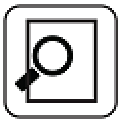
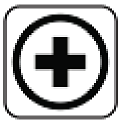
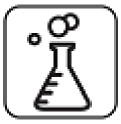
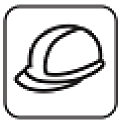
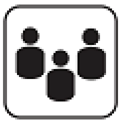
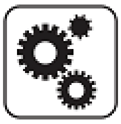
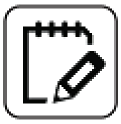
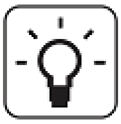
	Assessment / Activity		Multimedia
	Checklist		Practical
	Demonstration/ observation		Presentation/ Lecture
	Did you know?		Read
	Example		Safety
	Experiment		Site visit
	Group work/ discussions, role-play, etc.		Take note of
	In the workplace		Theoretical – questions, reports, case studies, etc.
	Keywords		Think about it

Module 1

Basic Principles of Surveying

Learning Outcomes
On the completion of this module the student must be able to:

- Describe the basic surveying terms and principles of surveying.
- Describe the following terms:
  - Surveying
  - Level plane
  - Horizontal plane
  - Linear measurement
  - Height measurement
  - Angular measurement
- Describe methods of fixing a point:
  - Trilateration
  - Intersection arcs
  - Rectangular offsets
  - Triangulation
  - Polar co-ordinates and control
- Describe the principle of working from the whole to the part.
- Explain the difference between accuracy and precision.
- Describe the characteristics of different types of errors.

1.1 Introduction

This module gives an overview of the field of surveying and a broad description of the various terms and methods used in surveying. It also explains the principle of working from the whole to the part and the importance of accuracy and precision. In addition it discusses the characteristics of various errors.

1.2 Surveying

Surveying may be defined as the art of taking measurements and making such field observations as are necessary to determine positions, volumes and areas of natural and man-made features on the earth 1s surface, and of representing the results on a map or plan to a suitable scale.

NOTE:
The surveying referred to above is also known as LAND SURVEYING, which must not be confused with other types of surveying such as quantity surveying, market research surveying, traffic surveying.

1.3 Plane surveying

In plane surveying all measurements made are either on a horizontal or a vertical plane, or are reduced to a horizontal or a vertical plane. The curvature of the earth is NOT taken into account because its effect is very small on small area surveys. (The limit of this area is about 30 000 hectares, which is roughly a square 17 x 17 kilometres).

Definition: Terms used in the definitions of Surveying and Plane Surveying

MAKING FIELD OBSERVATIONS is the measurement , and notation of that measurement; of all the necessary distances and angles, to the necessary degree of accuracy, with survey instruments.

MEASUREMENTS in surveying are resolved into two planes - the horizontal and the vertical planes, and there are four basic measurements to carry out:
(a) Horizontal distance
(b) Horizontal angle
(c) Vertical distance
(d) Vertical angle

The inclined Plane may also be required in some cases but that is beyond the scope of this course.

The measurements are required to establish the location of points and lines relative to one another for the purpose of making a map or a plan.

The word **FIELD** is used, in the broad sense of being anywhere outside the office, be it in open country, on a road network in a city or down a sewer.

NATURAL FEATURES are those brought about by the action of the natural processes of weathering, volcanoes. plate-tectonics , earthquakes and so on, rivers, hills, valleys, lakes, marshes and jungles.

MAN-MADE FEATURES include roads, bridges, dams, buildings, boundaries, surface and sub-terranian structures.

A **MAP** is the representation to scale, on a sheet of paper, of features on a plane surface. The scale is small: one unit of length on the paper represents a large number of units of length on the ground. Eg 1 mm = 20 000 mm would

be written as 1 in 20 000 or as 1: 20 000.
A **PLAN** differs from a map in one respect only and that is scale. A plan is a large -scale representation of a small area. For instance a house plan drawn to a scale of 1:100.

1.4 The purpose of surveying

Firstly a map or plan with (or without) profiles (sections) is drawn up from a survey and then with all the information about the natural and man-made features shown, the architects and/or designers can go to work on the design of the proposed building or structure.

The final plan they produce is then given to the surveyor who "SETS-OUT" the work to be done.

Definition: Setting-out
The ' reverse process of surveying - the surveyor has to transfer points and lines from a plan to the ground and this must be accurately done to avoid additional costs in materials, labour and possible boundary disputes. Again the work involves both the horizontal and the vertical planes.

1.5 Various methods of surveying

- Tape or chain surveys
  The tape is the basic and principal instrument. It is mainly used for large-scale detail surveying, particularly where the detail is geometrically regular. The location of points by measurement of distances from two known points to a new point, using a tape, is the basic principle involved.

  The word chain comes from the literal use of link chains; this will be discussed in more detail later).

- Direct leveling
  Determining the elevations of points relative to one another and/or to mean sea-level . When the survey is balanced no correction for the earth's curvature and atmospheric refraction is necessary.

- Indirect or trigonometric leveling
  It is also the determination of the elevations of points, but it is more precise. Vertical angles are measured from (and to) each of two points whose difference in elevation is required.

  The difference in elevation is half the horizontal distance between them multiplied by the sum of the tangents of the angles. In this procedure certain factors are applied to eliminate the effects of curvature and refract ion.

- Barometric leveling
  This is accurate to within approximately 300 mm and is sometimes used for contour surveying, but its use seems to be dwindling; due to variations of atmospheric pressure, corrections have to be regularly applied, rendering it a slower and less accurate method than direct or trigonometric levelling.

- Traverse (or transit) surveying
  Points are located by the measurement of distances and angles.

- Compass surveying
  Points are located by measurement of compass bearings (angles).

- Stadia (or tacheometric) surveying
  This is widely known simply as "TACHE" (pronounced "tacky"). The distances between points and the heights (vertical distances) relative to the instrument are measured optically. There are two main approaches, one uses a vertical graduated scale rod called a STAFF, and the other uses a horizontal rod, of fixed length with targets on each end, called a SUBTENSE BAR. The instrument (theodolite or tacheometer) has "cross-hairs" and "stadia-hairs" which are used as reference points against the staff or subtense bar.

- Triangulation surveying
  This involves the building up of a network of triangles from an accurately measured base line, by the measurement of angles only.

- Hydrographic surveying
  This type of survey is carried out where waterways, that is, rivers, lakes, harbours etc, are involved, to determine high and low water levels, depths of sand bars, rocks etc. These depths are called soundings. The map so produced is indispensible to sailors for safe navigation.

- Photographic surveying
  (a) AERIAL PHOTOGRAMMETRY, this is probably the most expensive form of surveying, but it is very useful for updating city plans and military and national surveys of large areas in great detail. Maps made by this process not only supply details such as manholes, light poles etc, but contours may also be plotted.
  (b) TERRESTRIAL PHOTOGRAMMETRY - Performs the same tasks as aerial photogrammetry, but the photographs are usually taken in pairs from the ends of a measured base line.

- Trilateration
  This is a very accurate form of surveying in which electro-magnetic distance-measuring instruments are used. It is similar to triangulation but must not be confused with chain surveying.

- Plane table surveying
  This technique is used for detail work in topographical surveys for small-scale maps, or detail for large scale engineering surveys. The plan or map is drawn as the observations are made; distances and angles are measured and plotted simultaneously.

1.6 Various types of surveys

- PLANE SURVEYING has already been defined.

- GEODETIC SURVEYING is a highly accurate survey in which the shape of the earth (the curvature) is taken into consideration, and it is used for national surveys primarily for fixing positions of boundaries and national grid reference points.

- TOPOGRAPHICAL SURVEYS for the location of the main natural and artificial (man-made) features of the area including hills, valleys, lakes, rivers, towns, roads, railways, dams etc.

- CADASTRAL SURVEYS for the preparation of plans showing and defining legal property boundaries.

- ENGINEERING SURVEYS preparatory, or in conjunction with, the execution of civil engineering works such as roads, railways, bridges, tunnels, dams, sewerage works and general construction.

- GEOGRAPHICAL OR EXPLORATORY SURVEYS

- GEOLOGICAL SURVEYS

- MILITARY SURVEYS

1.7 Horizontal, vertical and inclined planes

(Refer to **Figure 1.1** and **Figure 1.2**).

Definitions:

- A **PLANE** is a surface on which any straight line joining two points on it will touch the plane completely.
- A **HORIZONTAL PLANE** is one at right angles to the direction of gravity at any one point.
- **DIRECTION OF GRAVITY** is that from a point on the earth's surface towards the centre of the earth. It can be indicated by a plumb line.
- A **VERTICAL PLANE** is one at right angles to a horizontal plane.
- AN **INCLINED PLANE** is a sloping surface at ANY angle OTHER than a right

angle, to the horizontal or vertical planes.

- A **LEVEL SURFACE** is one which is at right angles to the direction of gravity at every point. A level surface is therefore curved, since the direction of gravity differs from point to point. An example of a level surface is the still surface of a large lake.

  Linear and angular measurements made during surveys are taken on horizontal, vertical or inclined planes .

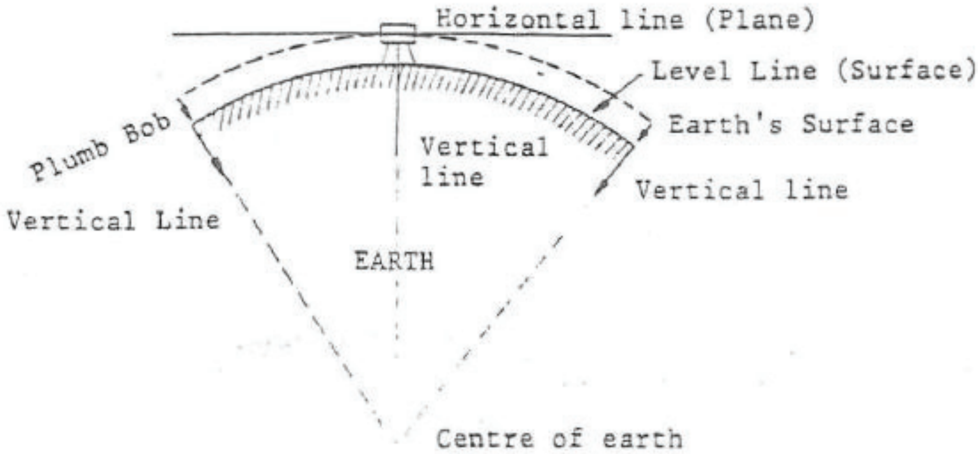


Figure 1.1 Cross-section through a sector of the earth

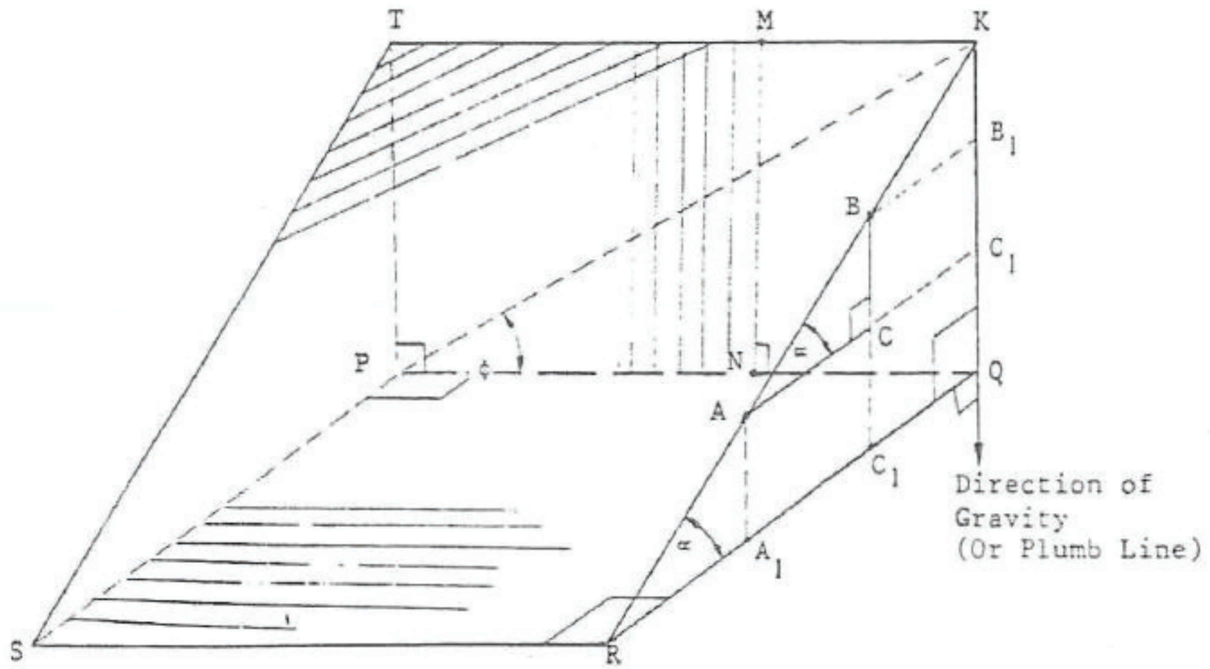


Figure 1.2 Horizontal, vertical and inclined planes

Referring to **Figure 1.2**:

(a) PQRS is a horizontal plane (at right angles to the direction of gravity). All lines joining points on this plane will be horizontal.

(b) TKQP is a vertical plane at right angles to the horizontal plane PQRS. Lines arawn parallel to the direction of gravity on this vertical plane will be vertical lines, eg line MN. (A line joining points K and P will be inclined at angle Ø° to the horizontal).

(c) TKRS is an inclined plane and the angle of inclination of the plane is ∝ ° to the horizontal plane. This angle is said to be the dip of plane TKRS and is always measured in a direction at right angles to a horizontal line drawn on plane TKRS. The angle ∝ is an angle in the vertical triangle RKQ and is therefore a vertical angle (measured in a vertical plane).

(d) RKQ i s a right-angled triangle in a vertical plane. AB is an inclined length measured along the line RK on plane TKRS.
Line AC (parallel to RQ) is the horizontal component of line AB. Knowing the length AB and the vertical angle ∝ we have AC = AB cos ∝ °.
If the difference in height between points A and B, ie distance BC, is known,
Then AC $= \sqrt{AB^2 - BC^2}$
Furthermore BC = AB sin 0
or BC $= \sqrt{AB^2 - BC^2}$

1.8 Horizontal and vertical inclined angles

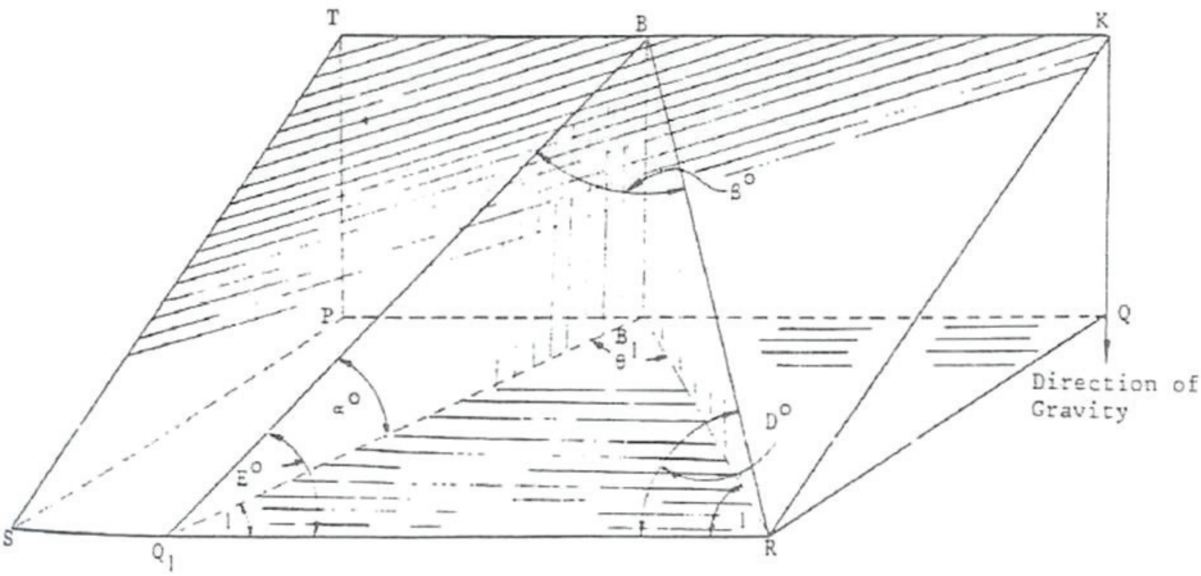


Figure 1.3 Horizontal, vertical and inclined angles

Referring to **Figure 1.3**, lines BQ_1 and BR are drawn on the inclined plane TKRS. B_1 is a point on the horizontal plane PQRS and vertically below B. Point Q_1 lies also on the horizontal plane PQRS.

The lines Q_1B_1, RB_1 and QR are all horizontal and on the same horizontal plane as PQRS. Triangle QB_1R is therefore one on a horizontal plane and angles Q_1, R_1 and θ must be horizontal angles.

Since both Q_1B and RB are lines on the inclined plane TKRS, triangle Q_1BR also lies on this inclined plane and therefore angles E, B and D are all inclined angles.

B_1 is a point vertically below B therefore BB_1 is a vertical line on the vertical plane TKQP. Since Q and B_1 lie on the same horizontal plane PQRS, triangle Q_1BB_1 lies on a vertical plane and is right-angled at B_1.

The angle $\propto$ in this triangle is therefore in a vertical plane and is called a vertical angle.

1.9 Basic surveying principles

The fundamental principles which form the basis of the various methods of surveying aim at establishing the relation between various points and lines in terms of angles and distances.

Ignoring the instruments used to collect the required data, the following methods of "fixing a point" provide the basis of point location, although some times the actual methods used in the field may be a combination of one or more of these methods due to obstacles in the path of the survey, but always try to keep the survey as simple as possible; that way fewer mistakes and errors will result.

i. By measurement of two distances from a base line of known length and location. The two measured distances are known as "TIES" or "TIE-LINES".

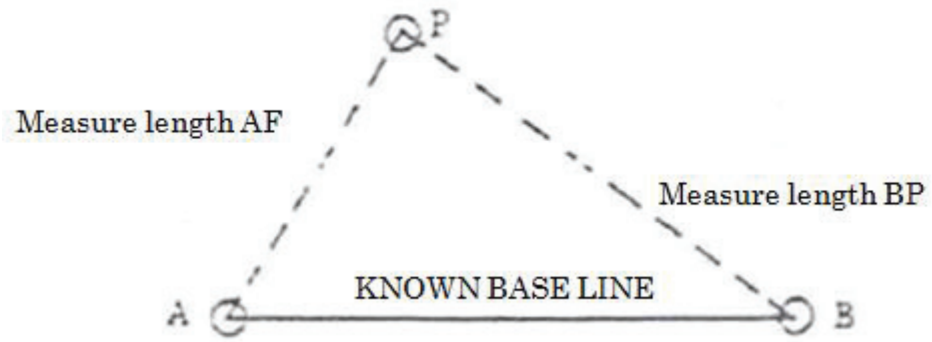


Figure 1.4a

ii. By measurement of the "PERPENDICULAR OFFSET" (or simply the "OFFSET ") from a known base line.

NOTE:
Perpendicular, or right-angled, or 90° all mean the same thing .

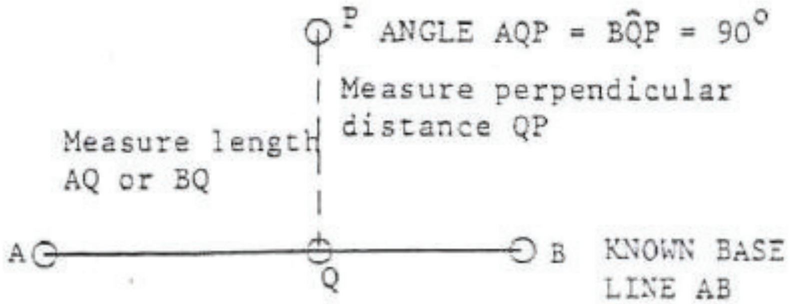


Figure 1.4b

iii. By the measurement of a length and a "direction". In this case the base line length is not a criterion as long as the exact location of the two ends is known.

	Definition: "Direction" The surveying term for angular measurement.

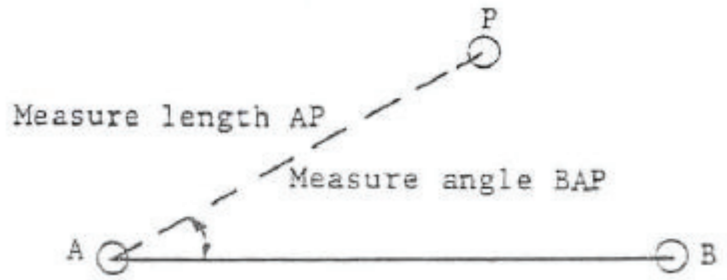


Figure 1.4c

iv. Similar to (c) by measurement of distance and direction but a different distance this time.

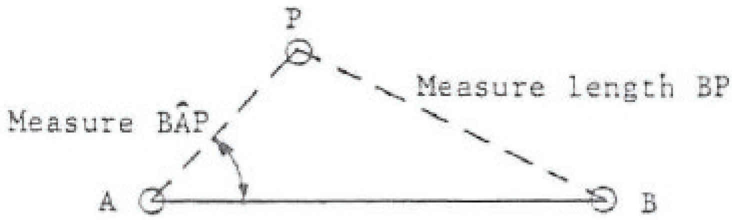


Figure 1.4d

v. By "FORWARD INTERSECTION" ie the measurement of two directions only, from two known points to the required point.

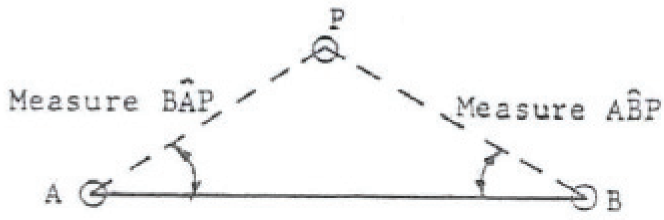


Figure 1.4e

vi. By "SIDEWAYS INTERSECTION" ie the measurement of two directions only, from one known point to the required point, then from the required point to a second known point.

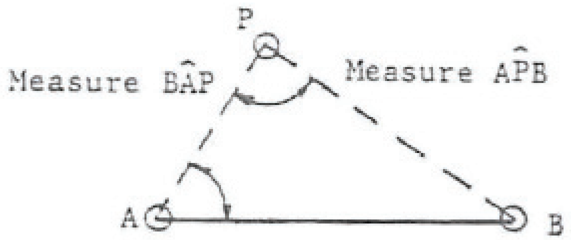


Figure 1.4f

vii. By "RESECTION" ie the measurement of the directions from the required position to three or more known points.

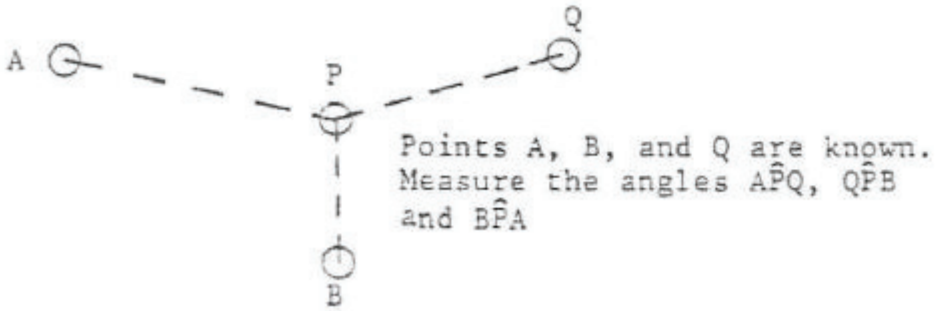


Figure 1.4g

NOTE:
Resection is beyond the scope of this course, but it is one of the basic principles of surveying.

1.10 Map, plan, section

Definitions:

A **MAP** may be defined as the representation (projected vertically) of the outline of the surface features of the earth onto a plane surface.

A **PLAN** is the representation (projected vertically) of features onto a HORIZONTAL PLANE surface and may be looked upon as a large-scale detailed map of a small area.

Linear measurements can be represented on the horizontal, the vertical and the inclined plane.

ON THE HORIZONTAL PLANE the measurement is a vertical projection onto a horizontal plane. An example is a plan of the surface and underground workings on a mine.

ON THE VERTICAL PLANE (Section) the measurement is a horizontal projection onto a vertical plane. Examples are cross, longitudinal or transverse sections taken through different positions on a plan.

ON THE INCLINED PLANE. The distances measured are merely drawn to scale straight onto the sheet of paper. An example is the stope sheet in mine surveying. Here the sheet of paper represents the plane on which the workings take place.

1.11 The qualities of a good surveyor

Besides a sound knowledge of the theory and practice of surveying, honesty is of prime importance. "Cooking the books" only leads to a bad reputation and double work. Soberness, reliability, sound judgment and consideration of subordinates are traits a good surveyor should maintain.

Neatness is important too; remember that the field book remains the property of the company and other people may need to refer to observations you have made. Initiative and resourcefulness should be applied to the solution of each new problem. Only accept information given or results obtained as trustworthy when verified.

1.12 Functions of the surveyor

The work of the surveyor may be divided into three groups:

- Field-work
- Office work
- Care and adjustment of the instruments

1.12.1 Field-work

The two main operations in field-work are:

(a) The measuring of distances and angles by the correct handling and manipulation of survey instruments.

(b) Recording the field notes according to a standard method.

Reconnaissance: Before making any measurements, carefully examine the area to be surveyed so that unnecessary back-tracking may be avoided and the survey is planned so as to cover the area effectively with built-in checks. A rough sketch diagram ("INDEX DIAGRAM") is prepared, showing the stations (reference points) selected and the lines to be measured.

Field-work is carried out for the following purposes:

- Establishing points and lines of reference from which all details of the survey can be located.
- Locating details. of the survey such as boundary lines, slope faces, buildings and any important structure, from the reference points or lines.
- Establishing directions for proposed tunnels, roads, railway lines and other structures or excavations.
- The fixing of inaccessible points.
- Determining the relative heights (elevations) of points or the vertical distance between points, and also the setting out of required gradients for tunnels, pipelines, drains and other features.

1.12.2. Office work

The office work consists of the following:

- Calculations and preparations that may be necessary to reduce the field measurements to a form suitable for plotting.
- The plotting, ie, drawing to scale, of measurements obtained direct from field operations or from the above mentioned calculations.
- Obtaining any required information from the plan by means of calculations or direct measurements.

1.12.3 Care and adjustment of the instruments

The maintenance of the various instruments will be discussed in depth later on in the course. A sound practical and theoretical understanding of survey instruments is necessary.

1.13 Scales

A map is a miniature picture of the country it represents, as far as a surface can portray a three-dimensional world. The map must be some chosen fraction of the true size of the area it pictures. Every length on the map must be this chosen fraction of the corresponding true horizontal length on the ground.

Definition: Scale of the map
The fraction, or ratio between the length of any line on the map and the horizontal length of the corresponding line on the ground, is known as the scale of the map.

The scale to which a map should be drawn depends mainly on the use to which the map will be put. The greater the degree of accuracy required, the larger must be the scale and inevitably the more costly the map is to produce.

1.13.1 Natural scales

The scale of a map may be expressed in different ways, the most convenient of which is known as the REPRESENTATIVE FRACTION (RF), the numerator of an RF is always unity, and the denominator denotes the number of times smaller the map is, compared with the area it represents. Thus, a scale or 1/100 000 means that one unit on the map represents 100 000 of the same units measured horizontally on the ground.

1.13.2 Engineer's scales

The scale may also be expressed as a statement of the relation between two different units of measurement, one referring to the map, the other to the ground measurement. Thus, one centimetre equals one metre, which is written 1 cm : 1 m. It will be seen there that the RF of 1 cm : 1 m is 1 : 100 which is the natural scale.

In the Metric system, only natural scales will be used for new work, but as plans previously drawn to engineering scales will remain in use for many years, familiarity with them is essential.

1.13.3 Divided scale line

An expression of the scale may also be given graphically, in the form of a Divided Scale Line. This is a straight line of any convenient length, equally divided into primary divisions, of which the first from the left is the zero division, while the first space is subdivided into secondary divisions, to facilitate the measurement of the distances by means of dividers. These distances represent those measured horizontally on the ground (see **Figure 1.5**).

1.13.4 Scale of map symbols

However truly the scale fraction may apply to the map as a whole, it cannot apply to all the detail. On a standard map whose scale is 1:25 000 for instance, roads which are 5 metres wide on the ground are represented by a width which scales 20 - 30 metres on the map. The sizes of buildings, monuments, windmills, etc, must be exaggerated, otherwise they would be insignificant.

1.13.5 Transformation of scales

The following examples will illustrate scale transformation:

(i) On a map scale of 1 : 200 000, what distance in metres, does 52 millimetres represent?

1 mm represents 200 000 mm

$\therefore$ 52 mm represents $\frac{52 \times 200\ 000}{1000}$ = 10 400 metres

(ii) On a map scale 1 : 50 000, what distance in kilometres does 8 centimetres represent?

1 cm represents 50 000 mm

$\therefore$ 8 mm represents $\frac{8 \times 50\ 000}{100 \times 1000}$ = 4 kilometres

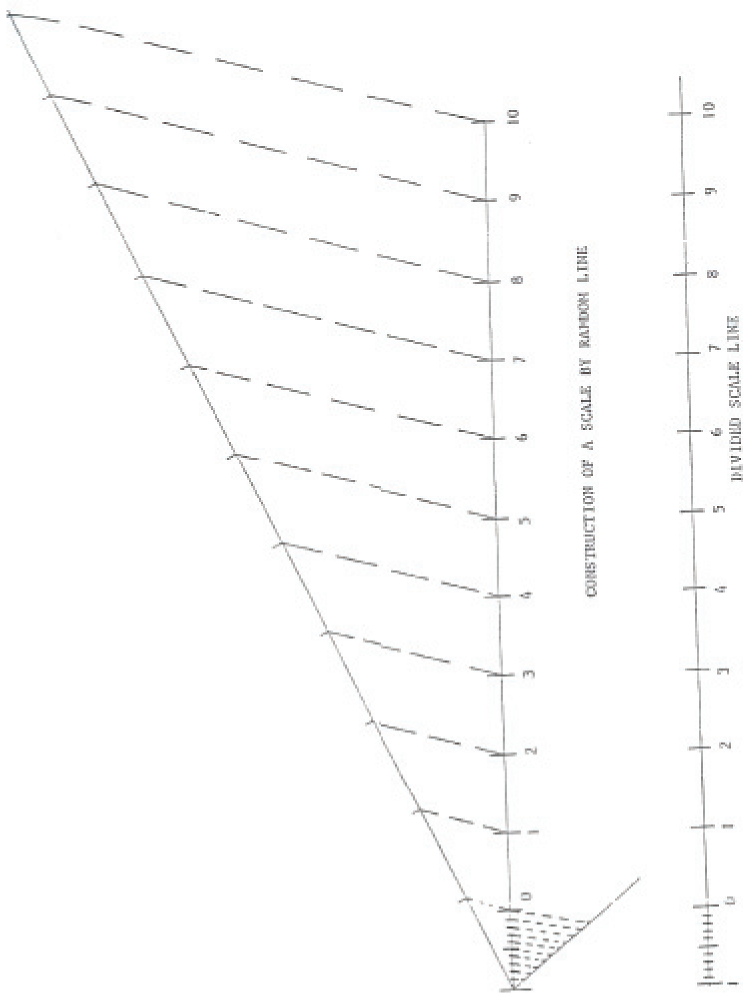


Figure 1.5

1.13.6 Accuracy of maps

The final form of a map or plan will depend upon the purpose for which the map is prepared.

The scale will depend on the use to which the map or plan will be put. The smaller the scale, the less accurately will it be possible to show the relative distances between points.

Factors influencing accuracy are:

- Control (framework)
- Scale
- Speed and cost
- Skill and experience

- Control (framework)
  Generally survey work may be divided into:

(a) the fixing of control points,
(b) the location of detail within the framework formed by control points.

In any survey the problem must first be considered as a whole. It is similar to that of constructing a building with a steel frame where the strength depends on the framework and the details inside the framework may be built with weaker members.

In survey this framework is treated in subsequent operations as being devoid of error. By this means accumulation of error is avoided and highly accurate and expensive methods are confined to that small portion of the work where they are essential.

Control is established by determining accurately the positions of a number of carefully fixed points covering the area to be surveyed.

Subsequent work is then connected to these control points, upon which the surveyed detail depends. Such a framework must be accurate to prevent the accumulation of any system of errors.

In South Africa the framework consists of a number of accurately co-ordinated trigonometrical beacons covering the whole Republic.

The maps of adjacent areas, surveyed upon this framework, will fit together to form an accurate, reliable, composite whole.

(Vertical control will be dealt with in Leveling.)

- Scale
  In topographical surveying, the co-ordinates of points are not calculated, they are determined graphically. The degree of accuracy with which this determination can be made depends upon the degree of accuracy to which plotting can be done.

  Plotting accuracy depends on how near to can be plotted. With care, using a fine its true position a point pricker needle, this distance is about ¼ mm.

  This provides an indication of the degree of accuracy to be maintained in the field work.

  Let us illustrate plotting accuracy by considering the following example:

Worked Example 1.1

A plan of an area, on a scale 1 : 10 000 is proposed. To what degree of accuracy must the surveyor work to represent detail accurately on the map?

Solution:

1 mm represents 10 000 mm

∴ ¼ mm represents ¼ × $\frac{10\ 000}{1000}$ metres = 2,5 metres

Thus, the plotting accuracy for the above plan is 2,5 metres.

Detail may, therefore, be located to the nearest 2 metres, to avoid plottable error.

The plottable error for a plan scale 1 20 000 is 5 metres. You should verify this answer.

- Speed and cost
  Accuracy is a function of speed and cost. Accuracy should therefore be balanced against costs and (or) the time available.

- (iv) Skill and experience

A surveyor has to learn accuracy first, then by practice, increase his speed of working. This will eventually result in skill and experience which are necessary to produce an accurate map rapidly.

The successful balancing of accuracy against cost and the time available needs initiative and judgment backed by experience.

Summary

Reviewing the field operations and applying a classification according to the degree of accuracy which they should exhibit gives the following results:
(a) Framework (highest accuracy)
(b) Interpolated control (intermediate accuracy)
(c) Location of detail (least accurate).

Each of these stages is dependent upon the preceding stage, and must be of sufficient accuracy to ensure that the location of detail in the final stage will be without plottable error.

The intelligent balancing of the accuracy of the work with the requirements of the job in hand is called Economy of Accuracy, and is the hallmark of the good surveyor in all branches of survey work.

1.14 Checks

No survey work is complete unless it is checked. Wherever possible, an independent check should be made on all stages of the work. The presence of a mistake or of errors of too large a magnitude in field measurements may not be detected until a considerable amount of subsequent work has been based upon them, in which case the work done will either have to be discarded or adjusted. This causes inconvenience and expense.

Checking consists of such additional measurements as are necessary to prove that the field work is correct within the allowable limits of error, or to prove that field measurements are mutually inconsistent, thereby detecting the presence of a mistake or of unpardonably large errors.

A check may be PARTIAL or ABSOLUTE. A PARTIAL check is one which provides verification of the work except in peculiar geometrical circumstances or except where a check itself may be wrong. An ABSOLUTE check precludes the possibility of any compensating mistakes from escaping detection.

1.15 Units of measurement

1.15.1 Linear measurement

The basic unit of linear measure is the metre. Kilometres, centimetres and millimetres will be in common use, but survey work will be confined to metres and decimals thereof. The relation between the metre and its decimals is given below. The abbreviated symbols appear in brackets.

Linear

1 metre (m) = 0,001 kilometre (km) 1 km = 1 000 m
1 metre (m) = 10 decimetres (dm) 1 dm = 0,1 m
1 metre (m) = 100 centimetres (cm) 1 cm = 0,01 m
1 metre (m) = 1 000 millimetres (mm) 1 mm = 0,001

Area

square metres (m^2); square centimetres ($c\ m^2$);
square decirnetres ($d\ m^2$); square millimetres (mm^2).

$1\ m^2 = 10^2 dm^2 = 100\ dm^2$ $1 = 0,01\ m^2$
$1\ m^2 = 100^2 cm^2 = 10\ 000\ cm^2$ $1\ cm^2 = 0,000\ 1\ m^2$
$1\ m^2 = 1\ 000^2 mm^2 = 1\ 000\ 000\ mm^2$ 1 mm2 = $0,000\ 001\ m^2$
1 are (a) = 100 m^2
1 hectare (ha) = 100 are (a) = 10 000 m^2

Volume

cubic metres (m^3) cubic decimetres (dm^3)
cubic centimetres (cm^3) cubic millimetres (mm^3)

$1m^3 = 10^3 dm^3 = 1\ 000\ dm^3$ ($10^3\ dm^3$)
$1m^3 = 100^3 dm^3 = 1\ 000\ 000\ cm^3$ ($10^6\ cm^3$)
$1m^3 = 1\ 000^3 mm^3 = 1\ 000\ 000\ 000\ mm^3$ ($10^9\ mm^3$)
$1m^3$ = 1kilolitre = 1 000 litres (10^3 litres)

Mass

1 kilogram (kg) = 1 000 grams (10^3 g)

1 000 kg = 1 m^3 = 1 000 litres (RD = 1)
1 kg= 1 dm^3 = 1 litre (RD = 1)
1 gram = 1 cm^3 = 1 millilitre (RD = 1)

1.15.2 Angular measure

(i) Sexagesimal measure
The unit of angular measure is the DEGREE, which is divided into MINUTES and SECONDS.

360 degrees = a full circle = 4 right angles
1 degree = 60 minutes (written 60')
1 minute = 60 seconds (written 60")

The above is known as the sexagesimal system.

(ii) Centesimal measure
In this system the unit of angular measurement is the GRADE and its decimals, ie, 400 grades = a full circle = 4 right angles.

Thus 100 grades = a right angle.

The divisions of a grade are centigrades and centi-centigrades.

1 grade = 100 centigrades
1 centigrade = 100 centi-centigrades.

The above is known as the centesimal system.

(iii) Circular or radian measure
A radi an (rad) is the angle at the centre of a circle, subtended by an arc equal to the rad ius of the circle.

$$
\begin{aligned}
2\pi r \text{ (circumference)} &= 360° \text{ (full circle)} \\
\therefore 2\pi \text{ radians} &= 360° \\
\pi \text{ radians} &= 180° \\
1 \text{ radian} &= \frac{180°}{\pi} \\
&= 57{,}295\ 78° \\
&= 206\ 265 \text{ seconds} \\
&= 3{,}141\ 59 \\
&= \text{the number of radians in a semi-circle} \\
1° &= 0{,}\ 017\ 45 \text{ radians} \\
1' &= 0{,}000\ 290\ 9 \text{ radians} \\
1'' &= 0{,}000\ 004\ 8 \text{ radians}
\end{aligned}
$$

The Sexagesimal and Radian Measures are the two systems used extensively in South Africa.

1.16 South African National Control Systems

The concept of control or framework was discussed previously. The purpose, ultimately, is to have a complete network of "trig beacons" (short for trigonometric) covering the whole country, being closely spaced in urban areas and more widely spaced in the rural districts. This task is not yet completed.

1.16.1 Horizontal control – The South Africa co-ordinate system

The GAUSS CONFORM (or TRANSVERSE MERCATOR) PROJECTION is the basis of the South African national co-ordinate system. It is a variation of the ordinary Mercator system, turned through 90° so that lines of longitude (meridians) can be used instead of the equator.

The system is a rectangular grid with a "Y-Axis" which runs parallel to the equator, with its zero at the CENTRAL MERIDIAN and its POSITIVE WEST of the central meridian and NEGATIVE EAST of the central meridian. The "X-Axis" runs from north to south, ie zero at the equator and positive south of the equator (in other words, always positive in South Africa).

The system is made up of belts, 2° of longitude wide. The central meridian is always an odd meridian, ie 15, 17, 29 etc and it is referred to as Lo 29, Lo 31 etc. Each central meridian has two boundary meridians, one to the east arid one to the west, and these will always be even meridians, for example, Lo 29 has longitudes 28° and 30°as boundary meridians.

Where the central meridian of a belt meets the equator it is called the origin of that belt.

The Trigonometrical Survey Office supplies the values of the Y and X-co-ordinates in metres.

When a survey is done the belt name (eg Lo 29) must be clearly marked, and the signs of all co-ordinates must be written down even if they are positive. This is to avoid any confusion, and in any case a co-ordinate without a sign is meaningless.

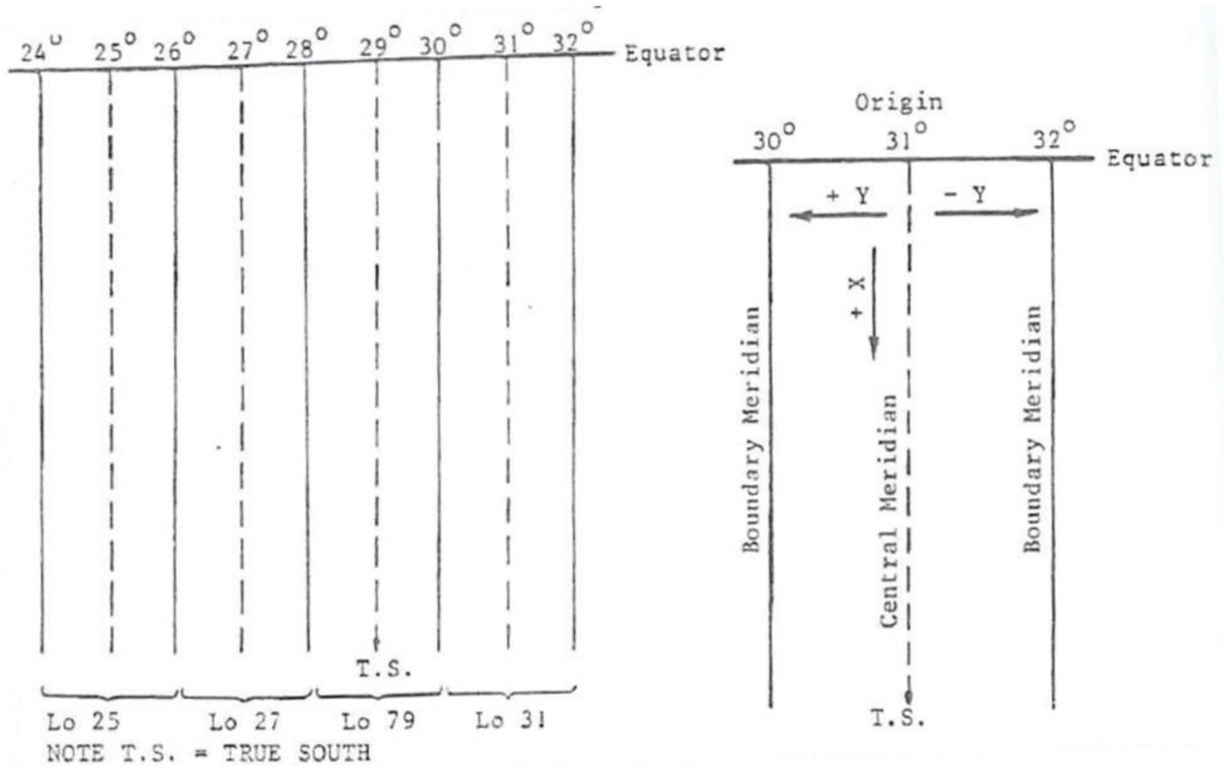


Figure 1.6 SA National co-ordinate system

1.16.2 Vertical control – heights of trig beacons above mean sea level

Trig beacons are also known as triangulation stations. Unless otherwise stated in trig lists the height is in metres above mean sea level.

Mean sea level has been accurately determined by observation over continuous periods for many years of tidal fluctuations at various points along the coastline.

The levels or elevations of the trig stations are determined by precise levelling.

Definitions:

MERIDIANS

These are imaginary line circles around the earth passing through the poles. All meridians intersect the equator and all parallels of latitude at right angles.

CO-ORDINATES

These values in metres of a point on the ground represented on a plan or map. They consist of an ORDINATE, the Y-value (which is always written first in surveying), and an ABSCISSA, the X- value

1.17 Mistakes

Mistakes are avoidable blunders, which occur from lack of concentration, carelessness, temporary mental lapse or simply from inexperience. Some typical mistakes are, noting down the wrong figures, reversing figures, using incorrect data in calculations or incorrect use of the instruments by inexperienced surveyors or assistants.

Mistakes are unpredictable and because there is no law governing them adjustments cannot be made; the survey simply must be repeated . Nobody is infallible, no matter how experienced, so all surveyors must always be on the look out for mistakes.

1.18 Errors

Errors may be divided into three categories:

(a) NATURAL ERRORS: These result from the effects of the forces of nature, namely, temperature, wind, atmospheric pressure, humidity, earth tremors and visibility or any other general prevailing natural conditions. Most of these sources of error can be compensated for if the exact conditions are known. Generally these conditions affect the accuracy of the instruments which are precision-made scientific apparatuses and therefore very sensitive.

(b) INSTRUMENTAL ERRORS: These errors are caused by imperfections of the instrument as a result of wear and tear or maladjustment or manufacturing defects. This type of error is usually constant for a particular instrument.

(c) PERSONAL ERRORS: Individual peculiarities and imperfections of sight and touch and actual mistakes give rise to this type of error.

1.18.1 Types of error

Three categories of sources of error have been defined. These sources give rise to different types of error, namely, mistakes, constant errors, systematic errors and accidental errors.

(a) Mistakes

(b) Constant Errors

Constant errors are those which, in all measures of the SAME UNIT, made under the SAME CIRCUMSTANCES, have the SAME MAGNITUDE AND ALGEBRAIC SIGN. For example, the employment of a tape which is permanently too long or too short will introduce a constant error into linear measurements.

Similarly faulty coll imation in the telescope, or uneven graduation of the divided circle will cause constant error in the measurement of angles.

Any circumstance, either in the apparatus itself, or external to the apparatus, which affects each separate measurement by the same amount gives rise to a constant error.

(c) Systematic Errors

Errors which depend for their magnitude and sign upon external circumstances, such as temperature, the slope of a surface, etc, are usually referred to as systematic errors.

In precise surveying, it is usual to avoid the effects of very strongly marked circumstances, such as extremes of temperature, strong winds and excessive refraction, by ceasing to observe when such conditions are present, but they will always be present to a greater or lesser extent.

(d) Accidental Errors
The small errors which remain in all recorded observations after mistakes and constant and systematic errors have been eliminated, are termed accidental errors.

In their effects upon the final value they are just as likely to produce a result greater than the correct quantity as to give a value smaller than it. Consequently , where repeated measurements of a quantity are recorded, and an average value is taken, accidental errors tend to be compensating in nature.

Constant and systematic errors, on the other hand, tend to produce a cumulative effect when the quantity is derived from a succession of faulty measurements added together.

Accidental errors, in general, have irregular causes, and their effects upon individual observations are considered to be governed by no fixed law connecting them with the circumstances of observation. It follows that, if the law governing an accidental error becomes known, the error ceases to be accidental.

1.18.2 Cumulative and compensating errors

When the value of a quantity is found by adding together the measurement of a number of smaller quantities, any source of CONSTANT OR/AND SYSTEMATIC ERRORS becomes a source of CUMULATIVE ERROR.

Under given conditions, a constant or systematic error always has the same sign. Any source of accidental error becomes a source of compensating error, since for each of the separate measurements, the sign is as likely to be a plus as a minus, ie the errors tend to balance each other.

1.19 Eliminating errors

It is impossible to eliminate discrepancies entirely; but their effects upon measurements may be greatly reduced by attention to certain principles of observation.

(a) MISTAKES
Mistakes can be avoided largely by adopting a methodical procedure in handling the apparatus and in recording the results. The procedure should also include definite stages of checking the work. Checking is an essential part of the work.

(b) CONSTANT AND SYSTEMATIC ERRORS
When recognised, these can usually be eliminated by suitable methods of observation or by calculation, as will be seen later during the course. In the case of systematic errors corrections may be calculated from known data such as the temperature of the tape, the angle of slope of the land, etc.

(c) ACCIDENTAL ERRORS
These cannot be eliminated entirely but their effects can be diminished very considerably by repeating the measurement several times and by adopting a mean value.

In considering whether to continue repeating an observation, the observer should bear in mind that it is a waste of time to repeat the same observation under the same circumstances, more often than is necessary, to ensure that no mistake has been made.

In obedience to this principle in observing, lines should be re-measured with different tapes, angles should be repeated upon different portions of the graduated circle , instruments should be thrown slightly out of adjustment, and re-adjusted in preparation for a new series of observations.

1.20 Methods of obtaining an average result

(a) SIMPLE ARITHMETIC MEANS
For the purpose of elementary surveying, it will usually be sufficient to adopt the average of a number of repeated measurements.

This average is simply the arithmetic mean, obtained by adding all the different results together and dividing the total by their number.

Means usually have to be rounded off to a given degree of accuracy, say the nearest number. It often happens that the mean falls exactly midway between two numbers, eg ½(4 + 9) = 6½. In such cases the figure should be rounded off to the nearest even number ie 6, and not 7 as is more usual.

(b) WEIGHTED MEANS
Under certain conditions it may be known or suspected that some of the observations are likely to produce more accurate results than others. In such cases it is usual to give more "weight" to the more reliably observations, ie, the observations with greater "strength" are allowed to influence the final adopted value more than are the "weaker ones.

The method of simple Means will be used extensively by students in thls grade ot surveying.

1.21 Conventional symbols for maps

The attached sheets show the various conventions for features to be plotted and shown on plans. You are advised to study these carefully for they may be very useful in plan drawing and interpolation (reading).

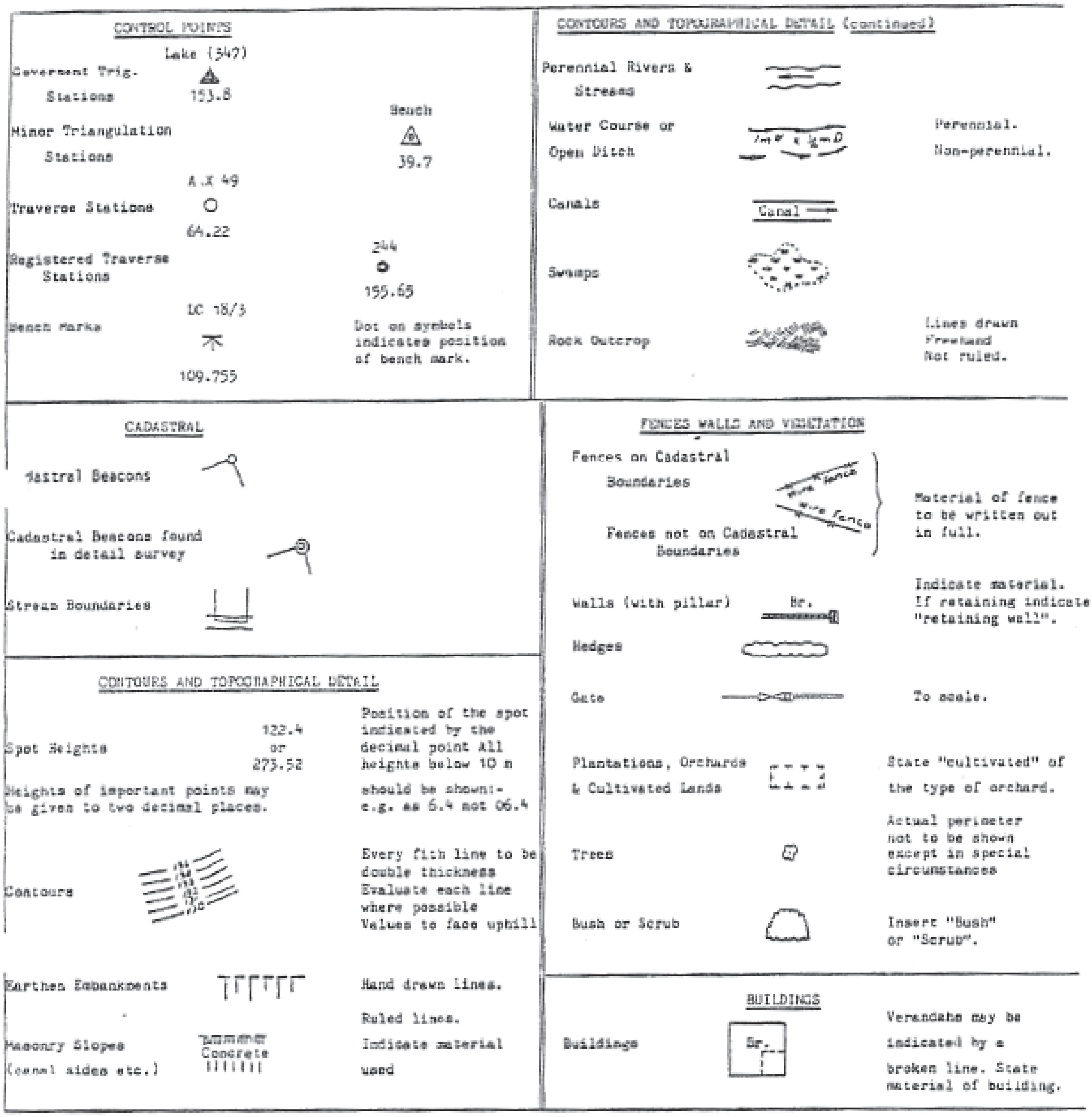


Figure 1.7a

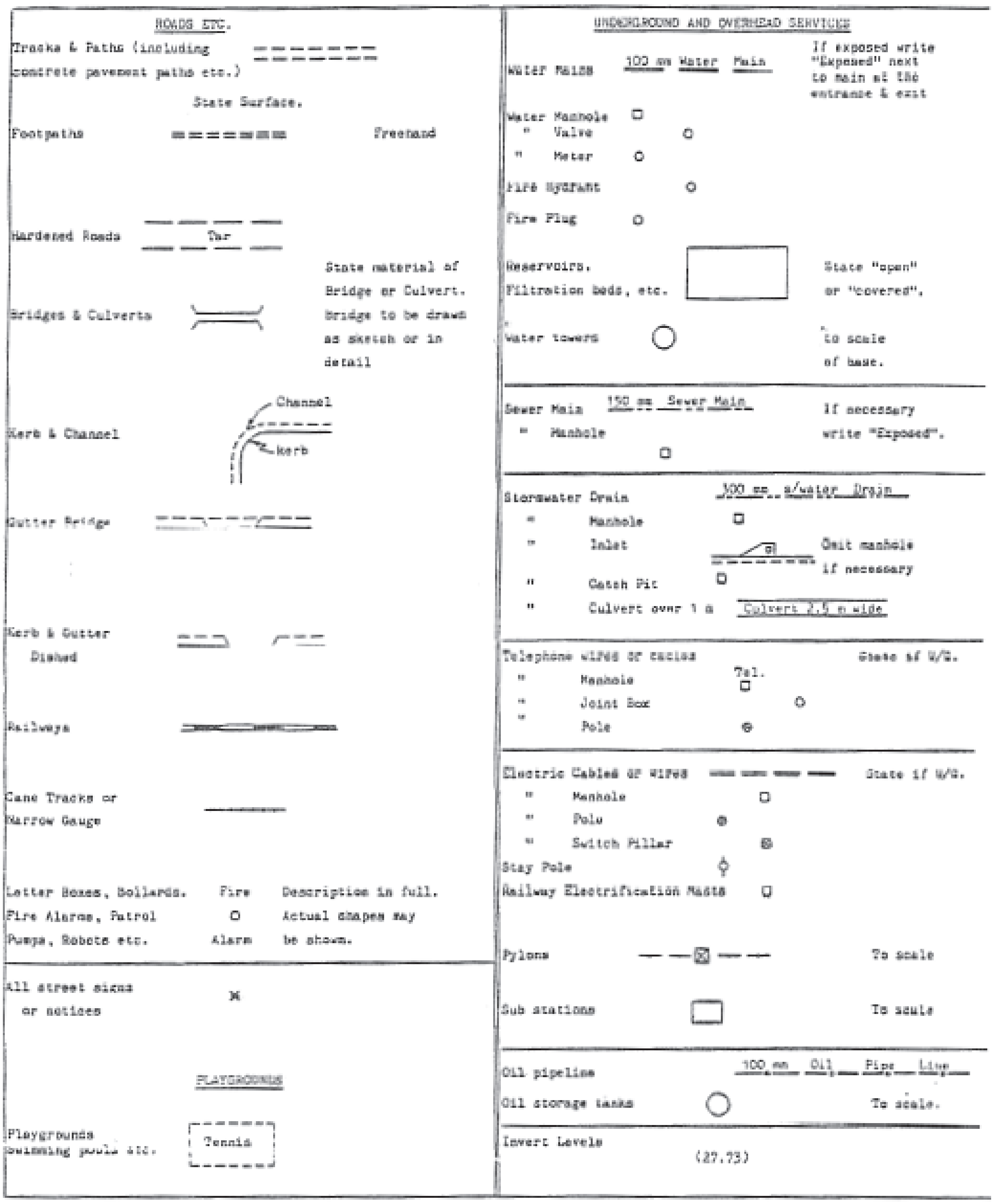

ROADS ETC.
Tracks & Paths (including concrete pavement paths etc.)
State Surface.
Footpaths
Freehand
Hardened Roads
Tar
State material of Bridge or Culvert. Bridge to be drawn as sketch or in detail
Bridges & Culverts
Channel
Kerb & Channel
Kerb
Gutter Bridge
Kerb & Gutter Dished
Railways
Cane Tracks or Narrow Gauge
Letter Boxes, Bollards, Fire Alarms, Patrol Pumps, Robots etc.
Fire
Alarm
Description in full. Actual shapes may be shown.
All street signs or notices
PLAYGROUNDS
Playgrounds Swimming pools etc.
Tennis
UNDERGROUND AND OVERHEAD SERVICES
Water Mains
100 mm Water Main
If exposed write "Exposed" next to main at the entrance & exit
Water Manhole
" Valve
" Meter
Fire Hydrant
Fire Plug
Reservoirs, Filtration beds, etc.
State "open" or "covered".
Water towers
to scale of base.
Sewer Main
150 mm Sewer Main
" Manhole
If necessary write "Exposed".
Stormwater Drain
300 mm s/water Drain
" Manhole
" Inlet
Omit manhole if necessary
" Catch Pit
" Culvert over 1 m
Culvert 2.5 m wide
Telephone wires or cables
State if U/G.
" Manhole
Tel.
" Joint Box
" Pole
Electric Cables or wires
State if U/G.
" Manhole
" Pole
" Switch Pillar
Stay Pole
Railway Electrification Masts
Pylons
To scale
Sub stations
To scale
Oil pipeline
100 mm Oil Pipe Line
Oil storage tanks
To scale.
Invert Levels
(27.73)

Figure 1.7b

1.22 Co-ordinates

In survey work, measured slope distances are transformed into their horizontal and vertical components. Only horizontal distances are used for plotting a plan, and these horizontal distances, combined with their directions, are transformed into two further components called CO-ORDINATES.

The two co-ordinates of a point are, in survey work, measured at right angles to each other, and they originate from zero lines called co-ordinate axes.

The position of any point on the earth's surface can be fixed by measuring its perpendicular distance from each of the two co-ordinate axes, the intersection of which is called the origin. A description of the point's position may be recorded accurately and concisely by means of its co-ordinates.

In mapping and survey work, the co-ordinates are usually named Y and X (in military mapping they are called Eastings and Northings). It is the usual practice for Y, the ORDINATE, to represent measurement from East to west and for X, the ABSCISSA, to represent the measurement from north to south.

It is a convention in surveying that the Y co-ordinate should be written first, followed by the X co-ordinate, as follows:

- 19 425,13 Y + 314 006,07 X.

Important Note:
A co-ordinate without its sign is meaningless and it must on no account be thought that, if the sign is +, it may be omitted.

The co-ordinates of a point are, therefore, the lengths of the perpendiculars supposed to be drawn from the point to the co-ordinate axes. See **Figure 1.8.**

On the South African co-ordinate system the Y co-ordinate is measured positively to the west of the origin and negatively to the east. The X co-ordinate is measured positively from the origin, in the positive direction of the X-axis (which is south) and negatively in the opposite direction. (See **Figure 1.9**)

In survey work, angles are always measured in a clockwise direction. The ZERO direction of the South African system is South. **Figure 1.9** also shows how a co-ordinate system is divided into quadrants, which are numbered 1 to 4 in a clockwise direction, starting from the one in which both co-ordinates are positive.

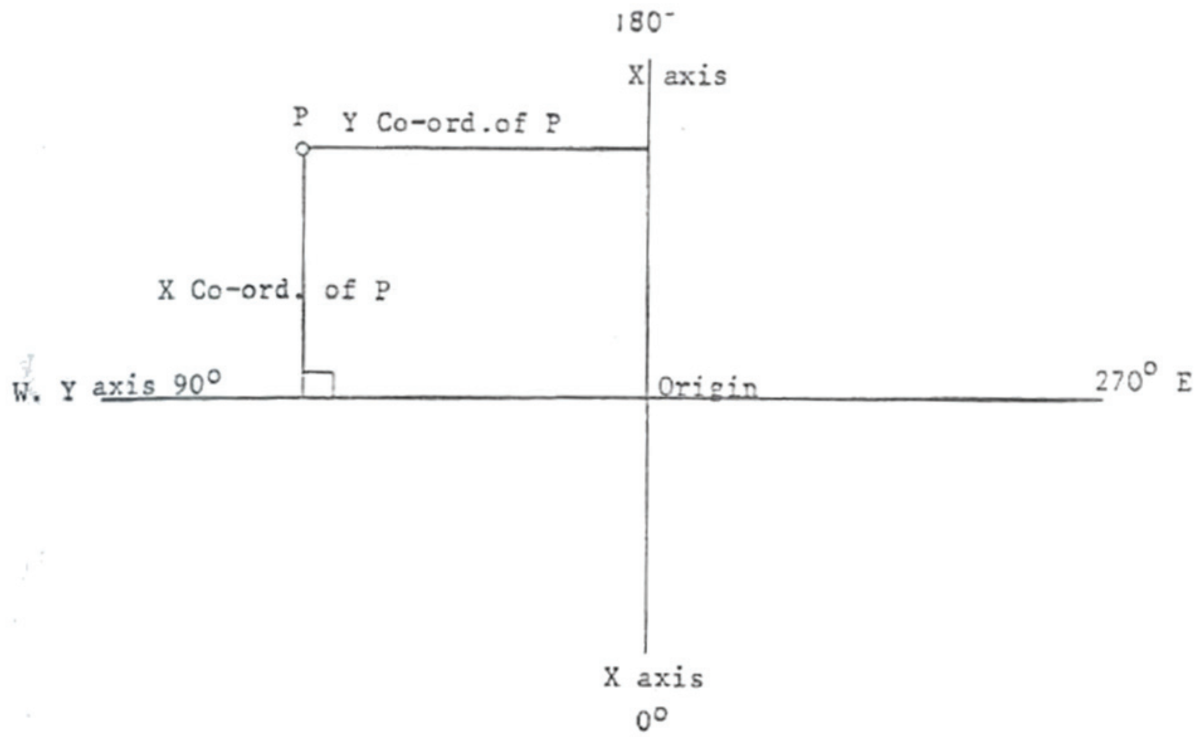

180°
X axis
P Y Co-ord.of P
X Co-ord. of P
W. Y axis 90°
Origin
270° E
X axis
0°

Figure 1.8

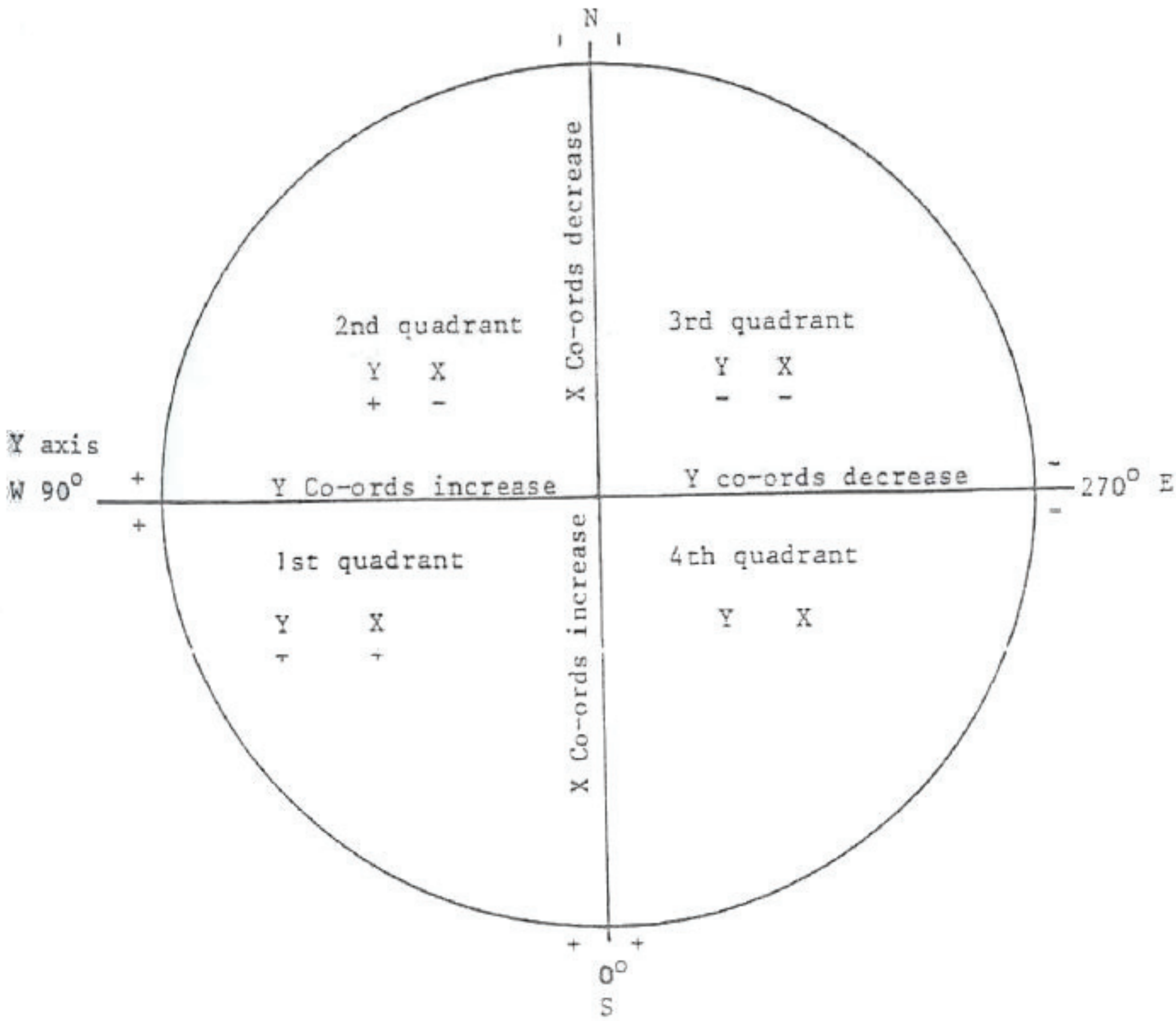


Figure 1.9

The tabulation below shows the signs of Y and X values in the 4 quadrants.

QUADRANT	QUADRANT NO.	Y CO-ORD	X CO-ORD
0 - 90°	1	+	+
90 - 180°	2	+	-
180 - 270°	3	-	-
270 - 360°	4	-	+

Table 1.1

In the calculation of co-ordinates the FA (Fundamental Angle) is used and not the direction. See **Figure 1.10** and **Table 1.2**.

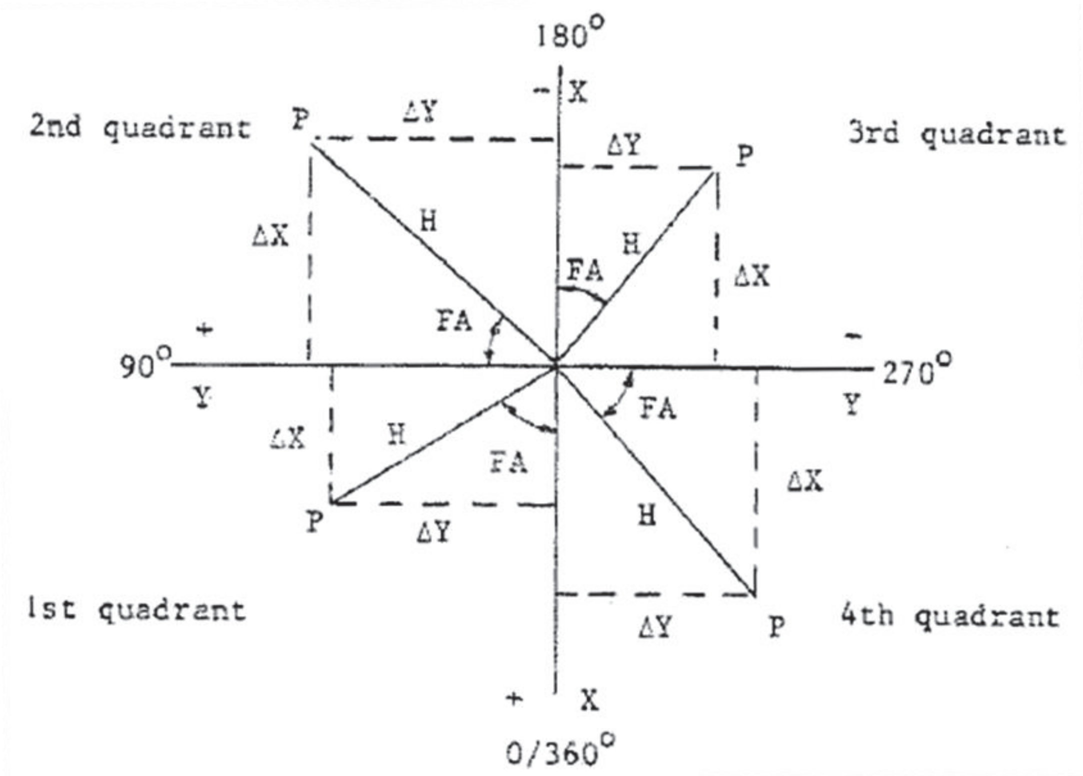


Figure 1.10

QUADRANT NO.	CHANGE IN Y CO-ORD	CHANGE IN X CO-ORD
1	+ ΔY = H.sin FA	+ ΔX = H.cos FA
2	+ ΔY = H.cos FA	- ΔX = H.sin FA
3	- ΔY = H.sin FA	- ΔX = H.cos FA
4	- ΔY = H.cos FA	+ ΔX = H.sin FA

Table 1.2

Note:
The symbol used for distance is H or S; Direction angle is D; Functional angle is FA or (more usually) A; Co-ordinate is symbolised by a square bracket, eg, [B] + 116 765,854 + 564 781,235

In survey work, the distance H is measured from a known point (the origin of the graph, (see **Figure 1.8**) and the direction is measured from some other known point or points, or from the X axis (if it can be accurately located, eg, with a compass).

Then the functional angle can be calculated, the changes in X and Y values are calculated and added algebraically to the co-ordinates of the known point to obtain the co-ordinates of the required point P (or whatever it is designated).

The above procedure is called the calculation of the POLAR, the direction instead of the functional angle may be used, but this can be a source of error when calculating the sin and cos of the angle. Also, the functional angle can always be calculated if the direction is known, but not vice versa.

1.23 Directions (Azimuth or bearing)

The direction of a line is the angle the line makes with the positive side of the X axis (ZERO DIRECTION), or an imaginary axis, and is always measured in a clockwise direction.

Let Y_1Y_1 and X_1X_1 in **Figure 1.11** be the original axes of a survey system, while Y_2Y_2 X_2X_2 etc. are imaginary axes parallel to Y_2Y_2 and X_2X_2, the original axes.

Then

(a) The direction of line OA = $\propto$ ° = X_1OA and it can be seen that line OA lies in the third quadrant with respect to the axes through 0. The direction of line OA must therefore be a value somewhere between 180and 270°.

(b) The direction of line AB = β° = X_2AB and here, line AB lies in the second quadrant with respect to the axes drawn through A. Direction A to B is therefore a value somewhere between 90° and 180°.

(c) The direction of line BC = Ø° X_3BC and in this case the line falls in the first quadrant with respect to the axes drawn through B, and the value must therefore be something between 0° and 90°.

Substituting the symbol x for the word "direction"; we have xAB, which is read: "direction AB".

The symbol, <, means "smaller than" and when we turn it around, thus, >, it means "greater than". The following examples based on **Figure 1.11** illustrate this point:

(i) OA = $\propto$ ° where 180° < $\propto$ < 270°.
The above reads: "The direction OA equals $\propto$ ° where $\propto$ is greater than 180° but smaller than 270°".

(ii) yAB = β° where 90° < β < 180°.

(iii) xBC = Ø°0 where 0° < Ø < 90°.

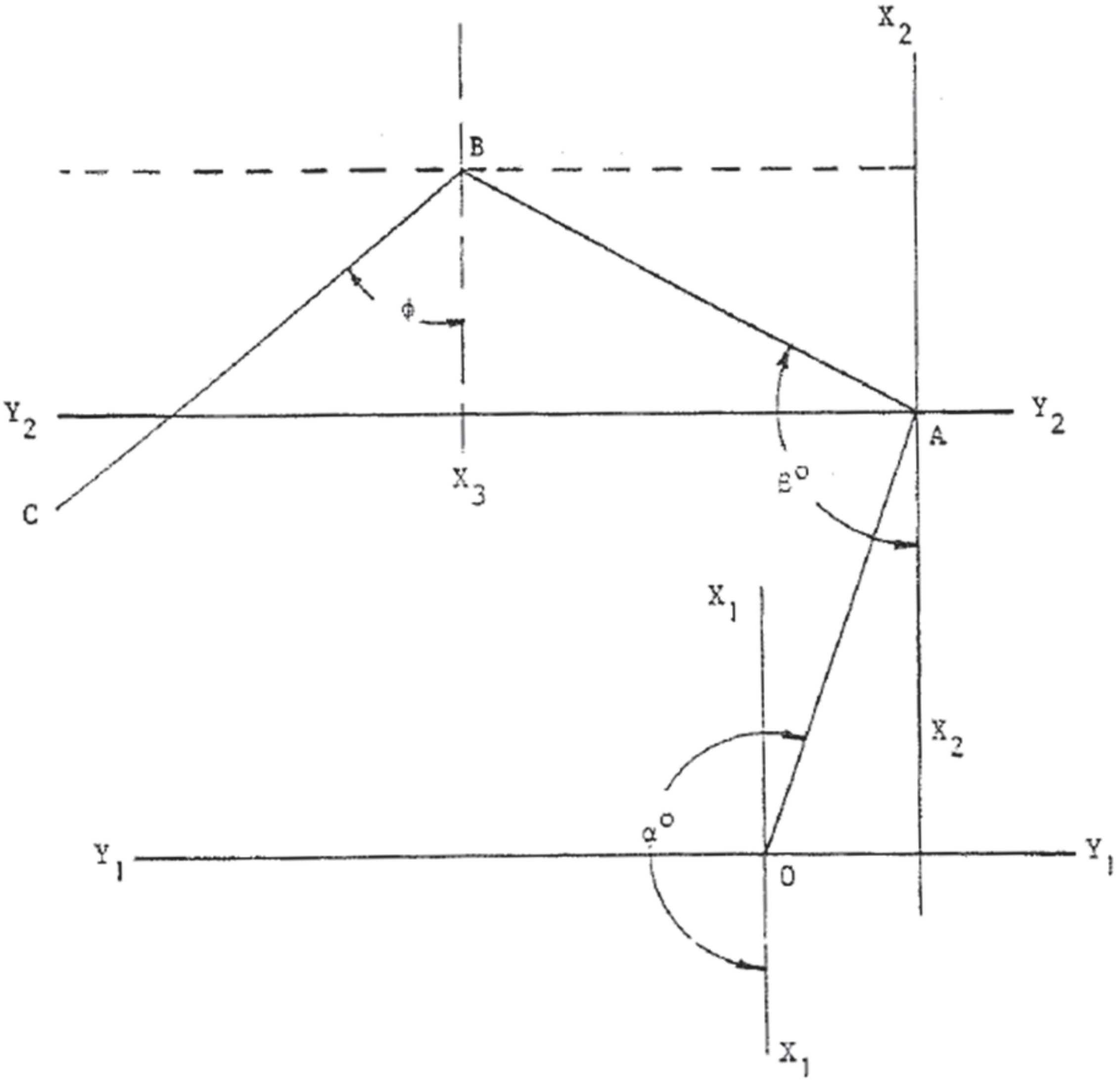


Figure 1.11

1.24 Relation between the two extremeties of a line

Let the direction of a line PQ (in **Figure 1.12**) be 40°, thus xPQ = 40°.
It can be seen that $X_1PQ = X_2PQ = 40°$ (alternate angles are equal), so that xQP = (180° + 40°) = 220°.

The relation is therefore ± 180° in all cases.

Another example: suppose the direction of a line R5 (**Figure 1.13**) 0 equals 330°.

Now XRS = RSX_1 = 30°

∴X_1SR = 180° - 30° = 150°

Thus xRS = 330°

and xSR = 150° (330° - 180°)

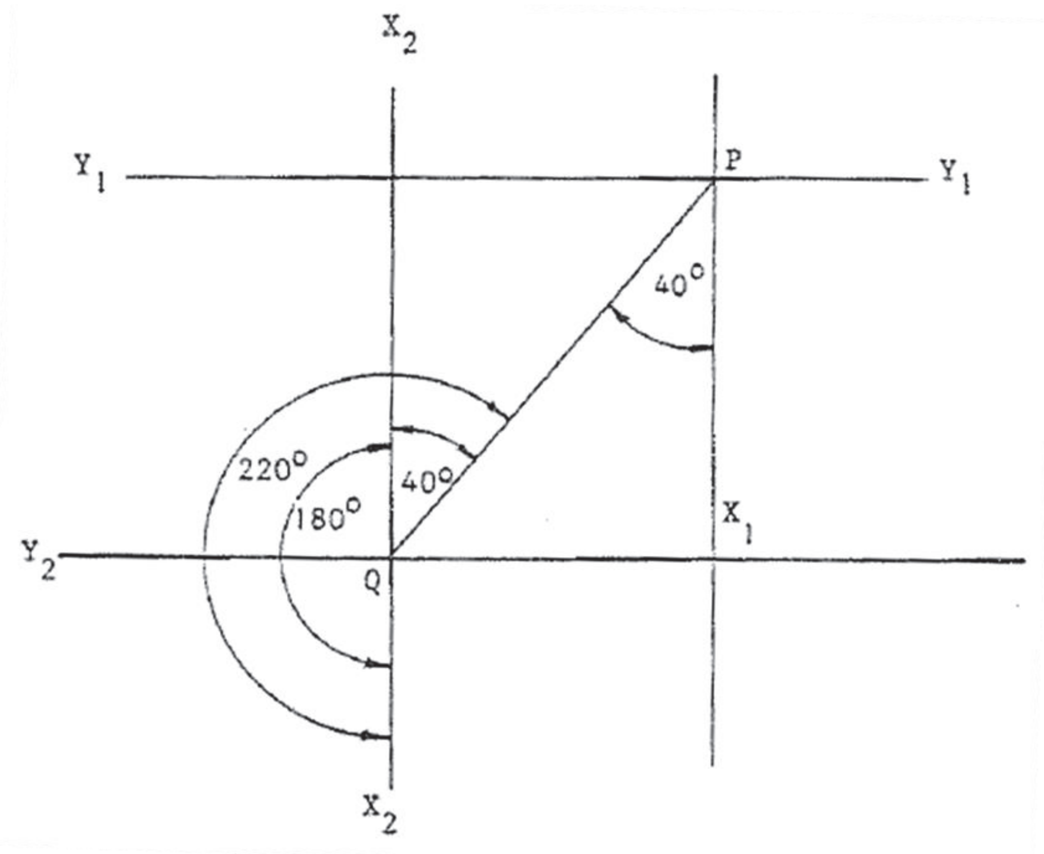


Figure 1.12

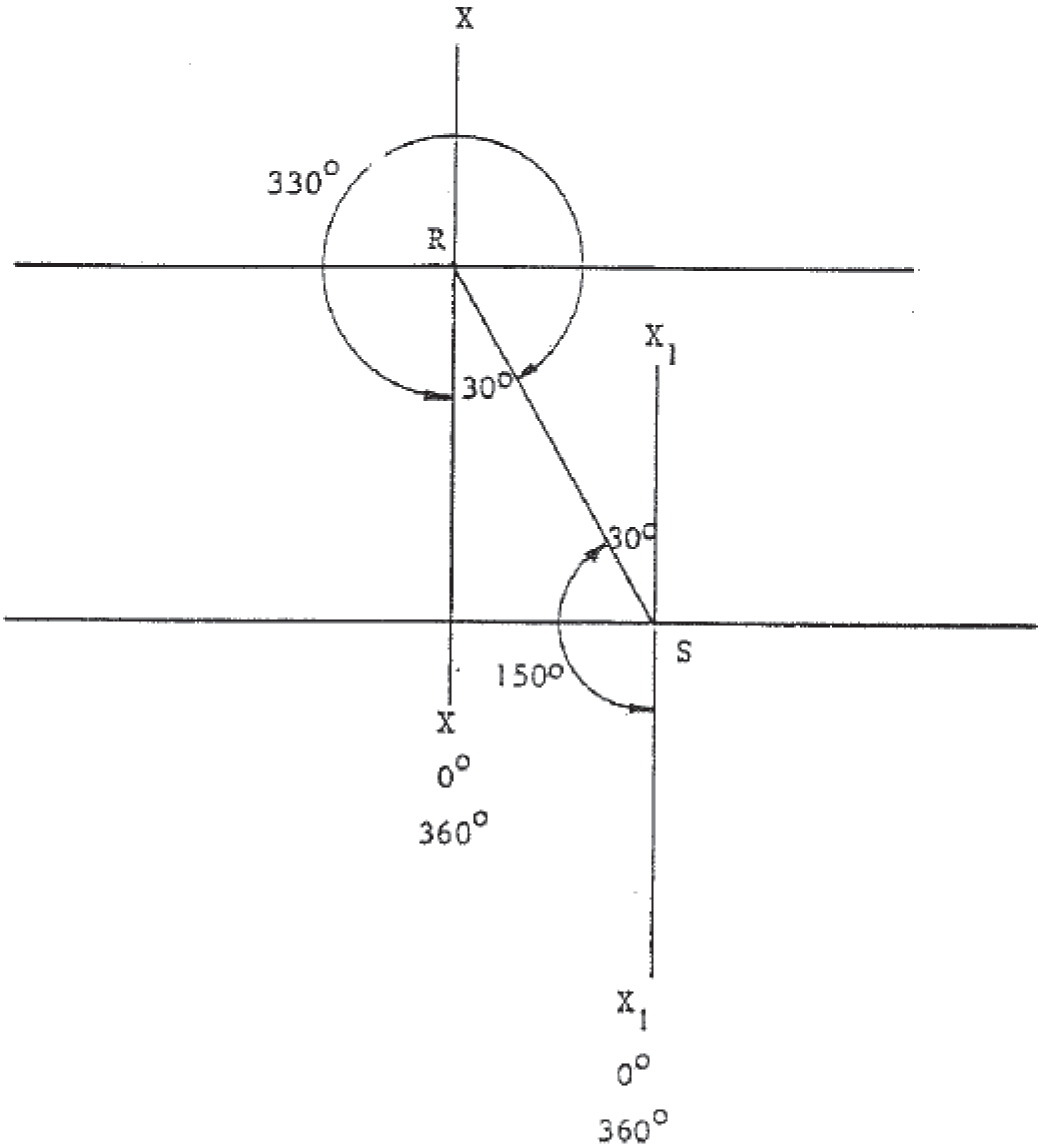


Figure 1.13

SR is known as the "reversed direction" of RS.
The following rules are useful when reversing a given direction:
(a) If the given direction is smaller than 180°, then add 180° to it.
(b) If the given direction is greater than 180°, then subtract 180° from it.

Examples:

AB = 20°	∴ BA = 200°
PQ = 197°	∴ QP = 17°
MR = 180°	∴ RM = 0°
TS = 179°	∴ ST = 359°

1.25 deduction of horizontal angles from given directions

In **Figure 1.14a** QP = 61° and QP = 118°.

By subtracting xQP from xQR, angle PQR is found, thus:
xQR = 118°
- xQP = 61°
$P\hat{Q}R$ = 57°

In **Figure 1.14b** it should be noted that line NM lies west of the zero direction while line NK lies east of it.

In this case then to find angle KNM, 360° is added to the directional value of NM, ie, xNM = 27° + 360° = 387°.

Now xNK is subtracted as before, from xNM in which case angle KNM will be found. The working is shown below:

xNM = 387°
xNK = 318°
∴ KNM = 69°

Generally
To find any required angle, subtract the smaller direction from the larger: the resultant angle will be that one measured clock1vise from the line having the smaller direction to the line having the larger direction.

The following points must, however, also be noted:
(i) Both directions should run outwards from the intersection point. (Reverse if necessary.)
(ii) (ii) If required, the smaller given direction may be changed to become the larger one by adding 360° to it (as was the case in **Figure 1.14b**).

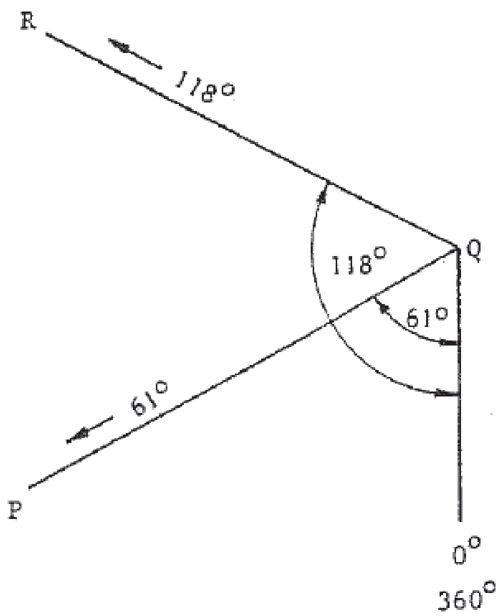


Figure 1.14a

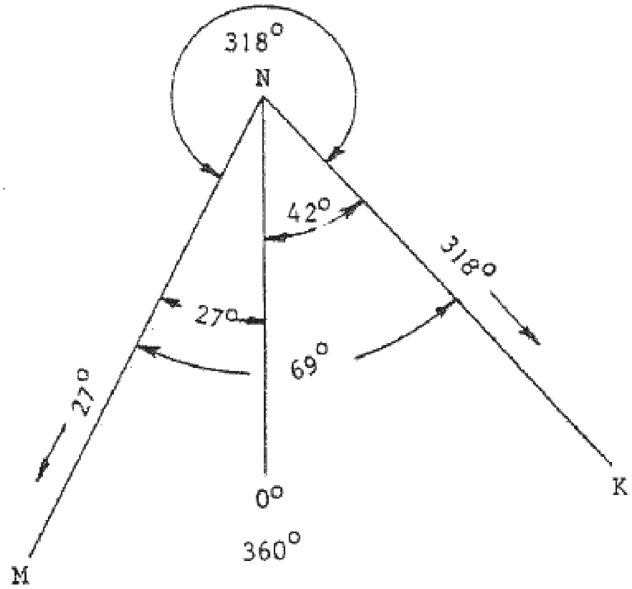


Figure 1.14b

1.26 Measurement of angles by theodolite

From the angles of direction, direction angles or directions measured by a theodolite, horizontal angles may be deduced. By definition, directions are measured in South Africa from the positive direction of the X-axis.

In some small surveys, however, assumed directions may be used, and, in setting out curves, deflection angles are employed, which are often reckoned from a zero direction assumed for the survey concerned.

1.26.1 Orientation

Whatever system is used, the instrument must be correctly ORIENTED.

In other words, the instrument, whether it is a plane-table with an alidade or a magnetic compass or a theodolite, must be set in such a way that it and the resulting survey can be related directly to the features on the ground, much the same as you would orientate yourself. with a map by pointing both the map and your eyes (the instrument in this case) to the north.

When using a theodolite on a traverse survey it is usual to set the horizontal angle at 0° on a known base line and then proceed with the survey. But when doing a survey which must be related to co-ordinates, the direction from the point you have set up at to the point you are orienting on is more likely to be any one of 360 angles and very seldom a round figure, eg, 176°53'28".

The methods of orienting instruments vary from instrument to instrument and will be discussed as each instrument is covered, some in this course and others in more advanced courses.

The instrument is now oriented and, when the reading is set to 0°, the telescope will point in the direction of the chosen zero. Readings to all other points will now be related to this chosen zero.

1.26.2 Precautions

As the accuracy of the whole survey depends upon the accuracy of the orientation, certain precautions are necessary, which must be observed at all times.

(1) Centre the instrument as accurately as possible over the instrument station.
(2) Always orient on the farthest possible points, consistent with good visibility. This is probably the most important rule in survey work.
(3) Always orient on an object which can be sighted with the utmost accuracy. For close work, such as traversing and setting out, where pegs are used and a sighting object is employed, a ranging rod is better than a tache staff, and a chaining arrow is better than a ranging rod.
(4) Always make sure that the sighting object is truly vertical.
(5) Try to keep lines of sight well clear of the ground to minimise the effects of irregular refraction close to the ground.

(6) Whenever possible, make a small mark on the top of the peg, so that the sighting object may be held at the same spot each time, and the instrument may be centred over this same spot. This is an easy matter with wooden pegs, where a small hole may be made with the point of the ranging rod or an arrow.
(7) When observing from a point, the position of which has not yet been fixed, it is usually not possible to check the orientation. At all other times, however, at least one additional known point must be sighted to check the orientation setting.
(8) Upon completion of the observations at a station, the orienting point must again be sighted to ensure that the instrument has not been moved accidentally.
(9) Always make final settings of tangent screws or micrometer drum with a clockwise movement, to counteract any play there may be in the mechanism, and to ensure that return springs are being compressed.

The above precautions are of the utmost importance and cannot be stressed too strongly. Most of these precautions are equally applicable to measurement, described below.

1.26.3 Measurement

Having oriented the instrument, it is simply necessary to sight the required points, bearing in mind the precautions listed above. The reading of the horizontal circle will give the direction to the point.

In South Africa most calculations are based upon directions, but, if the angle between two points is required, this may be derived by subtracting the smaller direction from the larger.

1.26.4 Setting-off angles

Directions are set off by setting the horizontal circle reading to the required direction. Angles may be set off by applying the required angle to the direction of the known point. If the angle is to the right of the known point, it is added. If it is to the left, it is subtracted.

When the work entails only the setting-off of angles, it is usual to use 0° as the orientation direction to the known point. When the known point is not used for orientation, the instrument may be so orientated that it would read 0° if the known point were sighted.

1.26.5 Accuracy

Although requirements vary tremendously, it is fairly safe to adopt the principle that, when a reasonably high standard of accuracy is required in the final results, angles and directions should be read to single seconds when the distance exceeds 300 metres, and to the nearest ten seconds for shorter distances.

This does not, however, preclude the use of instruments which may be read or estimated to only ten or even twenty seconds, for distances greater than 300 metres, for results may be greatly improved by the use of correct observing techniques.

For most normal purposes, single readings give sufficient accuracy, provided the instrument is in adjustment. Often however, readings are required to the greatest possible accuracy. In this case, the principles of transitting and multiple observations are employed.

The majority of theodolites are normally used in the "face left" position, ie, the vertical circle is to the left of the telescope. When we "transit" or "plunge" the telescope, ie, rotate it through 180° in the vertical plane, so that it points in the opposite direction, the instrument is brought into the "face right" position, ie, the vertical circle is to the right of the telescope, and the directions to all points are automatically altered by 180°.

The readings taken face left and face right will seldom differ by exactly 180°, but, by taking the means of these readings, certain errors of manufacture and adjustment are cancelled out.

A number of readings are taken to each point, alternatively in the face left and face right positions, to cancel out human errors, such as setting and reading errors, and any play there may be in any of the working parts. If, in addition, the orientation is altered by a predetermined amount after each set of readings, errors of graduation of the circle are reduced.

1.27 Example of orientation

Worked Example 1.2

The direction of a line A to B is known to be 120.06.00. With the instrument set up at B, the fallowing observations were recorded:

at B	
to A	300.05.20
to C	191.09.00.

Calculate:
(a) the orientation error (or correction, abbreviated, ct.)
(b) the correct direction (oriented) BC.

Solution and explanation:
The known direction AB is given from A towards B, but, since the instrument was set up at B, we need to reverse the given direction thus:

BA = 120.06.00 + 180.00.00 = 300.06.00.

For correct orientation, the reading towards A must equal 300.06.00, however, the observed direction = 300.05.20. The difference between true and observed is therefore +00.00.40, since the observed direction is smaller than the true direction.

Thus, error in orientation (orientation correction) = +00.00.40.

The orientation correction is now added to all other observed directions from A. In this case it is added to observed direction BC to give the correct (oriented) direction BC. The calculation is as follows:

Observed BC = 191.09.00
Orientation ct. = +00.00.40
Oriented BC = 191.09.40

1.28 Measurement of angles with a compass

The angles measured with a magnetic compass are called BEARINGS.

Bearings are true geographical angles of direction and are related to True North or True South.

Hence the bearing of a line and its direction on the co-ordinate system will only coincide when the point of measurement is situated on the Central Meridian. The word "bearing" should thus not be used when dealing with a rectangular co-ordinate system.

For short lines, their directions on the map are virtually the same as their bearings on the spheroid, and as a result, the shapes of small figures are similar to their shapes on the spheroid although, due to scale distortion, their areas are increased proportionally to their distances away from the central meridian.

1.29 Trigonometrical functions of directions

The principles introduced earlier are further developed to a broader usage of trigonometrical functions, where one must think in terms of all the basic functions of directions, combined with the algebraic signs generated by these functions in the various quadrants.

This principle is equally applicable to all angles between 0° and 360°.

In the following explanation the distance, $\underline{S}$, is always positive; D denotes the direction, while A denotes the functional angle.

In the first and third quadrants, the direction, in each case, has the same function as the functional angle, eg in the third quadrant, sin D = sin A.

In the second and fourth quadrants, however, the direction has the same function as the angle B, which equals the co-function of the angle A, eg in the second quadrant, sin D = sin B = cos A.

The method of derivation of the functions to be used, of the functional angles, ana particularly, of the applicable algebraic signs, is indicated below for sine, cosine and tangent of directions in the four quadrants.

The remaining functions, cosecant, secant and co-tangent are simply reciprocals, and require no further clarification.

FIRST QUADRANT

$\frac{+\Delta Y}{+S} + \sin A = \sin D$

$\frac{+\Delta X}{+s} = +\cos A = \cos D$

$\frac{+\Delta Y}{+\Delta X} = +\tan A = \tan D$

SECOND QUADRANT

$\frac{+\Delta Y}{+S} = +\cos A = \sin D$

$\frac{-\Delta X}{+S} = -\sin A = \cos D$

$\frac{+\Delta Y}{-\Delta X} = -\cot A = \tan D$

THIRD QUADRANT

$\frac{+\Delta Y}{+s} = -\sin A = \sin D$

$\frac{-\Delta X}{+S} = -\cos A = \cos D$

$\frac{-\Delta Y}{-\Delta X} = +\tan A = \tan D$

FOURTH QUADRANT

$\frac{-\Delta Y}{+S} = -\cos A = \sin D$

$\frac{+\Delta X}{+S} = +\sin A = \cos D$

$\frac{-\Delta Y}{+\Delta X} = -\cot A = \tan D$

For easy reference, the following table gives the functions to be used, together with their algebraic signs.

Function required	**1st Quadrant A = D**	**2nd Quadrant A = D - 90°**	**3rd Quadrant A = D - 180°**	**4th Quadrant A = D - 270°**
sin D	+ sin A	+ cos A	- sin A	- cos A
cos D	+ cos A	- sin A	- cos A	+ sin A
tan D	+ tan A	- cot A	+ tan A	- cot A
cosec D	+ cosec A	+ sec A	- cosec A	- sec A
sec D	+ sec A	- cosec A	- sec A	+ cosec A
cot D	+ cot A	- tan A	+ cot A	- tan A

Table 1.3

From a little study of this table it is easy to conclude that we can say:

ΔY always = S sin D

ΔY always = S cos D

and the signs of the differences will automatically be correct. You should try some examples for yourself to verify this fact.

By using the functions of directions, most rules and formulae can be stated without ambiguity, and the student is advised to adopt this method wherever possible, after he has become thoroughly familiar with the more elementary method.

1.30 Calculation of the polar

The formulae for the calculation of the co-ordinates of an unknown point has been discussed and it remains only to put everything together, and in a suitable form. The basic calculation is illustrated in the following examples:

Worked Example 1.3: Direction in 1st quadrant

Calculate the co-ordinates of a point P, from the following data:
(i) Direction BP = 65° 10' 00"
(ii) Distance BP = 80,00 m (Horizontal)
(iii) Co-ordinates B + 600,00 + 1 200,00.

Solution:

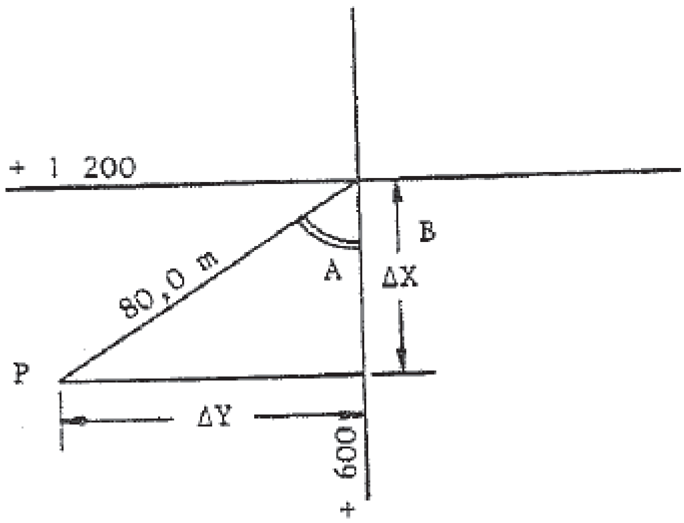


Figure 1.15

The Y co-ord of P = y co-ord. of B + ($\pm \Delta Y$)
The X co-ord of P = X co-ord of B + ($\pm \Delta X$)
The direction BP = 65° 10' 00"
∴ Functional angle (A) = 65° 10' 00"

ΔY is positive ie $+\Delta Y$ (Direction in 1st quadrant)

ΔX is positive ie $+ \Delta X$ (Direction in 1st quadrant)

$+ \Delta Y$	= S sin A	$+ \Delta X$	= S cos A
	= 80 sin 65° 10'		= 80 cos 65° 10'
log 80	= 1,903 090 0	log 80	= 1,903 090 0

+ log sin 65° 10'	= 9,957 862 6	+ log cos 65° 10'	= 9,623 228 7
	1,860 952 6		1,526 318 7
ΔY	= + 72,60	ΔX	= + 33,60
	Y		X
	[B] + 600,00		+ 1 200,00
	ΔY + 72,60		ΔX + 33,60
	[P] + 672,60		+ 1 233,60

The co-ordinates of P are thus + 672,60 + 1 233,60.

CHECK:

$\tan(\propto +45°) = \frac{\Delta X+\Delta Y}{\Delta X+\Delta Y}$ (for 1st and 3rd quadrants)

$\tan(65°10' + 45° = \frac{33{,}60+ 72{,}60}{+33{,}60-72{,}60}$

$\therefore -2{,}723 = -2{,}723$

Note:
In 2nd and 4th quadrants use:

$$\tan(\propto +45°) = \frac{\Delta X + \Delta Y}{\Delta X + \Delta Y}$$

Worked Example 1.4: **Direction in 2nd quadrant**

Calculate the co-ordinates of point R from:
(i) Oriented direction TR = 139° 30' 00''
(ii) Horizontal distance TR – 200,00 m
(iii) [T] + 625,00 + 78,00

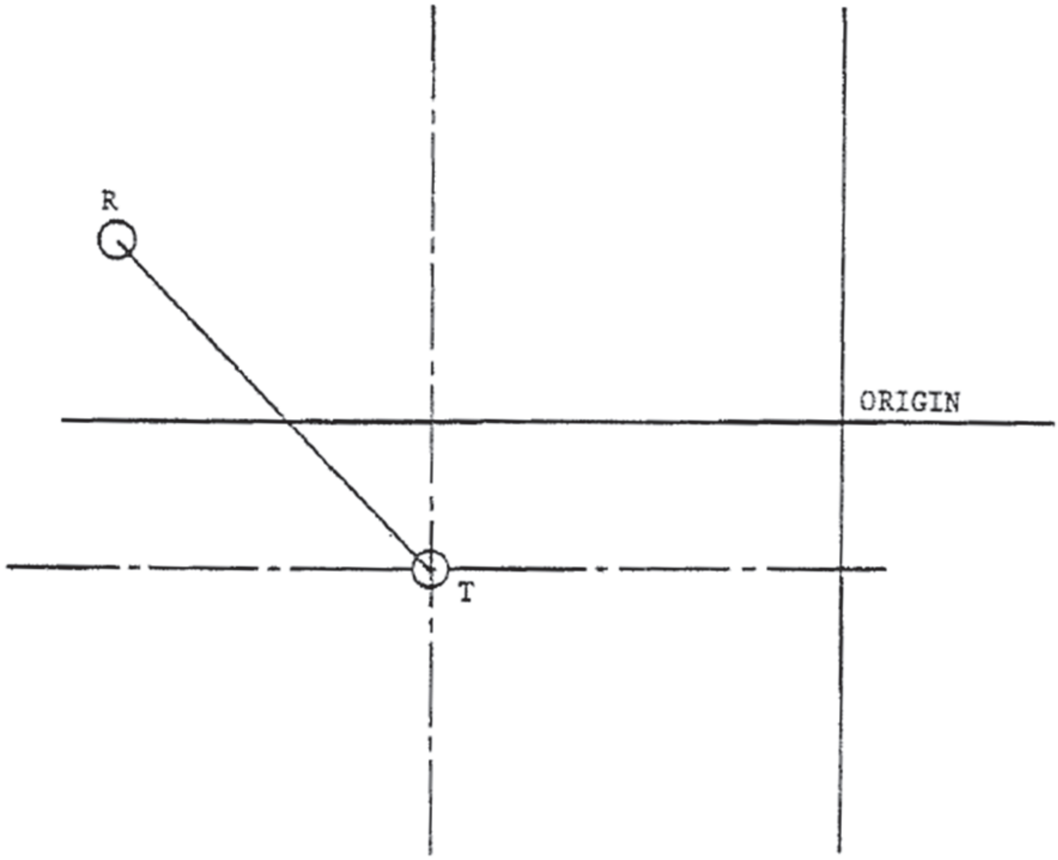


Figure 1.16

Solution:

(Direction) TR	= 139° 30' 00"	
(Functional angle) A	= 49° 30' 00"	(139° 30' - 90°)
Distance TR	= S = 200,00	

In the 2nd quadrant ΔY positive and ΔX negative

ΔY	= S cos Â	ΔX	= S sin Â
∴ ΔY	= 200 cos 49° 30' 00"	ΔX	= 200 sin 49° 30'
log 200	= 2,301 030 0	log 200	= 2,301 030 0
+ log cos 49° 30'	= 9,812 544 4	+ log sin 49° 30'	= 9,881 045 5
log ΔY	2,113 574 4	log ΔX	2,182 075 5
ΔY	= + 129,89	ΔX	= - 152,08
[T]	= + 625,00		+ 78,00
[R]	= + 754,89		+ 74,08

CHECK:

$\Delta Y + \Delta X = \sqrt{2}\, S.\sin(\theta + 45°)$

$+129{,}89 - 152{,}08 = \sqrt{2}\,.\,200{,}00\, Sin\,(139°\ 30' + 45°)$

$-22{,}19 = -22{,}19$

ANSWER: Co-ordinates R + 754,89 – 74,08

Worked Example 1.5: Direction in 3rd quadrant

Calculate the co-ordinates of a point Q from the following data:
(i) Oriented direction RQ = 220° 16' 00"
(ii) Horizontal distance RQ = 300,00
(iii) Co-ordinates of R + 754,89 - 74,08

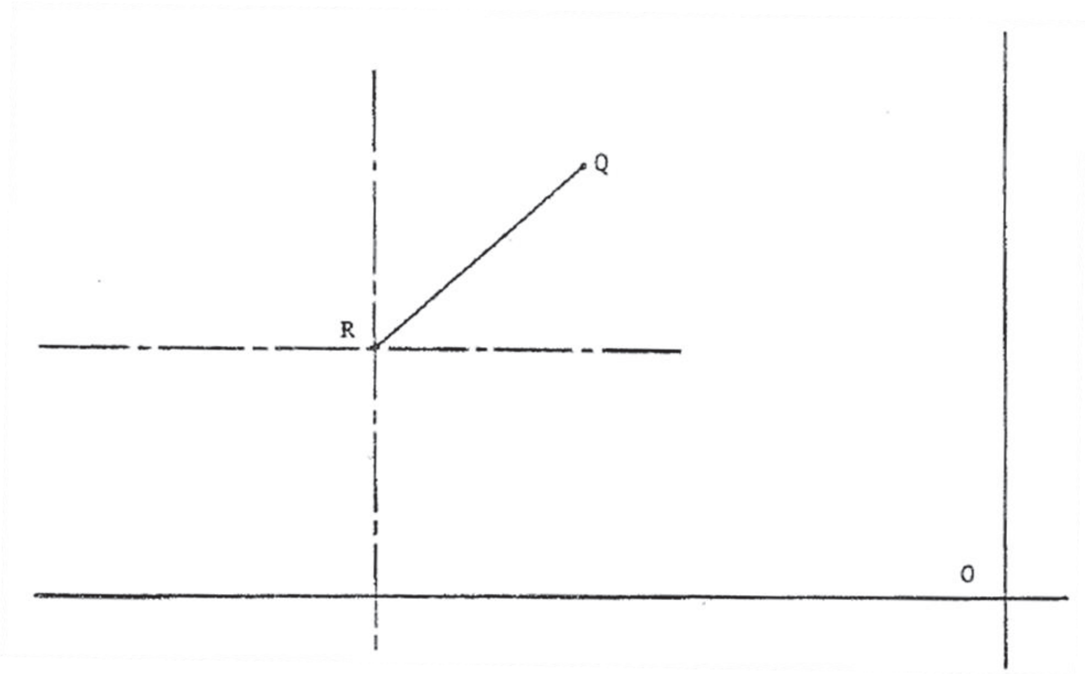


Figure 1.17

Solution:

(Direction) RQ = 220° 16’ 00”
Functional angle A = 40° 16’ 00”
Distance RQ = 300,00

In this example the direction falls in the 3rd quadrant, where both ΔY and ΔX are negative.

In the 3rd quadrant:

ΔY	$= S \cos \hat{A}$	ΔX	$= S \sin \hat{A}$
	= 300 sin 40° 16'		= 300 cos 40° 16'
log 300	= 2,477 121 3	log 300	= 2,477 121 3
+ log sin 40° 16'	= 9,810 465 0	+ log cos 40° 16'	= 9,882 549 9
	2,287 586 3		2,359 671 2
ΔY	= - 193,90	ΔX	= - 228,91
[R]	= + 754,89		- 74,08
[Q]	= + 560,99		- 302,99

Co-ordinates Q + 560,99 – 302,99

CHECK:

$\tan(\propto +45°) = \frac{\Delta X+\Delta Y}{\Delta X+\Delta Y}$

$\tan(40°16' + 45° = \frac{-228{,}91-193{,}90}{-228{,}91+193{,}90}$

$12{,}077 = 12{,}077$

Worked Example 1.6: Direction in 4th quadrant

Calculate the co-ordinates of a point V from the following data:
(i) Oriented direction QV = 323° 00' 00''
(ii) Distance QV = 150,00 m
(iii) Co-ordinates Q = + 560,99 - 302,99

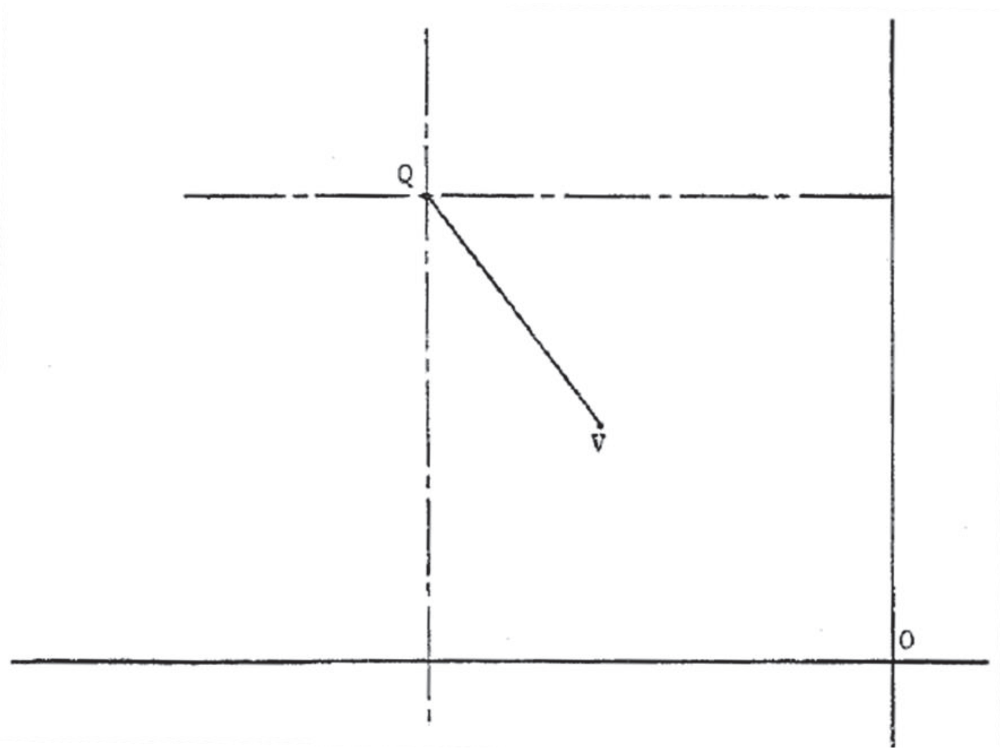


Figure 1.18

Solution:

Direction falls in the 4th quadrant where ΔY is negative and ΔX is positive.

QV = 323°
Â = 53°
Distance QV = 150,00

ΔY	= S cos Â	ΔX	= S sin Â
	= 150 cos 53°		= 150 sin 53°
log 150	= 2,176 091 3	log 150	= 2,176 091 3

+ log cos 53°	= 9,779 463 0	+ log sin 53°	= 9,902 348 6
	1,955 554 3		2,078 439 9
ΔY	= - 90,27	ΔX	= + 119,80
[Q]	= + 560,99		- 302,99
[V]	= + 470,72		- 183,19

Co-ordinates V + 470,72 – 183,19

CHECK:

$\Delta Y + \Delta X = \sqrt{2}\, S.\cos(\theta + 45°)$

$+119{,}80 + 90{,}27 = \sqrt{2}\,.150{,}00\, cos\,(323° + 45°)$

$210{,}07 = 210{,}07$

1.31 Revision

At this stage you should fully understand the following:

- The symbol D denotes the direction of a line.
- The symbol A denotes the functional angle of a direction.
- The symbol S denotes the horizontal distance between two points.

IN 1ST QUADRANT	LOOK UP
+ΔY = S sin D =	S sin A
+ΔX = S cos D =	S cos A
IN 2nd QUADRANT	
+ΔY = S sin D =	S cos A
-ΔX = S cos D =	S sin A
IN 3rd QUADRANT	
-ΔY = S sin D =	S sin A
-ΔX = S cos D =	S cos A
IN 4th QUADRANT	
-ΔY = S sin D =	S cos A
+ΔX = S cos D =	S sin A

1.32 Surveyors' staffs

Surveying staffs are made in varying lengths and for all possible purposes.

- Leveling staffs for line and area leveling;
- Invar staffs for geodetic control and high-precision leveling in civil and mechanical engineering;
- Industrial staffs (invar) for precise heighting in laboratories and workshops;
- Tacheometric staffs and Topographic staffs for stadia survey.

The staff is an important element in survey equipment; good results in leveling and tacheometry depend on good staffs as much as on good instruments. The staff must be precise and robust and it must be possible to read it without doubt.

Very often, instruments are blamed for bad results whereas in fact a low-quality staff is the sole reason for faulty reading. High-precision templates are used to graduate the staffs, thus all staffs are of the same quality. The factory tolerance for a graduation interval is ± 0,2 mm (± 0,02 mm for invar staffs) and the overall length of a 3 m staff, measured from its base plate, is accurate within± 0,8 mm (± 0,09 mm for invar staffs).

All Hild staffs are painted orange on the rear side so as to be highly visible on roads and construction sites.

A staff is a graduated rod made of wood or aluminium alloy, with a steel plate on each end so that the possibility of wear affecting results is reduced. Be sure not to buy an inferior staff.

Three basic types are:
(a) A telescop1c staff, made in 3 sections which telescope together for carrying convenience. The lengths range from 3 to 6 m extended (**Figure 1.19**).
(b) The solid two or three-section staff in which the sections are socketed together.
(c) The one or two-hinged staff, in two or three sections, respectively. These fold flat for transportation. Also growing in popularity is the addition of handles on the sides of staffs which help the staff man to hold the staff steady and vertical, usually with a staff bubble (**Figure 1.21**) or staff level.

Change plates (**Figure 1.23**) are used to put the staff on when the ground is soft or uneven.

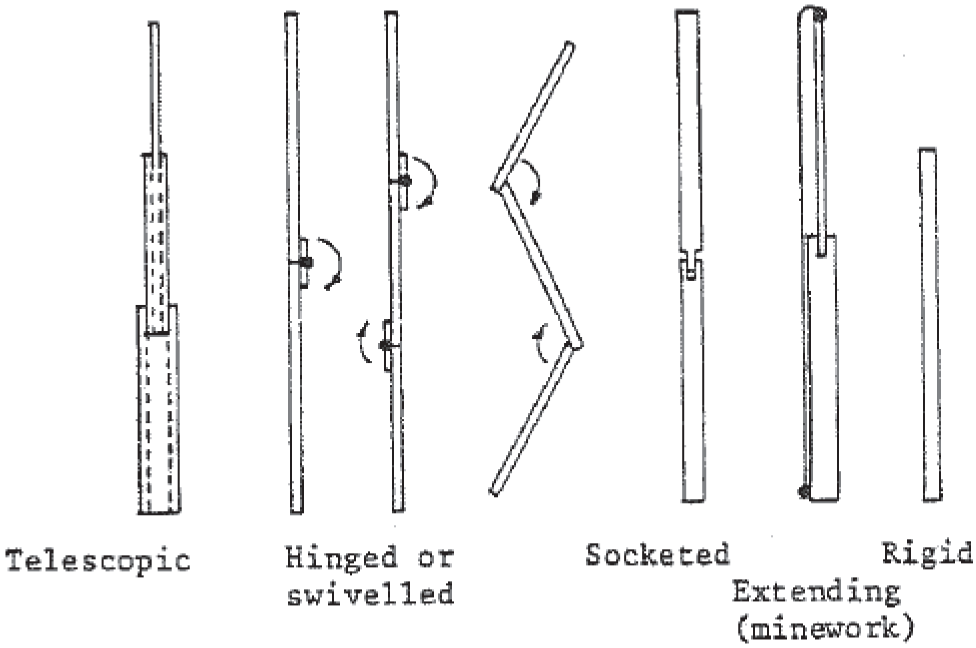


Figure 1.19 6.1 Staff arrangements

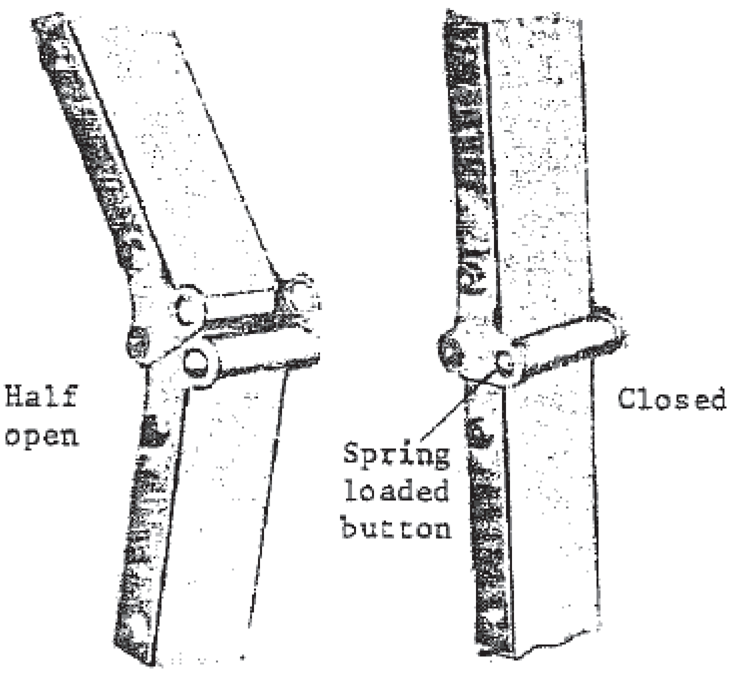


Figure 1.20 6.2 Typical hinge

Figure 1.21 6.3 Angle bracket staff levels

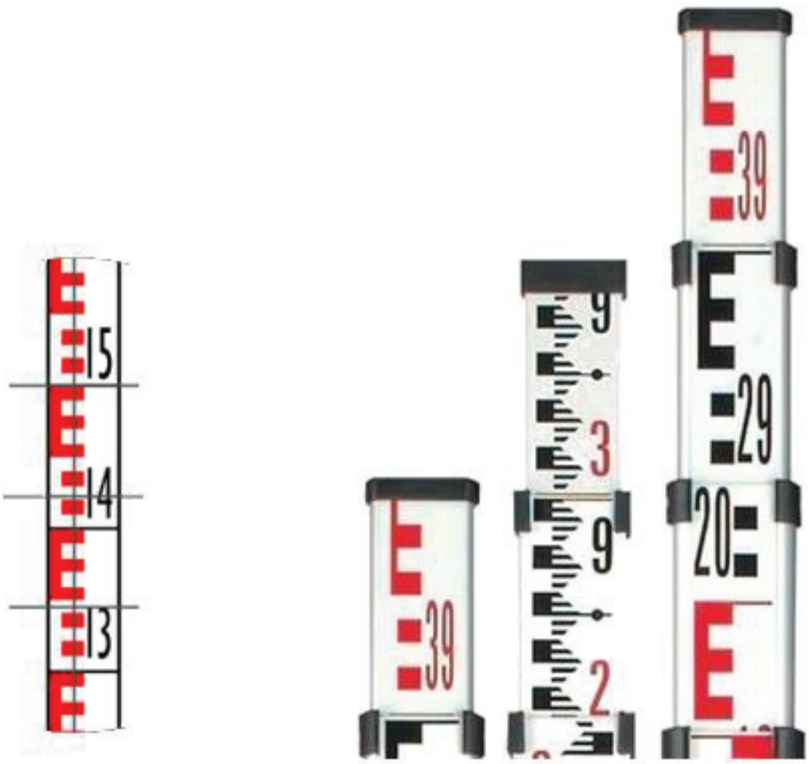


Figure 1.22 6.4 Patterns

Figure 1.23 6.5 Triangular change plates with chain, also for working in soft ground

There are many different patterns, too numerous to show.

Figure 1.22 shows some patterns and their applications. The "stadia" system used for tacheometry will be discussed later.

1.32.1 Other types of levelling staff

A variety of other types of staff have been made, the most notable probably being the "Philadelphia" pattern target staff. This is fitted with a target and a vernier scale, and the instrument-man directs the staff-man to move the target up or down on the staff until it is on the collimation line.

The staff-man then reads the staff and obtains the final place of decimals from the vernier scale. Popular in the USA, but little used in South Africa.

Normal leveling staves with inverted numbers are available - the figures appear the right way up when viewed through an inverting telescope.

These are more popular on the continent than in the RSA, and are unsuitable when used with an erecting telescope.

Some "self-reducing tacheometers" require specially designed staffs, but generally the choice of staff pattern is left to individual preference for the particular job in hand. Different colours are available and a surveyor will find from experience which colour and pattern suit him best.

Some surveyors have designed their own patterns and there are companies in the RSA that will make staffs to order (eg, Stewart Instrument Co). The important thing to do when buying a new staff, no matter whose design it is, is to check that the graduations are accurate.

1.33 The stadia system

This system is by far the most commonly used. The theory is more involved than that of the tangential system, but the field work is simpler, and the office work has been so mechanised that it is no more arduous.

In its simplest form, the instrument consists simply of a theodolite, the diaphragm of which is provided with two additional hairs, known as stadia hairs, one above and one below the horizontal cross-hair, and equidistant from it.

These hairs subtend a fixed angle at the centre of the instrument (or near it). If a graduated staff is held vertically in the line of sight, the hairs will be seen to subtend a certain length, which increases in proportion to the distance of the staff from the instrument.

Obviously then, if we know the proportion between the intercepted distance on the staff and the distance of the staff from the instrument, we can easily deduce the latter from the former. The factor by which the staff intercept must be multiplied to obtain the distance is almost invariably fixed at 100.

Some interesting variations of this principle will be discussed later.

Although the term can lead to some argument, any theodolite fitted with stadia hairs can, for practical purposes, be called a tacheometer or taché. A tacheometer is, in fact, a superior type of theodolite, and not necessarily, as is so often presumed an instrument of inferior accuracy.

Generally, however, the instruments used for tacheometry are of less angular accuracy than those used for, say, triangulation. This is possible, since distances are comparatively short. The instruments are cheaper, more rugged and faster in operation.

1.33.1 Theory of the stadia svstem

In the now obsolete external focusing telescope, the distance was measured to a point in front of the object glass, equal to the focal length of the object glass, so that a constant, equal to the focal length plus the distance from the optical centre of the object glass to the vertical axis of the instrument, had to be added to all readings.

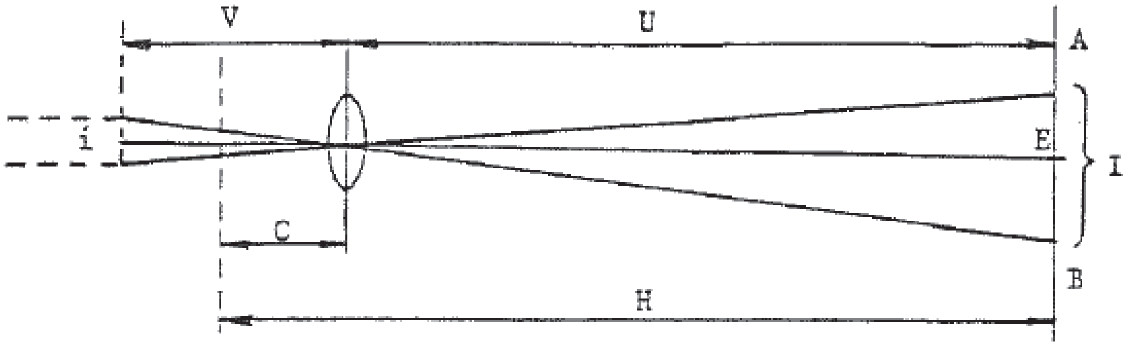
This is known as the stadia constant. This difficulty was later overcome by the addition of a special lens, the analactic lens, which caused the light rays to intersect at the vertical axis. In the short modern internal focusing telescope, however, the analactic lens is not required, for the arrangement of the lenses is such that the value of the constant is either nil or quite insignificant.

To simplify the mathematics, therefore, **Figure 1.24** shows the rays intersecting at the vertical axis, and passing through the object glass as straight lines.

The focusing and eyepiece lenses are omitted altogether. Although this simplification omits all the bends in the light rays, the theory remains basically similar, but is greatly simplified.

There are two sets of conditions which are encountered in normal practical tacheometry, and these will be dealt with separately.

Case 1: Staff Vertical: line of sight horizontal



f = focal length

Figure 1.24

f = focal length of object glass
i = distance between stadia hairs
I = staff intercept = AB
U = horizontal distance from the staff to the optical centre of object glass
V = horizontal distance from the hairs to the optical centre of the object glass
C = horizontal distance from the object glass to the vertical axis of the instrument
D = distance from the axis to the staff

$\frac{u}{v} = \frac{I}{i}$ (sides of similar triangle)

$\therefore$ U $= \frac{VI}{i}$ .... (1)

$\frac{1}{f} = \frac{1}{u} + \frac{1}{V}$ .... (2) (principal lens formula)

In (2), multiply throughout by the LCM which is fUV.

$\therefore$ uv = fv + fu

$\therefore$ u $= f + \frac{fu}{v}$ .... (3) (dividing by v)

Substitute (1) in (3)

$\therefore$ u $= f + \frac{fvI}{vi}$

$= f + \frac{fI}{i}$

Adding C

$\therefore$ U + C $= \frac{fI}{i} + f + c$

D $= \frac{fI}{i} + (f + c)$ (u + c = D)

Now, $\frac{fI}{i}$ is the stadia constant, K, which is 100 for modern instruments, (f +c) is called the additive constant which is used in the older types of theodolites with external focusing in which the objective moved.

This constant varied from 0,25 to 0,5 metres and had to be taken into account. In modern telescopes this constant is negligible and is omitted. The effect of curvature and refraction may be ignored for short distances.

Thus D = $\frac{fI}{i}$ = KI = 100 I (I = stadia intercept), when the sight is horizontal.

Worked Example 1.7

A tacheometer was set up at a peg A and the following readings recorded:
On staff at B

Top hair	= 4,205
Middle hair (axial)	= 3,405
Bottom hair	= 2,605

The sight was horizontal.
The multiplying constant, K, = 100. Instrument height at A = 1,30 m.

Given that the elevation of A = +430,250 calculate:
(i) the horizontal distance AB
(ii) the elevation of B.

Solution:

D	= KI
I	= difference between stadia readings
	= 4,205 – 2,605
Thus, D	= 100 x 1,600 m
D	= 160,00 m

To calculate the elevation of B

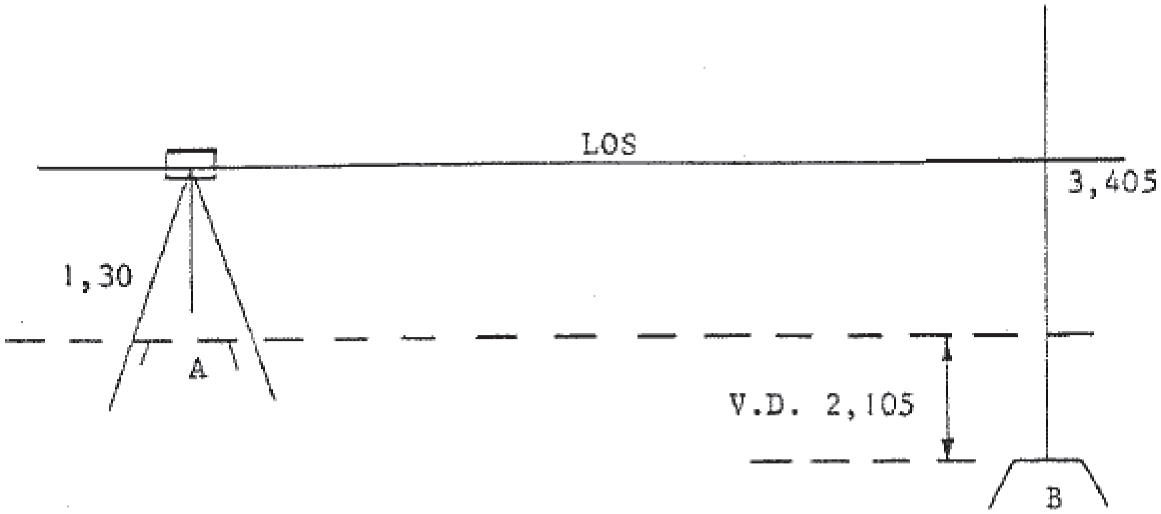


Figure 1.25

Since the line of sight is horizontal, the height difference is found directly by subtracting the instrument height from the middle wire reading.

	**Note:** When elevations are calculated, always use the MIDDLE WIRE READING.

Thus vertical difference A - B	= 3,405 - 1,30
	= 2,105
From A to B there is a fall of 2,105 m	= -2,105 m
Elevation A	= + 430,250
Vertical difference A - 8 - 2,105	
Elevation B	= + 428,145.

Case 2. Staff vertical: line of sight inclined

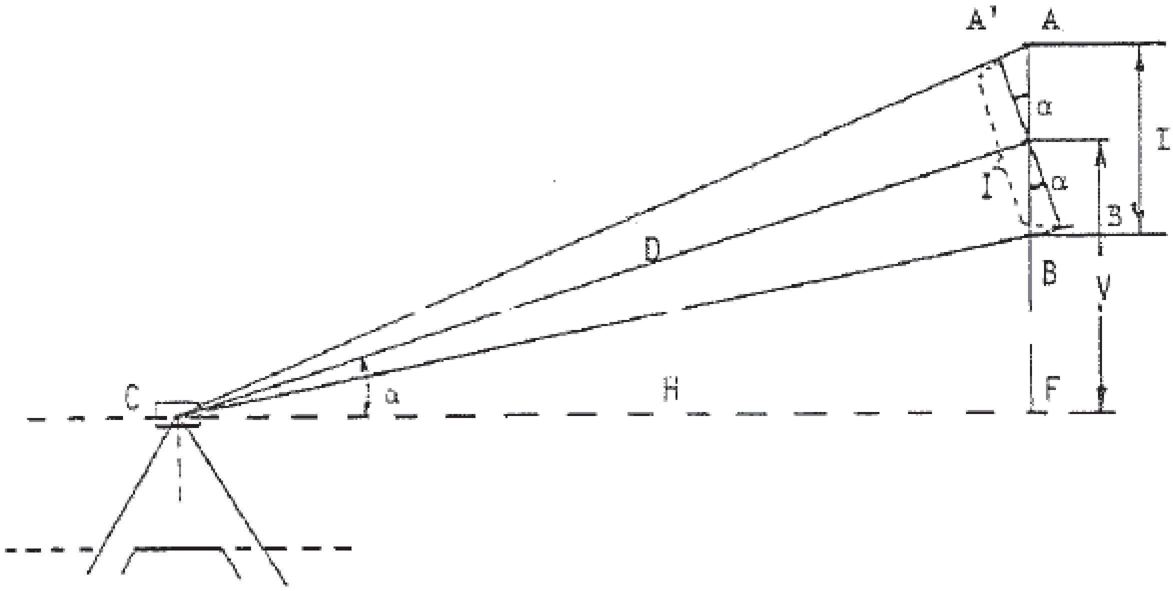


Figure 1.26

This, the more usual case, is a development of Case 1, and is illustrated in **Figure 1.26**. In this case D is a slope distance since the LOS is inclined.

The slope distance D must be multiplied by the cos of the angle of elevation or depression (a), to obtain the horizontal distance H, but as the staff is held vertically, and is, therefore, not normal (ie, perpendicular) to the line of sight CE, the staff intercept AB is greater than it would have been if the staff had been held normal to the 1 ine of sight, and consequently the distance derived from the formula D = 100 I is greater than the true distance CE or D in the figure.

The lettering of the figure is as for **Figure 1.25** with the following exceptions:

- D is the slope distance from the instrument to the point E on the staff.
- H is the horizontal distance from the instrument to a point F vertically below (or above) the staff, and on the same horizontal plane as C.

(a) Proof of formula for finding H

In the formula for horizontal distance D as in case 1 where the line of sight is horizontal, the intercept, I, is perpendicular (normal) to the line of sight.

In case 2 it is necessary to find the equivalent value (I') of I in a position to the line of sight.

In this case, then:

$$D = 100\,I' + (f + c) \text{ .... } (1)$$
$$I' = AE \cos \propto + BE \cos \propto$$
$$= (AE + BE) \cos \propto$$

$$\begin{aligned}
&= AB \cos \propto \\
&= l \cos \propto \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2) \\
\text{Substitute} \quad l' &= l \cos \propto \text{ in } (1) \\
\therefore D &= 100\, l \cos \propto + (f + c) \ldots (3) \\
\text{But } H &= D \cos \propto \ldots\ldots\ldots\ldots\ldots\ldots (4)
\end{aligned}$$

Substituting (3) in (4) we get:

$$\begin{aligned}
H &= \{100\, l \cos \propto + (f + c)\} \cos \propto \\
&= 100\, l \cos^2 \propto + (f + c) \cos \propto
\end{aligned}$$

In the case of a modern instrument we have

$$\begin{aligned}
H &= 100\ l \cos^2 \propto \\
H &= Kl \cos^2 \propto \ldots\ldots\ldots\ldots\ldots\ldots (NB)
\end{aligned}$$

(b) Proof of formula for finding V

The vertical length, V, is the difference between the instrument height and the middle wire (axial) intersection on the staff, ie, length EF in **Figure 1.26** (F is a point on the same height as C.)

In right-angled triangle CEF

$$\begin{aligned}
EF &= EC \sin \propto \\
&= D \sin \propto \quad (D = EF) \\
\text{but } 0 &= 100\ l \cos \propto + (f + c) \ldots\ldots\ldots\ldots (3) \\
\therefore EF &= \{100\, l \cos \propto + (f + c)\} \sin \propto. \\
&= 100\ l \sin \propto \cos \propto + (f + c) \sin \propto.
\end{aligned}$$

Note:

$$\begin{aligned}
\text{That } \sin \propto \cos &= \tfrac{1}{2} \sin 2\propto \\
\therefore EF &= V \\
&= 100\, l \times \tfrac{1}{2} \sin \propto + (f + c) \sin \propto \\
\therefore V &= \tfrac{1}{2} Kl \sin 2\propto + (f + c) \sin \propto \\
\text{Or} \quad V &= \tfrac{1}{2} Kl \sin 2\propto \text{ for modern instruments } \ldots\ldots \text{NB}
\end{aligned}$$

Note:

$$\begin{aligned}
\text{If } \propto \text{ is, say} &= 5^\circ \text{, then:} \\
\cos^2 \propto &= \cos^2 5^\circ \\
&= \cos 5^\circ \times \cos 5^\circ \\
&= 0{,}9\,962 \times 0{,}9\,962 \\
\text{and } \sin 2\propto &= \sin 2\,(10^\circ) \\
&= \sin 20^\circ \\
&= 0{,}3\,420.
\end{aligned}$$

The elevation of a point is found by adding together algebraically:
elevation of known point, instrument height, vertical length V, and middle wire reading. This will be seen clearly in the illustrative included in the lecture.

Summary

Case 1 Horizontal sight

(i) H = KI

(ii) V = NIL

Case 2 Inclined sight

(i) $H = KI \cos^2 \propto$

(ii) H = KI cos2

It may also be noted that $V = H \tan \propto$.

Worked Example 1.8

Reduce the following tacheometric observation (ie, calculate horizontal distance and elevation):

Set up at A	**Staff at B**
instrument height = 1,50	top hair
vertical angle A - B = + 10°00'00"	axial = 3,00
	bottom hair = 2,00

The multiplying constant, K = 100 and the elevation of A = + 450,00.

Solution:

Horizontal distance A – B $= KI \cos^2 \propto = H$

H $= 100(4{,}00 - 2{,}00) \cos^2 10°$

$= 200 \cos^2 10°$

log AB = log 200 + 2 log cos 10°

log cos 10°	= 9,993 351 5	log 200	= 2,301 030 0
2 log cos 10°	= 9,986 703 0	+2 log cos 10°	= 9,986 703 0
		log H	= 2,287 733 0
		H	= 193,97

Horizontal distance AB = 193,97 metres.

Vertical length, $V = \frac{1}{2} KI \sin 2\propto$

$= \frac{1}{2} \times 100 \times (4{,}00 - 2{,}00) \sin (2 \times 10°)$

$= \frac{1}{2} \times 100 \times 2 \times \sin 20°$

$= 100 \sin 20°$

= 34,20 metres

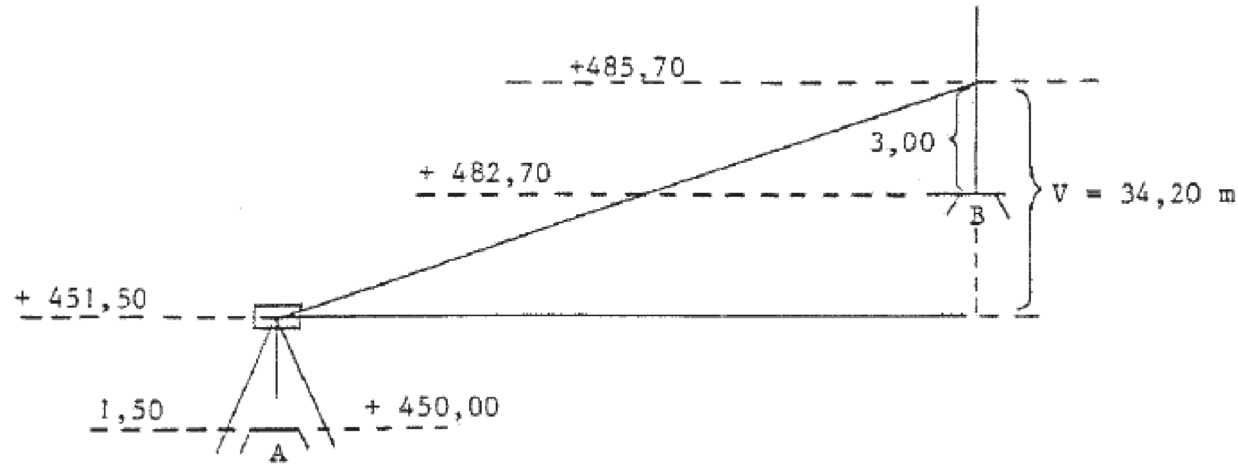


Figure 1.27

Calculation of elevation of B
See **Figure 1.27**

Elevation of A	= + 450,00
instrument height at A	= + 1,50
Height of instrument	= + 451,50
V	= 34,20 m
V	= + 34,20 (vertical angle +. from A to B)
Elev. of middle wire intercept	= + 485,70
Middle wire reading	= - 3,00
Elevation B	= + 482,70

Worked Example 1.9

Reduce the following tacheometric observation:

At P instrument height = 1,35
to $Q\frac{3,24}{1,78}$ …….. 2,51 …...... + 04°10'
K = 100;
elevation of Q = + 725,52

Solution:
(see **Figure 6.10**)
Here again we have to find the horizontal distance PQ as well as the elevation of P. Note that the elevation calculation must be reversed, because we have to work from Q back to the instrument station, P. The vertical angle P to Q = + 04°10', therefore from Q to P it is -04°10'. Also, 3,24 and 1,78 are the stadia readings while 2,51 represents the axial reading.

Horizontal distance PQ $= Kl \cos^2 \alpha$
$= 100(3{,}24 - 1{,}70) \cos^2 04°10'$
$= 100 \times 1{,}46 \cos^2 04°10'$

PQ	$= 146 \cos^2 04\ 10'$		
log cos 04°10'	= 9,998 850 6	log 146	= 2,164 352 0
		2 log cos 04°10'	= 9,997 701 2
2 log cos 04°10'	= 9,997 701 2	log PQ	= 2,162 054 1
		PQ	= 145,23 metres

Vertical length V = ½ KI sin 2α
= ½ x 100 x 1,46 x sin 08°20'
V = 73 sin 08°20'
log 73 = 1,863 3229
+ log sin 08°20' = 9,161 163 9
= 1,024 4868
V = 10,58

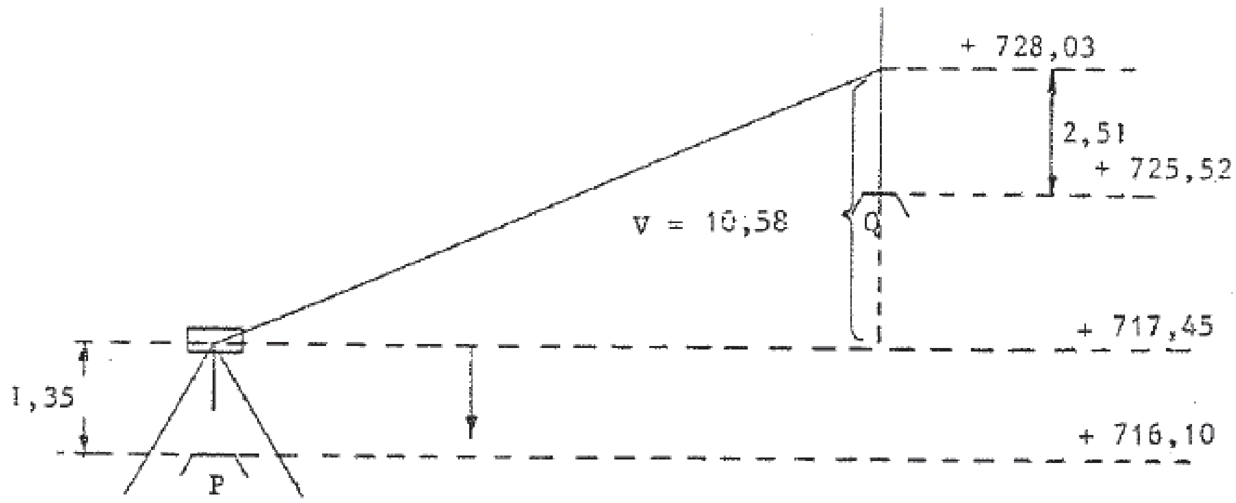


Figure 1.28

Elevation Q = + 725,52
Middle wire reading = + 2,51
Axial intercept elevation = + 728,03
V = - 10,58
Height of instrument = + 717,45
Height of instrument height = - 1,35
Elevation P = + 716,10

The above elevation may be checked by working from P forward to Q.

Answer:
Horizontal distance PQ = 145,23 m
Elevation P = 715,33 m

Worked Example 1.10

Calculate the elevation of R, from the following tacheometric observations:

Set up at A.

To P $\frac{1,60}{0,80}$ ………………….. 1,20 ………………. + 03°15'
to R $\frac{2,16}{1,60}$ ………………….. 1,88 ………………. + 07°40'
K = 100; elevation of P = + 1 015,00.

Solution:
The height of instrument can be calculated, using the observation to P. Having found it, the elevation of R may easily be found. The instrument height at A is not given since the elevation of A is not required.

Referring to **Figure 1.29**.
Let the vertical length A – P = V_1; α_1 = 03°15'
and the vertical length A – R = V_2; α_2 = 07°40'

V_1 = ½ K_1 sin $2\alpha_1$
V_1 = ½ x 100 x (1,60 – 0,80) x sin 06°30'
= ½ x 100 x 0,80 x sin 06°30'
= 40 sin 06°30'
log 40 = 1,602 060 0
+ log sin 06°30' = 9,053 858 8
log V_1 = 0,655 918 8
V_1 = 4,53
V_2 = ½ Kl sin $2\alpha_2$
V_2 = ½ x 100 x (2,16 – 1,60) sin 15°20'
= ½ x 100 x 0,56 sin 15°20'
= 28 sin 15°20'
Log 28 = 1,447 158 0
+ log sinn15°20' = 9,422 317 6
log V_2 = 0,869 475 6
V_2 = 7,40

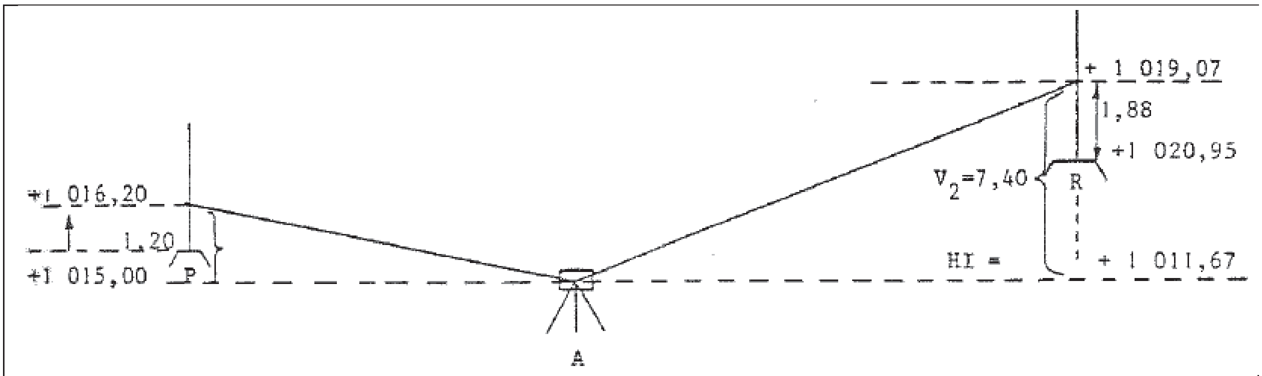


Figure 1.29

Elevation P	= + 1 015,00
Axial reading	= + 1,20
Elevation axial intercept	= 1 016,20
V_1	= 4,53
Height of instrument	= + 1 011,67
V_2	= + 7,40
Elevation axial intercept at R	= + 1 019,07
Axial reading at R	= + 1,88
Elevation of R	= + 1 020 95

Check by working from R back to P.

1.34 The level tube

1.34.1 Description and principle

Nearly all survey operations depend on the accurate determination of the direction of gravity, and therefrom, the horizontal plane at any point. The action of the level tube depends on the fact that the surface of a still liquid is a level surface.

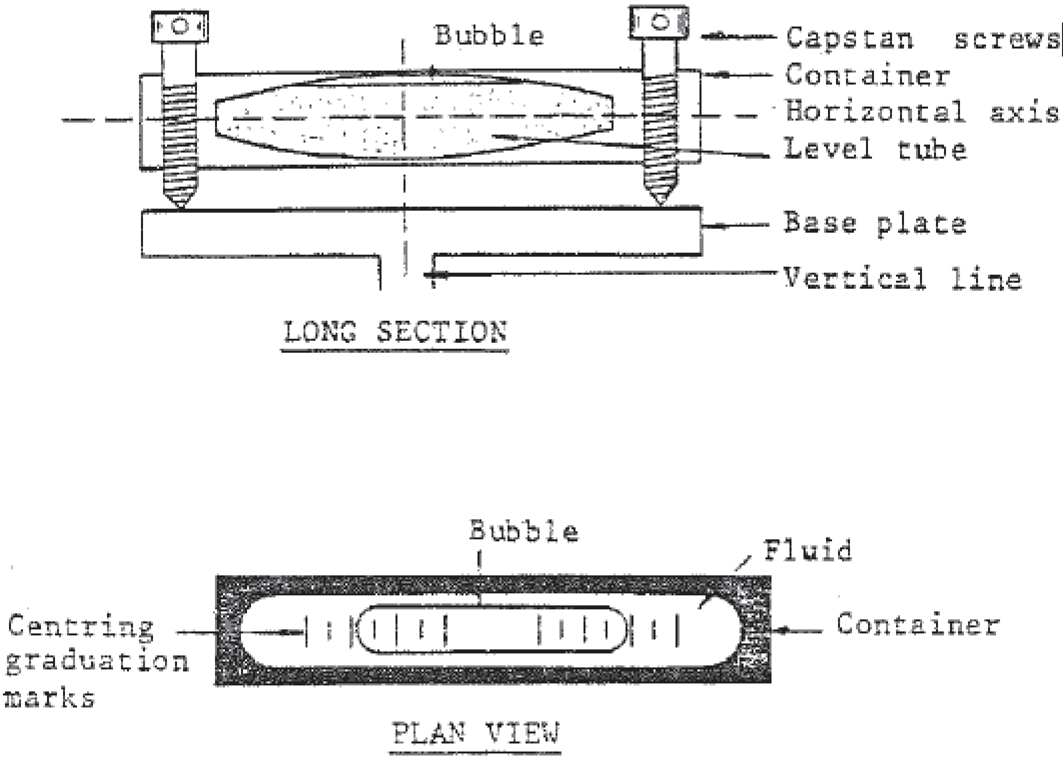


Figure 1.30

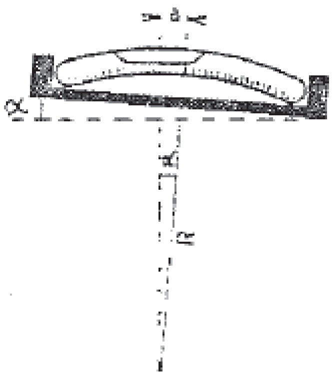


Figure 1.31

Spirit levels may be referred to as vials, level tubes, bubble tubes, or simply bubbles. The bubble tube consists of a glass tube, partly filled with a liquid such as alcohol, chloroform or ether, all of which have a lower viscosity and freezing point, but a higher coefficient of expansion than water.

The remaining space is filled with a mixture of air and the vapour of the liquid.

If a perfectly straight tube were used, and placed in a horizontal position, the bubble would simply become elongated and, if unable to extend over the whole length of tube, come to rest in any position, so that such a tube would be useless.

If, on the other hand, a curved tube is used, the bubble will always find its way to the highest part of the tube, due to the action of gravity.

The axis of the level tube is a tangent to the curve at the centre of the tube. If now the bubble comes to rest in the centre of the tube, the axis will be parallel to the surface of the liquid and, as this is truly horizontal, the axis will also be truly horizontal, ie, normal the plumbline.

The accuracy of the bubble is dependent upon its sensitivity, ie, the smallness of the angle through which the tube must be tilted to produce a visible movement of the bubble. See angle ∝ in **Figure 1.31**.

Sensitivity is dependent upon a number of factors. The greater the radius of the curve, the lower the viscosity and surface tension of the liquid, the greater the diameter of the tube and the length of the vapour bubble, and the smoother the inner surface of the tube, the more sensitive the bubble will be. Of these factors, curvature is the most important.

Stability is the degree of ease with which the bubble may be brought to rest and maintained in a given position. All the above factors influence stability in exactly the reverse way, ie, they decrease stabi1ity.

Sensitivity and stability, both desirable qualities, are therefore in direct opposition to each other, and a balance must be struck between the two to suit the purpose for which the bubble is required. A poor bubble may, however, be both unstable and not very accurate. The most likely reason for this probably is the use of a liquid which is too sensitive to changes in temperature.

In a builder's spirit level the curve of the tube is easily visible, but in a surveying instrument far greater accuracy is required, so that a curve of larger radius must be used. In this case a straight tube is used, and the inner surface is very accurately ground to the required curve.

This is usually done over the whole of the inner surface, so that the interior of the tube is barrel-shaped. Such a level tube may then also be used in the upside down position.

Graduations are usually etched into the top of the tube to assist in centring, or the bubble may be viewed through a system of prisms. One half of each end of the bubble is seen, and when the two halves coincide, to appear as one whole end of the bubble, the bubble is central.

Sensitivity is usually stated as the angle through which the tube must be tilted to move the bubble one graduation or two millimetres.

Heat affects the bubble in two ways. If more heat from the sun falls on one end of the tube than on the other, the liquid will become warmer at that end, resulting in a lowering of relative density, viscosity and surface tension, all of which tend to make the bubble creep towards the warmer end.

There is no remedy, without loss of sensitivity, for this difficulty, other than shading the bubble, and this is one of the reasons why all accurate survey work should be done under an umbrella.

Heat also makes liquid expand and, as the bubble is more compressible that the liquid, its volume will be reduced, thus shortening the bubble. This results in a loss of accuracy and, in extreme cases, may burst the tube. Shading the bubble will minimise but not cure this effect.

One method of maintaining the length of the bubble is by the use of a level tube consisting of two chambers joined together by a narrow opening. The smaller chamber contains some liquid and some air. By tilting the tube some of the liquid is allowed to run into the smaller chamber and some of the air into the larger.

In cold weather the process is reversed. In this way the proportions of air and liquid in the main chamber may be balanced to maintain the correct bubble length.

Reversal of the bubble (refer to **Figure 1.32**)

Adjustment of the bubble is accomplished by the process of reversal, and it is necessary first to understand the effects of reversal. The figure numbers referred to all fall under **Figure 1.32**.

The bubble tube may be reversed in two ways:
(a) by having the tube unattached to the base upon which it rests and
(b) by having it rigidly attached to the base.

In the former case, it is simply necessary to lift the whole tube assembly and rotate it through 180°, thereby reversing it end-for-end. in the latter case, the base must be rotated through 180°, usually about a spindle.

Figure 1.30 shows a bubble tube ab resting on, but not fixed to, a base AB such as a table top. The tube is provided with two adjustable supports, S1 and S2.

The bubble is central in the tube, indicating that the tube (or, more correctly, the axis of the tube) is horizontal. Due to the fact that the supports are of unequal length, however, the base is not horizontal, but tilted through angle t, off the horizontal.

This means also that, if the base were horizontal, the tube would be tilted through angle t, as in **Figure 1.32**.

Figure 1.32b shows the base in the same position, but the tube has been reversed to position ba. Now the tilt of the base and the inherent tilt of the tube are working together, so that the tube is now tilted off the horizontal by angle 2t.

Figure 1.32c shows the base horizontal, but the tube is tilted by angle t, as indicated by the bubble being off-centre to the right. Upon reversal to the ba position (**Figure 1.32d**), the tube is tilted in the opposite direction by the same amount, as indicated by the bubble being off-centre to the left by the same amount.

It will thus be appreciated that it is possible to obtain horizontality of the base, even if the length of the supports are slightly unequal, simply by ensuring that the bubble is off-centre, by the same amount, in both the direct and reversed positions, but in one position it must be to the right of centre, while in the other position it is to the left, ie, it must occupy the same position in the tube.

Note that it will always be off-centre towards the longer supports, in this case the b end.

While it is possible to obtain horizontality in this way, it is always more satisfactory to have tle bubble in proper adjustment.

As the adjustments required are generally very minute, any attempt to achieve equality in the length of the supports by measurement is out of the question.

The following procedure is, therefore, adopted:
Having established that maladjustment exists (**Figure 1.32b**), the base is tilted back towards the horizontal sufficiently to bring the bubble halfway back towards centre (**Figure 1.32d**), and then the supports are adjusted to bring the bubble to the centre of its run (**Figure 1.32f**).

Upon reversal (**Figure 1.32e**) the bubble should again occupy the central position. This-process may have to be repeated several times before complete adjustment, as illustrated by **Figure 1.32e** and **Figure 1.32f**, is achieved. It is most important to realise that half the error is corrected by tilting the base and the other half by adjusting the supports.

It is also important to realise that, to achieve complete horizontality of a plane surface, it is necessary only to achieve horizontality along two lines at right angles to each other. The surface will then be horizontal in all directions.

Where the bubble is attached to the base, conditions are somewhat different. **Figure 1.32g** shows the base AB which is free to rotate about a spindle S. The base, in this case, is not perpendicular to the spindle. The spindle being vertical, the base is tilted as before, but, due to the inequality of the supports, the bubble is central in the. tube.

Upon reversal, by rotation of the base to the ba position (**Figure 1.32h**), the direction of tilt of the base is reversed, but the angle of tilt remains the same. As the bubble tube rotates with the base, the positions of the supports, relative to the base, are unchanged.

The tube thus remains horizontal and the bubble remains central. The inequality in the lengths of the supports then merely compensates for the tilt of the base, and provided the spindle is vertical, the bubble wi11 remain central, irrespective of the tilt of the base.

Obviously then, a fixed bubble gives no indication of the horizontality of the base, but indicates only the verticality of the axis of rotation, which is represented here by the spindle.

Again, to ensure complete verticality of the axis, it is only necessary to ensure verticality in two bubble positions at right angles to each other.

In the theodolite, the base and spindle are the upper plate and vertical axis, respectively.

In **Figure 1.32i** to **Figure 1.32n** the spindle is shown perpendicular to the base, but lack of perpendicularity would have no material effect, other than to complicate the sketches.

Figure 1.32i shows the spindle and base tilted at angle t off the vertical and horizontal, respectively, but, due to the inequality of the supports, the tube is horizontal.

Upon reversal (**Figure 1.32j**), the angle and direction of tilt remain unchanged, but, the shorter support now being at the lower end, the tube is tilted through angle 2t, indicating lack of verticality of the spindle.

If the spindle is now brought to the vertical position (**Figure 1.32k**) the tube will remain tilted by angle t. Upon reversal (**Figure 1.32l**), the direction of tilt is reversed, but the angle remains the same, as indicated by the bubble being off-centre by the same amount, but on the opposite side, ie, it occupies the same position in the tube.

This condition indicates verticality of the axis, but again it is preferable to have the bubble in proper adjustment, and this is accomplished exactly as with the unattached bubble, except that the bubble is reversed by rotating the base.

Correct bubble adjustment is indicated by **Figure 1.32m** and **Figure 1.32n**, where the bubble is central in both the direct and reversed positions.

1.34.2 Adjustment of the plate bubble

This is the first adjustment to be carried out.

(1) Set up the instrument firmly, in the shade, and level. This leveling need not, and, if the bubble is badly out of adjustment, cannot be done very accurately.
(2) Set the bubble parallel to any pair of footscrews and adjust these to bring the bubble to the centre of its run.
(3) Rotate the upper plate to reverse the bubble.
(4) If the bubble does not return to centre, operate the footscrews to bring it halfway back to centre.
(5) Adjust the bubble adjusting screws to centralise the bubble.
(6) Again reverse the bubble and, if necessary, repeat the adjustment. This repetition may have to be carried out several times.

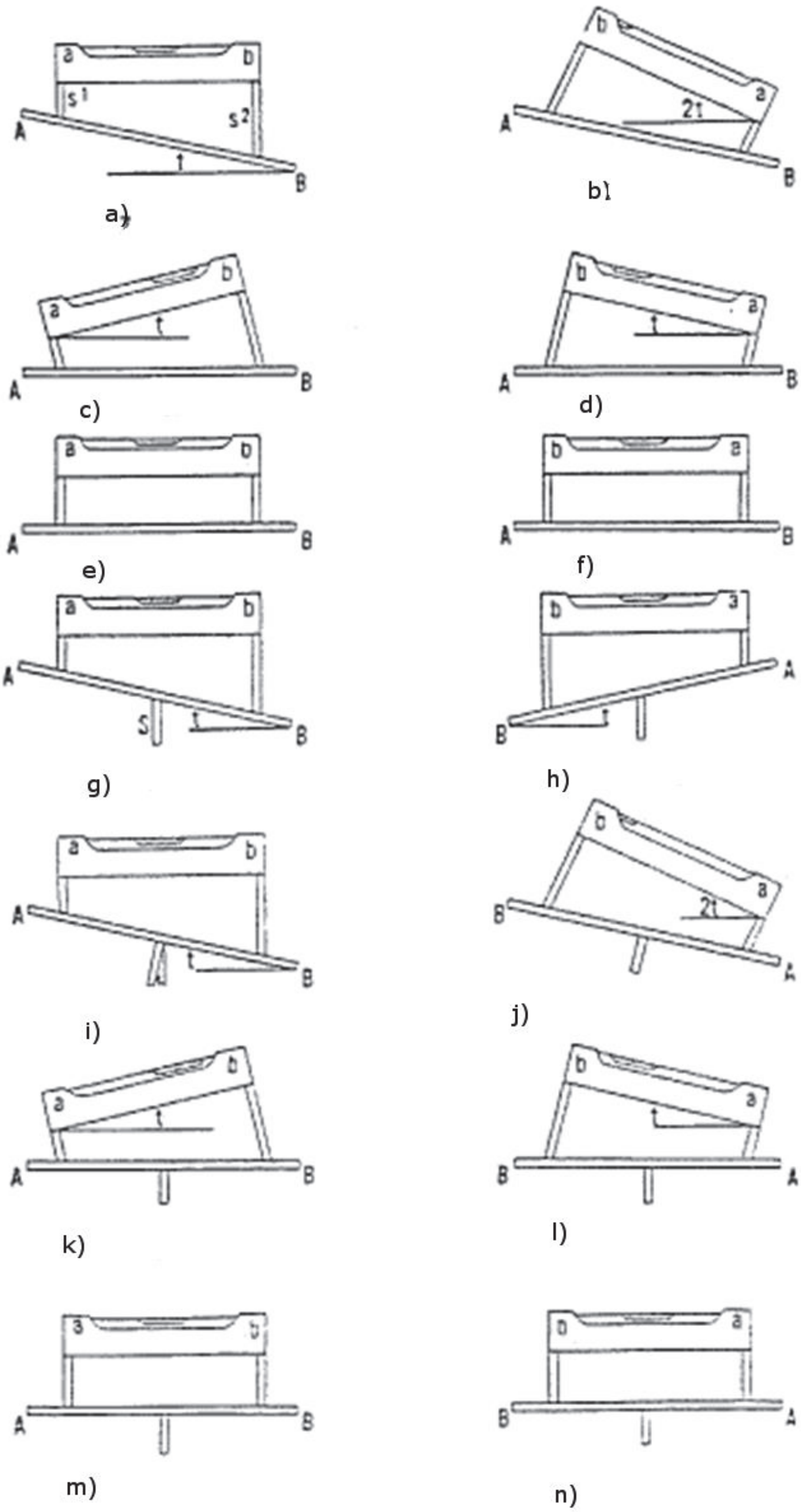


Figure 1.32

1.34.3 Calculation of sensitivity of a level tube

A base of about 50 metres is set out on level ground, by means of a steel tape. The instrument is set up at one end, and a staff held at the other end. With the bubble first near one end of its run, then near the other end, readings are taken on the staff. Let EF be the run of the centre of the bubble.

0 is the horizontal distance from instrument to staff. Let n be the length of staff intercepted between upper and lower lines of sight. Let n be the number of divisions through which the centre of the bubble is moved, and let R be the radius of curvature of the tube. (See **Figure 1.33**).

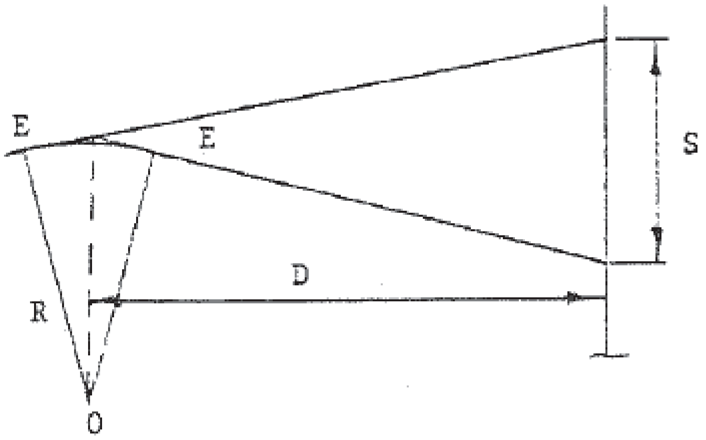


Figure 1.33

Thus:

Angular value of one division in seconds = $\frac{S}{Dn} \times 206\,265$.

Worked Example 1.11

Calculate the sensitiveness (angular value of one division) of a level tube when:

D = 50,00 m
S = 0,020 m
n = 4

Solution:

(a) Angular value/division $= \frac{S}{Dn} \times 206\,265$

$= \frac{0{,}02}{50 \times 40} \times 206\,265$

= 20 seconds to nearest

(b) If 1 division = 2 mm, the radius of curvature is calculated as follows:

$\propto \; = \frac{arc}{radius} \times 206\,265$

$\therefore 20 \; = \frac{2}{R} \times 206\,265\; mm$

$$\therefore R = \frac{1}{10} \times 206\,265 \times \frac{1}{1000} \, metres$$
$$= 20{,}62 \, metres$$

1.34.4 Coincidence levels

The vertical circle index level of Wild Theodolites (excluding the TIA and the TO) is of the coincidence, or split-bubble, type (see **Figure 1.34**).
By means of an ingenious system of prisms the two ends of the bubble are seen side by side in the bubble viewing eyepiece. The two halves are brought into coincidence to form a smooth curve by turning the index level setting screw.

Only when this coincidence setting has been made may the vertical circle reading be taken. To distinguish them from the other fine movement drive screws, the index level setting screws on all Wild Instruments have fluted edges.

Figure 1.34

1.34.5 Typical sensitivity values

(a) Level tubes
The lower the value the more sensitive the bubble, given in seconds per 2 mm. Some examples are: Wild 60''/2 mm; Fuji-Koh Sunray 45''/2 mm; Sokkisha 30''/2 mm; Zeiss 20''/2 mm.

(b) Circular bubble levels
These are less sensitive than level tubes and sensitivity values are in minutes per 2 mm. Some examples are: Wild 8'/2 mm; both Fuji-Koh and Sokkisha are 10'/2 mm, as is Zeiss.

1.35 Circular bubble

1.35.1 Some applications of the circular level

As shown, the circular bubble may be mounted on an angle bracket which is called a staff bubble, used for keeping the staff vertical. Some staffs are manufactured with a built-in bubble. Some tripods have circular bubbles on them for rough leveling.

Most modern Automatic Levels and Theodolites have them built in for rough leveling up of the instrument after the instrument has been attached to the tripod.

1.35.2 Adjustinq the circular level

With the instrument in perfect adjustment and properly set up, the bubble will remain centred in the engraved circle of the circular level as the instrument traverses in all directions in the horizontal plane.

If adjustment of the circular level proves necessary;

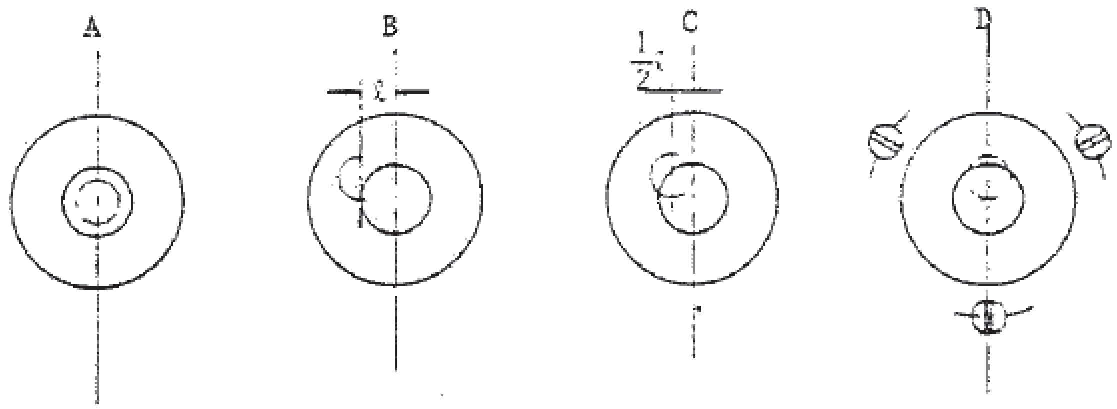
1. Set up the instrument. Use the leveling screws to bring the bubble of the circular level exactly to the centre of the engraved circle.

2. Turn the instrument to the direction in which the displacement of the bubble from the central position is greatest.

3. Reduce half of the displacement by adjusting the leveling screws and eliminate the remaining half of the displacement by adjusting the screws surrounding the circular level.

   It is best to adjust the bubble in one direction at a time; first (as shown in Fig. 7.6) from left to mid-line and then from top to centre, the movement in each direction being made. half by the leveling screws and half by the adjusting screws.

4. Rotate the instrument through 180° and repeat the process of stage (3) (above). Repeat stages 2 and 3 with the instrument traversed in as many directions as necessary until the instrument will traverse in all directions without displacement of the bubble.



A. First centralize the bubble in the engraved circle.
B. Turn the telescope through 180° or to the direction of greatest displacement.
C. Half the displacement is reduced by the leveling screws and .....
D. The remaining half of the displacement, in this direction, is eliminated by the adjusting screws.

Figure 1.35

Activity 1.1

1. Describe the difference, with the aid of sketches, between ties and offsets.
2. Name the basic measurements that are carried out in surveying.
3. Discuss the twofold object of surveying.
4. Give brief descriptions or definitions of the following:
   (a) Level surface
   (b) Map and plan
   (c) Triangulation and trilateration.

Activity 1.2

1. Discuss the field work of a surveyor.
2.
   i. A tree is 29,5 mm away from a pylon on a 1:2 000 map, what is the actual distance on the ground?
   ii. A trigonometric beacon is 1,25 km from where you are standing. How far from the beacon would you be on a 1:20 000 map (in millimetres)?
3. Calculate the plotting accuracy of the maps used in question 2.
4. What is the value of 95 grades 2 centigrade in
   i. degrees, minutes and seconds and
   ii. radians?
5. What is the difference between accuracy and precision in surveying? Explain in relation to each other.

Activity 1.3

1. Discuss the procedure you would adopt to reduce the possibilities of making mistakes.
2. Give a brief description of the transverse Mercator projection.
3. What are co-ordinates?
4. Briefly define weighted and arithmetic means.

Activity 1.4

1. What are "face left" and "face right" readings?
2. Define the following terms:
   (a) orienting ct.
   (b) co-ordinates

(c) the zero direction of the South African Lo system.

3. You set up a point A with co-ordinates - 1 058,47 + 310 248,17
   Your readings were:

Point	Hor. Dist.	Direction
A – B	453,15	35°56’23”

   Calculate the polar of point B.
4. What is the relation between the two extremities of a line?

Activity 1.5

1. Define BEARINGS, and state where they are used and where they should not be used.
2. State the general rule for the relation between distance, direction and the change in co-ordinate values from one point to another.
3. Calculate the polar given, [K] - 112,45 + 675,20 x KU = 123° 10' and KU = 46,25 m.
4. In this module you will find the information to calculate the JOIN (the reverse of the polar) between A and B, given [A] - 15,623 + 32,793[B] - 18,618 + 26,315. The information required in a join calculation is the direction from A to B and the distance from A to B.

Activity 1.6

1. Define "staff" and explain the difference between a leveling staff and a tacheometric staff.
2. What are change plates and staff bubbles used for?
3. You are conducting a survey and you read the top hair as 2,907; the middle as 2,464 the bottom as 2,021 and the vertical angle as + 10°00'00". Your instrument is 1,523 m above a bench mark. Find:
   (a) the horizontal distance from the bench mark to the point the staff is at, and
   (b) the height of that point above the bench mark.

Activity 1.7

1. Define stability, with respect to level tubes.
2. Describe the basic principles of a coincidence level.
3. Using a 86 m base, the difference between the upper and lower lines of sight is 23 mm and the bubble moves through two 2 mm divisions. Find
   (a) the sensitivity of the bubble in seconds/2 mm;
   (b) the radius of curvature of the bubble in metres.
4. List the adjustment steps for a plate bubble.

Self-Check

I am able to:	**Yes**	**No**
• Describe the basic surveying terms and principles of surveying.		
• Describe the following terms:		
o Surveying		
o Level plane		
o Horizontal plane		
o Linear measurement		
o Height measurement		
o Angular measurement		
• Describe methods of fixing a point:		
o Trilateration		
o Intersection arcs		
o Rectangular offsets		
o Triangulation		
o Polar co-ordinates and control		
• Describe the principle of working from the whole to the part.		
• Explain the difference between accuracy and precision.		
• Describe the characteristics of different types of errors.		
If you have answered ‘no’ to any of the outcomes listed above, then speak to your facilitator for guidance and further development.		

Module 2

Linear measurements

Learning Outcomes
On the completion of this module the student must be able to:

- Describe methods of direct linear measurement.
- Describe the uses of chains, tapes and bands.
- Describe the measuring sloping distances using chain, tape and band by:
  - Stepping; using a clinometer.
- Demonstrate the calculation of slope correction for distances measured on an incline.
- Demonstrate the graphical method used to correct distances measured on an incline.
- Demonstrate ranging and measuring over a hill and through a depression.
- Demonstrate measuring around a pond, across a river or busy road.
- Demonstrate measuring when a building obstructs vision.
- Give a description of equipment used for chain surveys.
- Describe linear survey methods.
- Demonstrate measuring offsets and ties by optical square and tape.
- Describe the recording of measurements taken in a field by a recognised booking method.
- Explain the identification and correction of fieldwork errors.
- Describe factors which govern chain survey framework.
- Demonstrate the application of chain survey principles to a small practical situation.
- Demonstrate the plotting of survey lines including all detail.

2.1 Introduction

In this module you will learn about linear measurements using chains, tapes and bands. You will also do calculations and learn how to measure when a building obstructs vision. This module covers slope correction and how to measure around a pond, across a river or a busy road.

2.2 Types of tapes

Tapes used in surveying are of three main types:

2.2.1 Cloth or linen tape
This is a painted and varnished strip of strong linen 16 mm wide and varying in length from 8 to 30 metres: The tape is graduated into metres, tenths and hundredths of a metre (some to ½ cm).

2.2.2 Metallic tapes
A "metallic" tape is a cloth tape into which fine brass or copper wires have been woven - it is a wired linen tape. This gives additional strength to the tape, makes it hard-wearing and reduces (but does not eliminate) stretching and shrinkage.

The usual width is 16 mm and the tapes are available in the following lengths: 8, 15, 30 and 60 metres. The tapes used on the mines are graduated in metres and tenths of a meter, although tapes graduated to hundredths of a metre are also available.

Metallic and cloth tapes are kept in hard leather or plastic cases (see **Figure 2.1**).

2.2.3 Steel tapes
The steel tape is a long thin ribbon of high quality tempered steel varying in length from 10 to 100 metres.

The 10 and 15 metre lengths are 9,5 mm wide, while the 30, 50 and 100 metre lengths are all 6,35 mm in width. Steel tapes are graduated in metres, tenths, hundredths and some up to thousandths of a metre.

In general, surveying measurements by steel tape are made to the nearest millimetre. Tenths and hundredths are read off while the third decimal figure is found by approximation.

The 10, 15 and 30 metre lengths are kept in steel or plastic cases (see **Figure 2.2**) while the longer lengths are wound on a steel (sometimes plastic or plastic-coated steel) frame, as illustrated in **Figure 2.3**.

The shorter steel tapes have their zero at the ring on the end while the longer tapes have a blank space between the ring and the zero mark.

A new development is nylon-coated steel tapes (**Figure 2.9**).

Steel tapes are correct at a standard tension (pull) of 6 - 10 kilograms force and at a temperature between 16°C to 21°c, these figures usually being specified for each tape.

Note:
1 kg force= ± 9,81 Newton's (symbol N). Newton scales are not available yet.

2.3 Maintenance and handling of steel tapes

Steel tapes are brittle and elastic. Being brittle they break easily and the following should be observed:

(1) Draw the tape out in a straight line in the direction in which it is coiled. (See **Figure 2.4**.)
(2) Ensure that there are no kinks or loops in the tape when using it.
(3) Do not allow vehicles to run over the tape, and avoid stepping on it.
(4) Being elastic, tapes stretch but return to their natural length unless strained. The standard pull (6 - 10 kg f) should not be exceeded.
(5) Steel tapes are liable to rust. Before putting a tape away after use, wipe it dry then wipe it with an oily rag.

2.4 Standardisation of steel tapes

Because of the purpose for which steel tapes are used they must be accurate, and if the tape is not correct the error must be known so that due allowance can be made for it.

An error in the tape can be revealed by comparing the tape with legally accepted standard lengths in South Africa. These are found in certain government buildings in Johannesburg, Bloemfontein, Pietermaritzburg and Cape Town.

There is no. specific government regulation requiring the periodic standardisation of steel tapes, but the necessity for this is enforced by the regulations dealing with the maximum error of closure allowable in a survey.

Note:
Every registered land surveyor must have a standardised tape which is given a number by the Surveyor-General and which must be given on all plans submitted for registration.

2.4.1 Standardising the tape

A number of metal blocks are let into the floor of a long passage-way each with fine marks on it; they are placed so that the required distances (eg, 0 - 100 m, etc.) are set out.

The temperature at which the tapes must conform with the established distance is indicated on each block, and each block is protected by a brass cover which is unscrewed and removed when tapes are to be standardised.

2.5 Miscellaneous notes on tapes

(1) The ranging rod (**Figure 2.5**) is an instrument which may also be used in conjunction with tapes in surface surveys. It is used to line up points in a straight line, or to mark the position of a point which is to be located. It is also used in traversing, triangulation and tacheometrical surveys.

(2) Spring balance for use with steel tapes This, as illustrated in **Figure 2.6**, is a tension handle with a spring balance graduated up to a pull of 10 kg force. It is used to ensure that the correct tension is-obtained v1hen measuring.

(3) Steel wind-up measuring tapes. This type of steel tape is used for measuring the depths of shallow boreholes (eg, water boreholes on farms). As illustrated in **Figure 2.7**, a detachable brass masspiece is included in the first portion of the tape with the zero at the bottom of the masspiece.

It is obtainable in lengths up to 60 m, and is obviously limited to boreholes up to this depth only. The depths of boreholes deeper than this are determined by measuring the boring column, which may consist of a long rope or steel rods

(4) Flexible spring steel rule. This is a flexible type of short steel rule which, when drawn out from its case, becomes rigid and remains stiffly extended due to the slightly concave shape it is forced to assume. Upon release it springs back or is easily pushed back into its case Figure 17.8(b). Available in 2, 3 and 5 m lengths.

Figure 2.1 "Metallic" woven measuring tape

Figure 2.2 Steel tape

Figure 2.3 Steel tape

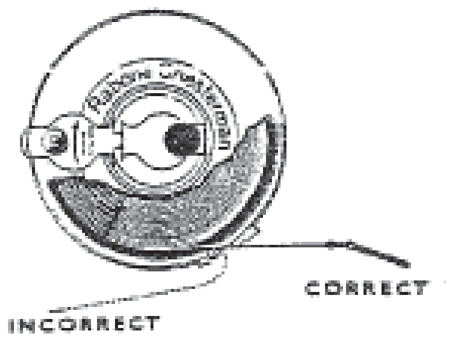


Figure 2.4

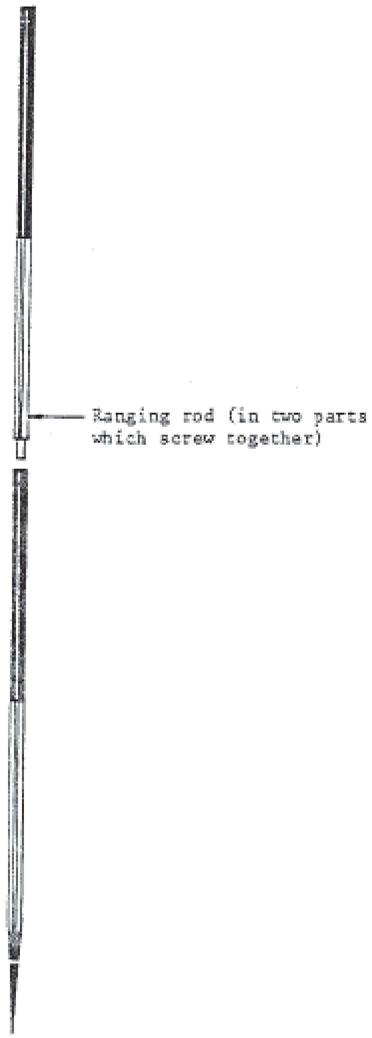


Figure 2.5

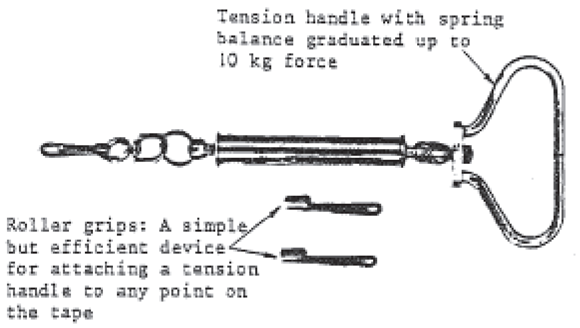
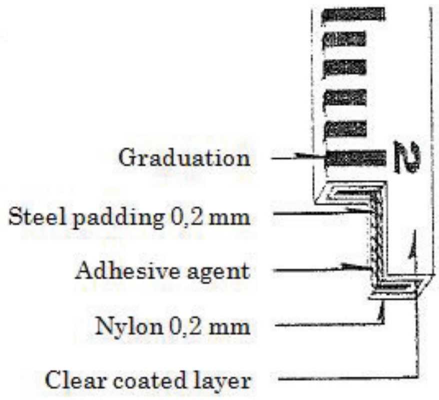


Figure 2.6

Figure 2.7 Steel tape for measuring borehole depths

Figure 2.8



13 mm x 0,2 mm thick (steel padding)
Tension of 10 kg
Plastic (ABS resin) open frame reel.

System	Length
Metric	50 m 100 m

Graduation
mm throughout on one side

Figure 2.9 Steel-nylon tape stilon wide

2.6 Incorrect tapes

When distances have been measured with tapes which are known to be, or subsequently found to be incorrect, the measurements and the results of any calculations based directly on those measurements (eg, linear areas and volumes) must be adjusted.

2.6.1 Permanently incorrect tapes

Tapes can become permanently incorrect in the following ways:

- A permanent shrinking or stretching of the tape after it has been manufactured and graduated, due to defects of the material, and assumed to be evenly distributed throughout the tape length.
- Inaccurate graduation during manufacture. (This error is usually very small.)
- The tape has become permanently stretched due to an excessive amount of tension (pull) applied every time it has been used.
- The tape has become shorter than its proper length due to:
  - a short piece broken off;
  - an overlap or gap resulting when repairing the tape after it has snapped;
  - a knot due to temporary repair when a metallic or cloth tape has snapped.

Temporarily and permanently incorrect tapes result in CUMULATIVE ERRORS, the magnitude of which can be calculated, so that the necessary adjustment can be made to a measured distance.

How the error is discovered
If a tape is incorrect, this will be discovered, and at the same time the amount of error ascertained when the tape is compared with a legally accepted standard length, ie, when it is standardized.

As it is not always possible for tapes to be frequently checked in this manner, a commoner practice is to compare the tape with a "master length" and to note any error.

This so-called master length is actually just an accurately graduated invar band (a tape of invar steel) which has already been standardised and whose length can be accepted, for all practical purposes, as correct.

How the error is expressed
When a tape is found to be permanently incorrect due to stretching or shrinking the error can be expressed in three different ways:

- The fraction of a metre (or number) that the tape is too long or too short, eg,
  - tape has stretched 0,2 metres,
  - tape is 0,3 metres too short.

  Here the length of the tape must be known.

- A fraction of the nominal (graduated or marked) tape length; eg, the tape is $\frac{1}{200}$ too long or too short.

  Here the length of tape is not required.

- A percentage of the nominal tape length, eg,
  - the tape is 0,5% too short,
  - the tape has stretched 2%.

2.7 The adjustment or correction of measurements

When a distance has been measured with a tape which is subsequently found to be incorrect, the adjustment can be made as follows:

correct distance = measured distance x $\frac{actual\ length\ of\ tape}{nominal\ or\ graduated\ length\ of\ tape}$

To find the actual length of tape

- If the tape is too long (stretched) add the error to the graduated length. For example: The 60 metres tape is 0,1 metre too long; actual length is 60,1 and graduated length 60 metres.
- If the tape is too short (shrinking, etc) subtract the error from the graduated length, e.g., the 60 metres tape is 0,3 metres too short; actual length is 59,7 metres and graduated length 60 metres.

Note:
If the error is expressed as a percentage, the graduated length is taken as 100; eg, the tape is 0,4% too long. Here the actual length is 100,4, while the graduated length is 100.

Worked Example 2.1

A distance was measured with a 100 metre tape and found to be 400 metres. It was later discovered that the tape was 0,05 metres too long. Calculate the correct distance.

Solution:

Correct distance = measured distance x $\frac{actual\ length}{graduated\ length\ of\ tape}$

= 400 x $\frac{100,05}{100}$ metres

= 400,200 metres.

Worked Example 2.2

A measured distance of 90,500 metres was obtained between two pegs, with a steel tape known to be 0,02% too short. What is the correct distance?

Solution:

Correct distance = $\frac{actual\ length}{graduated\ length\ of\ tape}$

= 90,500 x $\frac{99,98}{100}$ metres

= 90,482 metres

By examining the formula and examples just given the following facts are proved:

- If the tape is too long, the true length of the line will be longer than the measured length. (In the case where a definite given distance is to be set off, the definite given distance is the same as the true distance.)
- If the tape is too short, the true length of the line will be shorter than the measured length.

Study the following illustrations in order to obtain a clear understanding of these statements:

(a) An exaggerated case

A 50 m tape has stretched and is 10 m too long. The true length of the tape is therefore 60 m, but it is still marked as 50 m. Consequently, a distance of exactly one tape length would be read off and recorded as 50 m. It is obvious that the true length is 60 m (longer than the measured length).

(b) An exaggerated case
A 100 m tape has shrunk and is 10 m too short. The true length of the tape is therefore 90 m, but it is still marked as 100 m.

Remembering that the error is distributed throughout the length of the tape, the true or correct length of a measurement of 50 m obtained with this tape (ie, 1 the tape length) will be 45 m.

The statements are modified slightly when dealing with areas or volumes.

2.8 Adjusting factor

The fraction in the formula $\frac{actual\ length}{graduated\ length\ of\ tape}$ is called the adjusting factor.

1. When dealing with linear measurements (distances) adjustments due to incorrect tapes are solved as follows:

$$correct\ distance\ =\ measured\ distance\ \times\ adjusting\ factor.$$

2. When dealing with areas and volumes calculated from measurements taken with an incorrect tape, adjustments are made as follows:
   (i) $Correct\ area\ =\ originally\ calculated\ area\ \times\ adjusting\ factor^2$
   (ii) $Correct\ volume =\ originally\ calculated\ volume\ x\ adjustinq\ factor^3$.

   (Originally calculated areas and volumes are those calculated from measurements taken with incorrect tapes.)

2.9 Problems involving incorrect tapes

Worked Example 2.3

A base is measured with a 60 m steel tape and found to be 200,00 m. On standardising the tape afterwards it was found to be 0,05 m too long. Calculate the correct length of the base line.

Solution:

Correct distance = measured distance × $\frac{actual\ length\ of\ tape}{graduated\ length\ of\ tape}$
= 200,000 x $\frac{60{,}05}{60}$ metres
= 200,167 metres.

Worked Example 2.4

2. It is required to set off a 100 metre track on a sports field with a 60 m steel

tape known to be 0,03 metres too long. What distance must be measured off with this incorrect tape in order to get an accurate length of 100 metres?

Solution:
Here we require the measured distance while the correct distance is given (ie, 100 m) .

∴ by transposing the formula we get:

$$\text{Measured distance} = \text{correct distance} \times \frac{actual\ length\ of\ tape}{graduated\ length\ of\ tape}$$
$$= 100 \times \frac{60}{60{,}03}$$
$$= 99{,}950 \text{ metres.}$$

Note:
Tape is too long therefore true distance is longer than the measured distance. This once more proves the statement made earlier.

Worked Example 2.5

A plot of ground was measured with a steel tape which is 0,2% too short and its area was found to be 2 ha. Calculate the correct area of the plot, working to the nearest three decimals.

Solution:

$$\text{True or correct area} = \text{measured area} \times \left(\frac{actual\ length\ of\ tape}{graduated\ length\ of\ tape}\right)^2$$
$$\therefore \text{correct area} = 2 \times \left(\frac{99{,}8}{100}\right)^2 ha$$
$$= \frac{2 \times 9\,960{,}44}{10\,000{,}00}\ ha$$
$$= 1{,}992 \text{ ha}$$

Worked Example 2.6

4. A cylindrical-shaped storage dam was constructed on a mine and from measurements of its diameter and depth, the volume was found to be 5 500 cubic metres. The measurements were obtained by means of a metallic tape which, on comparing it with an accurate steel tape, was found to have stretched 2%. Determine:
(a) the correct volume of the dam in cubic metres
(b) the capacity in litres.

Solution:

$$\text{Correct volume} = \text{measured volume} \times \frac{actual\ length\ of\ tape}{graduated\ length\ of\ tape}$$

$= 5\,500 \times \left(\frac{102}{100}\right)^3$ m³
$= \frac{5\,500 \times 1\,061\,208}{100 \times 100 \times 100} m^3$
$= 5\,500 \times 1{,}061\,208$ m³
= 5 836,644 cubic metres
= 5 836 644 litres

2.10 Temporarily incorrect tapes

2.10.1 Temperature

A tape can become temporarily incorrect due to the expansion or contraction caused by variations in temperature. The measurement is adjusted each time the tape is used, and the degree of error varies according to the change in temperature above or below that at which the tape was graduated or standardized. This leads us firstly to a few definitions.

Definitions: Temperature

Standard temperature is the temperature at which the tape was graduated, and is usually between 16°C and 21°C. The exact temperature is supplied with the tape. The tape will be correct only at that temperature.

Coefficient of expansion is the amount by which a steel tape will expand or contract uniformly throughout its whole length, for a change of every 1°C in temperature.

This amount varies for different types of steel used in the manufacture of tapes but is generally between 0,000 011 and 0,000 013 per degree Celsius.

Temperature during measurement
The temperature of the tape is taken as that of the atmosphere and is measured in the normal way by means of a thermometer.

Note:
When dealing with comparatively short distances together with a small difference between standard and atmospheric temperatures, correction for temperature becomes insignificant and may therefore be ignored.

Temperatures in underground surveying are usually within the range at which the tape is correct. It is only in important underground surveys such as a connection between two shafts that the temperature factor should be taken into account.

Corrections to measured distances, due to temperature, are found by using the formula:

Correction = MD x COE x (T2 – T1)
where MD = measured distance
COE = coefficient of expansion
T2 = atmospheric temperature during measurement
T1 = standard temperature

The sign (+ or -) of the correction depends upon the sign of the factor (T2 – T1). It then follows that

correct distance = measured distance + correction.

The graduated length of tape is not required.

Worked Example 2.7

A base line was measured with a steel tape which was standardized at a temperature of 17°C and was found to be 160,000 metres.

If the temperature during the time of measurement was 21°C, what is the correct length of the base line? The coefficient of expansion is known to be 0,000 012.

Answer to the nearest three decimals.

Solution:

Correction = MD x COE x (T2 – T1)
= 160 x 0,000 012 x (21 - 17)
= 160 x 0,000 012 x 4
= 0,008 metre
Correct distance = 160 + 0,008 metres
= 160,008 metres

Worked Example 2.8

Calculate the correct distance A to B if:
(i) The measured distance = 250,000 metres
(ii) Standard temperature = 20°C
(iii) Temperature during measurement = 14°C
(iv) Coefficient of expansion = 0,000 011

Solution:

Correct distance AB = measured distance AB + correction
Correction = MD x COE x (T2 – T1)

= 250 x 0,000 011x (14-20) metres
= 250 x 0,000 011 x - 6
= - 0,016 5 metres
= - 0,017 to nearest three decimals
∴ correct distance AB = 250,000 + (- 0,017)
= 249,983 metres

Worked Example 2.9

It is required to lay off an exact distance P - Q = 200,000 metres with a steel tape standardised at 17°C. If the COE is 0,000 012 and the temperature during the time of measurement is 29°C, calculate the distance to be measured off under these conditions.

Solution:

Correct distance = measured distance + correction
Measured distance = correct distance - correction
Correction = MD x COE x (T2 – T1)
= 200 x 0,000 012 x (29 - 17)
= 200 x 0,000 012 x 12
= 0,029 m
∴ Measured distance = 200,000 - 0,029
= 199,971 metres.

The measured distance was an unknown when the correction was calculated and the true distance was therefore used in the correction calculation. This makes no significant difference to the value of the correction and may therefore be treated as correct.

2.10.2 Sag

When a tape is suspended in catenary and supported only at the ends, a correction for the resulting sag must be made.

A tape suspended between two points will be in the form of a curve due to the effect of gravity (known as a catenary curve) and the difference between the horizontal distance and the distance measured along the curve is called "correction for sag" or just "sag correction".

Consequently this correction is deducted from the measurement obtained in order to arrive at the true horizontal distance between the end points, or added to a required distance which is to be laid off.

Formula for sag correction
(a) Horizontal measurements

$$ct_s = \frac{4W^2L^3}{N^2} \text{ or } ct_s = \frac{W^2L^2}{24T^2}$$

where W = mass of tape in kilograms per metre
L = 1ength of tape suspended, in metres
N = tension or pull applied in newtons
T = tension or pull applied in kilograms force (kgf)
and true distance = measure d distance - sag correction
or measured distance = true distance+ sag correction.

(b) Inclined measurements

$$ct_s = \frac{4W^2L^3}{N^2}\ cos^2\theta$$

where θ is the vertical angle (angle at which the line slopes).

Worked Example 2.10

Determine the sag correction for the measured distance AB = 60,000 m, obtained by stretching a tape between the at the same height. A tension of 60 N is two points, which are both maintained on the tape, whose mass is 0,6 kg, and whose length is 60 m.

Solution:

(a) w = mass of tape in kg per metre

$$= \frac{0{,}6}{60}$$

$$= 0{,}01 \text{ kg/m}$$

$$ct_s = \frac{4 \times W^2 \times L^3}{N^2}$$

$$= \frac{4 \times (0{,}01)^2 \times 60^3}{N^2}$$

$$= 4 \times 0{,}000\ 1 \times 60$$

$$= 0{,}024 \text{ m}$$

The correction = - 0,024 m
true distance = 60,000 m- 0,024 m
true distance AB = 59,976 m

(b) Given a tension of 6 kgf

$$ct_s = \frac{W^2L^3}{24T^2}$$

$$= \frac{60^3 \times 0{,}01^2}{24 \times 6^2}$$

$$= 0\ {,}025 \text{ m}$$

Worked Example 2.11

The distance between two anchor towers of an overhead cableway across a gorge was found to be 180 000 metres, by means of a 200 m steel tape with a mass of 2,3 kg. If a tension of 90 N was applied to the tape, determine the correct distance between the towers.

Solution:

W = mass of tape in kg/m

$= \frac{2,3}{200}$

= 0, 0115 kg/m

$ct_s = \frac{4W^2L^3}{N^2}$

$= \frac{4 \times (0{,}0115)^2 \times 180^3}{90^2}$

= 0,000 529 x 8 x 90

= 0,380 m

∴ Correct distance between towers = 180 000 - 0, 380

= 179,620 m

Note:
Verify this answer by calculation using a tension of 9 kg.f.

Worked Example 2.12

Determine the sag correction given the following:

measured distance = 100 m
tension applied = 70 N (7 kg.f)
mass of tape = 0,01 kg/m
vertical angle = 60°

Solution:

(a) $ct_s = \frac{4W^2L^3}{N^2}\cos\theta$

$= \frac{4\times(0{,}01)^2\times100^3}{70^2} \times \cos^2 60°$

$= \frac{4\times0{,}000\ 1\times100^3}{70^2} \times 0{,}5^2$

= 0,02 m

Sag correction = 0,02 m

(b) Given a tension of 6 kgf

$ct_s = \frac{W^2L^3}{24T^2} cos^2\theta$

$= \frac{100^3\times0{,}01^2}{24\times7^2} \times 0{,}5^2$

= 0 ,02 m as before

Note:
Since many spring balances are still graduated in kilograms-force (kg.f) the following formula is more commonly used:

$ct_s = \frac{W^2L^3}{24T^2}$

	where T is the given tension in kg.f and 24 is a constant (1 kg.f = 10 N approximately).

2.10.3 Slope

Distances measured on an inclination (slope) have to be reduced to the horizontal except in certain cases such as in some mine measuring where the plotting of results is done on the same plane as that on which measurements were taken.

Formulae to reduce inclined distances to the horizontal (see Figure 2.10)

1. In a case where the vertical angle is known:
   horizontal distance = inclined distance x cosine of vertical angle.

2. In a case where the difference in elevation (V) is known.
   (a) Horizontal distance = $\sqrt{(slope\ dist.)^2 - (diff.in\ elev.)^2}$
   (b) Horizontal distance = slope distance - correction for slope where
   correction for slope (C_s) = $\frac{(diff.in\ elevation)^2}{2 \times slope\ distance}$

This correction is accurate only when the difference in elevation is small, ie, so as to give a vertical angle up to 8°.

3. From (1) above, we can write:
   S - D = S - S cos θ (θ = vertical angle)
   S - D = correction because s - correction = D
   ∴ Correction = S - S cos θ

⚠	**Note:** (1 - cos θ) = versine θ

∴ Correction = S versine θ

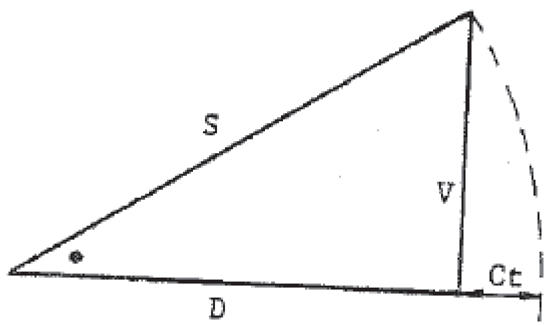


Figure 2.10

S = slope distance

D = horizontal distance
V = difference in elevation
θ = vertical angle = (VA)
Ct = correction

Worked Example 2.13

A distance of 100,000 metres was set off at a vertical angle of 6°, The difference in elevation between the two ends of the line was known to be 10,450 metres. Determine the horizontal line distance between the two points working to two decimals.

Solution:
Using formula 1

Horizontal distance = inclined distance x cos vertical angle
= 100 cos 6°
= 100 x 0,994 52
= 99,452
= 99,45 metres

Using Formula 2

$$Cs = \frac{(difference\ in\ elevation)^2}{2 \text{ x slope distance}}$$
$$= \frac{10{,}45^2}{2 \times 100}$$
= 0,55 m

∴ Horizontal distance = inclined (slope) distance-correction
= 100,00 - 0,55
= 99,45 metres

Horizontal distance $= \sqrt{(slope\ distance)^2 - (difference\ in\ elev)^2}$
$= \sqrt{100^2 - 10{,}45^2}$
$= \sqrt{9\ 890{,}80}$
= 99,45 m

Using formula 3

Ct = S versine θ
= 100 versine 6°
= 100 x 0,005 48
= 0,55

Horizontal distance = 100 – 0,55
= 99,45 metres

Note:
All calculations must be done by using Chambers' Tables. All figures (logs etc) must be shown.

4. step chaining

For measuring distance for topographical purposes with an order of accuracy of 1/1000 it is often more convenient to "step chain" as shown in **Figure 2.11**.

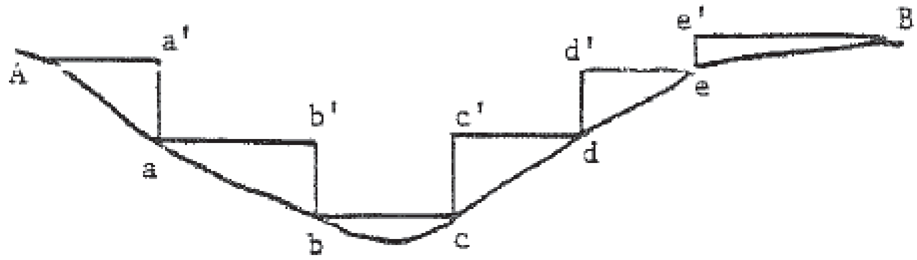


Figure 2.11

The horizontal distance between A and B in the sketch is required. The tape is held horizontally A-a', and at a' a plumb bob is suspended and the point a marked.

It may be difficult to determine the horizontal line, but making Aa' perpendicular to the plumb-bob string aa', may help.

The reading of the tape at a' is then held at a, and we continue to b.

Measuring up the slope from c toward B is more difficult, as the reading obtained at c must be held at c, with the plumb-bob held, to be at c and similarly at d and e.

A little practice at this will enable one to do it fairly satisfactorily, but it is usually possible to arrange matters so that step chaining may be done downhill.

2.10.4 Height above mean sea level

Figure 2.12 indicates how a line AB, measured at height H above mean sea level, is longer than its projected distance at mean sea level. Since the projection plane of the SA survey system (Gauss Conform – see later) is at sea level, a correction must be applied to measured distances, for accurate work.

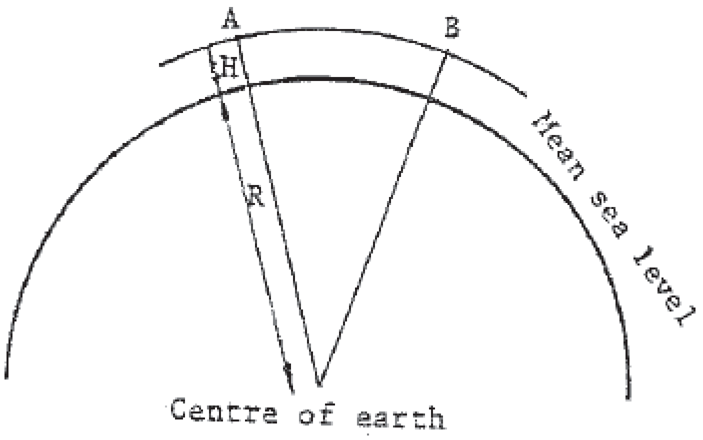


Figure 2.12

The formula for correction to mean sea level is:
Ct = DH ÷ R where D is the distance measured, R being the radius of the earth (accept 6 400 000 for calculation), and H the elevation of the line with respect to sea level .

Although seldom used in practice, this correction begins to be effective from about 300 metres above sea level and its omission will cause an error of about l/5 000 at an altitude of 1 500 metres.

Worked Example 2.14

A distance AB was measured and found to be 700,00 metres. The elevation of the line is 1 500 m above sea level. Calculate the projected distance AB at sea level.

Solution:

$$Ct = \frac{700 \times 1\,500}{6\,400\,000}$$
$$= 0{,}164 \text{ metres}$$
$$\therefore \text{distance AB at MSL} = 700{,}00 - 0{,}164$$
$$= 699{,}836 \text{ metres}$$

2.10.5 End alignment (Displacement at one terminal)
Referring to **Figure 2.13**, if it is not possible to measure directly between two points A and B, the difficulty might be overcome third point C, at right angles to the line AB at B.

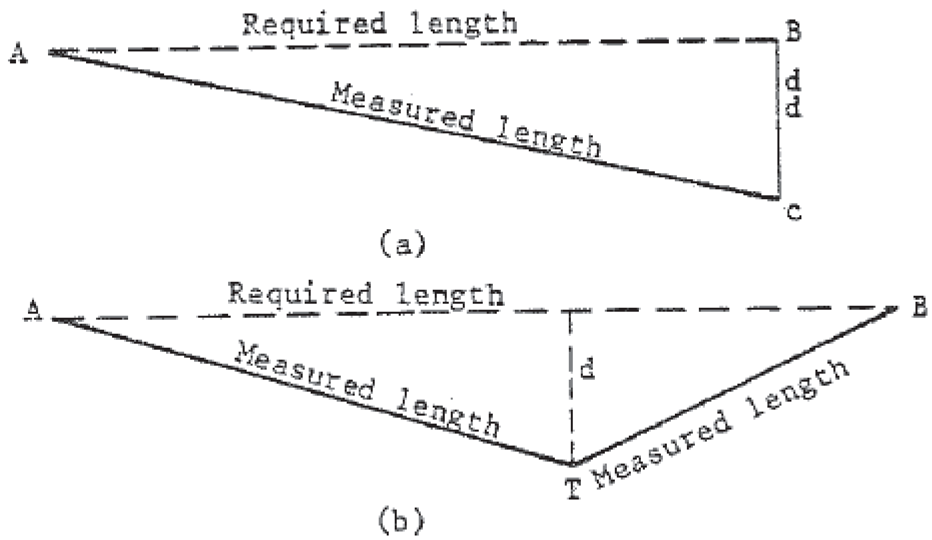


Figure 2.13

The distance AC as well as BC or angle A is measured. See **Figure 2.13a**. The required distance AB is then calculated.

In **Figure 2.13b**, the distances AT , BT and d are measured; from these AB can be calculated.

The above method (**Figure 2.13a**) may be used for displacement in the vertical plane as well.

2.10.6 Pull (or tension)

If the tension is greater or less than that at which the tape is standardised, the tape will be correspondingly too long or too short.

correction (C_p) for variation in tension in a steel tape is:

$$C_p = \frac{(P-Po)l}{AE}$$

where P = applied tension in N
Po = standard fens ion for the particular tape in N
l = length of the tape in mm
A = cross-sectional area of the tape in mm²
E = the Modulus of Elasticity (Young's Modulus) of stee 1. Usually taken as 200 000 N/mm²

Note:

If the applied tension is excessive it may well result in the tape being permanently stretched and possibly broken. If a high order of accuracy is required for a survey and the surveyor is using a tape of unknown standard tension, then use a tension of between 5 and 7 kg force (ie, 49,05 to 68,67 N) and be sure to note the actual tension used.

When the survey is completed the standard tension may be determined by measurement of a standard base line, with the tape used, applying a gradually increasing tension, until the length read off the tape agrees with the length of the standard base line, and then note the applied tension. This will be the standard tension for that tape.

Worked Example 2.15

A tape with cross-sectional dimensions of 6 mm by 0,5 mm was used to measure the distance from A to B. The result was AB " 49,726 m. The applied tension was 7 kg force and the standard pull for the tape is 56,5 N. Assume that all other corrections have been accounted for and calculate the actual distance measured.

Solution:

$$A = 6 \times 0{,}5 = 3 \text{ mm}^2$$

$$E = 200\,000 \text{ N/mm}^2$$

$$P = 7 \times 9{,}81 = 68{,}67 \qquad C_p = \frac{(P-Po)l}{AE}$$

$$Po = 56{,}5 \text{ N}$$

$$l = 49\,725 \text{ N}$$

$$\therefore C_p = \frac{(68{,}67 - 56{,}5)49\,726}{3 \times 200\,000}$$

$$= 1{,}008\,6 \text{ mm, say } = +1 \text{ mm}$$

$$\therefore \text{Actual distance} = l + C_p$$

$$= 49\,726 + 1$$

$$= 49{,}727 \text{ m}$$

Note:
The sign is "automatic", ie, if the applied tension is greater than the standard tension the sign of the correction will be positive, as illustrated above; if $P < P_o$ then C_p is negative.

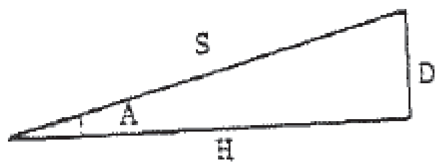
2.10.7 Sequence of corrections

Corrections and reduction of measured distances are always made in the following order:

1. Constant error correction, ie, incorrect length.
2. Correction for temperature.
3. Tension correction.
4. Sag correction.
5. Reduction to horizontal (slope correction).
6. Alignment.
7. Reduction to sea level.

ERROR	SOURCE	CORRECTION	MAKES TAPES TOO	IMPORTANCE IN ORDINARY CHAINING	STEPS TO ELIMINATE OR REDUCE
Systematic	Erroneous length	Varies	Long or short	Usually small but should be checked	Standardize tape and apply computed correction
	Temperature	C_T = MD x COE(T_2-T_1)	Long or short	Only significant in extremes of temperature	In precise work measure at favourable times and/or use invar tape.
	Tension (pull)	$C_p = \frac{(P-P_o)l}{AE}$	Long or short	Negligible	Use spring balance in precise work and apply computed correction
	Sag	$C_s = \frac{4W^2L^3}{N^2} = \frac{W^2L^3}{24T^2}$	Short	Large, especially with heavy tape	May be avoided on level ground by putting tape on ground. Otherwise apply computed correction.
	Slope (incline)	C_l = S versine θ	Short	Varies	Measure slope angle and/or elevation difference. Apply computed correction
	Alignment	See text	Short	Not usually serious	Be careful
	Reduce to sea level	See text	-	Not often applied	-
Accidental	Variable		$\pm$	Variable	Exercise care
		Note: See text for other formulae			

Table 2.1 Rods and corrections in chaining

CORRECTION FOR	FORMULA	EXPLANATION
Slope (Conversion of slope distances to horizontal distances.)	When A is known: H = S cos A or C = S(1 - cos A) = S versine A When D is known but A is not: $C = \frac{D^2}{2S} + \frac{D^4}{8S^3}$ (approx.) or A may be deduced from: sIn A = $\frac{D}{S}$	A = slope distance (given) H = horizontal distance (required) A = vertical angle of depression or elevation C = S - H = correction to be applied (always minus) D = difference in height 
Slope (Conversion of horizonta1 distances)	S = H sec A = $\frac{H}{\cos A}$ or C = H (sec A – 1)	As above except that horizontal distance is given and slope distance is required.

(For setting out)		C is always plus.
Sag (When taping near horizontal)	C = $\frac{W^2L^3}{24T^2} = \frac{W^2L}{24T^2}$	C = sag correction (minus for measuring, plus for setting out) w = weight of tape in kg per metre run L = length of tape under consideration W = weight of tape for length L T = pull applied in kg (tension)
Sag (on steep slopes) (Deformation of catenary)	C' = C $\cos^2 A$ This correction is seldom applied in practice.	C = correction for horizontal horizontal tape C' = correction for sloping tape A = vertical angle of depression or elevation
Temperature of steel tape	C = L x e x t Note: When t is +, C will be + When t is -, C will be -	L = measured length C = correction (plus or minus) e = coefficient of expansion of steel = 0,000 011 3 per 1°C t = increase or decrease in temp. from standard temp on °C
Height above (or below) datum	C = $\frac{SH}{r}$ or C = $\frac{S(H1+H2)}{2r}$ where H1 and H2 are the heights of the two ends of the line. In practice, C = 0,000 015 C = 0,000 015 7 x H per 100 m length of line. This correction is seldom applied in practice.	C = correction (minus for heights above datum plus for heights below datum) H = mean height of survey above or below datum in metres. S = horizontal distance r = radius of earth = ± 6 373 000 metres at 30° latitude.
End alignment: displacement of one end of tape. (Vertically or horizontally.)	As for conversion of slope distances to horizontal distances.	

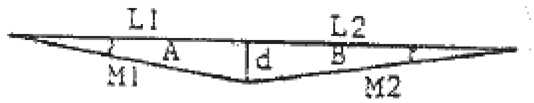
Internal alignment: displacement of tape at any point along its length. (Vertically or horizontally.)	Treat the two triangles formed separately and determine L1 and L2 as for end alignment. L1 + L2 is the required distance.	 L1 + L2 = required distance MI + M2 = measured distance d = vertical or horizontal displacement A and B = angles of displacement Either a or A and B must be known. The tape reading at the point displacement must be noted.

Table 2.2 Some variations on chainage corrections

2.11 Basic principles of chain surveying

Chain surveying is that branch of survey work which employs the tape as its basic and principal instrument. It is employed principally for large-scale detail surveying, and is eminently suitable for this purpose, particularly where the detail is geometrically regular.

The methods used are easily mastered, the amount of apparatus required is a minimum, and it is inexpensive, while the location of detail may be performed to a high degree of accuracy.

While tacheometry is undoubtedly the superior method when a large number of levels is required, it is safe to say that the possibilities of chain survey are not generally appreciated in South Africa.

When a large number of levels is not required, and in city work, the surveyor would be well advised to consider whether chain survey would not be the best method. It could also be combined with tacheometry or other methods to advantage in certain circumstances, such as when complicated building details have to be located accurately.

In chain surveying, linear measurements only are taken in the field, and by their means a rigid framework is established, which can be plotted to any required scale. The only straight-sided plane figure which can be accurately plotted when the lengths of its sides are known, is a triangle.

The area to be surveyed is, therefore, covered with a network of triangles whose sides are measured, and the network is then plotted. The triangles themselves must be "well-conditioned", to enable an accurate plot to be made, and the use of triangles with an apex angle less than 30° or greater than 150° should be studiously avoided to comply with this condition. The principle used then, is that of TRIANGULATION, and the solution of the triangles (in this case the plotting) is by trilateration, ie, the solution of triangles from the lengths of the sides only.

2.12 Checking

In conformity with the conclusions at which we arrived after our examination of the question of checking, it will be necessary for us to make such additional measurements as will be required, either to prove or disprove our results, so that the resulting plan will be an accurate representation, to scale, of the area surveyed, within the permissible errors of plotting.

Consider the **Figure 2.14a** each of whose sides have been measured. The available data is insufficient to enable the figure to be plotted. If, however, the diagonal 80 (tie-line) is measured, the figure may be plotted, but should any of the measurements be incorrect, the mistake would pass undetected.

A suitable check would be to measure the second diagonal AC (check line), noting the reading on the tape when measuring the diagonals, at their intersection E.

If it does not tally, check the plotted measurements and if no error is found, re-measure the lines in the field.

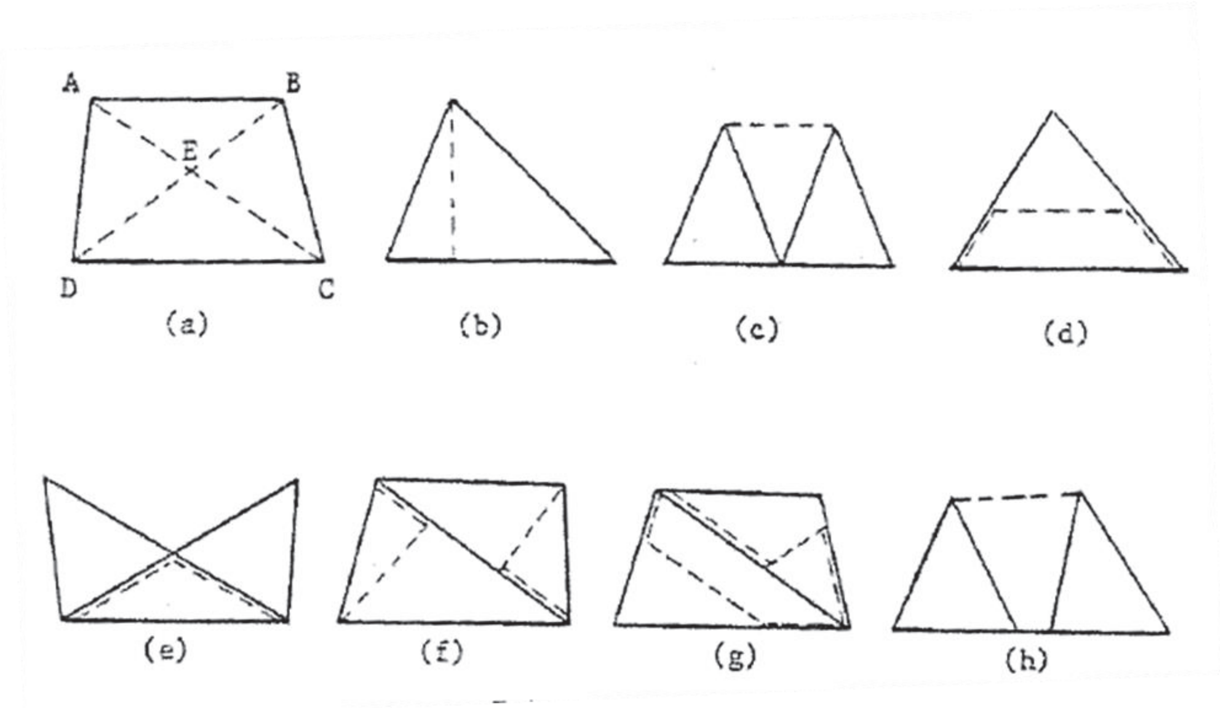


Figure 2.14

Figures 2.14b to **2.14h** indicate alternative methods of applying checks. Check lines are shown broken.

Each triangle in the survey must be checked, and the check line should, wherever possible, be so chosen that it will also serve for the measurement of offsets, which will be explained later.

None of the above checks is absolute, and compensating errors may occur. The possibility is so remote, however, that absolute checks are unwarranted and uneconomic. By a careful examination of the records, it should be possible to localise a mistake discovered in plotting.

It may even be possible to work around this weak link, but it is always advisable to re-measure the line in the field, especially as offsets may be affected.

The checks shown in **Figure 2.14a** to **2.14c** are stronger than the remainder, and should be applied to the main triangles, while any of the check illustrated may be applied to the subsidiary triangles.

2.13 Field work

2.13.1 Reconnaissance

Before any measurements are made, the area to be surveyed should be examined carefully and stations selected and marked so as to cover the area with a system of well-conditioned checked triangles. The intersection of diagonals is done at the same time, and the points so determined are also marked.

During the selection and marking of stations, an "INDEX DIAGRAM" is prepared, which shows, in diagrammatic form, all the chosen stations and the lines which are to be measured. No attempt is made to draw the diagram to scale.

2.13.2 Selection of stations

The principles to be observed in making a reconnaissance may be summarised as follows:

- Economise in the use of survey lines by eliminating as far as possible, all lines run without offsets.
- Where possible, the longest line in the survey should run from end to end of the area and roughly through its middle, to form a backbone upon which to build the main triangles.
- The main survey lines should be built up in the form of well-conditioned triangle based upon the backbone, and embracing as much as possible of the area to be surveyed. If possible, these lines will follow the hedges, fences, etc, closely, but, as they form the sides of the main triangles, their "condition" should not be sacrificed for this consideration.
- Detail lines are built up on the main survey lines, in the form of small checked triangles whose sides follow the fences and detail closely, so that the length of offsets will be kept short.

- Each triangle of the survey must be checked in a manner suitable to the importance of the triangle.
- Offsets should be kept as short as possible, particularly in locating important detail.
- Survey lines should lie over the more level, clear ground where possible.
- Care should be taken to measure the angles of elevation or depression of all survey lines whose inclination is too steep to be neglected at the scale used, and to reduce the measured slope distances to their horizontal equivalents.

2.13.3 Offsets and tie lines

A field bounded by straight fences could be surveyed very simply by a system of triangles and check lines, as shown in **Figure 2.15a**.

The geometrical figure ABCDE is divided by the tie lines BE and BD, into the three triangles ABE, BDE and BCD, which are checked by the checklines and APO and CQ.

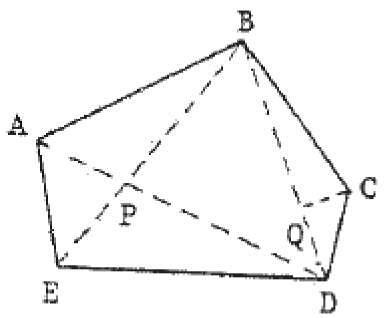


Figure 2.15a

In the case of a similar area bounded by irregular features, the problem, basically the same, is modified as shown in **Figure 2.15b**, by the necessity for locating the position of the irregular features by means of measurements, known as offsets, taken at right angles to the survey lines, and from recorded points along them.

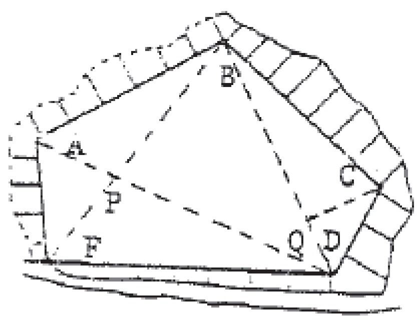


Figure 2.15b

Offset distances must be kept as short as possible to preserve the accuracy of locating detail, since the right angle to the survey lines is usually estimated by eye. Suggested maximum offset lengths under these conditions are, 5 metres on a scale of 1/1 000 and 2,5 metres on a scale of l/500.

This distance may be increased where the right angle is laid out instrumentally, and is subject to modification in relation to the importance of the detail being located. For indefinite features, offsets up to 30 metres are tolerable, but in no case should they exceed the length of the offset tape.

Where this is likely to happen, introduce an additional survey line close to the detail. Any point may be located accurately by erecting a small checked triangle to it.

2.13.4 Running survey lines

By this is meant the routine of chaining the line, and locating the detail adjacent to it, by means of offsets.

Having aligned the tape along the survey line and inserted the forward arrow, the fore-chainman leaves the tape lying on the ground and returns to assist the rear chainman with the offsetting.

He takes the ring of the offset tape (zero end) and holds it at the various points of detail to the right and left of the survey line, as directed by the rear chainman, who measures the offset from the survey line, estimating the perpendicularity of the offset tape by eye or optical square.

The rear-chainman calls out the offset distances and the chainages from which they are projected, to the booker, who repeats them as a check and records them in the field book. The distances along the survey line at which fences, streams, hedges, etc., are intersected by it, must also be read and recorded. This will be discussed in detail in the following lectures.

Notes

(a) In locating a straight line such as, for instance, a fence or hedge, an acceptable fix would be to locate two terminals, with a check offset to the middle of the line. Additional checks are, however, desirable on long lines.
(b) Offsets should be taken in order of their chainages, and, before the tape is moved it is necessary to make sure that no offsets have been overlooked.
(c) A common tendency is to take too many offsets. No attempt should be made to record irregularities too small to be shown on the scale of the final map.
(d) On flat ground, offset measurements should be made along the surface of the ground. On slopes a plumb-bob must be used.
(e) The perpendicularity of the offset tape to the survey line may be estimated by swinging it through a small arc with the zero end of the tape as centre.

The smallest offset tape reading against the survey line is the perpendicular distance, and it gives the point of perpendicularity on the line tape.

(f) The following figures, calculated for a plotting accuracy of ¼ mm, indicate the degree of accuracy required in measurements:

Scale 1/500. Measure to the nearest 0,125 m.
Scale 1/1 000. Measure to the nearest 0,25 m.

2.14 Field book

The art of keeping any field book is so to arrange the entries, in a logical, orderly, systematic and legible manner, that any other surveyor, with no knowledge of the surveyor responsible for the survey, or of the area surveyed, may produce a map or p 1 an from the recorded data, just as faithful and as accurate as that which could have been produced by the responsible surveyor.

There is a conventional manner in which entries are made in a field book, employing a convenient system which is universally recognised by surveyors. A description of this system follows.

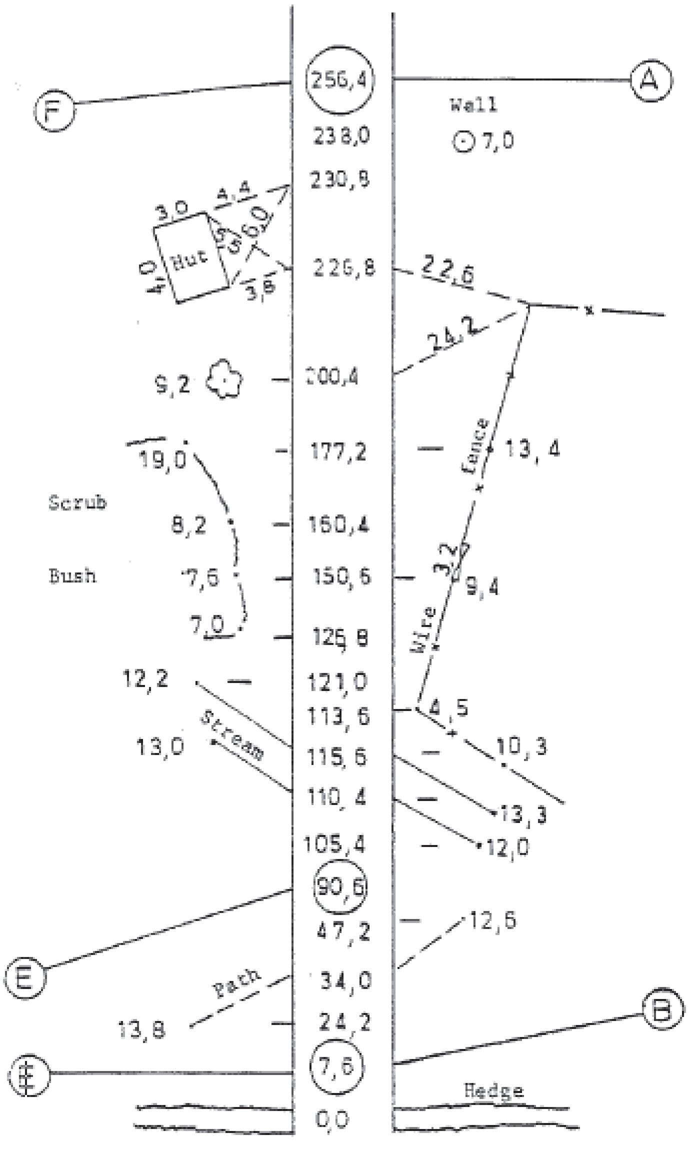


Figure 2.16

The field book varies in size from 10 cm x 15 cm to 13 cm x 23 cm, the pages being ruled lengthwise centrally, with two lines about 15 mm apart. These lines represent the tape, and measurements taken along the tape are recorded within this column.

Entries are commenced at the bottom of the page and progress upwards, thus corresponding with the natural direction of the chaining, and right and left sides in the book correspond with those in the field when looking in the direction of the chaining.

Great care must always be taken to hold the field book in its true position in relation to the line being worked along, to ensure that the record of measurements etc, are made on the correct side of the field book centre line.

If this rule is not strictly adhered to, detail may be plotted on the wrong side of the line. There is often no other check to guard against this possibility. As the various features within offsetting distance are reached, the surveyor sketches them in the space upon the appropriate side of the column in the field book, and enters the chainage and length of each offset in the manner shown in the example (**Figure 2.16**).

When a feature crosses the chain line, care must be taken to sketch it in such a fashion that it enters the field book column at one side, and leaves it at the other, at the same chainage figure, so that, if the column were cut out and the two remaining halves of the field book joined together, the detail and lines, as sketched, would form a continuous whole. This is illustrated in the specimen field book page (**Figure 2.16**).

Each survey line should be started upon a fresh page in the field book. The number of the survey line and the names of its terminals are written at the bottom of the page. Measuring may commence at the terminal of the line or at some detail feature before it.

The chainage at the terminals must, in any case, be circled in the column, as shown in **Figure 2.16**. The directions of survey lines meeting the line being measured should be sketched, and the chainage at the junction circled, while the number of each are recorded outside the column against it.

Offset distances are recorded outside the column and upon the appropriate side of it opposite the relevant chainage.

The field book must include:
1. Title page
2. Index
3. Index diagram
4. Recording of the field work
5. Separate sketches of detail too intricate to show on the chainage pages.

The title page will give:
- an accurate description of the area surveyed,
- a list of the tapes and/or other apparatus used,
- name of surveyor, chainman and booker,
- date of survey.

Index diagram:
This consists of a diagrammatic sketch, not to scale, of the survey lines.
It shows:
- the number of each survey line,
- the name of each survey station,

- the length of each survey line, written against the line and reading in the direction of measurement so that the whole skeleton may be plotted from the diagram,
- a reference to the field book page number where detail surveyed from each line is recorded.

2.15 Recording of the field work

In addition to the previous instructions upon booking, the following notes are appended.

The ideal field book is produce d by paying careful attention to:

- neat figures and legible printing,
- clear sketches,
- clearness in representing the points to which offsets have been taken,
- add explanatory notes wherever necessary, to avoid ambiguity,
- cross-index wherever possible,
- leave nothing to the memory,
- use a hardish pencil (about 2H) and keep it sharp,
- keep the field book clean.

2.16 Sketches

To the inexperienced surveyor, the making of satisfactory sketches is often troublesome. The tendency is to allow insufficient room for sketches of intricate detail, so that dimensions cannot be entered legibly. No attempt should be made to sketch strictly to scale.

The sizes of complicated parts are exaggerated, the curvature of flat curves being increased, while angles which are nearly 90° or 180° should have their divergence emphasised. Long straight lines, on the other hand, should be shown shortened.

All the dimensions of buildings and similar detail should be measured. Even if the two ends of a wall are fixed, the length of the wall should still be measured, so that the building can be separately plotted if necessary.

Dimensions must be clearly and unambiguously shown, and the need for this should be kept in mind when drawing the outline sketch.

2.17 Plotting the survey

Drawing instruments, protractors, set squares and scales are required. It is useful to have a parallel rule and a set of curves.

A pencil plot is made first. The longest line is first drawn and its total length, as well as the positions of the intermediate stations, is carefully scaled.

Stations are marked with a pricker or fine pencil point, and circled. The triangles are then plotted with intersecting arcs and proved by check lines. The whole framework must be plotted and checked before the filling in of detail is begun.

The detail is then put in by plotting the offsets. An offset scale is very convenient for plotting these. The plan is finally inked in using conventional symbols and colours.

2.18 Baseline measurement

A base line is the foundation or "starting line" of the survey of an area, whether it be the geodetic triangulation of a new country, or a subsidiary and unconnected survey of a small field.

Triangulation surveying consists in covering the country with a network of triangles, all built up from one accurately and carefully measured base line.

Therefore, only one measurement of distance is required; the remaining measurements being those of angles at the base of each triangle.

To make a plan or local map of a small area a surveyor starts off by drawing a line on paper, in the correct compass direction, to represent the distance between two accurately measured pegs or beacons in the field and then fixes the position of (locates) all other points in relation to the originally established base line.

A base line must be measured with extreme accuracy, for the relative positions of all other points depend upon this measurement.

In geodetic triangulation the highest precision of all is required. Since the development of invar steel, with its low coefficient of expansion, base lines are measured using invar tapes or wires in catenary.

Note:
Sophisticated ancillary equipment is also available on the market.

Tripods are used at the terminal marks and these carry measuring heads with micrometers for fine measurements. The measuring heads may be replaced by a theodolite and targets, for accurate alignment and slope measurement, or by an optical plummet to transfer the terminal marks to a ground mark.

The tape is tensioned by passing it over a pulley on a straining trestle. A mass attached to the end of the tape exerts the required tension.

Corrections already described (constant temperature, sag, etc) are made to the measurements, while the total corrected length is finally reduced to the equivalent distance at mean sea level. The accuracy obtainable in this class of work is of the order of 1 in 500 000 or better.

2.19 Field problems in surveying

Many of the problems which are discussed here will undoubtedly suggest the use of a theodolite or some other optical instrument. It must be clearly understood, therefore, that we are dealing here only with problems in which the measurements or directions, for the purposes required, can be obtained to a sufficient degree of accuracy by using tapes only.

2.19.1 To extend or prolong a line

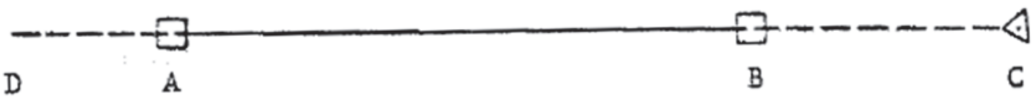


Figure 2.17 Method of extending a survey line

The line AB is to be extended beyond B.

Method

Two rods are placed at points A and B which are already fixed, and the surveyor, standing a short distance back from A (approximately 3m), "lines in" a third rod at the required point C, ie, he sights through the two rods at A and B and directs the rod at C to the left or to the right until all three are in line. C is the new point, and A, B and C are now in one straight line.

2.19.2 To set out a right angle using a tape only

(To set off from a given point a line perpendicular to a given line.)

Two simple methods

1. Application of the "3:4:5" rule

This mathematical rule states that if the lengths of the sides of a triangle are in the ratio of 3:4:5, one of the angles must be a right angle. The proof of this is based on the principle of Pythagoras, but it must be remembered that any multiple of these numbers may be used and also any combination of three numbers such that-

(One number)^2 = sum of the squares of the other two numbers.

For example

3 : 4 : 5 :	5 : 12 : 13
30 : 40 : 50	8 : 15 : 17
12 : 16 : 20	20 : 21 : 29
15 : 20 : 25	28 : 21 : 35

6 : 8 : 10

Practical details of the method
It is required to set off from point C a line at right angles to the line AB. Assume three men available. 12, 16 and 20 metre lengths of metallic tape can be used.

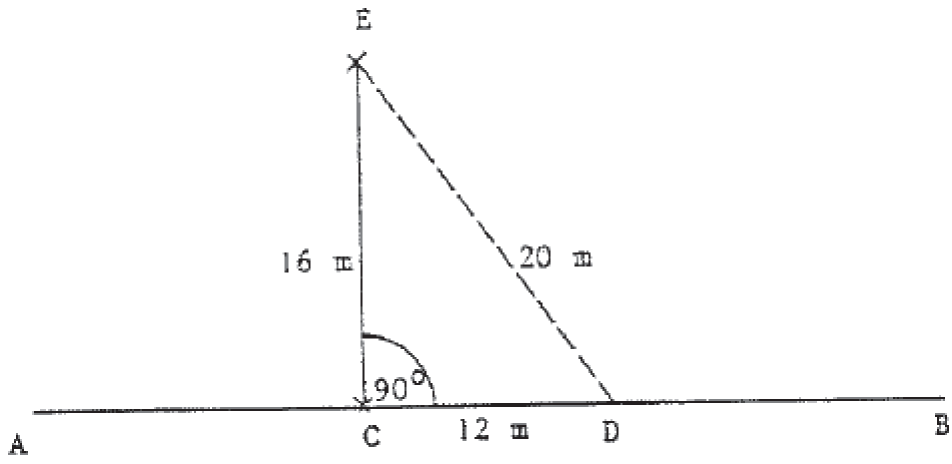


Figure 2.18 Setting our a right angle

A point is established 12 m along the line AB from C. One man holds the zero of the tape at C, a second man holds the 36 m (20 + 16) mark at D, and the thid man, holding the 16m mark of the tape, moves outwards, and when the two sections of the tape are fully extended he marks the point E.

The line CE is now at right angles to the line AB.

⚠	**Note:** By securing the 0 and 36 m marks of the tape at points C and D, respectively, one man can do all the work.

If a steel tape is used, it cannot be bent sharply enough to form the vertex of the right-angle triangle at E; consequently a slight modification must be made in the form of a loop at E.

With a loop of 4 m at E, the 40 m mark of the tape is held at D.
See **Figure 2.19**.

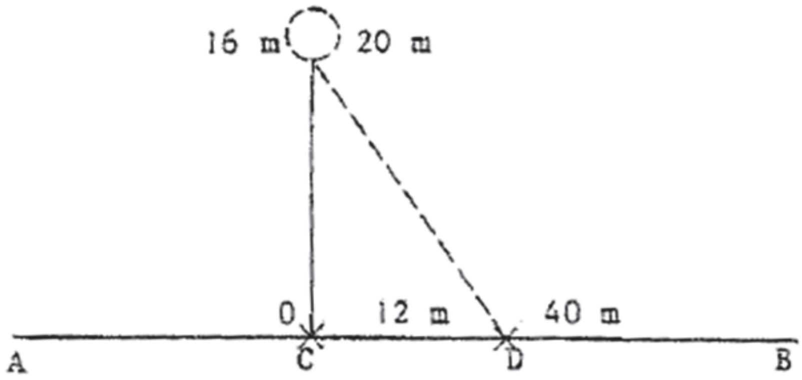


Figure 2.19 Setting out a right angle, using a steel tape

When using any of the other combinations of numbers to produce a right-angled triangle, suitable adjustments must be made so that the tape is held at the required positions.

2. Another method of setting out a right angle by means of tapes only

This is based upon the geometric construction used for erecting a perpendicular to a given line at a given point.

The practical application of this method for use in the field is as follows:

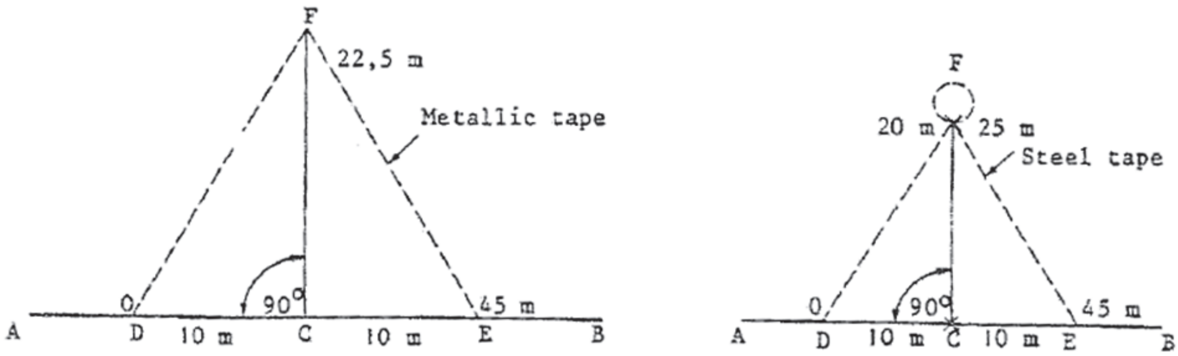


Figure 2.20 Setting out a right angle using metallic and steel tapes

From point C mark off equal distances to D and E on the line AB (say 10 metres).

With the zero at D, secure a suitable number on the tape, say 45 m, at E, and then, holding the centre of the tape, move outwards until both sections are fully extended to establish point F.

The line CF is at right angles to AB from point C.

Note:
If a steel tape is used, make a loop by bringing the 20 m and 25 m marks together and stretch the tape out to F as illustrated.

3. To drop a perpendicular onto a line from a given point outside it
Method 1

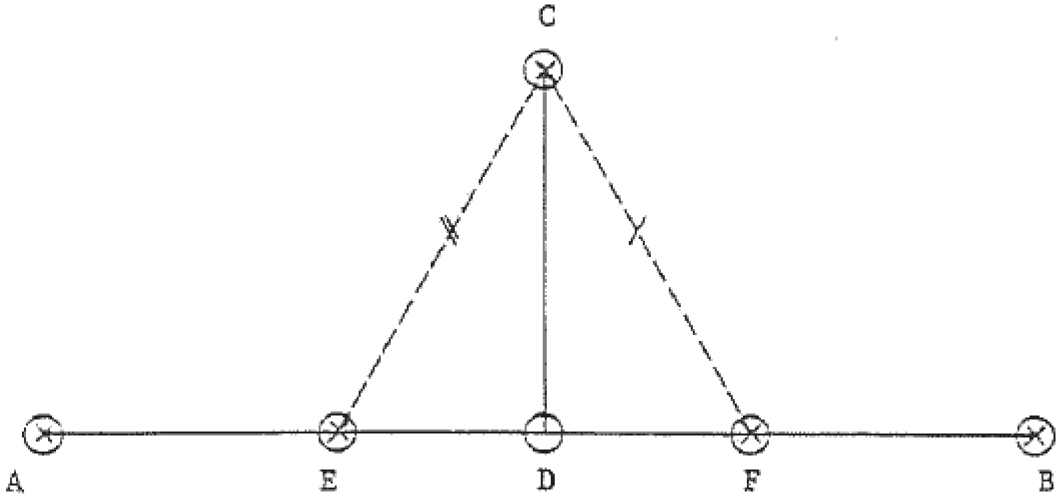


Figure 2.21 A perpendicular onto the line AB from a given point C outside it

From the given point C establish two points E and F on the line AB which are of equal distance from C. (See **Figure 2.21** - the field equivalent of swinging on an arc as in a geometrical construction.)

Find the point D midway between E and F. Then CD is the required perpendicular onto the line AB.

Discussion
A certain aspect of the descriptions already given and those which are to follow, which may be worrying the learner surveyor, is the relation between the layout and method of the survey as shown on plan or in the sketches of these notes, and the actual conditions outside in the field.

On the plan, actual lines are drawn between points representing the survey pegs, but all that exists outside are a number of wooden or metal pegs driven into the ground. There are no "lines" on the rough ground surface and the surveyor should constantly have in mind the layout of the survey as indicated on plan when working in the field.

We shall, therefore, repeat the method given above for dropping a perpendicular onto a "line" from a given point outside it, but shall describe the actual field operations which must be carried out in order to determine the position of peg D.

Note:
Remember: All that exists of this "line" in the field are the two pegs A and B marking the extremities of it, together with a small peg in the ground representing point C.

The procedure is as follows:

(i) The surveyor stands at peg A, one of his assistants holds the zero of the tape at peg C, and another assistant runs out the tape to the required distance (10 m, 20 m .or 60 m, according to the scale of operations in the field).

(ii) The second assistant with the tape stretched out to the predetermined distance moves towards peg A and is directed by the surveyor at A, who is looking towards peg B, to place the end of the tape in such a position that it is in line with A and B. Peg E is then driven into the ground.

(iii) The same procedure is followed again with the assistant on the other side closer to peg 8, and, under constant directions from the surveyor at A, he places the peg F in the ground.

(iv) The tape is then stretched out between E and F and a peg D is driven into the ground exactly half-way between them. The pegs C and D mark the position of the survey line which is perpendicular to the line represented in the field by pegs A and B.

Remember that the surveyor is able to direct his assistants in their movements,. because he has a clear picture in his mind of the conditions as illustrated in **Figure 2.21**. All that appears outside in the field is a series of pegs driven into the ground as illustrated in **Figure 2.22**.

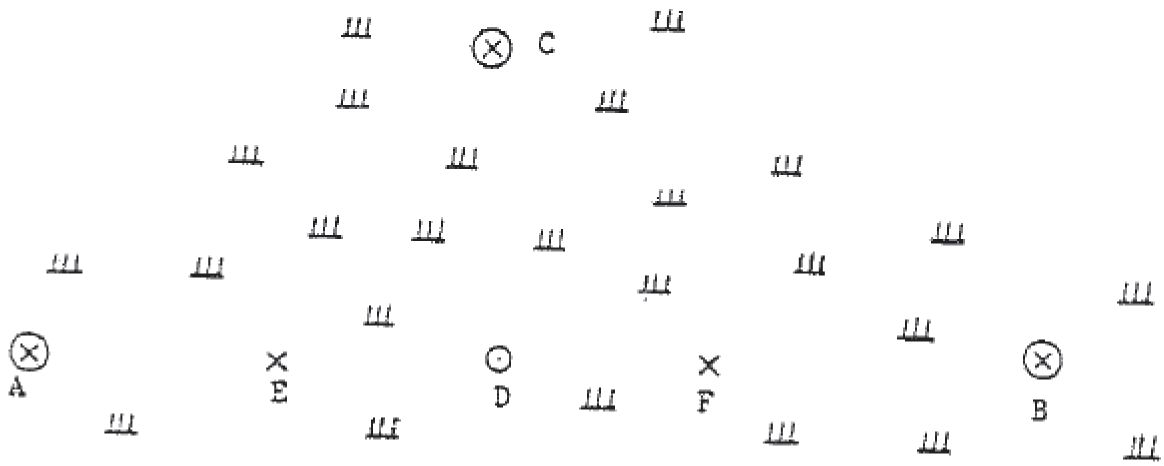


Figure 2.22 Positions of pegs setting out a perpendicular from C onto the line AB

Method 2

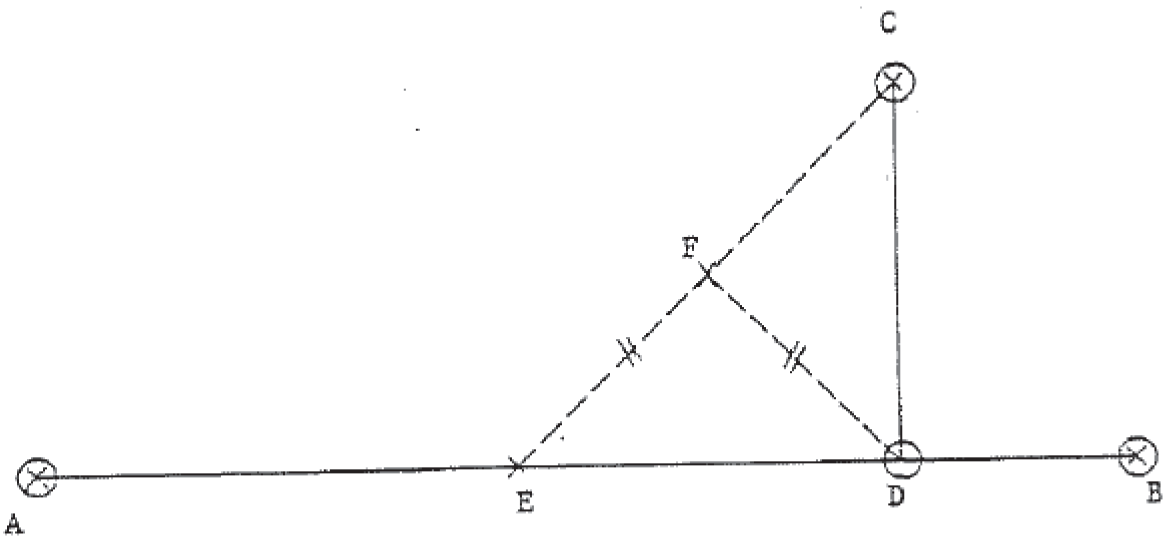


Figure 2.23 A perpendicular onto the line AB from a given point C outside it

Take any point E on the line AB.
Find the midpoint F on the line EC.
With F as zero establish a peg D on the line AB so that FD = FE.
CD is the required perpendicular from peg C onto the line AB.

2.19.3 Field equipment

The equipment considered here is used in several survey techniques, although it is treated in this section from the point of view of the chain survey or linear measurement methods only.

Distance measuring equipment and accessories

Land chain

Rathborne's original chain was for the measurement of land and its area, and the length was two or three poles (1 pole = 16½ ft).

Gunter then developed a four-pole chain 66 ft in length and divided into 100 wire links. This allowed rapid area measurement, since ten into square Gunter's chains gave exactly one acre of 43 560 square feet.

As engineering developed, the need for measurements in feet resulted in the Engineer's Chain, 100 ft in length and divided into 100 wire links.

Despite these changes, the actual construction of the chain remained more or less the same - 100 main wire links, with three small ring links between each for flexibility. Brass handles are attached to the ends, and the nominal length or the chain is measured to the outside of the handles.

Brass tallies are attached at every tenth link, and marked by teeth or indentations to show how many tens of links between the tally and the nearest end of the chain. Thus, if a tally is marked with two prongs or teeth, it is 20 links from the nearest end of the chain and 80 links from the other.

The others are at 10 or 90, 30/70, 40/60, and the tally at the centre, 50 links from either end is a simple circular disc.

Obviously, a common source of error in chain measurement is mis-reading - such as noting 20 units when it should be 80 units.

The metric land chain is constructed as described but made in 1engths of 20, 25, 30 or 50 m overall. The preferred length is 20 m, since this simplifies booking and reading as compared with the other versions.

On a 20m chain, tallies are placed at every 1 and 5 m and each link is 0,2 m. A British Standard Specification is being prepared but at present details depend on the manufacturer.

The tally markers are yellow, with no markings and are attached to the middle connecting ring at every whole metre position. Attached in the same way is a red tally marker of a different shape and with raised or engraved numerals for each 5 m position. See **Figure 2.24**.

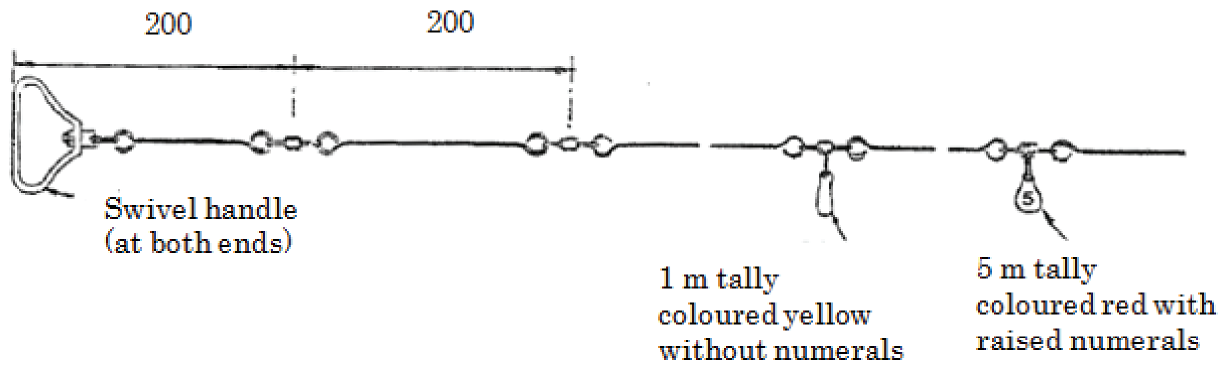


Figure 2.24 Land chain markings

Steel band chain

This is a continuous ribbon of steel, furnished with brass handles at each end, and was the logical development from the chain when manufacturing techniques permitted production of steel strip in long lengths.

The band is more accurate than the link chain, but more easily damaged and greater care is required in its handling.

For "land" work, replacing the land chain, the best variety is probably 20 m long, in widths of 13 or 16 mm. Lengths of 25, 30 and 50 mare also available.

Bands may be of blued steel, graduated by brass studs at every 0, 2 m, numbered at every 1,0 or 2,0 m, or alternatively of bright steel with divisions etched on one or both sides. The simple stud marking is probably the best for chain-type work.

Handles are generally included in the measurement, but not always - inspect the chain before use. The nominal length of chains and bands is usually engraved in the brass handles. When not in use, the steel band is wound onto a steel cross or drum.

Surveyor's band
This is a lighter, smaller section band than the band chain, still more accurate but also even more susceptible to damage. The type most suitable for engineering work is the narrow band, width 6 mm, in lengths of 30, 50 or 100 m.

Invar measuring tape
The equipment listed earlier is used for measurement along the surface of the ground, ie, it is fully supported, and must not be lifted off the ground and allowed to hang in loops. Surface measurement always has built-in errors small changes in surface levels, grass or bushes pulling the band off line, and so on.

In order to avoid these troubles, very accurate measurement is made by supporting the ends of the tape or band so that it is kept clear of the ground altogether.

The tape or band then hangs in the form of the curve known as the catenary, and in order to take full advantage of this method the material of the tape must be more stable in temperature movement than ordinary band steel.

Invar steel, having a coefficient of expansion of the order of 0,000 000 4 per degree Celsius, is the material used, and since such bands are never dragged along the ground and always kept clear of the surface, they are made of light section and are often termed tapes rather than bands.

Widths typically 3 mm or 6 mm, lengths 20, 24, 30, 50 or 100 m, with terminal marks (ends of the stated length) engraved at about 0,3 m from each end.

Usually ten 1 mm graduations are engraved each side of the terminal mark.

Various accessories are required, including tape thermometer, spring balance regulate tension applied, etc.

Summary of equipment for "long" measurement
- The chain for work of low order accuracy, where there are rough conditions to met with - ideal in ploughed fields, and vehicle wheels will only bend a link usually and this can be straightened.
- The band chain for better accuracy. It withstands a fair amount of rough usage, but will break if "kinked" or a wheel runs over an upstanding edge.
- The surveyor's band for better accuracy still, but it must not be dragged along rough surfaces such as a band chain, and care is necessary about oiling and drying or the graduations will become unreadable.

- The Invar tape used in catenary for measurement of the highest accuracy.
- "Short" steel tape- basically similar to the linen tape, lengths of 10, 15, 20, 25 or 30 m, widths 9,5 or 12,7 mm. Graduated at every millimetre, may be of blued steel with etched markings, or nickel-plated with black markings, or white enameled with black markings.
  Used in chain survey for important detail or short lines. Used in building survey where precise measurements are required, e.g., positions of steelwork, setting out steelwork, and so on.
- Pocket rule - small steel tape, 2 or 3 m, contained in a D-shaped case. When pulled out, the tape remains rigid. Useful on building work, particularly for "inside" measurements, since the 51 mm case width can form part of the measurement.
- Surveyor's folding rod - wooden lath, fourfold (four lengths joined by three hinges) or twofold, length 2m opened, width 25 mm, graduated in millimetres both sides. Particular field of use is in building survey for odd lengths', heights, set-backs, etc.
- Folding boxwood rule - the traditional "carpenter's rule", length 0,6 or 1,0 m width 35 mm. Useful standby if no rod available.
- Chaining arrow - steel wire pin, length 0,4 mm with a loop at one end. Used for marking the position of the end of a length of chain or band on the ground.

Should have a piece of red cloth tied into the loop to avoid losing the arrow in grass or bush. A set of five or six arrows should be used with a 20 m band or chain.

Other equipment covered previously includes ranging rods, optical squares, hard level, clinometers, linen and plastic, (including fibreglass) tapes and the magnetic compass.

2.19.4 Accuracy of distance measuring methods

METHOD	USUAL PRECISION	USE	INSTRUMENT FOR ANGULAR MEASUREMENT WITH SIMILAR PRECISION
Pacing	1/100 – 1/200	Reconnaissance; small-scale mapping; checking tape measurements.	Indian clinometer; hand compass.
Stadia	1/300 – 1/1 000	Rough traverses; location of details for mapping; checking more	Surveyor's compass; theodolite; telescopic

		precise measurements.	alidade of plane table.
Ordinary chaining	1/1 000 – 1/5 000	Traverses for land surveys and for control of route and topographical surveys; ordinary construction work.	20” Theodolite (reading angles in both directions)
Precision chaining	1/10 000 – 1/30 000	Traverses for city surveys; base lines for triangulation of intermediate precision; precise construction work.	10” Repeating theodolite.
Base line measurement	1/50 000 – 1/1 000 000	Triangulation of high precision for large areas, city surveys, or long bridges and tunnels.	10” Repeating theodolite; electronic direction measurement instrument.

Table 2.3

2.20 Catenary taping

However carefully surface taping is carried out there will always be errors from slight irregularities in the ground surface and small deviations in alignment. For the highest accuracy, these must be eliminated by suspending the tape in a catenary curve clear of the and all obstructions, then applying all appropriate corrections.

A tape to be used in catenary should be standardized in catenary, then if held at the correct tension there will be no need to correct for sag.

When a tape has been standardized on the flat, but is used in catenary, a correction for sag will be necessary. A more comprehensive be referred to for details of such a correction.

Alignment, slope and temperature must all be observed more carefully. Alignment is checked by theodolite rather than the naked eye, slope is measured by theodolite or by leveling the ends of tape lengths, temperatures must be more frequently and carefully measured.

Various field methods are used, with varying standards of precision, the best probably giving 1/200 000 accuracy or better.

In the most elementary form of catenary taping, the tape is held about waist high by the two chainmen, one using a spring balance, and the tape terminal marks are transferred to the ground by plumb-bob. Better methods use stakes placed at the terminal marks, with the tape passed over the top of the stakes.

2.21 To set out and/or measure angles in the field by means of tapes only

The field work for the setting out of angles by means of tapes is an application of the geometrical and trigonometrical principles involved in the plotting of angles on plan.

Three such methods can be used in the drawing office.

TWO other methods of plotting angles are by means of:
(i) a protractor;
(ii) co-ordinates.

However, these last two methods obviously do not apply in this case. It must be remembered that, while the simplest and most commonly used method of setting out or measuring angles in the field is by means of the theodolite (or other angle-measuring instruments such as a stereometer or compass), there may be many occasions where these instruments are not available, but the surveyor is still required to carry out the task with the simple means in his possession, viz, a tape, chaining plans, and a book of mathematical tables.

The following methods may be useful:

2.21.1 Chord method

Mathematical principle as applied to the plotting of angles on plan.

By this method the angle is constructed by two intersecting arcs of different radii which are drawn from the two ends of a base line as illustrated in **Figure 2.25**.

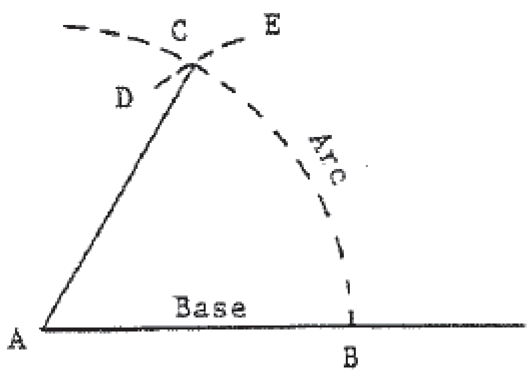


Figure 2.25 Plotting of the angle bag by means of two intersecting arcs

With A as centre and radius, the predetermined length of the base line (say 10 units) describes an arc BC. With B as centre and radius equal to AB x unit chord of the required angle (ie, the chord of the required angle for a radius of 1 unit) describe a second arc DE to cut the first arc in C.

Join AC. Then BAC is the required angle

Discussion
The chord of an angle
Study the following figure:

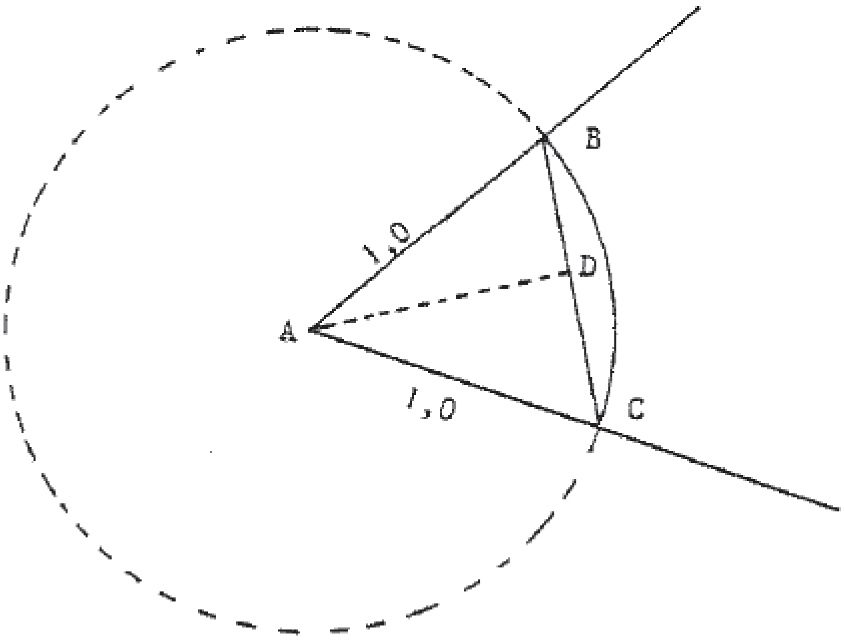


Figure 2.26 Chord of the angle BAC

From the above it can be seen that AB and AC represent two radii of a circle whose centre is A. As B and C are two points on the circumference of this circle, the straight line BC represents the chord of the angle BAC.

With the radii equal to unity, a fixed scale of chord lengths for angles from 0° - 90° can be determined and is available in any book of trigonometric tables.

Worked Example 2.16

Chord of 30° with unit radius = 0,517 638 0.
If a table of chords is not availab1e the chord can be calculated as follows:

Chord for angle of unit radius = 2 sin ½ (angle), ie, 2 sin BAD
∴ Chord of 30° per unit radius = 2 x sin 15°
= 2 x 0,258 819 0
= 0,517 638 0

Plot an angle of 38° by means of the chord method.

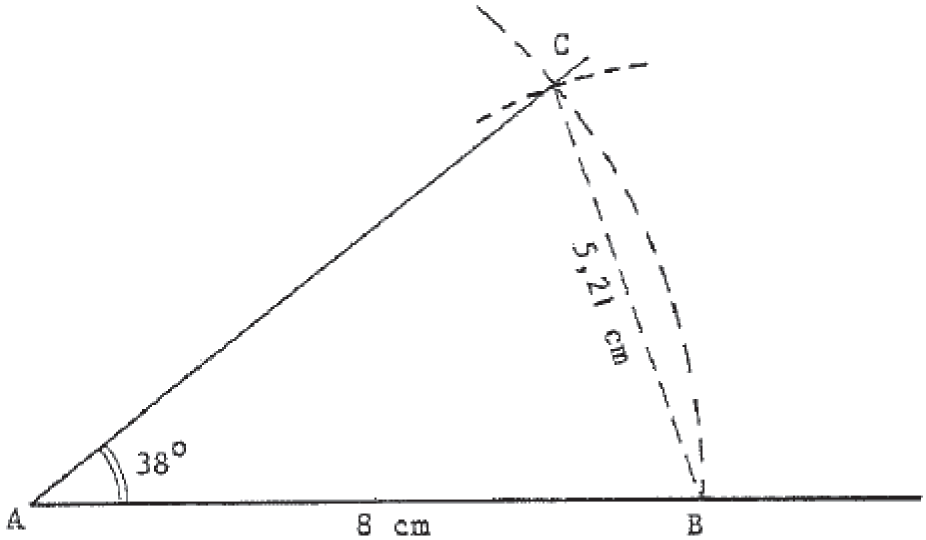


Figure 2.27 Plotting an angle of 38° by means of the chord method

With A as centre and the length of the base line AB as radius (say 8 cm), describe an arc BC.

With B as centre and radius = 8 x chord 38°
= 8 x 0,651 136 4
= 5,21 cm

describe a second arc to cut the first arc in C.

BAC is the required angle
(Check. Radius x 2 x sin ½ (angle) = 8 x 2 x sin 19°
= 8 x 2 x 0,325 568 2
= 5,21

To set out an angle in the field by means of the "chord method"
Required: to set out an angle of 38° from the survey line AB using a 60 m tape.

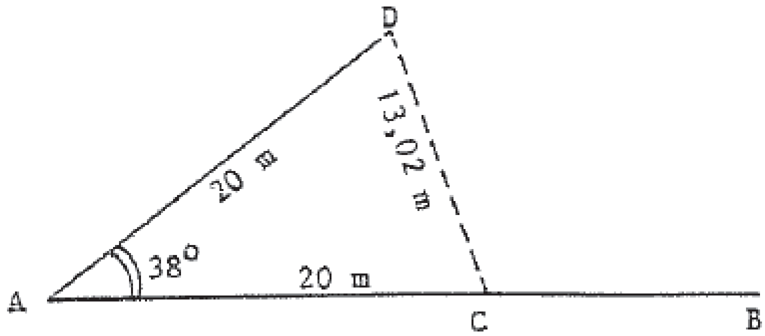


Figure 2.28 Setting out an angle of 38° by means of the chord method

(i) Mark off a convenient distance AC (say 20m) on the established survey line AB.
Chord of 38° with radius 20 = 20 x chord 38°
= 20 x 0,651
= 13,02 m

(ii) Hold the zero of the tape at A, and the 33,02 mark at C (33,02 = 20 + 13,02).

(iii) Holding the 20 m mark on the tape, move outwards sections until both of the tape are straightened out. Mark the point D.

(iv) Then BAD is the required angle of 38°.

2.21.2 Tangent method

This method is based upon the mathematical principle that, in a right-angled triangle, the $\frac{perpendicular}{base} = tangent\ of\ the\ angle$

(ie, Base x tan angle= perpendicular.)

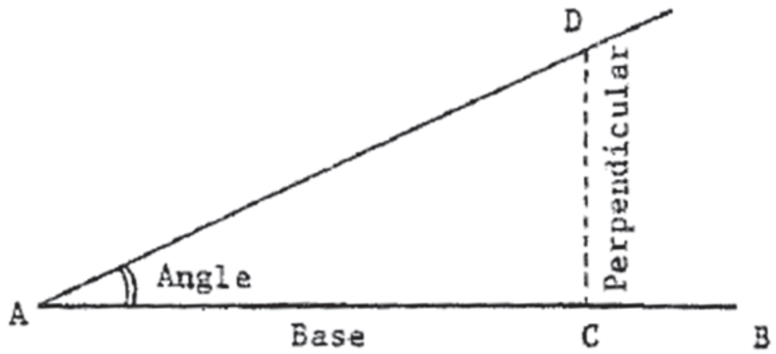


Figure 2.29

(i) On the given survey line mark off a base AC of any convenient length.

(ii) At C set off a line perpendicular to the given survey line. Use the 3:4:5 method.

(iii) From the table of natural tangents determine the tan of the required angle and multiply it by the length of the base.

(iv) On the perpendicular just established, mark off a distance CD= base tan of required angle.

(v) DAB is the required angle.

Note:
If the required angle is greater than 45°, set out the complement of the angle, ie, 90° - angle, as per sketch.

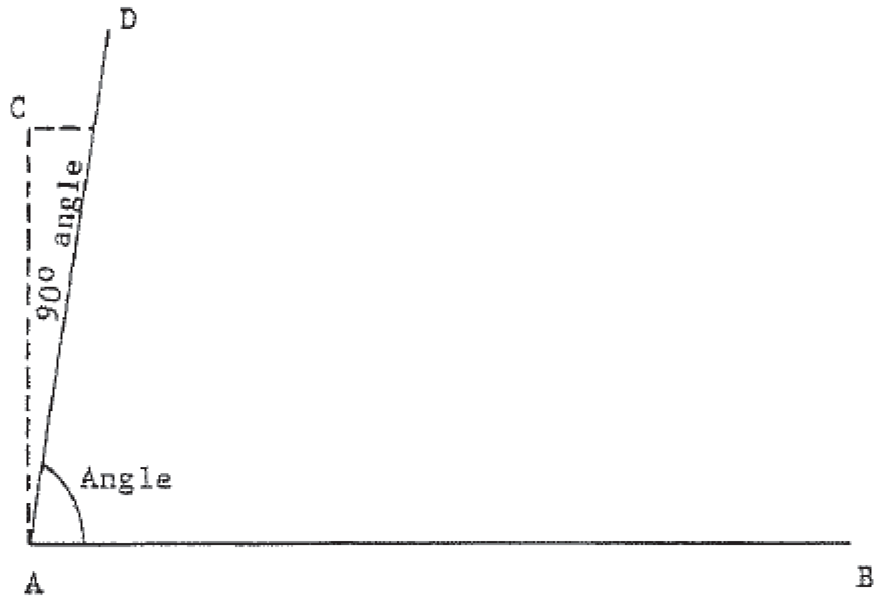


Figure 2.30

If the angle to be set off is 75° erect a perpendicular AC and then set off the angle DAC = 90° - 75°
= 15°

2.21.3 Sine - cosine method
Mathematical principles involved

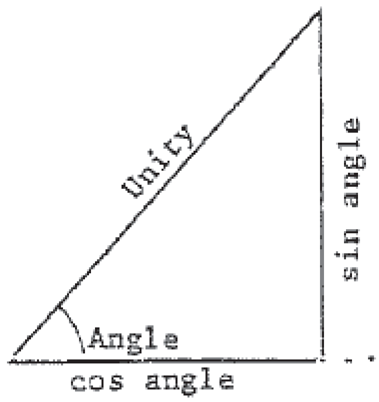


Figure 2.31

In **Figure 2.31**, if the hypotenuse of the right-angled triangle is unit, the cosine of the required angle represents the base and the sine of the required angle represents the perpendicular.

Practical method to be used in the field
Required: to set off an angle of 35° by means of the sine-cosine method.

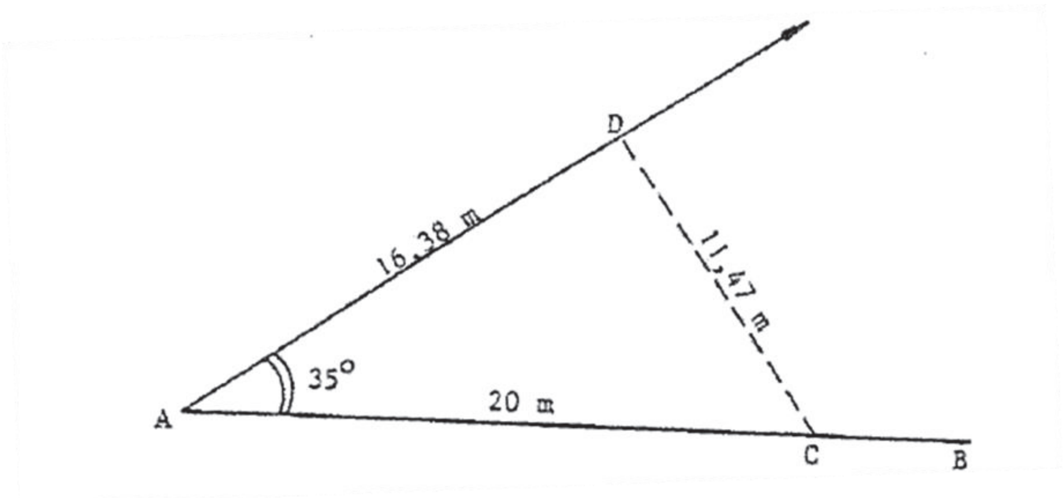


Figure 2.32 Setting out an angle of 35° by means of the sine-cosine method

(i) On the survey line AB mark off any convenient distance AC (say 20m).
Mathematical calculation
sin 35° x 20 = 0,573 5 x 20 = 11,47
cos 35° x 20 = 0,819 15 x 50 : 16,38

(ii) Place the zero of the tape on A and the 27,85 mark on c.
(27,85 = 11,47 + 16,38)

(iii) Holding the tape at the 16,38 mark, move outwards unti1 both sections are fully extended. Mark this point D. Then DAC is the required angle.

Note:
When the angle is between 45° and 90°, its complement (90° - angle) may be set out as in the tangent method.

2.22 To set off large angles by means of tapes only

When large angles are required to be set off proceed as illustrated in **Figure 2.34** with any one of the three methods already described.

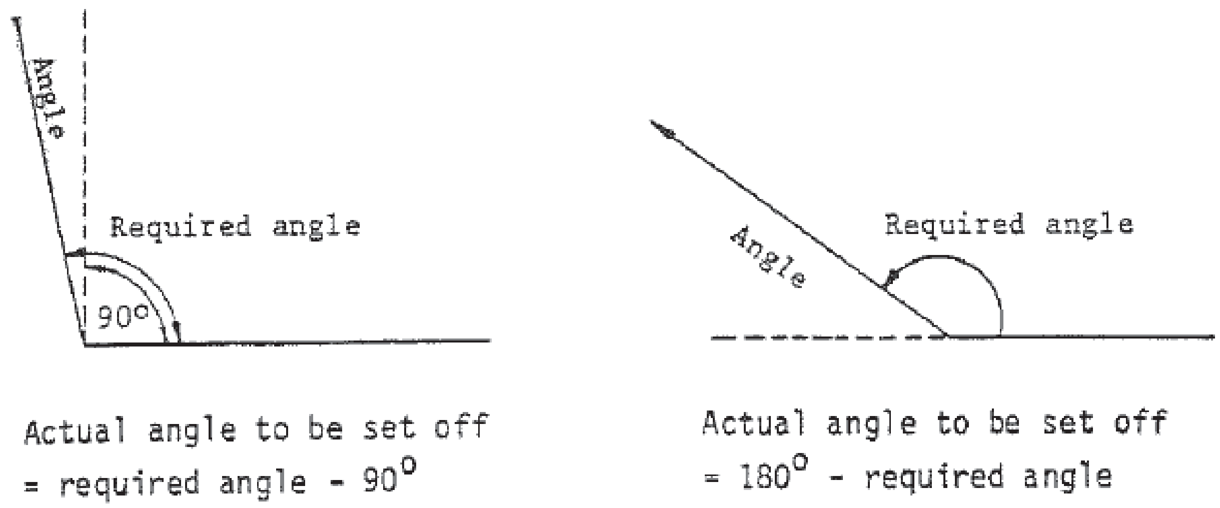


Figure 2.33

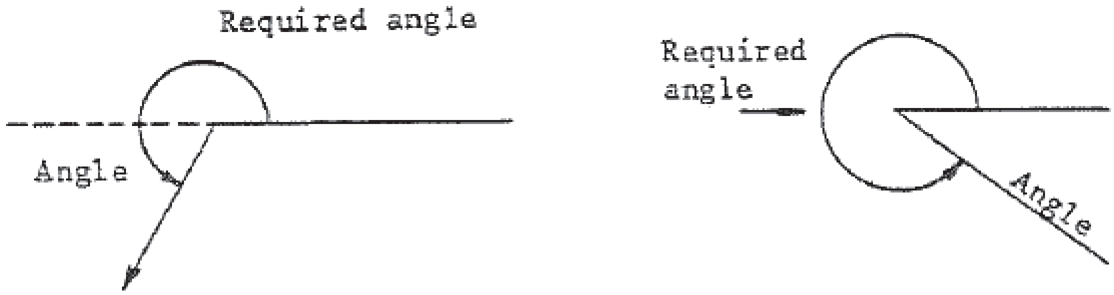


Figure 2.34 Method of setting off large angles

2.23 Typical angular measurement problems

The angle between two given survey lines may be measured with a tape b y any of the three methods just described but in a somewhat reversed order.

Worked Example 2.17

Two fences meet an angle and the surveyor, who has only a tape at his disposal, is required to determine this angle of intersection.

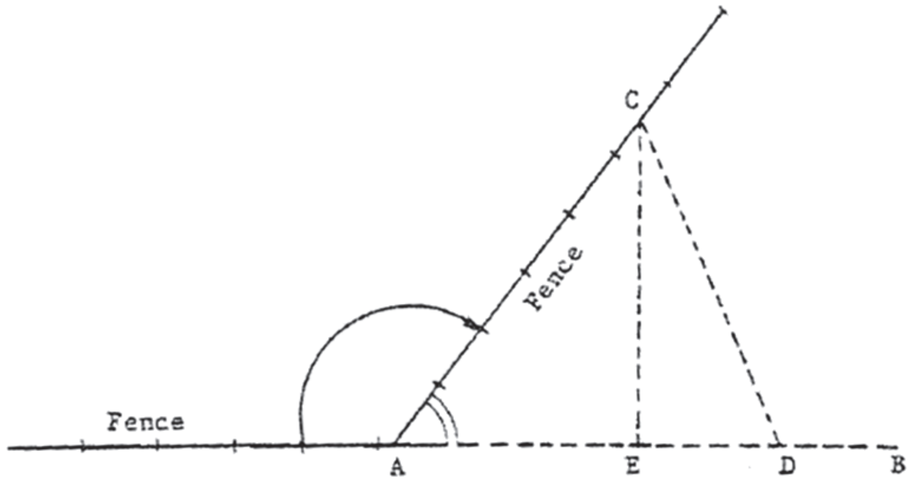


Figure 2.35 Intersection of two fences at A

Solution:

As the angle of intersection of the the two fences is greater than 90°, extend the line of one of them to B as illustrated. You are then required to determine angle CAB. The three methods which can be employed are described as follows:

1. Chord method
   (i) From A set off equal distances AD and AC;
   (ii) measure the distance CD;
   (iii) angle CAD (ie, angle CAB) = angle whose chord = $\frac{CD}{AD}$

2. Tangent method

(i) From C drop a perpendicular CE onto the line AB;
(ii) measure the distance AE and CE;
(iii) angle CAB = angle whose tan = $\frac{CE}{AE}$

3. Sine-cosine method
(i) From C drop a perpendicular CE onto the line AB;
(ii) measure the distance AC, AE and CE;
(iii) angle CAB = angle whose sin is $\frac{CE}{AC}$
= angle whose cos is $\frac{AE}{AC}$ (check)

If sufficient care is applied to the measurement of the distances required in the methods just described, a considerable degree of accuracy can be attained in either the setting out of angles or the measurement of angles, the error often not exceeding 5', which is considerably less than that which can be plotted on a plan.

With care and a clear understanding of what is required, the lack of an angle-measuring instrument should present no difficulties to the resourceful surveyor when faced with field problems such as these.

To find the distance between two points which are hidden from one another by an obstacle, and to extend a survey beyond that obstacle
Method 1
By setting off right angles.

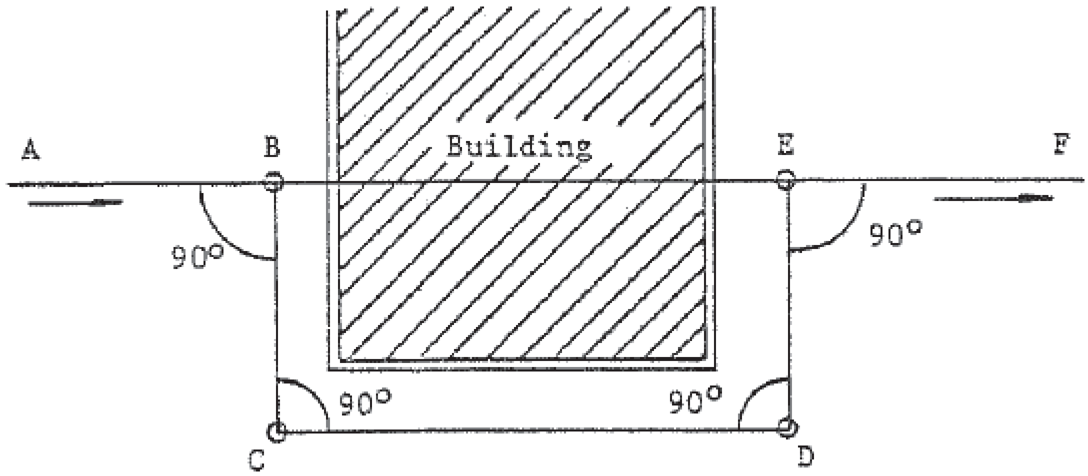


Figure 2.36 Running a line beyond an obstacle

Procedure
The survey line is brought from A up to point B.
At B, set off a line BC at right angles to AB.
At C set off a line CD to clear the obstacle, and at right angles to BC.
At D set off DE at right angles to CD and equal in length to BC.
At E set off a line EF at right angles to DE.
The direction EF is the extension of the survey line and distance CD = BE.

Method 2
By using a random line.

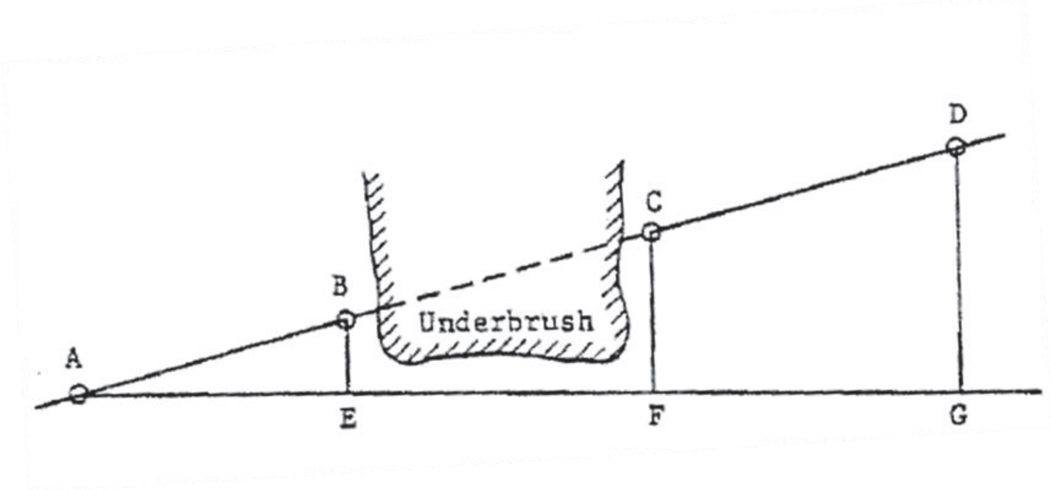


Figure 2.37 Running a line beyond an obstacle

Procedure
1. The survey line is brought up to point B on the one side of the obstacle which, for example, may be a thick growth of underbrush through which it; intended to clear a path for the continuation of the survey line.
2. From a point A on the main survey line set off a straight line AG to pass the obstacle as closely as possible (known as a random line).
3. From point 8 drop a perpendicular BE onto the line AG.
4. Mark a point F on the line AG and then set off perpendiculars from and G on the line AG.
5. Points C and 0 on these perpendiculars will give the position and direction of the survey line on the .other side of the obstacle, the required lengths FC and GO being calculated as follows:
   (a) From the similar triangles ABE and ACF
   $\frac{AF}{AE} = \frac{FC}{BE}$ (in similar triangles the ratios of corresponding sides are equal)
   $\therefore FC = \frac{AF}{AE} \times BE$
   (b) In similar triangles ABE and ADG
   $\frac{AG}{AE} = \frac{GD}{BE}$
   $\therefore GD = \frac{AG}{AE} \times BE$
   (c) To determine the distance BC on the survey line, take the two similar triangles ABE and ACF
   $\frac{AF}{AE} = \frac{GD}{BE}$
   $\therefore GD = \frac{AG}{AE} \times AB$
   and BC = AC – AB

To run a survey line between two points on opposite sides of a hill when neither point can be seen from the other

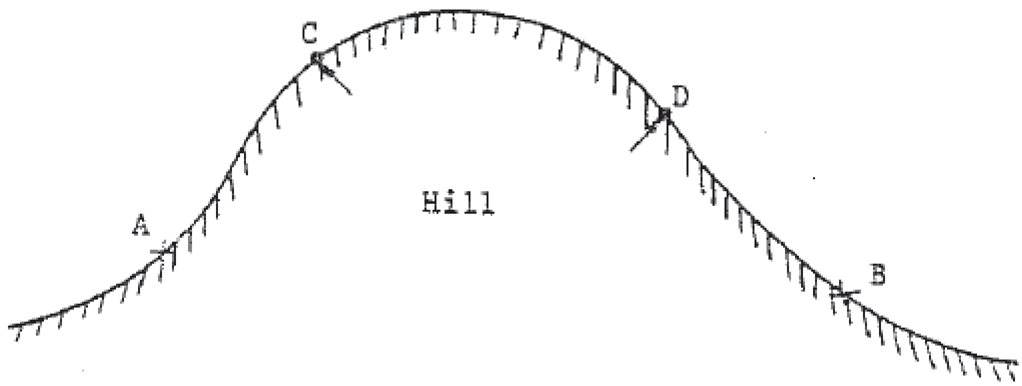


Figure 2.38 Running a survey line over a hill

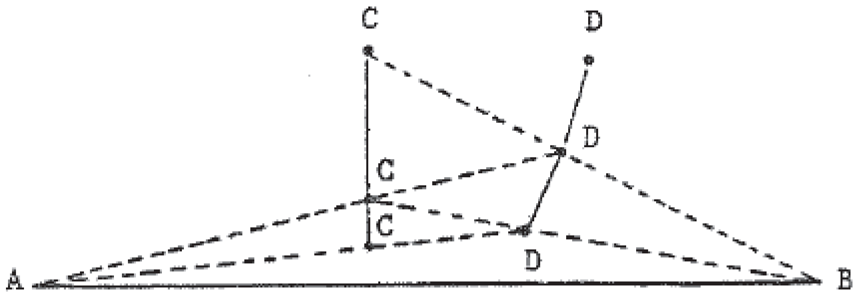


Figure 2.39

Let A and B be the two established points with the top of a hill between them. One assistant "C" moves as far down the hill towards A as he can while still being able to see point B (or a rod at B).

The other assistant "D" moves as far down the other side of the hill towards B while still being able to see point A.

C and D are both probably out of line but they proceed to line each other in alternately. C lines D in with himself and B. D then lines C in with himself and point A and they continue shuffling each other in until each is satisfied that the other is in line with the distant points A and B.

The distances AC, CD and DB are then measured.

To run a line through a given point parallel to a given line
Referring to **Figure 2.40**, it is required to run a line through point C parallel to the line AB.

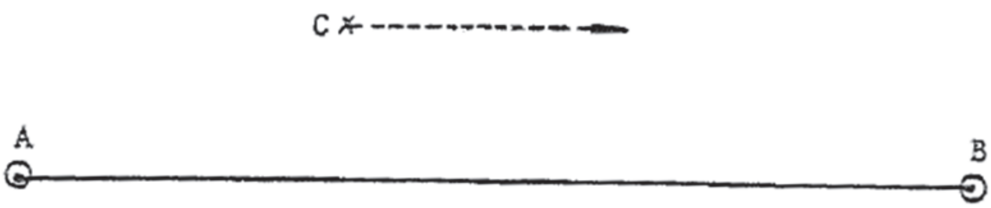


Figure 2.40 Running a line through a given point parallel to a given line

From point C drop a perpendicular CD onto the given survey line. At any other point E on the line AB set off a perpendicular EF of equal length to CD.

Then CF is a line parallel to AB through point C.

Method 1
By means of perpendiculars.

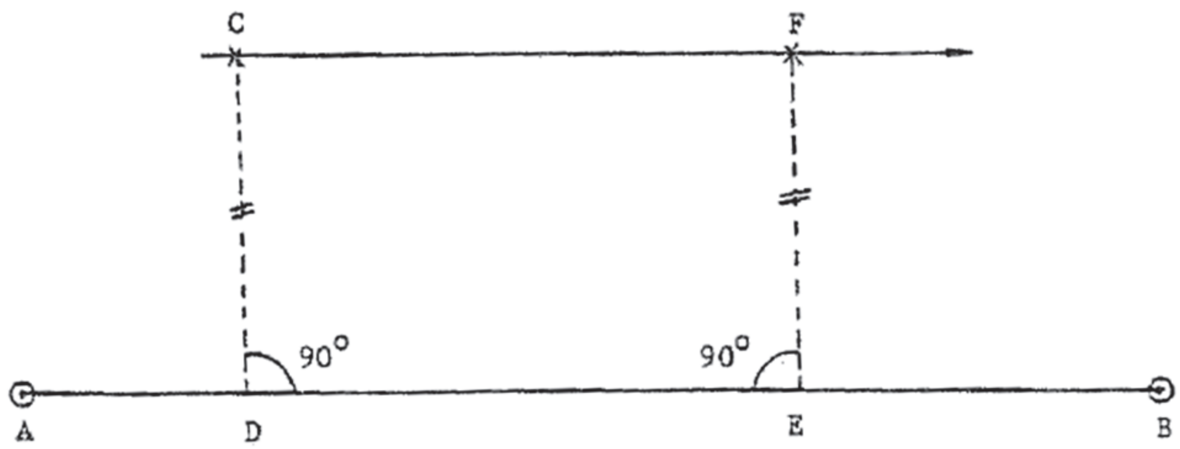


Figure 2.41

Method 2
By means of diagonals.

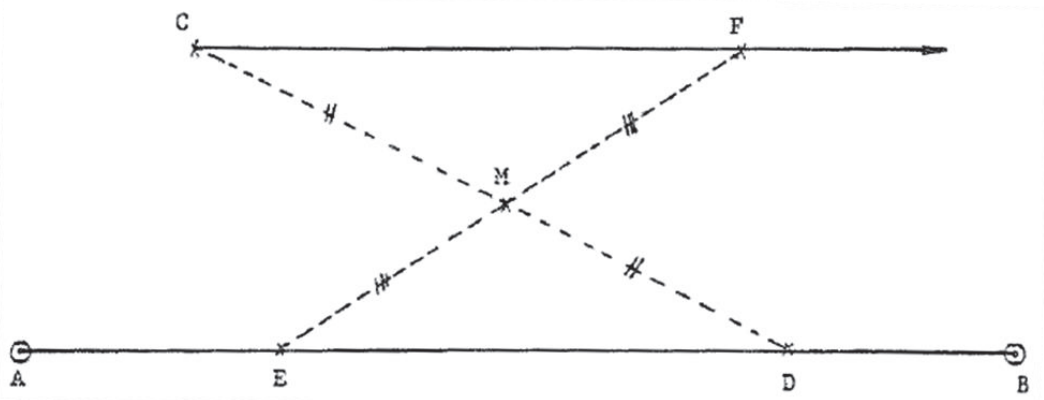


Figure 2.41 Running a line through a given point parallel to a given line

Take any convenient point D in the given line AB and find the mid-point M on the 1ine CD.

Take a second point E on the line. AB, and run a line EMF through M making MF = M.E.

Then CF is a line parallel to AB through point C.

To find the distance between two obstructed points when both are accessible

Method 1
By means of intersecting diagonals.

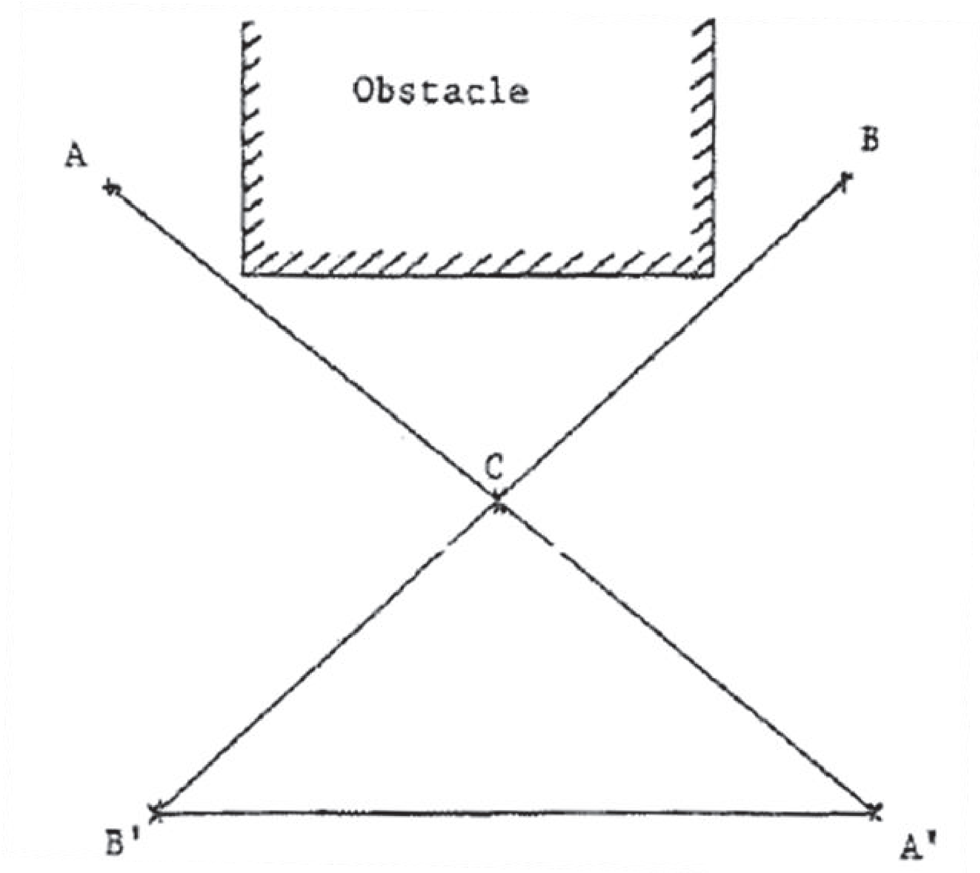


Figure 2.42 Finding distance between two obstructed points

Assume a point C through which lines from both A and B may be set off.

Extend the lines AC and BC to points A' and B'' respectively, so that
AC = CA' and BC = CB',
then A'B' = AB.

Method 2
From similar triangles.
Establish point C in a position so that AC = BC
Place points D and E on the lines so that CD = CE.
Measure distance DE.

Then AB = $\frac{AC \times DE}{CD}$ (from the similar triangles ABC and DEC).

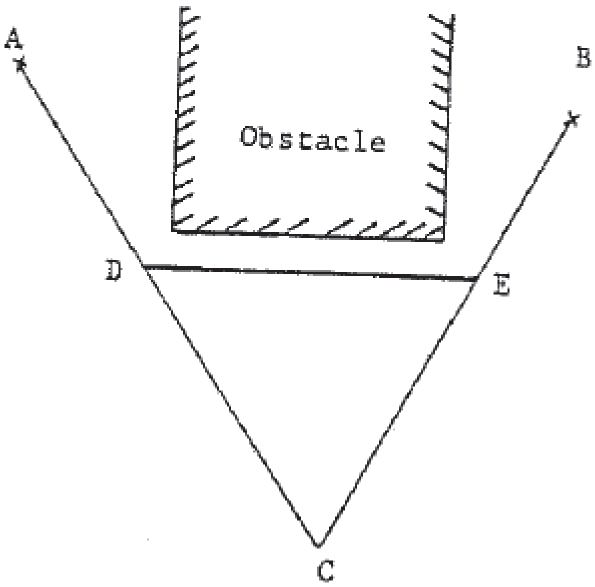


Figure 2.43 Finding distance between two obstructed points

To find the distance between two points when one is inaccessible

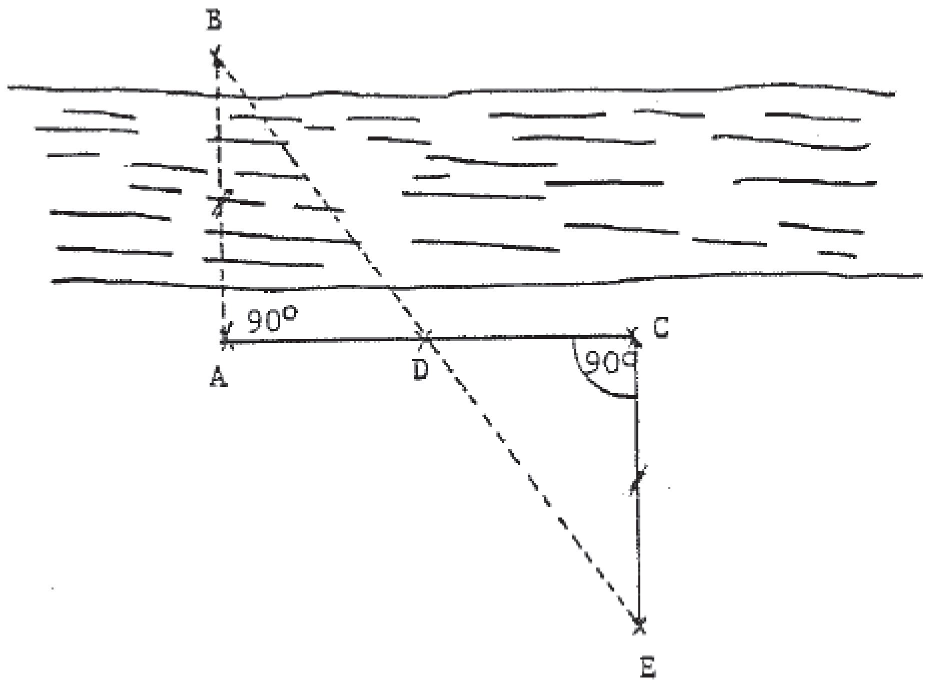


Figure 2.44 Finding distance between two points when one is inaccessible

Set off the line AC any convenient length but at right angles to AB. The simplest method is with a tape.
Find D, the mid-point of AC.
At C set off a line at right angles to AC.
Move along this line until points D and B are in line.
This position is point E.
Then CE = AB.

Note:
AC may be at any angle to AS, provided that the same angle is set off at C.

To find the distance between two points when both are inaccessible

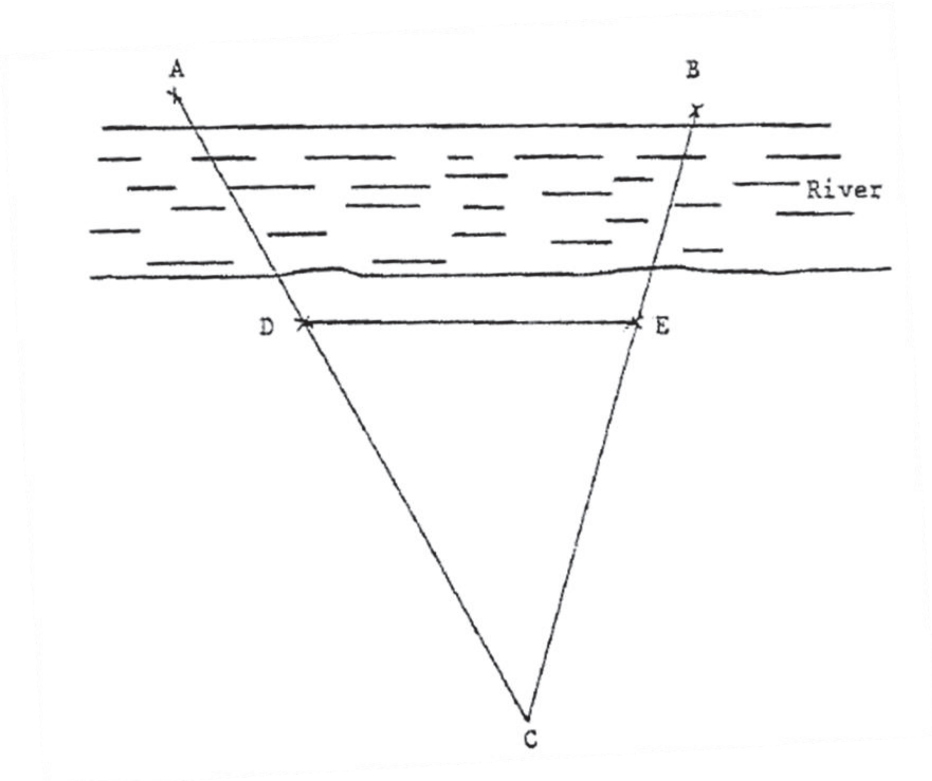


Figure 2.45 Finding distance between two inaccessible points

Worked Example 2.18

A and B are two points on the same side of a river and you are required to determine the distance between them from the opposite side.

Method

(i) Establish a point C some distance back from the river bank.
(ii) Determine the distances CA and CB by the method described in the previous problem.
(iii) Take a point D on the line AC and measure the distance CD.
(iv) Establish point Eon the line BC so that DE will be parallel to AB, and thus form two similar triangles CAB and CDE. In these two triangles,

$$\frac{CE}{BC} = \frac{CD}{AC}$$

$$\therefore CE = \frac{CD \times BC}{AC}$$

(v) Measure the distance DE.
(vi) Then in these same two similar triangles,

$$\frac{AB}{DE} = \frac{AC}{CD}$$
$$\therefore AB = \frac{AC \times DE}{CD}$$

2.24 Sources of error in taping

- The application of excessive tension (possibly accidentally and not noticed) will give the tape a "SET", permanently affecting its accuracy. Do not jerk it or put your weight behind it.
- The ends of some tapes do not start at zero and if you, as the surveyor, fail to point this out to your chainman, you may find all your results are 200 mm too short, or whatever the case may be.
  This could be called a mistake.
- Steel tapes (including metallic) are susceptible to kinking if misused and these kinks can be a further source of error. Don't step on a tape or allow vehicles to drive over it.
- Twisted tapes. Always see to it that for each measurement the tape is perfectly straight.
- If the tape is not standardized or you do not know whether it is or not, either check it against a known base line or a standardized tape.
- Incorrectly applied corrections will also affect results. Be careful.
- In catenary taping work a non-vertical ranging rod or marker could result in quite large errors, especially an accumulation of a series of such measurements over a long distance. Use staff bubbles or similar devices to ensure verticality.
- Sources of other errors have been covered in previous lectures. Remember them and the methods of correcting or avoiding them.

2.25 Mistakes in chaining

As in any other form of surveying, mistakes can only be kept to a minimum by careful and conscientious work. Their effects cannot be calculated or allowed for. Some common examples you should try to avoid are set out below.

- Miscounting the number of chain lengths measured in a line.
- Misreading the chain links or tallies.
- Reading numbers incorrectly, eg, "6" for "9".
- Incorrect booking, eg, chainman calls "fifty-three" and you book it as 50,3.
- Measuring the slope of the ground instead of the slope of the tape.
  There can be a considerable difference between the two.
- General mistakes as listed in previous lectures.

2.26Plotting of chain surveys

Plotting the survey is usually best done by the surveyor because of his familiarity with the details of the work, but he must also be reasonably skilled as a draughtsman.

2.26.1 Draughting equipment

The following list gives the main items required for a chain survey plot. In emergency, some might be omitted, and for ink work the list would have to be extended. Pencil work only is covered here.

- Drawing board plus T-square, or large drawing table
- Steel straight-edge
- Set squares, 45 and 60 degrees
- Protractor, 360 degrees
- Scales as appropriate
- Bearn compasses
- 150 mm compasses/dividers
- Springbow compasses/dividers
- Steel pricker with fine needle point
- French curves
- Soft and hard erasers
- Pencils, H, 2H, 3H, 4H
- Drawing pins
- Paper weights

When plotting building surveys, the board and T square are essential, but the T-square is of little use on a chain survey except to rule the sheet margins and to rule guide lines for the lettering of notes.

The steel straight-edge is used to rule long lines, and a parallel rule would be useful. The parallel rule is not so popular today.

Beam compasses are needed for drawing long intersecting arcs in the plotting of triangles.

The steel pricker is used to mark points accurately and finely on the paper - at small scales, a pencil mark might cover a scale area of several metres.

French curves are used to draw smooth curves to connect points of detail. When plotting a road or a railway, a set of railway curves is useful.

Paper weights may be handy for flattening a rolled-up sheet of paper, or to steady a loose drawing.

2.26.2 Plotting the framework

- Decide on the scale to be used, calculate the approximate dimensions of the final plan, and attach a suitable sized sheet of paper to the board or table.

- Decide on the layout of the drawing on the paper - generally, north is placed towards the top of the sheet, but this is not essential.

- Using the steel straight-edge, draw a single, long line with a fine hard pencil, approximately in the desired position of the base line of the survey. Mark a station point on one end of this line with the steel pricker, and circle the mark lightly in pencil.

- Add the station identification letter or name lightly beside the station mark. (All stations are marked in this way - the fine puncture of the steel point is easily overlooked.)

- From the marked station, set out all the station points along the base line, scaling the distances noted on the chain line in the field-book.

- Mark each station by pricker, circle and letter. Markings must be light, since very often in a chain survey the stations and lines are only required for plotting, and when all detail has been plotted they may not be required again.

- Plot all the triangles of the framework off the base line, by swinging arcs of the appropriate scale lengths. Do not set the compass points to radius against the face of the scale rule -this rapidly ruins a scale.

- A light line should be drawn the full length of the board at the bottom of the sheet, then all the arc lengths required set out to scale along this line. The compasses' points may be set to the necessary distance on the line and not on the scale.

- When the triangles have been plotted by arc intersection, scale the lengths of the check lines off the drawing. Compare these lengths with the lengths obtained in the field; they should agree.

- If check line lengths do not all agree, check the plotting for errors, and check the field-book entries. Correct any errors found, but if no errors appear, it indicates that the fault lies in the field-work.

- A return visit to the site and some re-measurement may be needed. When the whole of the framework has been plotted and "proved", mark all station points and draw all chain lines in lightly.

- When the framework is complete, the detail may be plotted.

2.26.3 Plotting the detail

Plot all the detail from one chain line, complete and draw in, then move on to the next line, and repeat until the detail plotting is finished.

Offsets
Using the pricker, mark the chainages of all the offset points along the first chain line. Offset distances to the right or left of the chain line may be plotted in either of two ways, depending on how the field measurement was made.

If the offset right angle was measured by eye in the field, set it out on paper by eye also, and scale off the distance to the detail point and mark.

If the offset right angle was obtained by instrument in the field, erect a perpendicular at the point on the chain line, using a set square against the drawn line, then scale the offset distance along the perpendicular and mark the detail point. As an alternative to this, offset scales are faster, if available.

Ties
Plot ties by marking the chainages on the chain line, then swinging scale-length arcs to intersect at the point of detail. Mark the intersection.

2.26.4 Drawing-in-detail
When a line of detail has been plotted, draw the details in carefully, and accurately. Where a straight fence or hedge is fixed by tie lines, a minimum of two tie lines will have been used at each end of the straight. The fence, etc, should be drawn as a simple straight line between the two ends.

Where a curved hedge or similar line has been fixed by a number of offsets, do not connect the offset points by straight lines – they must be connected by a smooth curve representing the actual curve on the ground as nearly as possible. In such problems, the selection of the original offset points is criticial.

Building outlines must be drawn carefully, using the plotted points and the building dimensions noted in the field-book. Buildings may be lightly hatched.

Where detail is too small to draw the scale, or the general nature of an area of ground is to be shown, use appropriate conventional symbols; these can be found in a previous section.

2.26.5 Completing the plan
When all detail and symbols have been inserted, add any necessary notes.

For example, north point and magnetic declination of the area if required, the scale,, and then, depending on your company's policy, the firm's name, the job number and description, surveyed by ---, plotted by ---, traced by ---, checked by ---, etc, and space for dates of completion after each name.

Activity 2.1

1. The distance between two fixed points was measured with a 20 metre metallic tape and recorded as being 93,641 metres. This tape was subsequently checked against an accurate steel tape, and was found to be 0,06 per cent too long. Calculate, to three places of decimals, the true distance between the fixed points.
2. You are required to mark off a 300 metre distance with a 100 m long tape and you know that your tape is 0,03 m too short. What measurement must you make with this incorrect tape so that an accurate 300 m distance may be laid out? Answer to three places of decimals.
3. The true area of a plot of ground is 6,435 hectares. When measured with a certain tape, its area was calculated to be 5,993 hectares.
   (a) Was the incorrect tape too short or too long?
   (b) By what percentage was the tape too short or too long?
4. Calculate the correct volume of a reservoir if the measured volume is 1 250 cubic metres and the tape used to find the measured volume was 0,5% too long. Give your answer to the nearest cubic metre.

Activity 2.2

1. A base line was measured with a steel tape which was standardised at a temperature of 15 °C, and was found to be 173,328 metres. If the temperature during the time of measurement was 25 °C what is the correct length of the base line? Work to the nearest three decimals. (Assume COE = 0,000 010.)
2. Calculate the sag correction under the following conditions:
   Measured length = 75,000 m
   Mass of tape = 11 g per metre
   A tension of 80 N was applied
   Vertical angle= 0° (cos 0° = 1)
   Use both formulae to check your answer.
3. Calculate the horizontal distance AB if the inclined distance AB = 73,426 m and the vertical angle= 12° 15' 11''. Use the formula $D = S \cos \theta$.
4. What effect do the following have on the length of a tape?
   (a) Sag
   (b) Alignment
   (c) Slope
   (d) Tension
   (e) Temperature

Activity 2.3

1. Between what limits should the apex angles be in triangulation surveys and what are the triangles called when this condition is complied with?
2. If the scale of the plan is to be 1:500, then the survey must be measured to the nearest, "how many" metres?
3. Given ΔPQR with PQ = 45 m; QR = 60,05 m and RQ = 41,23 m.
   Carry out a check as in **Figure 2.14b** and give your answer in metres to two decimal places. **Hint:** find angle P first and the rest should follow.

Activity 2.4

1. What precision and instruments would be associated with:
   (a) pacing and
   (b) Stadia methods of surveying?
2. What is a surveyor's folding rod and where is it used?
3. Briefly discuss the "3:4:5" rule and its applications.

Activity 2.5

1. Show how you would measure the distance between two points when both are inaccessible.
2. What is the most elementary form of catenary taping.
3. Describe how the tangent method, of setting out an angle, is carried out.

Activity 2.6

1. Give a comprehensive list of the mistakes which may be encountered in chaining.
2. Discuss the plotting of offsets.
3. In tape surveying what is a set and what effects could it have onresults?

Self-Check

I am able to:	Yes	No
• Describe methods of direct linear measurement.		
• Describe the uses of chains, tapes and bands.		
• Describe the measuring sloping distances using chain, tape and band by:		
o Stepping; using a clinometer.		
• Demonstrate the calculation of slope correction for distances		

measured on an incline.		
• Demonstrate the graphical method used to correct distances measured on an incline.		
• Demonstrate ranging and measuring over a hill and through a depression.		
• Demonstrate measuring around a pond, across a river or busy road.		
• Demonstrate measuring when a building obstructs vision.		
• Give a description of equipment used for chain surveys.		
• Describe linear survey methods.		
• Demonstrate measuring offsets and ties by optical square and tape.		
• Describe the recording of measurements taken in a field by a recognised booking method.		
• Explain the identification and correction of fieldwork errors.		
• Describe factors which govern chain survey framework.		
• Demonstrate the application of chain survey principles to a small practical situation.		
• Demonstrate the plotting of survey lines including all detail.		
If you have answered ‘no’ to any of the outcomes listed above, then speak to your facilitator for guidance and further development.		

Module 3

Height measurement

Learning Outcomes
On the completion of this module the student must be able to:

- Describe the sources of vertical control.
- Describe using a datum as reference for bench marks.
- Describe the use of maps to obtain the position of control points.
- Describe the establishment of bench marks giving reason for use.
- Give definitions of levelling terms.
- Describe levelling instruments.
- Give a brief description of traditional levelling methods and instruments including the spirit, water and Cowley levels:
  - dumpy level
  - tilting level
  - automatic level
- Describe how to check the accuracy of levelling instruments.
- Describe how to take the reading of the metric levelling staff.
- Demonstrate recording and calculating reduced levels by "rise and fall" and "collimation" methods including inverted staff and application of the required checks and corrections.
- Describe the following levelling methods:
  - Flying
  - Grid
  - Reciprocal
  - Cross sectional
- Describe the sources of induced and instrumental errors.

3.1 Introduction

Leveling may be defined as the operation of determining the differences in height between points, or the relative heights of points on the earth's surface. A level line is one which is of constant or uniform height relative to mean sea level, and is therefore a curved line concentric with the mean surface of the earth. See **Figure 3.1**.

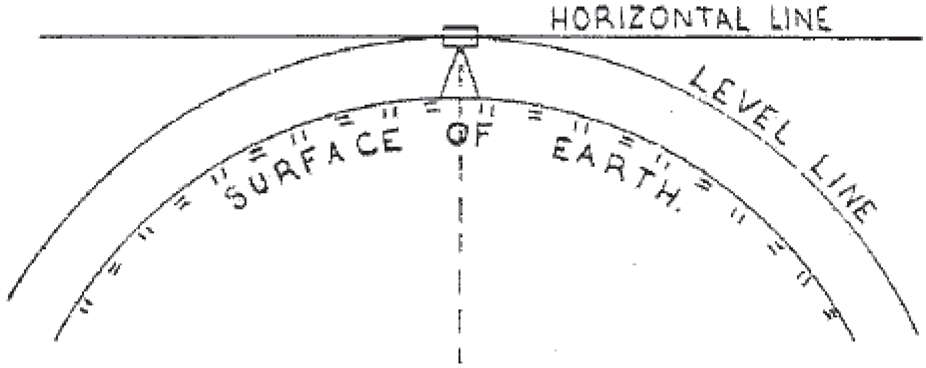


Figure 3.1

A level line is therefore one which lies on one level surface and is normal (at right angles) to the direction of gravity at all points in its length.

A horizontal line "through a point is tangential to the level line passing through the same point, and is normal to the direction of gravity at that point.

A DATUM PLANE is an imaginary level surface on which all points are assumed to have an elevation (height) of zero. The elevation of' any other point is its vertical distance above or below this surface.

Mean Sea Level (MSL) is a common and convenient datum used all over the world. It is often, however, convenient to assume some other datum.

3.1.1 Methods of leveling

- Direct, or Spirit leveling. A spirit level in some form is used.
- Barometric leveling. Elevations are obtained directly by means of a barometer.
- Indirect leveling (trigonometrical). The difference in elevation between two points is calculated from an angle and a distance.

The following sections deal with spirit leveling while that branch called tacheometry deals with indirect leveling.

Barometric leveling will not be dealt with in this manual.

3.2 Instruments

3.2.1 The dumpy (Surveyor's) level

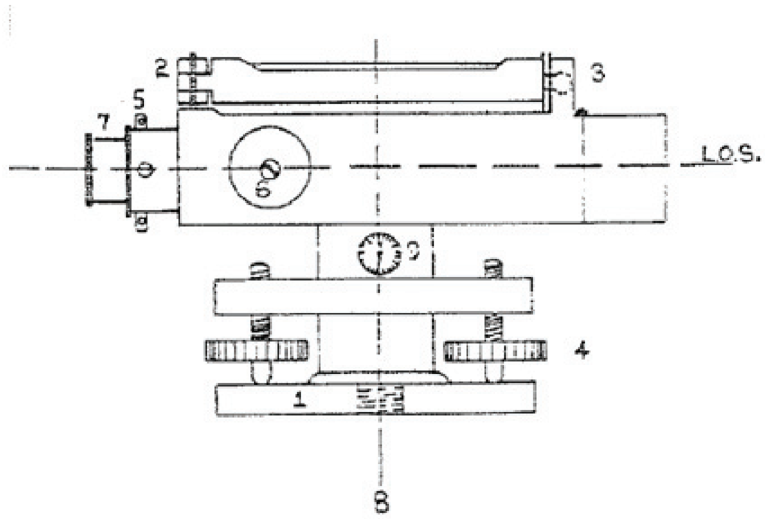
This is an instrument designed to furnish a horizontal line of sight. It consists essentially of a bubble tube attached to a telescope, the axis of the bubble tube and the line of collimation being parallel to each other.

> **Note:**
> A line of sight is the imaginary line passing through the intersection of the cross hairs and optical centre of the objective lens. When the instrument; is in adjustment the line of sight is known as the line of collimation.

The instrument is provided with leveling screws by which the bubble is centred and the line of collimation brought into a horizontal plane.

The modern form of dumpy level (see **Figure 3.2**), sometimes called the solid dumpy, has the vertical spindle and the telescope barrel cast in one piece.

This gives maximum rigidity. The telescope is focused internally.



1. Base plate
2. Level tube adjusting screw
3. Hinge
4. Footscrews
5. Diaphragm adjusting screw
6. Focussing screw
7. Eyepiece
8. Vertical axis
9. Horizontal adjusting screw for sighting

Figure 3.2

To set up the surveyor's level:
Press the tripod firmly into the ground and attach the level to it by means of the screw provided for this purpose.

The instrument will be level if the bubble remains in the same position when the telescope is revolved.

The instrument will be in perfect adjustment if:
(a) The bubble remains in the central position when the telescope is rotated horizontally.
(b) The bubble tube axis is parallel to the line of sight.

Put the telescope parallel to any pair of footscrews and bring the bubble central by using both these footscrews.

Note:
The bubble will move in the same direction in which the left thumb moves.

Now turn the telescope through an angle of 90° and use the third footscrew only, to bring the bubble central in this position.

Repeat the process until the bubble remains central in these two positions.

Now turn the telescope through 180°. If the bubble still remains central, the instrument is level. If not, say the bubble takes up a position two divisions off central position, then bring back the bubble by one division using the footscrews.

Note:
Remember that the apparent error here, is equal to twice the true error.

This particular instrument will be level with the bubble in a position one division off central position. Repeat the leveling process by bringing the bubble one division off central position.

Check the operation. The bubble should remain ore division off central position (in the same position) when the telescope is revolved.

Note:
Plane AB is horizontal although the bubble tube axis is not parallel to it, and remember that the telescope and this base (AB) is casted in one piece so that the telescope is parallel to AB.

Adjustment of the surveyor's level
Object of the adjustment:
- The bubble should remain central (for convenience only) when the telescope is rotated horizontally.
- The line of sight should be parallel t:o the bubble tube axis.

Bubble tube adjustment
Set up the instrument exactly midway between two blocks or pegs A and B, which are about 60 metres apart and on fairly level ground.

Level up the instrument as described, Suppose the bubble remains one division off central when the telescope is revolved, take up this one division by raising or lowering the adjustable end of the bubble tube until the bubble takes up the central position.

Check the leveling and adjustment. The bubble should now remain central on horizontal rotation of the telescope.

Adjustment of line of sight
Having completed ~he bubble tube adjustment, the instrument is carefully leveled up and readings are taken on staves held at pegs A and B. See **Figure 3.3**.

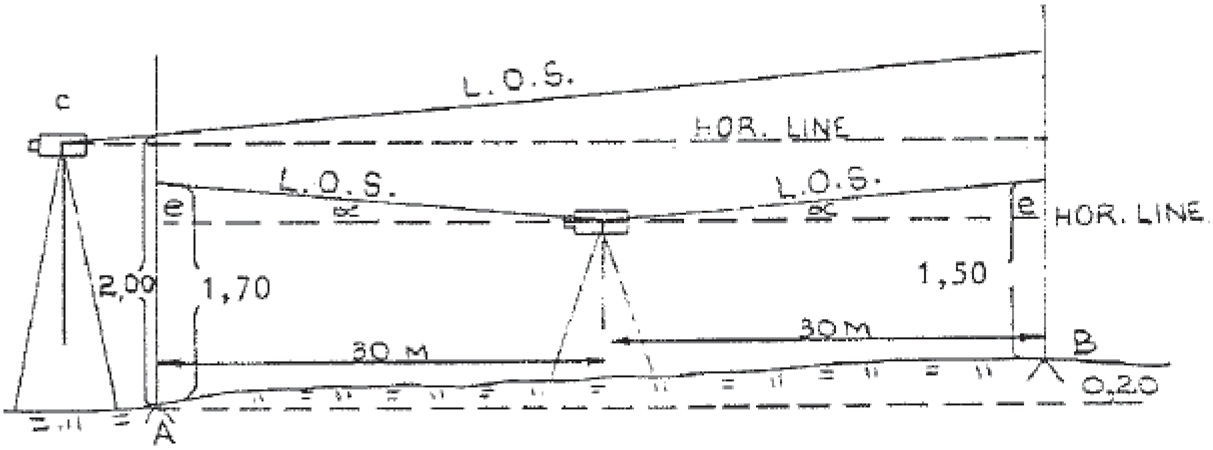


Figure 3.3

Suppose the following readings are obtained:
At A = 1,70
At B = 1,50

Suppose also, that the line of sight (LOS) makes an angle '∝' with the horizontal plane through the telescope.

The staff readings at A and B will both contain an error 'e' because the distances from the instrument to A and instrument to B are equal (ie 30 m).

The true reading at A should therefore be 1,70 – e, and that at B be 1,50 – e.

The vertical distance from A to B is found by subtracting the reading at B from the reading at A, thus:

True vertical dist. A - B = true reading at A - true reading at B
= (1,70 - e) - (1,50 - e)
= 1,70 - e - 1,50 + e
= 1,70 - 1,50
= 0,20

From the above, it can be seen that the true vertical distance between two points can be found with an incorrectly adjusted instrument, by setting up midway between the two points.

Having found that the true vertical distance A - B = 0,20, the instrument is now set up close to and behind one point, say A, and carefully leveled up again say ± 2 metres behind A).

When distance C - A equals zero.

A reading is now taken onto a staff held at A. Say this reading = 2,00. Knowing the vertical distance A-B the reading at B is calculated:

Reading at B = Reading at A + VD
= 2,00 + (-0,20)
= 1,80

This reading should be intersected on the staff at B if the line of sight is horizontal. If not, the cross hairs are adjusted by means of the capstan screws holding the diaphragm in position until the reading of 1,30 is intersected.

The line of sight is now perfectly horizontal, the bubble central which means that the bubble tube axis and LOS, are parallel.

Thus, for any future work, the bubble is brought central (ie: its axis horizontal) by means of the footscrews; the line of sight will be horizontal and leveling work may be carried out.

The circular bubble on a level is used for rough leveling before the precise leveling is begun. This circular bubble is adjusted after the bubble tube and line of sight adjustment. The capstan holding the circular bubble in position, are adjusted until it is central,

3.2.2 The automatic level

This instrument represents the latest and possibly greatest advance in visual leveling since the introduction of the telescope.

In this remarkable instrument, the level tube is dispensed with, Only a small circular bubble for rough leveling is supplied, and this need not even be leveled with very great care. A small glass prism (see **Figure 3.7**), suspended inside the telescope, swings freely within certain limits.

When the instrument is roughly leveled, the prism settles in such a position that the rays of light entering the telescope, are bent so that the line of sight is truly horizontal. The automatic level is thus a surveyor's level where the bubble tube is dispensed with and a suspended prism fitted in its place. See **Figures 3.4**, **3.5** and **3.6**.

Advantages of the automatic level

- The instrument very quickly settles in-co the observing position.

- Less time is wasted due to extraneous circumstances, such as the effect of the sun on the bubble.
- No umbrella is required for shading.
- The instrument itself gives a good indication of when operations should be suspended due co wind conditions, as the image becomes unsteady, and the leveler is unlikely to remain under an erroneous impression that he is still doing good work.
- Fatigue is very greatly reduced.
- Speed is very greatly increased.
- Generally, a superior class of accuracy is attained

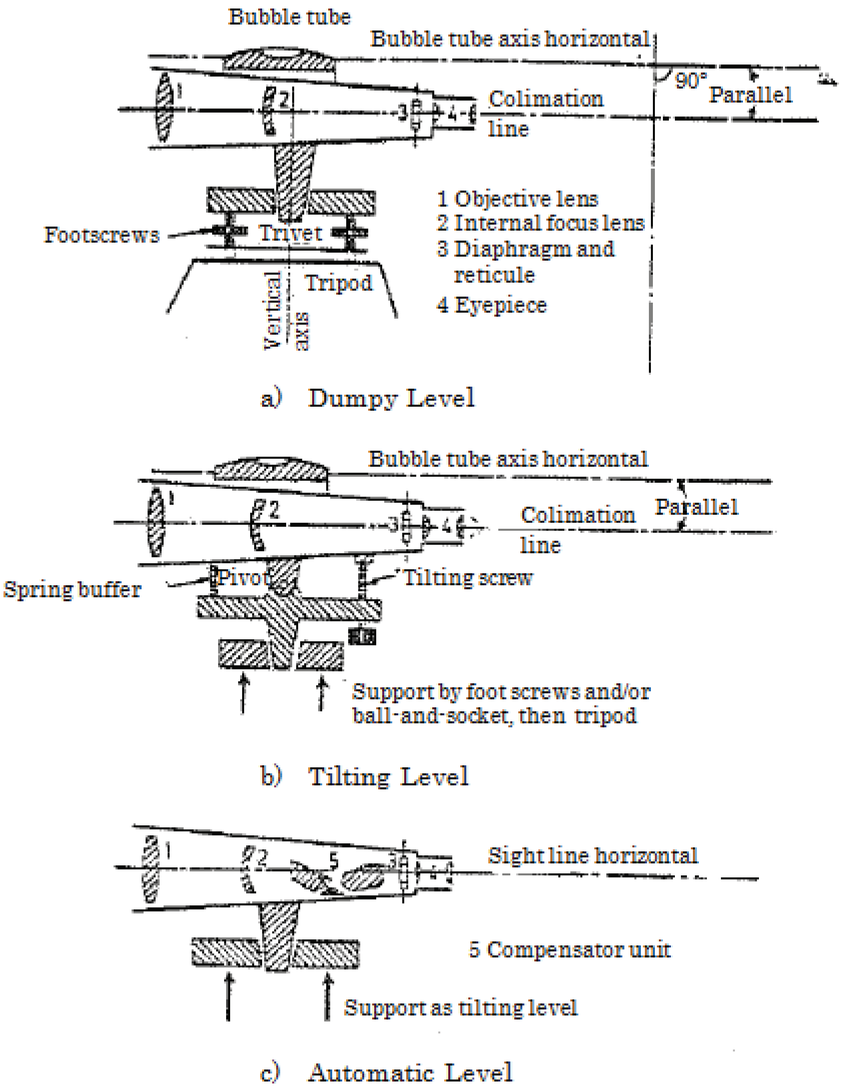


Figure 3.4

Figure 3.5 Automatic Level

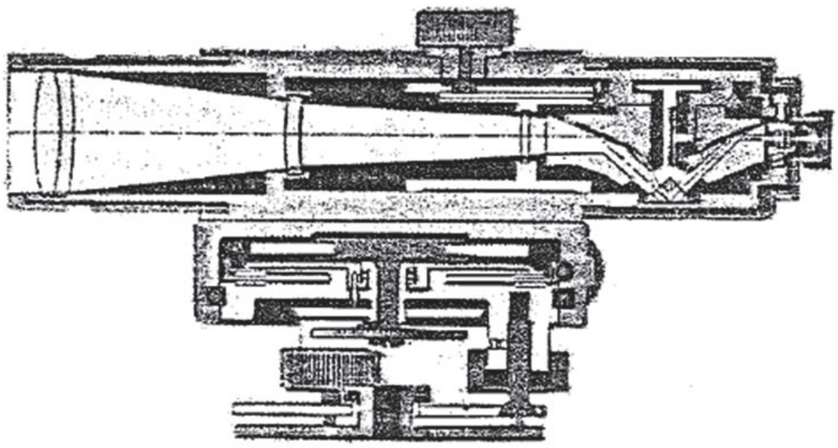
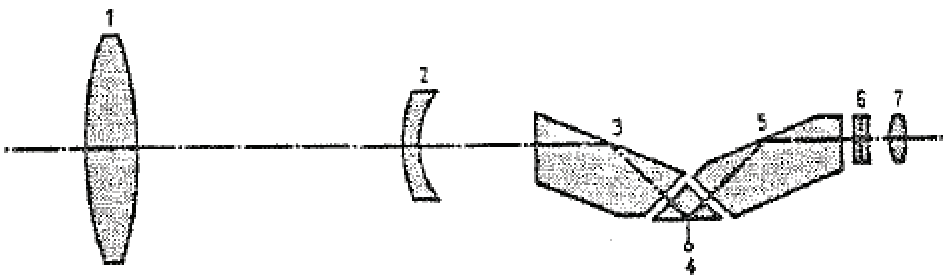


Figure 3.6 Section through automatic level



1. Objective lens
2. Focussing lens
3. Prism
4. Swinging prism
5. Prism
6. Eyepiece

Figure 3.7

3.2.3 The Water Level

One distinct method of leveling, which falls into the category of differential leveling, is by means of the water level.

This is a simple but effective method of leveling, which can be made to perform a most useful function under certain circumstances.

The instrument consists of a rubber tube with a graduated glass or clear plastic tube at either end and suitable clamps or stands. It is filled with water, and acts on the principle that the water will rise to the same level in each tube. Differences in height may be read off the graduations.

It is particularly useful in building operations when fitting gutters, shuttering etc., and when working in cramped conditions. Its use is mainly confined to special applications, but the surveyor should not consider himself superior to bearing in mind a method which may, under certain circumstances, solve otherwise insuperable difficulties.

A water level may be improvised from any flexible pipe up to 30m in length, transparent plastic garden hose is ideal. Levels have been carried over a fair distance, with excellent results, using only a length of hose a marked ranging rod for backsights and a level staff for foresights.

Provided air bubbles are excluded from the tube, this is a very precise method of leveling.

This technique is sometimes referred to as "hydrostatic leveling". Typical applications include checking levels of brick courses, floor screeds and ceilings, another more recent application is checking the levels on sliding formwork as used in silo and high-rise lift core construction.

3.2.4 The Cowley Level

Figure 3.8 The Cowley level

When surveying on a small scale for such things as domestic drainage work and leveling, the Cowley Level is a very useful instrument. It is very simple to use and for distances up to approximately 30 m, accuracy to within 6 mm can be obtained.

The instrument consists of a metal box 140 mm x 120 mm x 50 mm mounted on a lightweight metal tripod by a vertical pin (**Figure 3.8**). A clamp inside the casing is released when the instrument is placed on the pin and the level is then ready for use.

If the tripod cannot be used conveniently, the cast aluminium stand provided can be used with the level as an alternative to the tripod (**Figure 3.9**). The optical system employed is shown in Figure **3.10**.

It consists of two sets of mirrors set side by side, one completely fixed and the other inclusive of a swivel or pendulum mirror.

The image seen through one system is seen against the image on the other and when the two halves of the target bar (**Figure 3.11**) coincide, the line of sight is horizontal.

It is the pendulum mirror within the right hand system which remains in a horizontal position irrespective of how the level is tilted (to within certain limits) that allows an accurate reading to be taken.

The target bar and staff are sighted when in use and an operative moves the target bar up or down until the two halves coincide. A reading is then taken from the side of the staff which is graduated in 5 mm markings as shown in **Figure 3.12**.

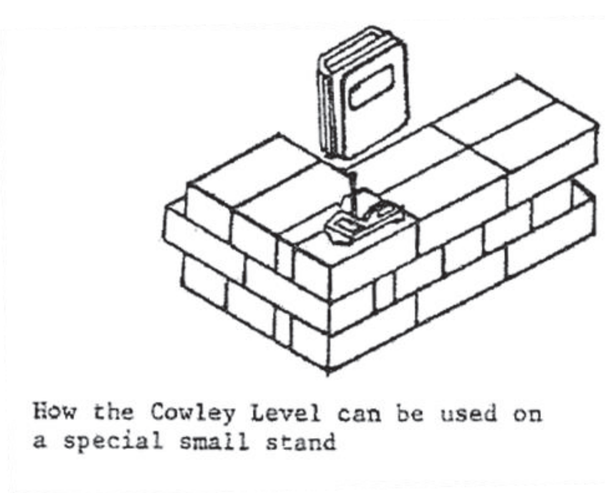


Figure 3.9

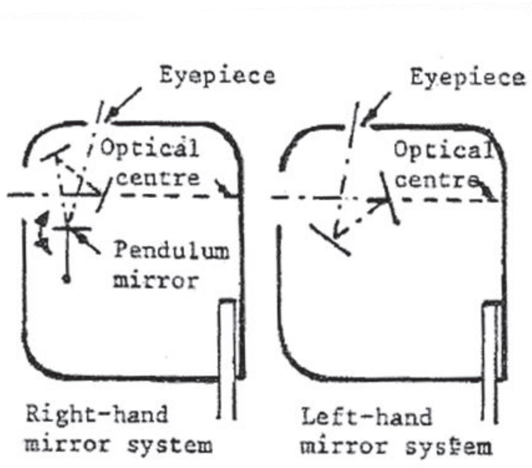


Figure 3.10

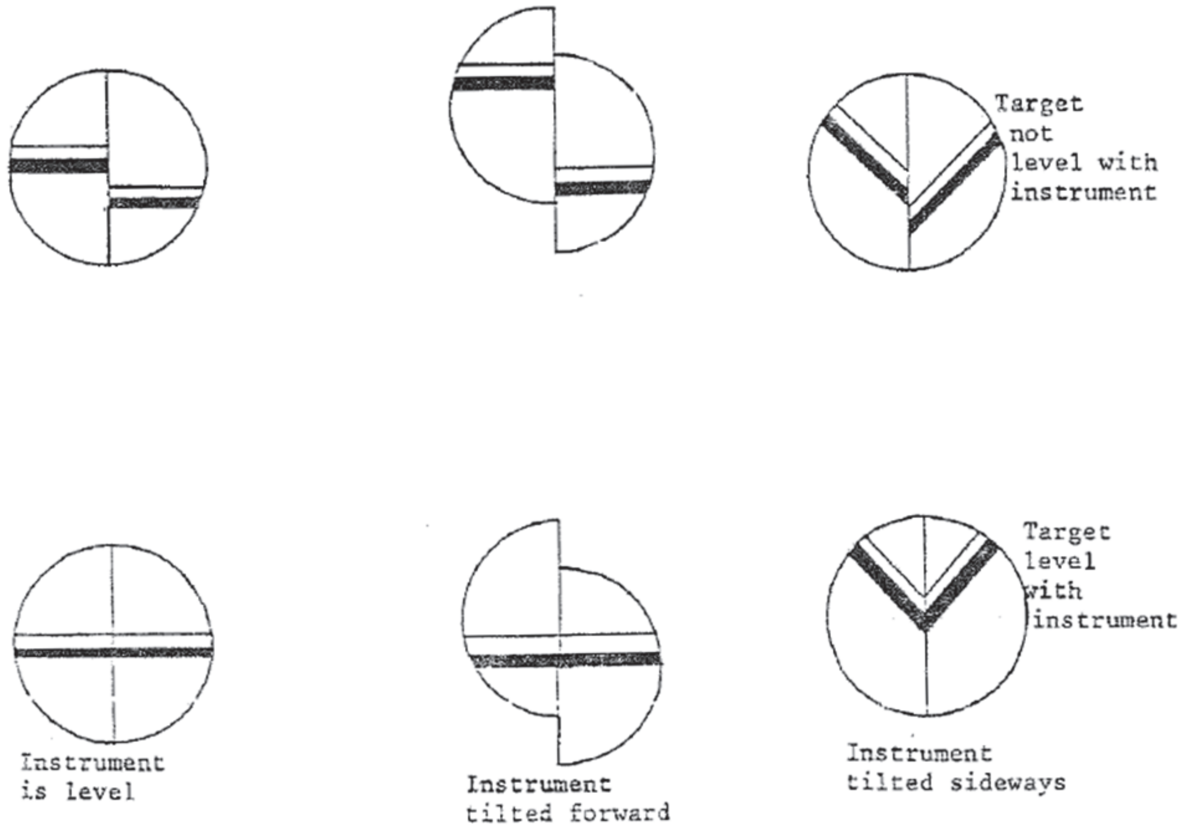

Target
not
level with
instrument
Target
level
with
instrument
Instrument
is level
Instrument
tilted forward
Instrument
tilted sideways

Figure 3.11

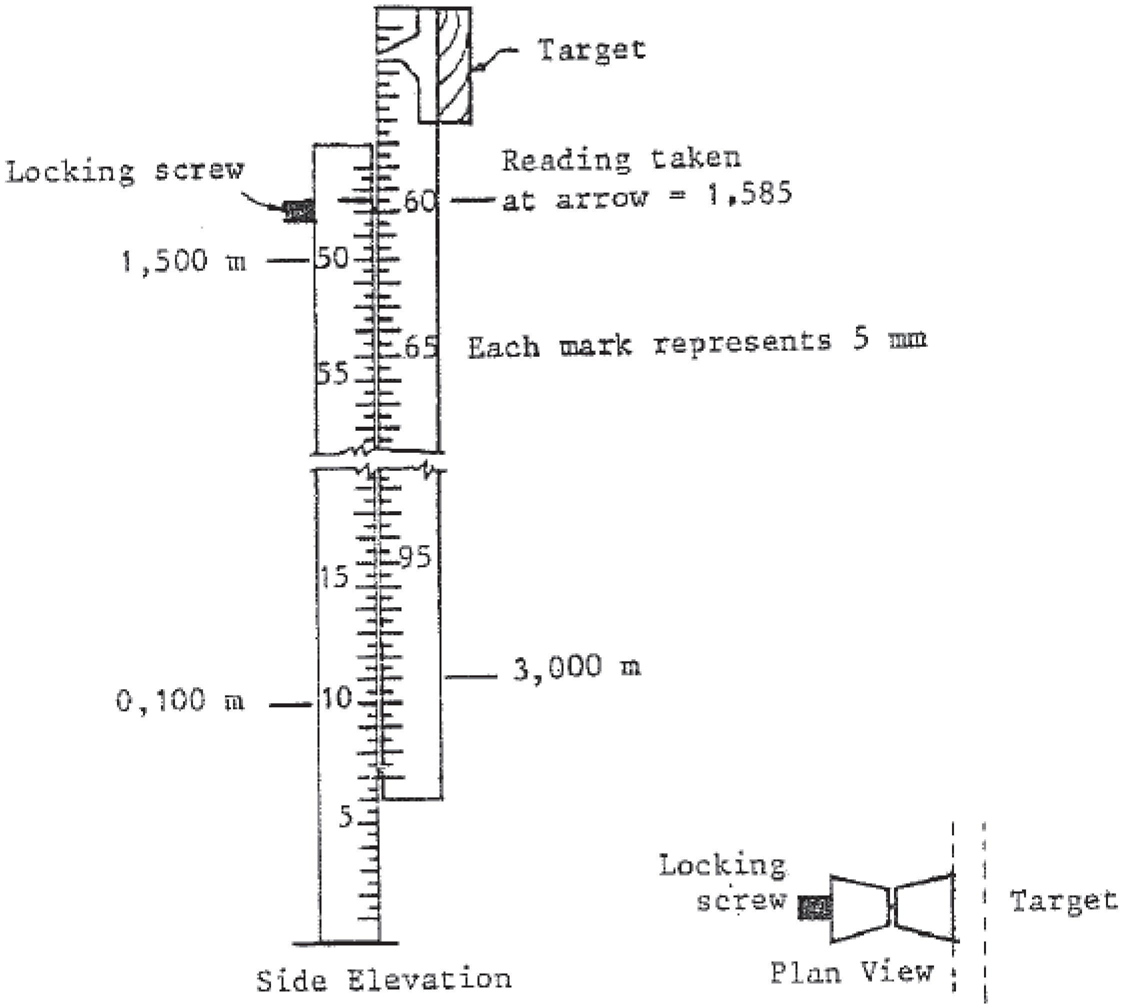


Figure 3.12

3.2.5 The Cowley Slope attachment

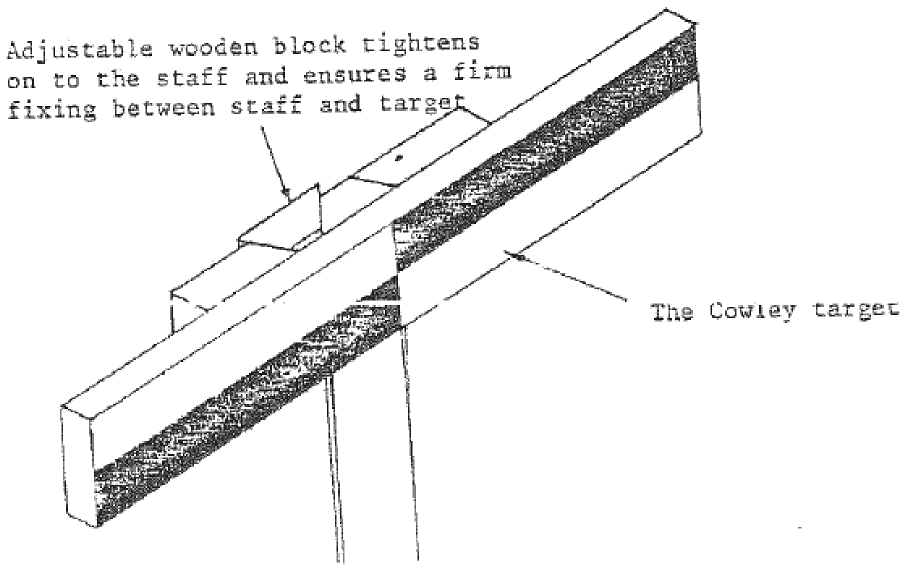


Figure 3.13

Figure 3.14 The Cowley Level with slope attachment

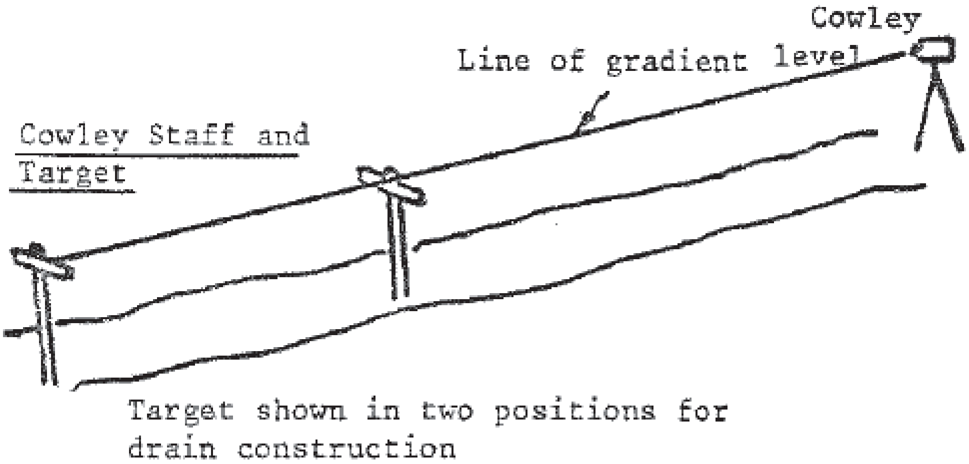


Figure 3.15

A recent useful accessory to the Cowley Level is the Slope Attachment, shown in its fixed position (**Figure 3.14**). The object of this fitting is the easy determination of gradients without the need to set up sight rails (explained later).

In operation the level is set in front of the gradient required with the attachment fitted as shown and the selected gradient set. The Cowley staff and target are then positioned at the other end of the slope. (See **Figure 3.15**.)

An operative then moves the target in the usual manner until the two halves are coincident. When this has been done he moves towards the level, setting pegs at regular intervals as required. It must be emphasised that the target halves must be coincident at each peg so that the pre-selected gradient is formed along the pegs.

3.2.6 The precise (Tilting) level

The tilting level has for many years been the most generally favoured level, but is now losing popularity in favour of the automatic level. Basically it is similar to the surveyor's level, but the telescope is not rigidly fixed to the vertical axis.

It is attached to the vertical axis by means of a hinge, which allows a small movement of the telescope in the vertical plane.

This means that the vertical axis need not be truly vertical when the line of sight is truly horizontal. The front end of the telescope is spring loaded, so tha the rear end bears down on a lifting device known as a gradienter, which is controlled by a very fine pitched tilting screw.

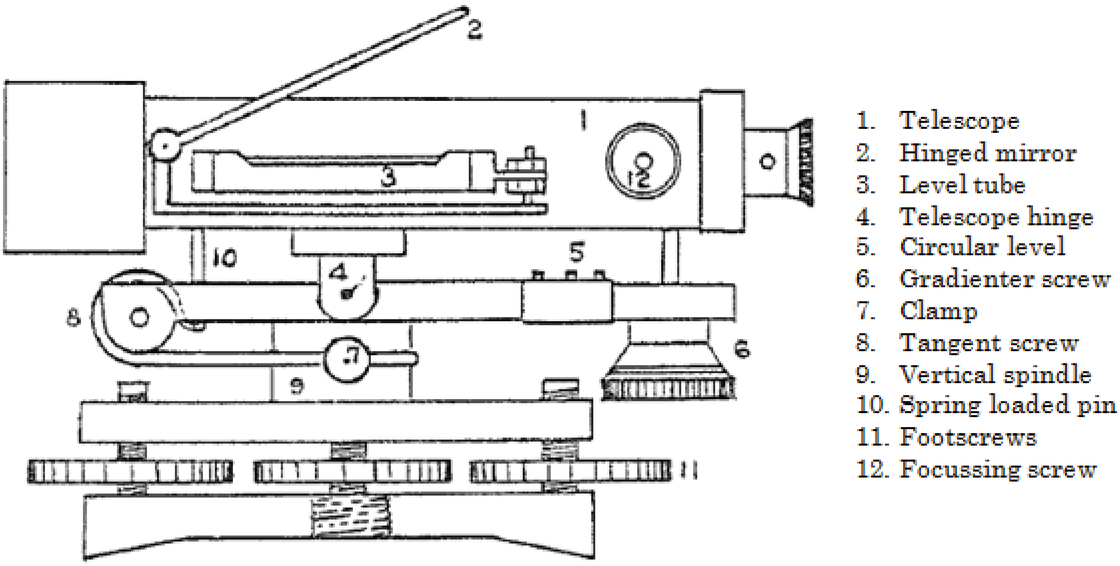


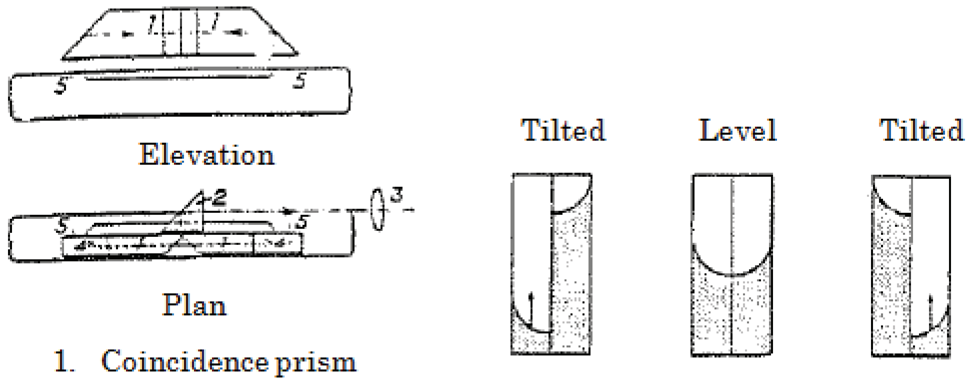
Figure 3.16 Tilting level

This gradienter is sometimes graduated in the form of a micrometer drum to indicate the gradient (or slope) of the line of sight. **Figure 3.16** represents a tilting level and **Figure 3.17** a gradienter micrometer,

Figure 3.17 Gradienter

The tilting screw is capable of imparting a precisely controlled movement to the telescope. By means of this device, the bubble may be centred with extreme accuracy, The bubble is usually viewed through a system of prisms, so that an image or one half of each end of the bubble is seen in adjoining planes.

When these two halves are brought into co-incidence, so that they appear as one full end of the bubble, the bubble is truly centred. See **Figure 13.3**.



1. Coincidence prism
2. Reader prism
3. Magnifier
4. Parts of bubble reflected
5. Reference marks

Figure 3.18

The telescope can be "tilted by means of the gradienter micrometer, to a required angle or grade. The vertical scale of the micrometer counts the number of revolutions and the horizontal scale gives the fraction of a revolution. In **Figure 3.17**, one revolution tilts the line of sight by 1 : 1 000.

Then :

$$Gradient = \frac{Drum\ revolutions}{1\ 000}$$

Setting up of the tilting level
Press the tripod firmly into the ground and attach the instrument by means of the screw provided for that purpose. Level up by means of the circular bubble.

Then for every pointing, before the reading is made, bring the two ends of the split bubble into coincidence by means of the gradienter micrometer.

Test and adjustment of the tilting level.
Having found the true vertical distance between two points A and B as described previously, set up close to, say, point: A.

Level up the instrument, take the reading an the staff held at A and calculate what the reading should be at B. Now, sight B to see if the calculated reading is intersected.

If not;

1. Make sure that the gradienter micrometer reads ZERO.
2. Adjust the cross hairs until the calculated reading is intersected.
3. Raise or lower the adjustable end of the bubble tube until the bubble remains central.

Note:
Parallax should be eliminated before any test is carried out!

3.2.7 The Wye level
This instrument has a telescope which may be removed from its supports or wyes and reversed end for end or rotated through 180° about its longitudinal axis. By taking readings in the reversed telescope positions, instrumental error is cancelled out.

The bubble may be attached to either the telescope or supporting stage. This type of level has been largely superseded by the reversible tilting level.

3.2.8 The reversible tiling level
In this type the telescope is mounted so that it can be rotated about its longitudinal axis, in order to cancel out any instrument error. See **Figure 3.18**. Having taken a direct reading (R1), the telescope is revolved through 180° and a second reading (R2) is taken. The mean reading $\frac{R1+R2}{2}$ is then used for leveling reductions.

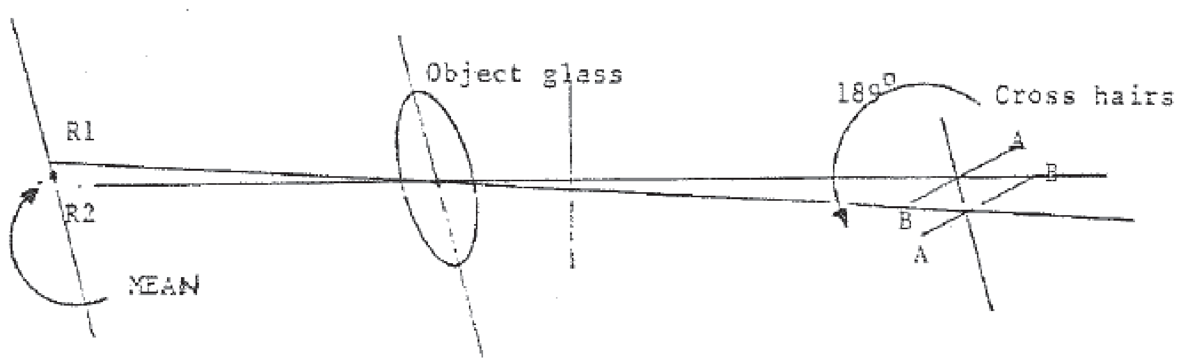


Figure 3.18

3.2.9 The parallel plate micrometer
Even with a good telescope and a staff marked in fine divisions, staff readings cannot be made finely enough by simple telescope for precision leveling demanding accuracy such as 0, 5 mm/km or so.

The plane parallel plate micrometer is an attachment for a level which typically permits the determination of level staff readings to 0,1 mm directly, and by estimation to 0,01 mm (0,000 01 m). This degree of accuracy would never be used in engineering surveying, only in scientific leveling.

The device is simply a piece of glass with parallel plane faces, placed in front of the telescope objective and supported on horizontal pivots with t:he plane faces at right angles to the collimation line.

Since glass refracts (bends) a ray of light entering it, rotation of the plane parallel plate causes the collimation line to be raised or lowered while still remaining parallel to its original path.

The physical constants of the glass being known, the vertical displacement of the collimation line can be calculated for a known tilt of the plate. The plate is tilted by a micrometer screw which registers the displacement of the collimation line rather than the amount of tilt.

The simplest version, often used as an optional attachment to a level, has a displacement scale engraved on the edge of the micrometer screw operating the plate.

When the device is permanently "built-in" to the level, the plate is generally linked up to an optical scale viewed in an eyepiece alongside the telescope eyepiece.

Figure 3.19 shows the system used on the Wild N3 precise level.

The total vertical displacement possible is 10 mm, and the eyepiece scale is graduated 0, 1,2, ... to 10, each number representing 1 mm of vertical displacement. Each division is further sub-divided into ten parts of 0,1 mm, and these may be sub-divided by eye to 0,01 mm.

Operation is extremely simple - after carefully focusing and leveling the instrument, turn the micrometer operating screw until the central horizontal cross-hair cuts a 10 mark on the observed staff, note that reading, and add on the reading from the micrometer scale.

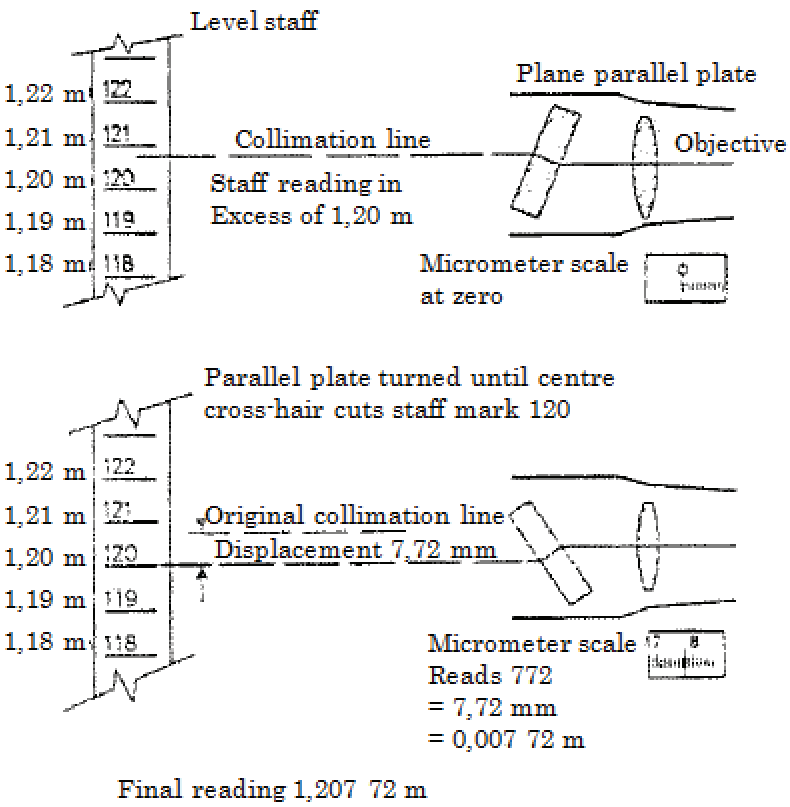


Figure 3.19 Parallel plate micrometer

On drum instruments, take the reading from the edge of the micrometer drum.

In example: Read 1,20 and book as 1,20
Read 772 and add to give 1,207 72 m

Precise staves are commonly marked at every 10 mm by a line 1 mm thick. Since this is difficult to "bisect", precise level reticules are marked with two 'wedge' lines, and the staff mark is centred between these rather than bisected.

Figure 3.20 shows the view seen in the N3 telescope; on the left a normal staff at long range read by centre hair and on the right the graduations of a precise staff at short range using the wedge lines".

Unless the plane parallel plate micrometer can be clamped when not required, ordinary leveling with an instrument of this type requires great care in use, to ensure that the collimation line does not get disturbed by accidental manipulation of the micrometer drum in mistake for the telescope focusing screw.

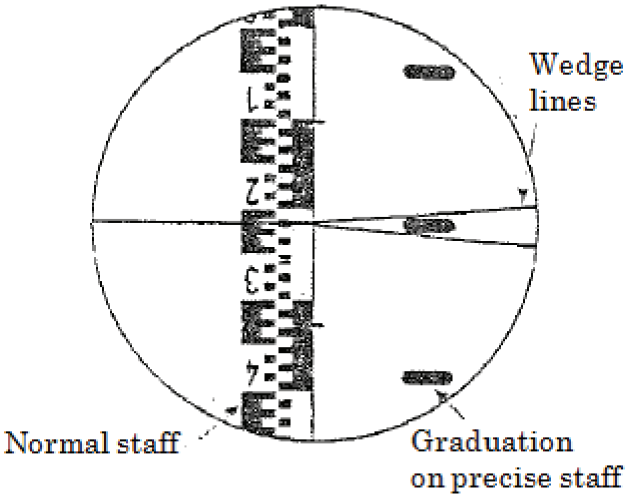


Figure 3.20

3.3 Leveling terms

3.3.1 Benchmark

This is a relatively permanent point of reference, the elevation of which, with respect to some assumed datum is known.

It is used either as a starting point for leveling or as a point upon which to close as a check. A trigonometrical benchmark consists of a brass stud let into concrete. A washer is pinned between the stud and the concrete, and on this the number allocated to the benchmark is stamped.

- *Stations* are points whose elevations are to be ascertained, or points that are to be established at a given elevation. A station is where the leveling rod is held.

- *Set-up* is where the leveling instrument is set up.

- *Height of Instrument (HI)*
  For any particular set-up this is the elevation of a plane of sight with respect to datum.

> **Note carefully:**
> The height of the instrument in leveling is not the height of the telescope above ground, nor is it the height above any station.

- *Backsight (BS)*
  Is the first reading taken at a set-up; it is a sight taken on a station held at a point of known elevation to ascertain how far the plane of sight collimation) is above or below that point, thus establishing the height of instrument with respect to the assumed datum.

- *Foresight*

Is the last reading taken at a set-up, and is a sight taken on a staff held at a point of unknown elevation to ascertain how far the point is below the line of sight in the case of surface surveying, and above the line of sight in the case of underground surveying or tunnel surveying, where the point is above the plane of sight. (It could also be a sight taken onto a point of known elevation in order to 'close' the leveling traverse).

- *Turning Point (TP)* (also sometimes called a change point (CP)
  This refers to a point to which both a foresight and backsight are taken on a line of direct levels, (traverse). The foresight is taken from one set-up in order to determine the elevation of the turning point, the backsight from the next set-up in order to determine the new height of instrument.

- *Intermediate sight (IS)*
  These are all other sights taken from a particular set-up to determine the elevation of these points. Many intermediate sights may be taken from one set-up, but in traversing only a backsight and foresight are taken.

- *Traversing*
  When the difference in elevation between two points is required and these points are far apart, a number of set-ups are necessary, as illustrated in **Figure 3.25**. This procedure is called traversing.

3.3.2 Lengths of backsights and foresights

The maximum length of a backsight and foresight will depend upon the character of the terrain, the atmospheric conditions and the optical qualities of the telescope.

In ordinary work, using a staff where a metre is divided into decimetres and centimetres it is advisable to limit the maximum lengths to 100 metres. Over a distance longer than this the thickness of the cross hairs appears to become the same as ± 5 mm, ie half of the centimetre graduations and it becomes difficult to decide on the correct intercept of the cross hair on the staff.

Other errors also increase rapidly over longer distances. In very precise work the distance should be limited to ± 30 metres.
Observations on the staff are made with the telescope properly leveled, and the differences of reading so obtained represent the relative levels of the several points, the actual height of the instrument being immaterial.

The principle is illustrated by the following simple example:

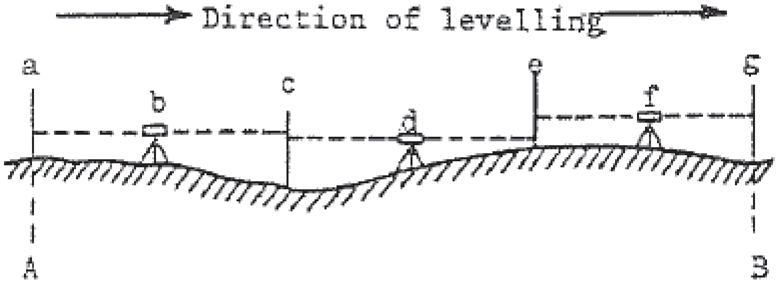


Figure 3.21 Traversing

It is required to determine the difference of level between A and B; a, c, e and g are the staff positions, and the level is set up successively at b, d and f.

The results obtained are:-

	Backsight	Foresight
from station b	1,231 (on a)	2,408 (on c)
from station d	1,905 (on c)	0,317 (on e)
from station f	1,103 (on e)	1,811 (on g)
	4,239	4,536
		4,239
Difference of Level		0,297

The sum of the foresights exceeding the sum of the backsights indicates that the point A is higher than the point B by 0,297 metres.

3.3.3 Reading the staff

The rod must be held plumb when it is read. This is not so easy. It must coincide with the vertical cross-hair of the telescope. The observer can indicate to the rodman how to swing the rod until it is in the same plane as the cross-hairs, but it may still not be vertical.

Some rods are fitted with a circular bubble. Verticality is then obtained comparatively easily, but these bubbles get damaged during transport and become unreliable.

The following method is usually adopted:

- The rod is moved slowly backwards and forwards.
- As the rod is moved away from the vertical position, the reading, as observed by 'the level man, will become greater and greater but, as it is moved towards the vertical position, it will become smaller and smaller.
- The rod will be vertical when the reading is a minimum.
- This reading is recorded.
- The level man can also signal to the rodman when this position is reached.
- He will then keep the rod in that position.

3.3.4 Refraction

A ray of light passing from a lighter to denser medium is deviated off its path. This is called refraction. If the density of the air through which a ray of light is passing varies from place to place on that path, the ray will suffer refraction and incorrect readings will result.

3.3.5 Curvature of the earth

Since the surface of the earth approximates to that of a sphere, the direction of gravity is not the same at any two places.

The line of sight of the telescope, when leveled, is always at a tangent to the earth's circumference at the position of the instrument.

It follows, consequently, that in any extended leveling operations it is necessary to take account of the earth's curvature unless the surveyor is able to arrange that back- and foresights are of equal length, in which case errors due to curvature cancel one another.

From the diagram it will be obvious that, if the sights x and x’ are of equal length, the errors due to curvature y and y' must be alike.

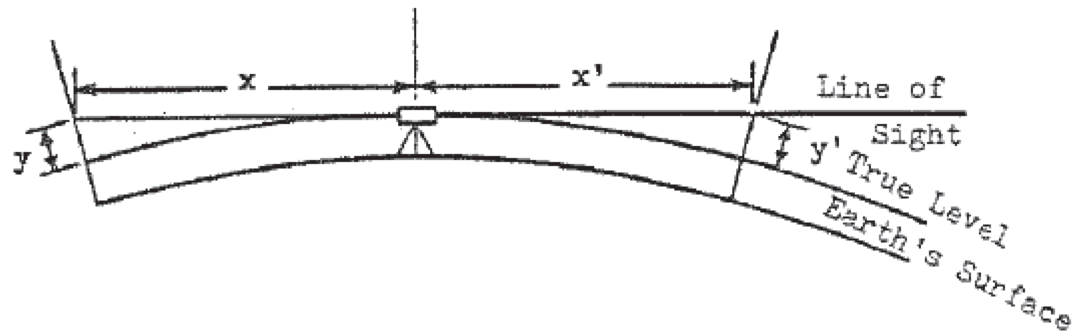


Figure 3.22

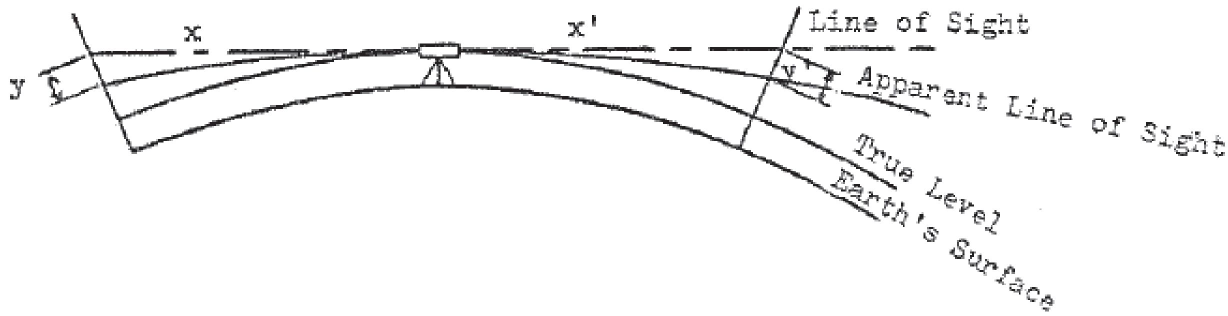


Figure 3.23

The correction for curvature is modified by another cause, namely refraction. Light traversing the atmosphere in a direction that is tangential to the Earth’s surface is bent or refracted towards the earth by reason of the increasing density of air.

This has the effect of causing the staff reading to be less than if there had been no refraction, but here again, if the back- and foresights are of equal length, the errors due to refraction (other than refraction due to local causes) cancel out. Viz: if x = x', then y = y' (ie the errors due to refraction will be equal).

3.4 The theory of leveling

When the instrument is leveled-up the line of sight will be horizontal, if the instrument is in adjustment. In the work that follows we will assume that the line of sight is always horizontal.

Where points lie below the line of sight

Abbreviations: BS = Backsight
FS = Foresight
IS = Intermediate sight

Consider the following case:

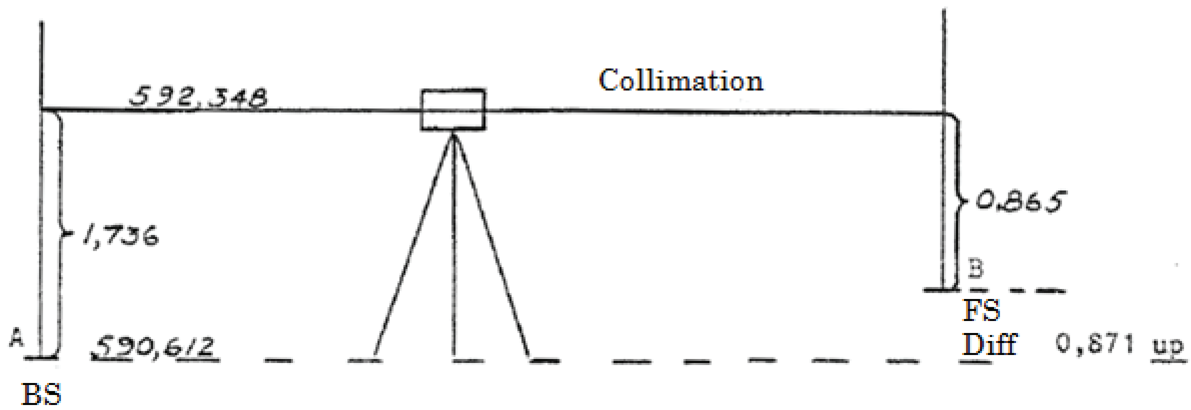


Figure 3.24

In **Figure 3.24** a BS reading was taken on a staff held at and was found to be 1,736 and at B 0,865. Assuming that the elevation of point A is 590,612 metres above Mean Sea Level (which is abbreviated to AMSL), then the elevation of the collimation will be 590,612 + 1,736 = 592,348 AMSL and point B lies 0,865 m below the line of collimation, so the elevation of B will be

592,348 - 0,865 = 591,483

There is however, another method of finding the elevation by studying the figure it will be obvious that the difference in elevation between A and B will be the difference in the two readings.

Ie 1,736 - 0,865 = 0,871

Therefore the elevation of B = 590,612 + 0,871
= 591,483 as before

From the above we notice that the backsight reading is plus and the foresight reading is minus, because to get the collimation, we have to rise 1,736 metres and then B is 0,865 metres lower than the collimation.

Note:
The elevations of points above datum is prefixed by a plus (+) sign, while those below datum are prefixed by a minus (-) sign.

3.5 Booking of leveling observations

Whenever levelling is done, the surveyor should note in his fieldbook his name, the working place and the date.

In the case of very precise levelling a note should also be made as to the weather conditions eg, sunny, no breeze or windy etc.

Let us now observe a levelling traverse in which it was required to find the elevation of a point E from a known point A, whose elevation is 741,250 metres AMSL (**Figure 3.25**)

Important Note!
In practise the levelling traverse would be runback from E to A to serve as a check.

You should study the figure well; the survey started at A, a point of known elevation, and there were four set-ups to arrive at point E.

There are therefore a series of backsights and foresights which are booked as shown in the field notes.

3.5.1 Field Notes

Surveyor : A Smith   Date: 16.10.12
Working Place : Survey of point E.

BS	FS	REMARKS
1,612		Point A
0,352	0,701	TP B
1,336	1,430	TP C
1,201	0,620	TP D
	1,503	Point E

Table 3.1

Remarks
TP = Temporary point also known as a change point

Note the following carefully:
The first backsight reading is written in the first column under BS. The first foresight reading is written in the next line under the FS column, but the next BS reading is writen in the same line in the BS column, since both readings were taken to the same points.

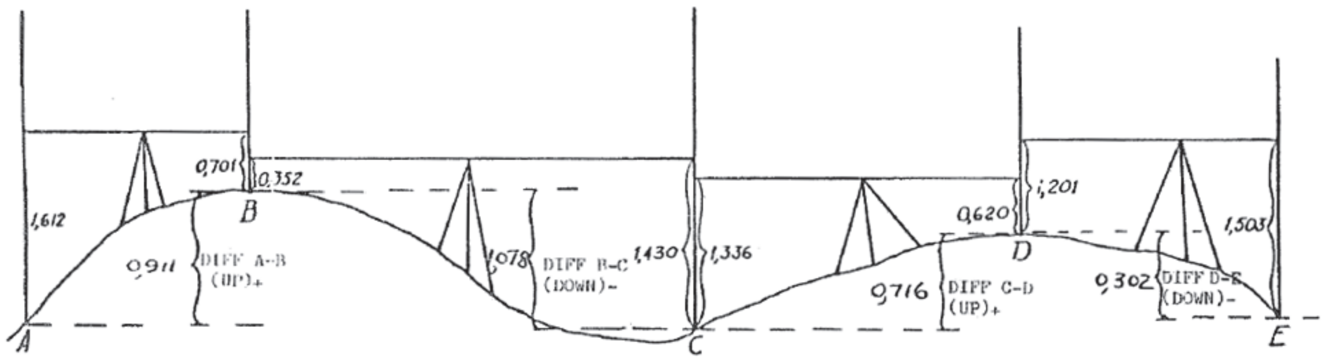


Figure 3.25

3.5.2 Rise and fall of reductions

- When both points are below the line of sight:
  Large reading to small reading gives a "Rise".
  ∴ Small reading to a large reading gives a "Fall".
- When both points are above the line of sight:
  Large reading to a small reading gives a "Fall".
  Small reading to D large reading gives a "Rise".
- From a point below the line of sight to a point above the line of sight gives a "rise" and from a point above to a point below gives a "fall".

BS	FS	RISE	FALL	ELEVATION	REMARKS
1,612				741,250	Points A
0,352	0,701	0,911		742,161	B
1,336	1,430		1,078	741,083	C
1,201	0,620	0,716		741,799	D
	1,503		0,302	741,497	Point E
+4,501	-4,254	+1,627	-1,380		Totals
-4,254 +0,247		-1,380 +0,247		+0,247	DIFFERENCE

Table 3.2

Checks
The checks are carried out as follows:
- Add all the backsights and foresights and find the difference, as shown in the last three lines at the bottom of the tabulation.
- Add all the rises and falls and find the difference. This difference must agree with the difference between backsights and foresights.

- The difference in elevation between A and E must be the same as the previous two differences. Subtract elev. A from elev. E.
  – Backsights are plus
  – Foresights are minus
  – Rises are plus
  – Falls are minus

3.5.3 Collimation method

BS	FS	COLLIMATION	REDUCED ELEVATION	REMARKS
1,612		742,862	741,250	Points A
0,352	0,701	742,513	742,161	B
1,336	1,430	742,419	741,083	C
1,201	0,620	743,000	741,799	D
	1,503		741,497	Point E
+4,501	-4,254			Totals
-4,254				
+0,247			+0,247	DIFFERENCE

Table 3.3

In the collimation method the height of the collimation is obtained by adding the first backsight reading to the elevation of the BS point A. The foresight is subtracted to give the reduced elevation of the turning point B.

Now the backsight reading to B is added ro the elevation of B to obtain new collimation elevation and so on. It will be noticed in this method that there is only one check.

Note:
These checks only check the calculations and do not check the field work.

Important Note!
Study these methods well. They are the basis of all the work that follows.

3.5.4 Leveling notes with intermediate sights

It is often necessary during a leveling survey to take intermediate sights to points. These points may be of importance and their elevations may be required.

In **Figure 3.26** we have shown two set ups with intermediate sights. This is merely a demonstration, but it is necessary to show how these are booked and calculated.

We shall do it by means of both the "rise and fall" and “collimation" methods. It is absolutely essential to do the checks. Although the whole process in both methods merely involves addition and subtraction, many students find difficulty with the method of booking and the steps involved once the result have been recorded. The elevation of the point A at which the survey was started in 1 090,710 metres AMSL.

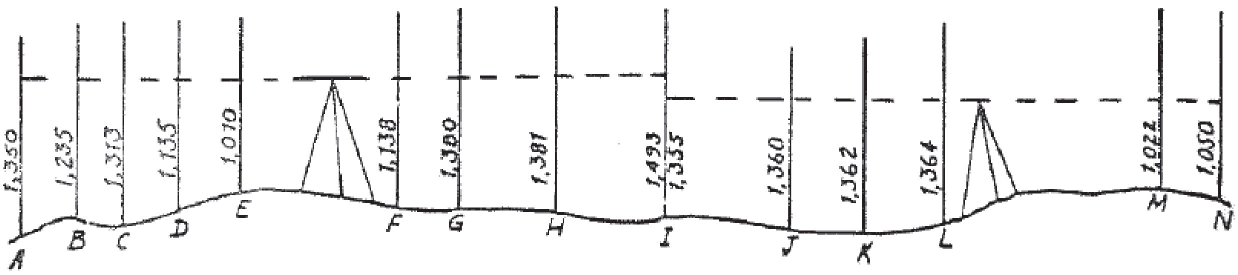


Figure 3.26

a) Rise and Fall method

BS	IS	FS	RISE	FALL	ELEVATION	REMARKS
1,350					1 090,710	Point A
	1,235		0,115		1 090,825	B
	1,313			0,078	1 090,747	C
	1,135		0,178		1 090,925	D
	1,070		0,065		1 090,990	E
	1,138			0,068	1 090,922	F
	1,380			0,242	1 090,680	G
	1,381			0,001	1 090,679	H
1,355		1,493		0,112	1 090,567	I
	1,360			0,005	1 090,562	J
	1,362			0,002	1 090,560	K
	1,364			0,002	1 090,558	L
	1,022		0,342		1 090,900	M
		1,050		0,028	1 090,872	N
+ 2,705		-2,543	+ 0,700	- 5,538		TOTALS
- 2,543			- 0,538		1 090,710	A
+ 0,162			+ 0,162		+ 0,162	DIFFERENCE

Table 3.4

Note the following:

- The difference between the first backsight and the first intermediate sight is first obtained and added or subtracted from the elevation or point A, depending on whether it is a rise or fall.
- The second step is to find the difference in elevation between the first and second intermediate sights and adding or subtracting from the elevation of the first intermediate sight and so on.

Important Note!
This difference must not be added or subtracted from the original point elevation.

We could find the elevations of all the intermediate sights by finding the difference between the backsight and each intermediate sight in turn, and add or subtract each difference to or from the elevation of the backsight. This practice must be avoided, as it does not allow for checking as in the method shown above.

b) Collimation method

BS	IS	FS	COLLIMATION	ELEVATION	REMARKS
1,350			1 092,060	1 090,710	Point A
	1,235			1 090,825	B
	1,313			1 090,747	C
	1,135			1 090,925	D
	1,070			1 090,990	E
	1,138			1 090,922	F
	1,380			1 090,680	G
	1,381			1 090,679	H
1,355		1,493	1 091,922	1 090,567	I
	1,360			1 090,562	J
	1,362			1 090,560	K
	1,364			1 090,558	L
	1,022			1 090,900	M
		1,050		1 090,872	N
+ 2,705		-2,543			TOTALS
- 2,543				1 090,710	A
+ 0,162				+ 0,162	DIFFERENCE

Table 3.5

In this method, the collimation elevation of the first set-up is obtained by adding the backsight reading to the station elevation. All intermediate sights and foresight elevations are obtained by subtracting the respective readings from the collimation.

At the second set-up, the elevation of the new collimation is obtained by adding the second BS reading to the elevation of the turning point I and then proceeding as before.

This method is extremely useful when many observations are made from one set-up.

The rise and fall method is more convenient when traversing.

By the rise and fall method there is a check on the reduction of each intermediate sight and any error in the reduction of these affects the next reduction, and is, therefore, shown up in the check.

On the other hand, by the collimation method there is no check on the reduction of the intermediate points, so that all important points should be made turning points. The field work should be carried out in both directions.

3.6 Methods of Leveling

3.6.1 Flying Levels

Figure 3.27 depicts a situation where a certain reduced level has to be transferred from one position to another. Obviously this should be done by using the minimum number of readings and change points.

Flying levels are used, which means that only a Backsight and a Foresight reading are taken from each instrument position. No intermediate readings are necessary as the objective is to transfer a level quickly.

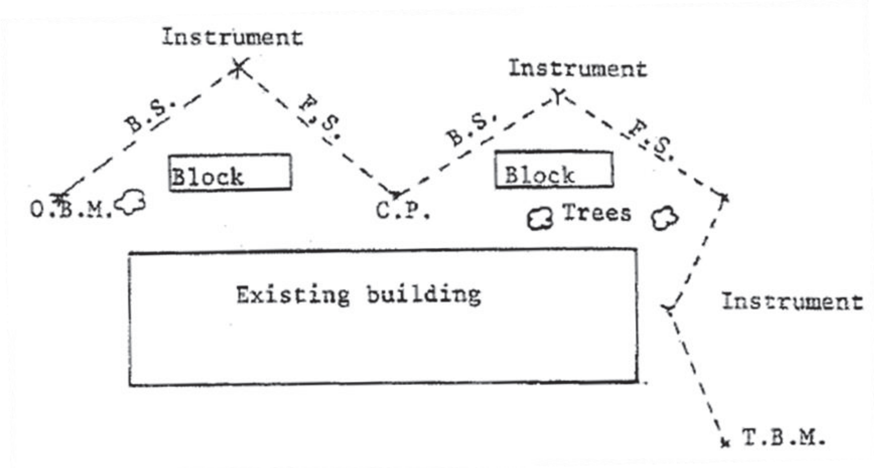


Figure 3.27 Sketch shows situation where flying levels are necessary in order to transfer an ordinance level of a site TBM

3.6.2 Grid levelling (the indirect or interpolation method)

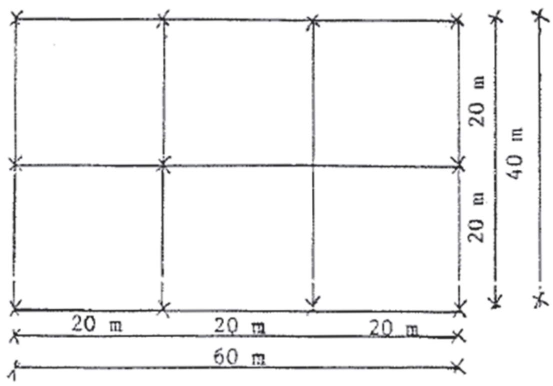


Figure 3.28

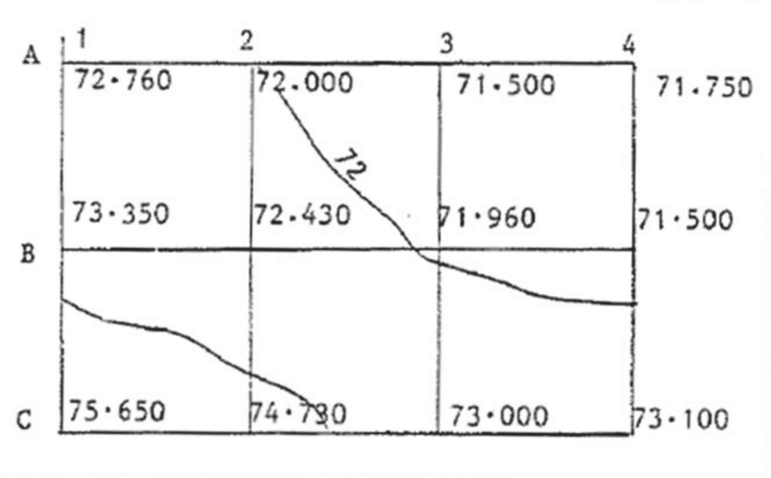


Figure 3.29

The first step in this procedure is to set out a 'grid' over the site. **Figure 3.28** shows a site plan with points 20 m apart in all directions.

The total area of this parcel of land is 60 m x 40 m x 2 400 m².

	Definition: To 'grid' a site This means to set up lines and points where the distance between the lines and points is equal, so that every point is the same distance apart from a neighbouring point (excluding diagonals). Reduced levels are then established in the normal way for each point on the grid.

Figure 3.29 shows the site mentioned above.

Letters are used for the horizontal lines and numbers for the vertical lines, so that each point on the grid has its own name, eg A1, B1, C2, etc.

Alternatively each point may be numbered consecutively, ie 1, 2, 3, 4, 5, 6, etc.

Assume that the 72.000 m contour has to be drawn on the plan. To establish a point where to begin, one has to examine the perimeter of the grid and find a point representing 72.000.

In this example A2 is exactly 72.000 and therefore the beginning point of the contour line is easy to find. It is more usual to find the contour line beginning between points of the grid and one looks for two successive grid points, one of which is below and the other above the required contour line height.

The 72.000 contour must pass through the square on the right or the square on the left. Check the established reduced levels of both squares and it will be found that the line passes between points B2 and B3 (72.430 and 71.960 respectively).

The exact position is found by interpolation and it is assumed that the ground slopes uniformly from B2 to B3. Should there be any sudden rises or falls between the grid points these must be surveyed separately and marked clearly on the plan.

To establish the position of the contour:
(a) Find the difference between 72·430 and 71·960 = 0·470.
(b) Find the difference between the contour line and 72·430 = 0·430
(c) Take (a) as the denominator and (b) as the numerator and calculate this fraction of the grid:

$$\frac{0.43}{0.47} \times 20 = \frac{8.60}{0.47} = 18.300\ m$$

∴ point of contour 72.000 is 18.300 m from B2. From this position the line passes between points B3 and C3, then B4 and C4, to disappear out of the site. The position between B3 and C3 is found thus:

$$73.000 - 72.000 = 1.000$$
$$73.000 - 71.960 = 1.040$$
$$\therefore\ \frac{1.000}{1.040} \times 20 = \frac{20}{1.040} = 19.230\ m\ (from\ C3)$$

All contour lines plotted by this method are found in a similar manner. It must be stressed that for normal building purposes the Interpolation Method is accepted as the simplest and quickest. The accuracy obtained is usually acceptable.

3.6.3 Reciprocal Leveling

When it is not possible to make backsight and foresight distances from the instrument position equal, such as in taking levels across a wide river, the technique of reciprocal leveling may be used.

In Figure 3.30, the level is first placed at Position A, then position B, and level staves are placed at 1 and 2. The distances from A to 1, and B to 2, should be equal. From A, readings are taken to 1 and 2, and from B to 1 and 2. Readings are noted as A1, A2, B1, B2. Differences (A2- A1), and (B2- B1) calculated.

These will not be the same, but their mean gives the difference in level of points 1 and 2, provided the atmospheric conditions have not changed between sets of readings.

(Zeiss (Oberkochen) have developed 'Valley Crossing Equipment', incorporating two of their Ni2 levels, specifically for this type of problem.)

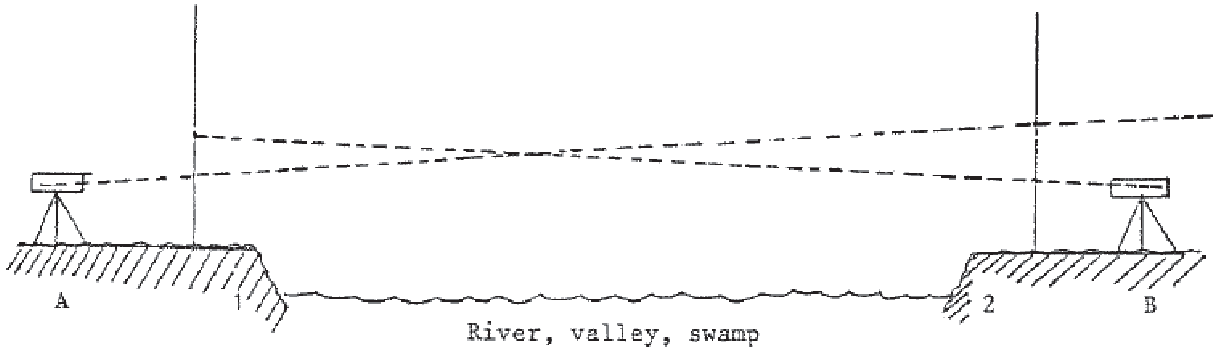


Figure 3.30

Where equalizing sight lengths or reciprocal leveling are impractical, a correction for curvature and refraction must be applied to the staff readings in high accuracy leveling.

The Earth's curvature causes the staff readings to be too large while the effect of refraction is to curve the sight line down. Under average conditions, the correction to a staff reading to eliminate both may be taken as $K^2/15$ m, where K is the sight length in kilometres.

3.6.4 Cross-section Leveling

Where the proposed construction is of considerable width the longitudinal section information must be supplemented by cross-sections.

A cross-section is a profile of the ground at right angles to the longitudinal line, serving mainly to allow the calculation of volumes of earthworks. Cross-sections are not usually taken for pipelines, but are required for roads, railways, and canals.

Cross-sections must be taken at regular intervals along the centreline, the same regular points generally as used for the longitudinal section. The distance apart depends upon the nature of the ground, perhaps 20m on broken ground, or even 100m on gentle slopes.

Occasional sharp changes in ground configuration, such as rock outcrops, may necessitate extra sections at other non-regular points.

The centre-line should be pegged at all points where cross-sections are to be taken, the pegs being driven to ground level and marker pegs placed beside them for identification. The pegs are best placed before the longitudinal section is taken, then the peg levels provide a comparison between long-section and cross-section levels if the two tasks are done separately. (Cross-section leveling acting as the check-leveling for the long-section leveling.)

Alternatively, both long-sections and cross-sections may be leveled at the same time and checked by flying levels.

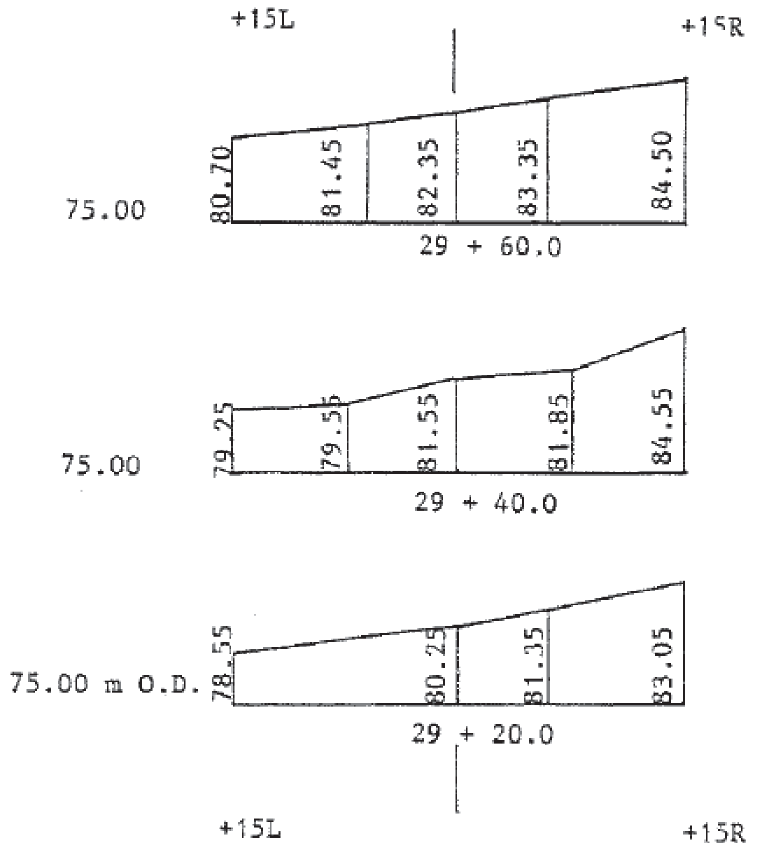
A cross-section is identified by the longitudinal section chainage at its centre, and distances on the cross-section are measured and noted as Left or Right of centre-line. The actual width of a cross-section is fixed by consideration of the construction width, and the width of land reserve available.

The distances left and right are measured by glass-fibre tape, direction usually being judged by eye like offsets. Levels are taken at the centre-line, at all changes of slope, and at the extreme width of section. Flat ground may only need three levels, broken ground perhaps twenty or more.

The actual leveling is normal series-leveling, and usually one cross-section is completed at a time, then on to the next, and so on.

Very steep side-slopes may need two or more set-ups per section, and then it may be faster to take the downhill levels for two or more sections from one set-up and their uphill levels from another set-up.

Booking must be done very carefully in this case to avoid mixing the levels of the two sections.



Cross-sections for proposed road
(ground as existing)

Original scales: Horizontal and vertical 1 : 200
(All measurements in metres)

Figure 3.31

An alternate, and sometimes faster method for steep slopes is to level by theodolite using a sloping collimation line.

When plotting cross-sections, it is normal to use the same scale both horizontally and vertically. Generally the scale is that used for the verticals on the longitudinal section for the same job.

In the two examples quoted earlier, the cross-sections would probably be plotted at 1 : 100 and 1 : 200 respectively.

3.7 Reduction of elevations of points below datum

Study **Table 3.6** carefully.

BS	IS	FS	RISE	FALL	ELEVATION	REMARKS
1,585					-1 561,079	BM 1
	1,271		0,314		-1 560,765	a
	1,158		0,113		-1 560,652	b

	1,954			0,796	-1 561,448	c
2,073		2,134		0,180	-1 561,628	d
	0,655		1,418		-1 560,210	d
		1,981		1,326	-1 561,536	BM 2
+ 3,658		- 4,115	+ 1,845	- 2,302		
		+ 3,658		+ 1,845	--1 561,079	Totals
		- 0,457		- 0,457	- 0,457	Difference

Table 3.6

Important Note!
BM = Bench mark which is a fixed point of known elevation, with regard to some fixed datum plane.

In this example, BMI is 1 561,079 m below the datum plane.

Notice that the tabulation remains exactly the same as was used previously.

Once we have our rises and falls these are added to, or subtracted from, the previous elevation as before, and is done algebraically.

Consider the calculation of the first point a.

Elevation of BM1	-	1 561,079	
Rise from BM1 -a	+	0,314	(rises are plus)
∴ Elevation of a	-	1 560,765	

Elevation of point c

Elevation of point b	= -	1 560,652	
Fall from b - c	= -	0,796	(falls are minus)
Elevation of c	= -	1 561,448	

Collimation method

BS	IS	FS	COLLIMATION	ELEVATION	REMARKS
1,585			1 559,494	-1 561,079	BM 1
	1,271			-1 560,765	a
	1,158			-1 560,652	b
	1,954			-1 561,448	c
2,073		2,134	- 1 559,555	-1 561,628	d
	0,655			-1 560,210	d
		1,981		-1 561,536	BM 2
+ 3,658		- 4,115			
		+ 3,658		BM1-1 561,079	Totals
		- 0,457		- 0,457	Difference

Table 3.7

Remember:

- The lower a point is, the further it is from the datum plane and it's elevation therefore becomes numerically bigger,
- The elevation of the line of sight is established by measuring up from a point of known elevation, ie, elevation of line of sight, or height of instrument = Elevation + BS
- The elevation of the other points is obtained by measuring down from the line of sight, ie Elevation = Collimation - FS

3.8 Inverted staff

During leveling operations in tunnels the staff is often inverted on pegs in the hanging wall.

The above rules hold good if it is remembered to use the proper signs.

For example, suppose the BM is a survey peg in the banging wall of a tunnel, and the elevation of this peg is -2 047,000 and the reading on the inverted staff is 1,022.

then -2 047,000 + (-1,022) = Collimation
or Collimation = -2 048,022

Also, if a foresight be taken onto another peg in the banging wall and the FS reading is 0,728,

then -2 048,022 - (-0,728) = Elevation
ie, Elevation of station = -2 047,294.

The inverted staff is denoted by means of a bar over the reading thus $\overline{0{,}728}$.

Figure 3.32 represents three set-ups of a level, at points SI, S2, and 53.

There are two turning points B and C and one intermediate sight at D.

Looking at the figure it can be seen that there are two methods of finding the elevation of B from A.
(1) By calculating the collimation and then the elevation of B from collimation.
(2) By calculating the rise and fall between A and B.

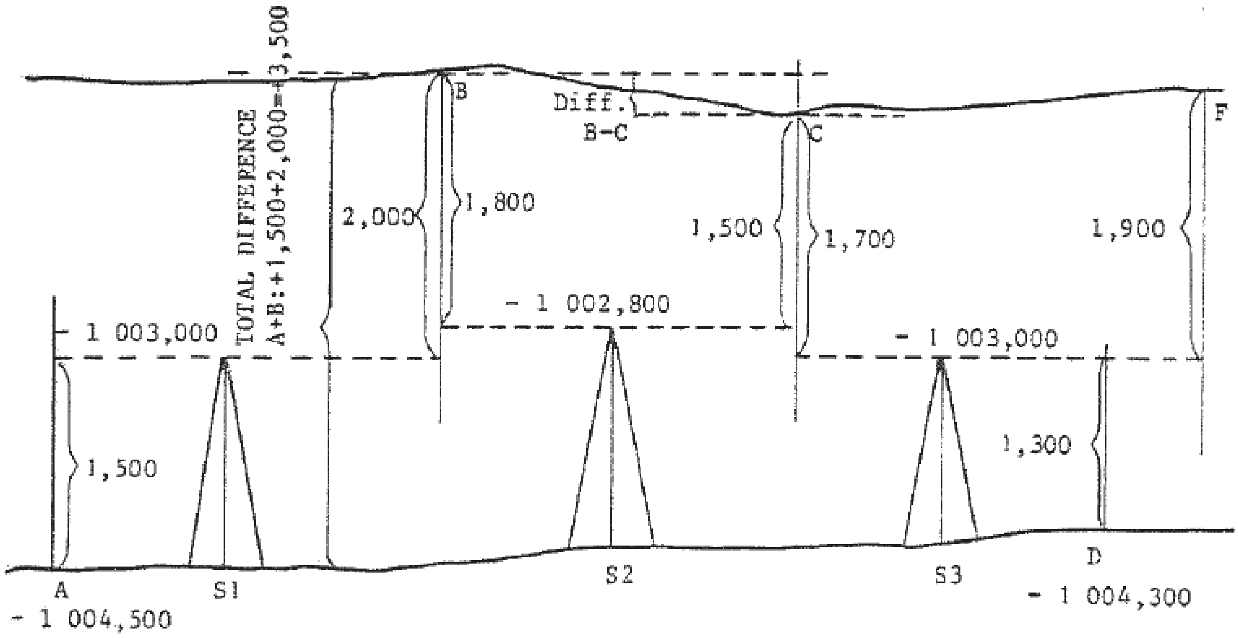


Figure 3.32

In this instance the BS at A is 1,500 and the FS at B is + 2,000 due to an inverted staff reading of 2,000, so there is a rise of 3,500 m,
ie the elevation of B = -1 004,500 + 3,500 = - I 001,000.

Similarly between B and C there is a fall of (BS) – 1,800 and (FS) + 1,500 = - 0,300 so the elevation of C = (- 1 001 ,000) + (- 0,300) = - 1 001,300.

When levelling in a drive or tunnel and sights are taken to hanging wall and footwall, the method of calculating and booking is the same, but inverted staff readngs are written with a bar drawn over the staff reading as shown in **Table 3.8**.

BS	IS	FS	RISE	FALL	ELEVATION	REMARKS
0,521					-1 597,548	BM Rail
	$\overline{1,167}$		1,688		-1 595,860	At start
	1,079			2,246	-1 598,106	10
	$\overline{1,006}$		2,085		-1 596,021	Rail @○9
0,564		1,250		2,256	-1 598,277	○9
	0,731			0,167	-1 598,444	Rail @ LP
	$\overline{1,445}$		2,176		-1 596,268	Rail @○8
	1,490			2,935	-1 599,203	Rail @○6
		$\overline{0,570}$	2,060		-1 597,143	○6
+ 1.085		- 1,250	+ 8,009	- 7,604	-1 597,548	
		+ 0,570				
$\overline{-0,680}$		- 0,680	- 7,604			
+ 0,405			$\overline{+0,405}$		+ 0,405	CHECK

Table 3.8

Note:

- There are three checks on the calculations in this method.
- A bar drawn over a staff reading indicates it has been inverted.
- Backsights are + and with bar.
- Foresights are - and + with bar.
- Rises are + and falls are -.

It would help if you drew a section of the drive and showed the set-ups and readings to the hanging and rail.

Let us consider how the elevations of the various points were obtained.

The elevation of the BM is – 1 597,548

BS reading to rail is 0,521 and the next reading to the hanging wall is 1,167

The total rise from the rail to peg 10 is therefore 0,521 + 1,167 = 1,688.

∴ Elevation of 10 = -1 597,548 + 1,688
= -1 595,860

Since peg 10 is in the hanging wall and the rail at peg 9 is on the floor the total fall from peg 10 to rail at peg 9 is 1,167 + 1,070 = 2,246 and since it is a fall it is -2,246.

Elevation of 10	= -1 595,860
	- 2,246
Elevation of rail	= -1 598,106

You should reason along similar lines and calculate the elevations of the other points to make sure you follow the method.

After a certain amount of practice the calculations can be done mentally.
Also do the problem using the collimation method.

3.9 General

During leveling every effort must be made to ensure that readings are as accurate as possible. The bubble must be centred with the utmost accuracy and readings must be taken with the greatest of care, only when the staff is truly vertical.

To ensure the verticality of the staff, a small circular bubble is most useful. This must be kept in proper adjustment.

Some leveling staves have these bubbles built in, but equally suitable bubbles may be obtained which are held against a corner of any staff and have the advantage that they may be put to other uses.

When no bubble is available, the staff man must be trained to swing the staff backwards and forwards along the line of sight, ensuring that the swing extends both forward and backward of the vertical position. The cross hairs will appear to move up and down the staff.

The staff will be truly vertical when the lowest reading is obtained, With a little practice it is quite easy to take this lowest reading.

Results are always given to three decimal places of a metre.

3.9.1 Application

Suppose it is required to level a line of pegs, starting at a bench mark of known elevation and trying on or closing on to a similar bench mark

(a) Let the staff be held on the bench mark.

(b) Set up the level at a convenient distance from the bench mark, ensuring, by estimation, that the line of collimation will intersect the staff, ie, the difference in elevation must not be such that the line of collimation is either too high or too low to intersect the visible portion of the staff. The line of sight must also be clear of obstructions. At the same time, ensure that a foresight of approximately the same length can be obtained.

(c) Let the change point be placed at the foresight position and the staff held upon it. (The staff man should be trained to do this without supervision).

(d) Level the instrument by means of the circular bubble.

(e) Sight the first staff so that the vertical cross hair falls just beside the graduations.

(f) Level the bubble with absolute accuracy,

(g) Read the exact position at which the horizontal cross hair intersects the graduations.

(h) Check the bubble.

(i) Book the reading in the first line of the backsight column in the book.

(j) Check the staff reading.

(k) Check the booking.
The order of steps (e) 100 (k) is important and these steps must be carried out for each reading.

(l) Repeat steps (e) to (k) for the staff at the foresight position, but book the reading in the second line of the foresight column.

(m) Wave the rear staff man to move to the next position and allow the fore staff man to relax.

(n) Carry the instrument to a convenient position forward of the foresight position which now becomes the backsight position. The first staff man moves to the new foresight position and places his change point.

(o) Repeat the entire process booking the backsight reading in the second line and the foresight in the third line. In this way the two readings to each point are booked in the same line of the appropriate columns.

(p) Proceed in this manner until the first peg is reached, where after the pegs are treated in the same way as change points, the change points being used only when either the distance or the difference in height or both are too great to allow one set-up between pegs.

Continue through to the check bench mark where the final sight will be a foresight.

For the final foresight use the same staff as was used for the first backsight to cancel index error in the staves. Always begin and end level runs on the same staff. This will often require an additional set of readings but not necessarily an additional set-up as the final set-up may be treated as two set-ups, an additional change point being placed in any convenient position.

When only leveling change points, it is often convenient for the observer and the chainman to count paces to ensure equal length backsights and foresights.

When change points are not being used it is usual to estimate the mid-point between successive pegs.

If the pegs are close together or awkwardly spaced it is often not convenient to use a particular peg as a change point. In such cases, a single reading is taken to the point after the backsight has been sighted and before the foresight. Such readings are called intermediate sights and are booked in a separate "Intermediate" column between the backsight and foresight columns.

No other readings are booked on the same line, so that the lines containing the backsight and the foresight for that set-up will be separated by a number of lines equal to the number of intermediate sights.

An intermediate sight is unchecked and must never be accepted unless an independent check has been applied. This will be dealt with under "Checking".

The designations or descriptions of points are entered in the "Remarks" column. Points where change points are used should be marked "CP" to show that a peg number or the like has not been omitted.

Note:
The reliability of a benchmark on which a run starts or ends, should always be checked by leveling to at least one other bench mark and comparing the results so obtained with the given values.

3.9.2 Checking
It is not sufficient that the level at the final bench mark agrees within allowable limits, as a compensating mistake may have occurred. For this reason, it is absolutely essential that all runs be leveled at least twice, preferably in opposite directions.

When there is no closing bench mark, the run should be leveled at least three times.

As intermediate sights are used, both as backsight and as foresight any error in reading will not show as an error in closure, so that these sights are quite unchecked. It is, therefore, essential that all intermediate sights be checked, either from another set-up on the same run or from the check run.

3.9.3 Adjusting
In **Table 3.9** below, the leveling traverse was started on BM1 and closed on BM2. The final elevations of these two benchmarks are known. It is therefore necessary to adjust the leveling results in order to balance the traverse.

The closing error, is found by subtracting the Reduced level (unadjusted) of the closing point (BM2) from its final elevation. The adjustment for each set-up is then found as follows:

adjustment/set-up = $\frac{e}{n}$ where n is the total number of set-ups.

STATION	BS	IS	FS	RISE	FALL	REDUCED LEVEL (UN-ADJUSTED)	FINAL ELEVATION	ADJ/SET-UP	SET-UP
BM1	1,500					+125 000	+125 000		
1		1,650			0,150	+124 850	+124 852	+0,002	1
2		2,405			0,450	124,095	+124,097	+0,002	1
3		3,505			1,100	122,995	+122,997	+0,002	1
4	1,700		2,000	1,505		124,500	+124,502	+0,002	
5		1,250		0,450		124,950	+124,954	+0,004	2
6	2,000		1,800		0,550	124,400	+124,404	+0,004	2
7	1,800		3,200		1,200	123,200	+123,206	+0,006	3
8		2,165			0,365	122,835	+122,843	+0,008	4
BM2			2,120	0,045		122,880	+122,888	+0,008	4
	+7,000 -9,120 -2,120		9,120	2,000 -2,120	4,120	-125,000 -2,120			

Table 3.9

In the above example the closing · error, e = +122,888 (given) - + 122,880 = +0,008.

There were 4 set-ups.
Thus error/set-up = $\frac{+0,008}{4}$ +0,002

This error (+0,002) is spread progressively, starting from set-up 1. All elevations reduced from set-up 1, change by +0,002 (up to and including the foresight ie peg 4).

The elevations reduced from set-up 2 change by +0,004 . Please note that the backsight of set-up 2 (ie peg 4) is excluded

The correction is made up to and including the foresight (peg 6).

The elevations reduced from set-up 4 change by +0,008. It can now be seen that the elevations of all pegs between BM1 and BM2 are in perfect balance with the elevations of the benchmarks.

Since the sign of the closing error is very important, it should, once again be noted that the error, e, is found by always subtracting the unadjusted Reduced Level of the closing point from the final elevation of that point.

In the case where more than one run between end benchmark have been made, each run is balanced first and a mean value between the balanced runs taken as the final elevations.

3.10 Gradient

If the horizontal distance between two points is known and we have determined the difference in elevation between the points, then the gradient can be calculated.

$$Gradient\ = \frac{Difference\ in\ elevation}{Horizontal\ distance}$$

Let the difference in elevation between A and B be 5 m and the horizontal distance be 100 m. Let B be the lower point.

Then gradient = $\frac{5}{100}$, or 1 in 20 down-grade from A to B, or 1 in 20 up-grade from B to A.

2.11 Plotting of profiles

A profile is a vertical section of country, drawn to show all the elevations and depressions along any chosen line.

For the construction lines, roads, cuttings, drains etc, it is frequently necessary to elevate points, usually in a straight line, or along the proposed cutting etc, so that a profile can be plotted to a convenient scale. Horizontal distances between points must also be recorded.

Once the profile has been plotted, the observer can see the nature of the ground in its true perspective at a glance, and the engineer can obtain very useful information, as we shall see later.

The plotting of the profile must be done to a definite scale. Only when the horizontal and vertical scales are the same, will we have a true representation of the area surveyed. This is not always convenient, as horizontal distances are usually large as compared with vertical differences in elevation.

In such cases we use different scales for the horizontal and vertical plotting. This gives an exaggerated view of the profile and is usually of more value to the engineer.

Illustrative Example 3.1

The following leveling notes were taken for a drain between points A and B. The drain has to have an even grade from A to B. How much cut or fill has to be done at each point?

Plot the profile of the points. Horizontal distances from A to each point are given.

STATION	BS	IS	FS	DISTANCE (METRES)	REDUCED LEVEL
A	3,383			0	71,597
1		2,228		40	
2		1,347		60	
3	1,649		2,265	80	
4		1,682		120	
5		0,640		130	
6	0,043		2,639	150	
B			2,243	165,750	

Table 3.10

Now re-copy these notes and reduce them by the rise and fall method.

The Collimation method may also be used.

STATION	BS	IS	FS	RISE	FALL	RL	DISTANCE (M)
A	3,383					71,597	0
1		2,228		1,155		72,752	40
2		1,347		0,881		73,633	60
3	1,649		2,265		0,918	72,715	80
4		1,682			0,033	72,682	120
5		0,640		1,042		73,724	130
6	0,043		2,639		1,999	71,725	150
B			2,243		2,200	69,525	165,750
	+5,075		-7,147 +5,075 -2,072	+3,078	-5,150 +3,078 -2,072	-2,072	CHECKS

Table 3.11

Note Abbreviation: RL stands for Reduced Level

The gradient

There is a total fall in elevation from A-B of 2,072 metres and the total distance is 165,75 metres.

Down grade from A to B $= \frac{2,072}{165,75} =$ 1 in 80

This means that for every 80 metres the elevation of the drain will fall by 1 metre. It is often more convenient to express the gradient as a percentage.

ie, Gradient $= \frac{2,072}{165,75} \times 100$

= 1,25% downgrade from A to B.

2.12 Calculation of grade elevations, and cut and fill

STATION	CALCULATION OF DIFFERENCE IN GRADE	RL	GRADE ELEVATION	CUT	FILL
A		71,597	71,597	-	-
	$\frac{1}{80} \times 40 = 0{,}500$				
1		72,752	71,097	1,655	-
	$\frac{1}{80} \times 20 = 0{,}250$				
2		73,633	70,847	2,786	-
	$\frac{1}{80} \times 20 = 0{,}250$				
3		72,715	70,597	2,118	-
	$\frac{1}{80} \times 40 = 0{,}500$				
4		72,682	70,097	2,585	-
	$\frac{1}{80} \times 10 = 0{,}125$				
5		73,724	69,972	3,752	-
	$\frac{1}{80} \times 20 = 0{,}250$				
6		71,725	69,722	2,003	-
	$\frac{1}{80} \times 15{,}750 = 0{,}197$				
B		69,525	69,525	-	-
		578,353 563,454 14,899	563,454		

Table 3.12

The grade is calculated from point to point rather than from the starting point. If all the calculations are done from point A it is easy to make a mistake with the intermediate points, but when working from point to point this error is avoided as the grade elevation of the last point must then check with the grade at that point.

When the elevation of the point is higher than the grade elevation it is a cut, and when the elevation is lower than the grade elevation then there is a fill.

Checks

Calculate cut and fill.

Add all the elevations of the points and all the grade elevations; the difference between these two totals must be equal to the difference between the total cut and total fill.

The grade can also be calculated by using the percentage gradient and multiplying it by the distance.

eg

A to 1

40 metres at 1,25% = -0,500

1 to 2

20 metres at 1,25% = -0,250

and so on.

This will give identical differences as in the example.

Usually the whole calculation is done in one operation as will be shown in the next example.

Illustrative Example 3.2: Plotting the profile

Select a suitable horizontal scale, say one in a thousand ie, 1 mm = 1 mm and a vertical scale of one in a hundred ie 1 cm = 1 m. Now select a suitable datum, say 60 metres. Draw horizontal line AB = 165,750 to the horizontal scale and mark off the distances for each point. Now plot the (vertical distances) elevations of each point at right angles to the horizontal line.

Join the points A and B. This line represents the drain. It will be noticed that there is a cut at each of the points 1 to 6; had any of the points plotted below 1:he line there naturally would have been a fill at such point.

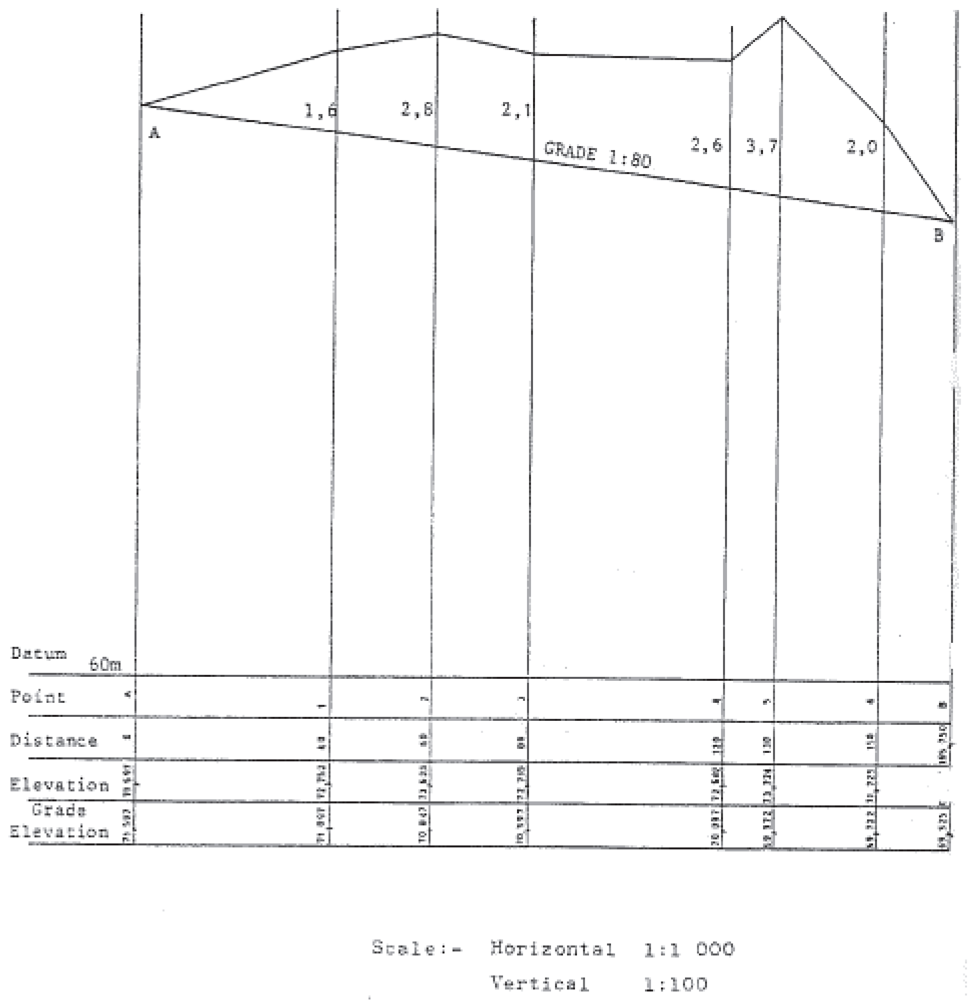


Figure 3.33

Worked Example 3.1

The following notes were taken during a leveling survey of a portion of a proposed railway line. The rails, when completed must be on an even grade from A to H, and the present elevation of A and H will remain unchanged.

(a) Complete the leveling notes by means of the rise and fall method.
(b) Calculate the amount o:f cut or fill necessary at each point between A and H.

STATION	BS	IS	FS	ELEVATION (METRE) AMSL	HORIZON L DISTAN (M)
A	2,865			+ 413,50	0
B		1,830			60
C	1,645		1,100		130
D		2,345			195
E		3,930			265
F	2,500		1,920		335
G		3,475			420
H			1,555		487
CHECKS					

Table 3.13a

Solution:

STATION	DIS-TANCE	BS	IS	FS	RISE	FALL	ELEVATIO N	DISTANCE x GRADE	GRADE ELEVATION	CUT	GILL
A	0	2,865					413,500	$\frac{1}{200}\times 60 = 0{,}300$	413,500	-	-
B	60		1,830		1,035		414,535	$\frac{1}{200}\times 70 = 0{,}350$	413,800	0,735	-
C	130	1,645		1,100	0,730		415,265	$\frac{1}{200}\times 65 = 0{,}325$	414,150	1,115	-
D	195		2,345			0,700	414,565	$\frac{1}{200}\times 70 = 0{,}350$	414,475	0,090	-
E	265		3,930			1,585	412,980	$\frac{1}{200}\times 70 = 0{,}350$	414,825		1,845
F	335	2,500		1,920	2,010		414,990	$\frac{1}{200}\times 85 = 0{,}425$	415,175		0,185
G	420		3,475			0,975	414,015	$\frac{1}{200}\times 67 = 0{,}335$	415,600		1,585
H	487			1,555	1,920		415,935		415,935	-	-
CHECKS		+7,010		-4,575	+5,695	-3,260	3 315,785		3 317,460	1,940	3,615
		-4,575			-3,260				3 315,785		1,940
		+2,435			+2,435		+2,435		1,675		1,675

$$\text{GRADE} = \frac{2{,}435}{487} = 1:200\ (upgrade)$$

Table 3.13b

Worked Example 3.2

Reduce the following leveling notes by the "rise and fall" method and adjust the closing error.

BS	IS	FS	FINAL ELEVATION	POINT
2,680			204,110	BM 1
	0,875			a
	0,980			b
1,665		0,430		c
	1,440			d
1,010		0,625		e
	1,690			f
2,455		1,225		g
	3,575			h
		3,880	205,720	BM 2

Table 3.14a

In this 'type of reduction we have two columns for the elevations viz:- Reduced Level and Final Level.

Solution:

BS	IS	FS	RISE	FALL	REDUCED LEVEL	FINAL LEVEL	POINT
2,680					204,110	204,110	BM 1
					-0,010		
	0,875		1,805		205,915	205,905	a
					-0,010		
	0,980			0,105	205,810	205,800	b
					-0,010		
1,665		0,430	0,550		206,360	206,350	c
					-0,020		
	1,440		0,225		206,585	206,565	d
					-0,020		
1,010		0,625	0,815		207,400	207,380	e
					-0,030		
	1,690			0,680	206,720	206,690	f

					-0,030		
2,455		1,225	0,465		207,185	207,155	g
					-0,040		
	3,575			1,120	206,065	206,025	h
					-0,040		
		3,880		0,305	205,760	205,720	BM 2
+7,810		-6,160	+3,860	-2,210			CHECKS
-6,160			-2,210				
+1,650			+1,650		+1,650		

Table 3.14b

From the reduced level of BM2 it can be seen that there is a error in the leveling of -205,720 + 205,720 = -0,040.

This closing error must be distributed uniformly through the levels, and it is assumed that 'the error bas been the same at each set-up. So their error is divided by the number of set-ups, thus:-

$$\frac{-0{,}040}{4} - 0{,}010$$

Note:
The number of BS give the number of set-ups.

There is in this case therefore an error of 0,010 at each set-up, which is cumulative, ie all reduced levels from the first set-up must be adjusted by -0,010, and all reduced levels from second set-up by 0,020 and so on.
In this example the error taken is larger 'than would occur in practice.

There is usually a limit set to the error in leveling for different types of work and these vary from ± 30 $\sqrt{K}$ mm to ± 5 $\sqrt{K}$ mm where K is the distance in kilometres.

Sources of errors

Source	Precaution
1. Permanent adjustments of the instrument faulty	Check instrument at intervals, adjust as needed. If cannot adjust, make sure that distances from instrument to backsight staff position and foresight staff position are the same - errors will cancel out.
2. Temporary adjustments faulty - including bubble not centred when reading staff, parallax not eliminated, tripod sinking in soft ground, instrument moved due to surveyor leaning on or kicking tripod	Check all operations carefully, do not lean on or kick the tripod, etc.
3. Faulty staff holding (a) Readings too large with a non-vertical staff	Check laterally for vertical by seeing if staff is parallel to central vertical cross-hair. To ensure vertical in the other direction, use staff bubble or have the staff man swing the staff slowly towards the level and away- note the smallest reading at the central crosshair. This is generally only used at change points or important intermediates
3. (b) Readings too large if staff held on soft ground	Select firm ground, or use a change plate.
4. Climatic effects- High wind may cause tripod shake, or dislodge staff	Avoid leveling in high winds, or use windshield around the instrument and stay-rod' on the staff.
Sun near horizon may make sighting impossible	Avoid sighting near the sun, use telescope ray shade.
Direct heat of sun may cause differential expansion of instrument parts, in particular the bubble tube.	Use umbrella in high temperatures.
Heat shimmer may make the staff graduations appear to 'bounce'.	Avoid sight rays grazing the ground.
Raindrops on objective may make sighting difficult or impossible.	Use ray shade and umbrella.
5. Curvature of the earth's surface combined with the bending down of the sight line due to refraction by the atmosphere.	Error negligible in ordinary leveling - eliminated in any case by equalizing backsight and foresight lengths.
6. Reading errors - including the commonest of reading stadia hair instead of central hair, reading the decimals correctly but omitting the metre figure, reading up instead of down, and common errors of 0,1.	Care, attention, and practice. Read, book and read again.
7. Booking errors - entry in wrong column, forgetting to book a reading, transposing the digits of the reading when booking.	Care, attention, and practice. Read, book and read again.

Table 3.15

Activity 3.1

1. Define:
   (a) Leveling
   (b) Line of sight
   (c) Datum plane.
2. State four advantages of an automatic level.
3. 3. Sketch the following three levels, showing their main components and differences:
   (a) Dumpy level
   (b) Tilting level
   (c) Automatic level.

Activity 3.2

1. (a) Give a brief description of the tilting level.
   (b) Explain how you would set up a tilting level.
2. Give a brief comparison on the construction of the surveyor's-,tilting- and automatic levels.

Activity 3.3

1. Reduce the following leveling observations using the Rise and Fall method.

BS	IS	FS	REMARKS
0,957			BM A
	0,634		b
	1,131		c
	1,774		d
	0,524		e
1,844		1,695	TP 1
	2,195		f
	1,387		g
	0,978		h
	1,463		i
	1,558		j
2,048		2,262	TP 2
	1,792		k
	2,664		L
	1,963		m
	0,975		n
	1,265		o
		2,463	BM p

Table 3.16

2. Compare the two methods of reducing leveling notes. State for what type of work each method is preferred.
3. Briefly define flying levels and reciprocal levels.

Activity 3.4

1. Complete the leveling tabulation below. Show all checks.

STATION	HOR. DIST. BETWEEN STATIONS(IN METRES)	BS	IS	FS	RISE	FALL	ELEVATION
A	0	1,350					
B	20,5		1,915				
C	30,5	0,750		1,445			
D	42,5	2,665		2,035			-676,630
E	50,0		2,010				
F	30,5	2,900		0,505			
G	20,5		2,250				
H	5,5			1,680			
CHECKS							

Table 3.17

2. The following leveling notes were obtained from a traverse along the centre line of a proposed road:

STN.	HOR. DIST. FROM A	BS	IS	FS	FINAL ELEVATION
A	-	4,50			94,20
B	50		4,00		
C	150		2,05		
D	300	3,32		0,42	
E	400		2,28		
F	450	0,26		1,54	
G	500		2,22		
H	560	0,20		4,24	
J	650		1,08		
K	750		3,90		
L	800			4,00	92,20

Table 3.18

Calculate the final elevation of each point.

Activity 3.5

1. Reduce the following leveling notes by means of the rise and fall method, Show all checks.

STATION	BS	IS	FS	REDUCED LEVEL
A	1,65			
B		1,12		
C		+2,50		
D		+3,00		
E	+2,20		+2,45	
F			1,75	

Table 3.19

Note: (+) means inverted staff.

2. Complete the leveling tabulation below. Show all calculations clearly. Assume an even gradient from grade elevation at A to Grade elevation at H.

STATION	HOR. DIST. BETWEEN STATIONS (IN METRES)	BS	IS	FS	RISE	FALL	ELEVATION	GRADE ELEVATION	CUT	FILL
A	0	1,350						-675,250		
B	20,5		1,915							
C	30,5	0,750		1,445						
D	42,5	2,665	2,010	2,035						
E	50,0						-676,630			
F	30,5	2,900	2,250	0,505						
G	20,5									
H	5,5			1,680						
								-674,250		
CHECKS										

Table 3.20

Work to the nearest 0,005 m.

Self-Check

I am able to:	**Yes**	**No**
• Describe the sources of vertical control.		
• Describe using a datum as reference for bench marks.		
• Describe the use of maps to obtain the position of control points.		
• Describe the establishment of bench marks giving reason for use.		
• Give definitions of levelling terms.		
• Describe levelling instruments.		
• Give a brief description of traditional levelling methods and instruments including the spirit, water and Cowley levels:		
o dumpy level		
o tilting level		
o automatic level		
• Describe how to check the accuracy of levelling instruments.		
• Describe how to take the reading of the metric levelling staff.		
• Demonstrate recording and calculating reduced levels by "rise and fall" and "collimation" methods including inverted staff and application of the required checks and corrections.		
• Describe the following levelling methods:		
o Flying		
o Grid		
o Reciprocal		
o Cross sectional		
• Describe the sources of induced and instrumental errors.		
If you have answered 'no' to any of the outcomes listed above, then speak to your facilitator for guidance and further development.		

Module 4

Angular measurement

Learning Outcomes
On the completion of this module the student must be able to:

- Describe the basic construction of the theodolite.
- Describe the various types of theodolite.
- Describe the use of the theodolite to measure horizontal and vertical angles.
- Demonstrate recording the readings form the theodolite.
  - Explain tacheometry, calculation of horizontal and vertical components.

4.1 Introduction

Basic field operations performed by a surveyor involve linear and angular measurements. Through application of mathematics (geometry and trigonometry) and spatial information knowledge, the surveyor converts these measurements to the horizontal and vertical relationships necessary to produce maps, plans of engineering projects, or Geographical Information System/ Land Information System (GIS/LIS).

The highway surveyor must be adept at making the required measurements to the degree of accuracy required. Various types of engineering works require various tolerances in the precision of the measurements made and the accuracies achieved by these measurements.

The use of common sense and development of good surveying practice in all phases of a survey cannot be overemphasized. All conditions that may be encountered in the "real world" during the actual field survey cannot be covered in any manual.

A manual may specify certain techniques, such as a certain number of repeated operations, to achieve a required accuracy. The surveyor must then often use judgment based on the equipment being used and the field conditions encountered, to modify those techniques.

Some field conditions (heat waves or wind for example) may make it impossible to perform some operations to a consistent degree of accuracy

4.2 The telescope

4.2.1 Functions

The telescope used in surveying instruments performs two basic functions.

(a) It provides a magnified image of the sighted object, so that it will appear to be closer to the observer than it actually is, thereby allowing it to be viewed in greater detail than is possible with the unaided eye.

(b) It provides a sighting mark which may be centred on the chosen object, thus enabling the telescope to be pointed with great accuracy.

It is quite possible to construct a theodolite without a telescope, using simply some form of sighting device such as the sights of a rifle, but as function (a) above would be absent, it would not be possible to perform function (b) with any reasonable degree of accuracy.

4.2.2 Basic principles

Basically, the survey telescope consists of two convex lenses, as shown in **Figure 4.1**. The lens 0 is known as the objective, objective lens or object glass, while E is the eyepiece.

The optical of a lens is a point situated, in the case of convex and concave lenses, at or near the geometrical centre of the lens, and the principal axis of the lens is a line perpendicular to the lens and passing through the optical centre.

The optical centres of the lenses 0 and E are shown at Cl and C2, respectively.

In a telescope, the principal axes of the object glass and the eyepiece should coincide, and this common line Cl C2 is known as the optical axis of the telescope.

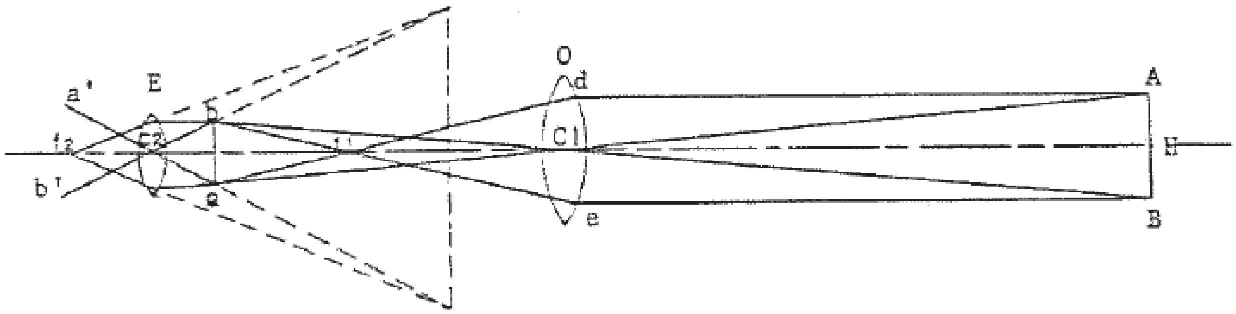


Figure 4.1

Light rays striking a glass surface obliquely are bent, or refracted, on entering and on leaving the glass, i.e., at each glass-to-air surface. This does not apply to rays perpendicular to the glass, which pass through as straight lines.

Although not strictly true, it may be assumed, for practical purposes, that all rays passing through the optical centre of concave or convex lenses are un-refracted. In fact, they emerge parallel to, but slightly displaced from the original path.

All rays parallel to the principal axis of a convex lens, such as Ad and Be in the figure, are refracted in such a way that they converge upon a point f1, on the principal axis. This point is known as the focus or principal focus, and the distance from the focus to the optical centre is the focal length of the lens.

In order to understand how an image of the sighted object is formed in the telescope, it must first be understood that any object consists of a vast number of points, and that light rays are reflected from these points, making the object visible.

Let us now examine only two of the rays emanating from A. The ray striking the lens at d follows the path A-d-fl-a, and the ray through Cl follows the path A-Cl-a.

Similarly, all rays from A which strike the lens will converge on a, forming an image of the point. Similarly, the rays from B will converge on b, and rays from all other visible points will converge to corresponding positions, thus forming a complete picture of the sighted object, but, as is clear from the figure, the image will be inverted.

This is known as a real image for, if a screen were placed in the plane ab, the image would be visibly projected. This plane, in which all points equidistant from the lens are in sharp focus, is called the focal plane. It will be seen later that, for varying distances, the position of the focal plane varies.

Let us now examine the effect of the eyepiece lens E.

Rays of light emanating from the image ab (which are, of course, simply continuations of rays originating at AB) pass through lens E. The parallel rays from a and b converge at f2, the focus of lens E, while the rays aC2 and bC2 pass straight through the optical centre.

As the aperture of the eyepiece .is only the size of the pupil of an average human eye, most of the rays passing through the lens will enter the pupil of an eye placed close to the lens, and the rays will appear to emanate, not from a and b, but from a' and b'.

As indicated in the figure these apparent rays intersect at a' and b', forming an erect image of ab and hence an inverted image of AB. This is not a real image, but a virtual image. All other points on the sighted object are similarly seen in the plane a'b'.

If now a sighting mark (usually in the form of cross-hairs) were placed in the plane ab, it too would be seen in the plane a'b', and the intersection of the hairs would appear to be superimposed over a definite point on the sighted object.

If the sighting mark is also situated on the optical axis CI C2 of the telescope, this may then be pointed, with extreme accuracy, towards any chosen mark.

4.2.3 Parallax and focusing

It is essential that the graticule be placed exactly in the focal plane for two reasons:

(a) The eyepiece lens must be situated at a certain distance (dependent upon the eyesight of the observer) from the objects viewed through it to obtain a clear image and, as both graticule and focal plane image are being viewed at the same time, it must be at the same distance from both. Should the two not be in the same plane, a blurred image of one or the other or both will result.

Definition: Graticule
A network of lines representing meridians and parallels, on which a map or plan can be represented.

Provision is made for moving the eyepiece backwards and forwards to compensate for varying eyesight. This is usually accomplished by mounting the eyepiece in a threaded sleeve, so that a finely controlled movement may be applied.

By screwing the eyepiece backwards or forwards, the graticule is brought into focus, i.e., a clear, well-defined image of the cross-hairs is obtained.

Note:
The eyepiece is not used for focusing the sighted object.

(b) Should the cross-hairs and the sighted object not be seen in the same plane, it will be noticed that, if the eye is moved up and down or from side to side, the cross-hairs will appear to move in relation to the object.

This condition is known as parallax and is illustrated by **Figure 4.2** where AA is the plane in which the cross-hairs are seen and BB is the plane in which the sighted object is seen. With the eye in position El, the intersection of the hairs C would appear to be superimposed upon the point PI. If the eye is now moved to E2, C would appear to move to P2.

The position of the cross-hairs in relation to the object is thus dependent upon the position of the eye, and it is impossible to determine the point towards which the telescope is pointing with any degree of exactitude.

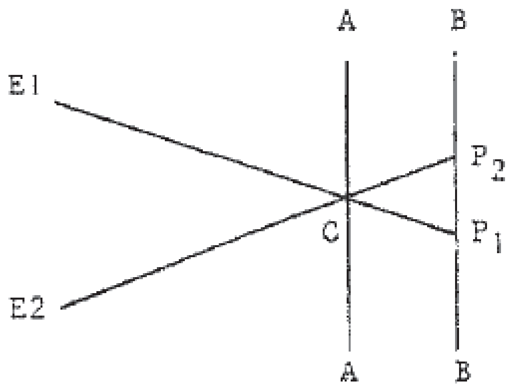


Figure 4.2

It might be thought that either the length of the telescope or the focal length of the object glass could be so fixed that the real image of the object would always fall on the graticule plane, in which case the requirements for both clear vision and parallax would be met.

Unfortunately, this is not so, for the position of the real image is dependent upon the distance of the object from the object glass.

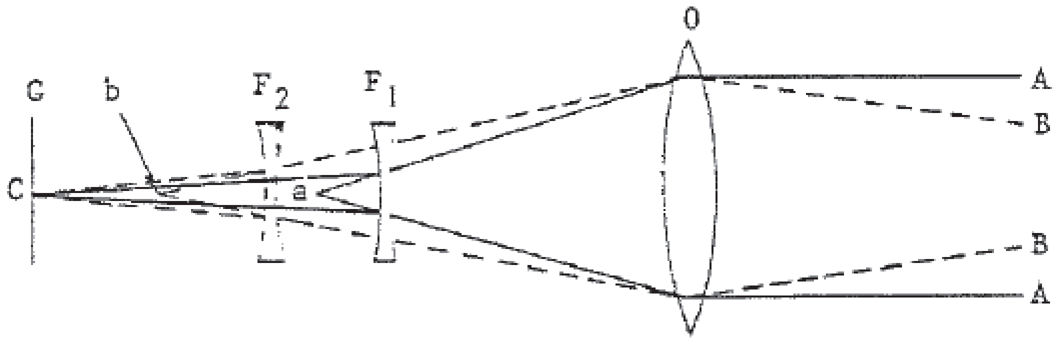


Figure 4.3

In **Figure 4.3** the rays from a very distant object A, such as a star, may be regarded as being parallel. They are refracted by the abject glass 0 and intersect at a, as shown by the solid lines, forming a real image at or near the focus of the lens.

The rays from a closer object B, on the other hand, follow the path of the broken lines and form an image at b so that, if the relation between the graticule plane and the object glass remained fixed, the telescope could be used only over a given distance and would be useless for varying distances.

The simplest means of overcoming this difficulty is to mount the object glass in a tube capable of sliding backwards and forwards within the main telescope tube.

This principle was used on earlier theodolites, where a focusing screw actuated a rack and pinion to impart a controlled movement to the object glass. In certain rare cases, the eyepiece/diaphragm assembly was capable of movement in relation to a fixed object glass.

Either way, it is possible to bring the graticule and focal planes into coincidence, simply by varying the distance between graticule and object glass to suit the distance to the observed object. This is known as an external focusing telescope.

This system, however, suffers some serious defects:
1. Wear between the two tubes enables the object glass to wobble, which causes shifts in the position of the image and thus errors in reading.
2. It is difficult to seal the telescope against dust and moisture.
3. An object glass with a comparatively great focal length must be used, making the telescope long and ungainly.
4. Variation in the position of the object glass causes changes in the balance of the telescope.
5. This system suffers certain disabilities in the optical measurement of distance.

This led to the development of the internal focus in telescope, as used on more modern theodolites, where focusing is achieved by means of a third lens, known as the focusing lens, which is situated inside the telescope tube, between the object glass and the diaphragm.

This lens may also be moved backwards and forwards by means of a similar focusing screw or a sleeve which rotates around the telescope tube near the eyepiece.

In this case, a concave or plano-concave lens is used. The effect of a concave lens is the opposite to that of a convex lens, i.e., rays of light passing through the lens are scattered or diverged.

Figure 4.3 shows the focusing lens in two positions, F1 and F2. In position F1, the rays from the distant point A are refracted so that they no longer intersect at a, but at C in the graticule plane G.

In position F2, the rays from the nearer point B are prevented from intersecting at b and again intersect at C. In this way the image of the sighted object may be made to coincide with the graticule plane for any distance.

In practice all telescopes have a minimum focusing distance, ie, objects closer than a certain distance, which may vary between about one and five metres, cannot be brought into focus.

For practical purposes, the focusing lens may be regarded as a means of increasing the focal length of the object glass, so that the two together may be considered a single lens system v1ith a variable focal length.

This telescope may be very effectively sealed, requires an object glass of shorter focal length, thus making the telescope shorter and more convenient, and removes the disabilities in optical distance measurement.

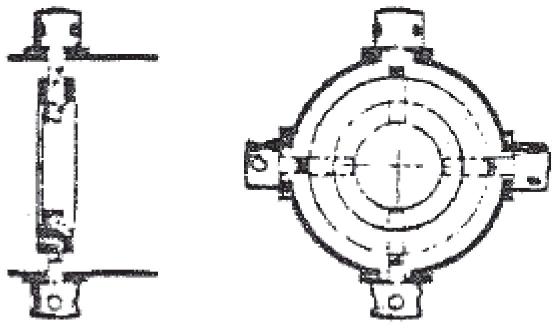
Now only two of the drawbacks of the external focusing principle remain, and these have, in fact, been reduced to negligible proportions.

4.2.4 The diaphragm and reticule

The diaphragm carrying the reticule, or graticule, of cross-lines is a flanged brass ring held in place in "the telescope tube by four capstan-headed screws, as shown in **Figure 4.4**.

These are screwed into the flange, but pass through slots in the telescope tube, so that, when slackened, they permit movement of the diaphragm horizontally and vertically as well as a small rotation about the axis of the telescope.

The heads of the screws, instead of being exposed, may be protected by covers to prevent their being turned accidentally.



Diaphragm

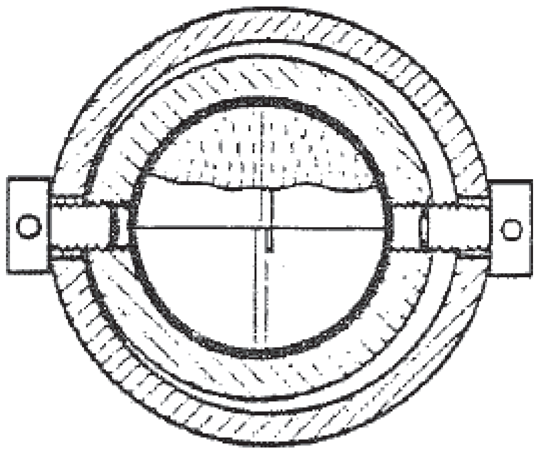


Figure 4.4

4.2.5 Magnification and resolving power

Magnification is the number of times larger an abject appears to be, when sighted through the telescope, than when seen with the unaided eye. For a given object glass, the magnification may be increased or decreased by fitting eyepieces of greater or lesser" power.

In general, it may be said that, with a magnification of say 20 times, an object appears as it would if viewed direct from one twentieth of the distance, i.e., an object 1 000 metres away appears as it would to the naked eye at a distance of 50 metres.

This is only partly true, for it applies to perfect observing conditions, which are virtually never met with. At 1 000 metres there is more intervening atmosphere than at 50 metres, and this causes obstruction and distortion of light rays.

The poorer the observing conditions, the more this rule break down, and the effects are intensified by the telescope, so that a stage is often reached where an object visible to the naked eye cannot be seen at all through the telescope.

The resolving power of a telescope is a measure of its capacity to distinguish between points situated close together, and, therefore, of the accuracy with which it may be pointed, and may be stated in seconds of arc .

From the foregoing, it is evident that resolving power is dependent upon magnification, brightness of the image and definition, ie, the sharpness with which focusing can be accomplished which is, in turn, dependent upon the absence of aberration and upon observing conditions. (Aberration is a defect.)

Note:
While resolving power is increased by an increase in magnification, such an increase also leads to an intensification of the effects of poor observing conditions and, without a corresponding increase in the size of the object glass, to loss of brightness.

It may well be that, under average conditions, the resolving power of a low-powered telescope, with good definition and brightness of image, caul d be greater than that of a higher-powered telescope with poorer definition and brightness of image.

The magnification of the telescope should, therefore, be so chosen that a balance is struck between the various requirements, and should not be greater than can be usefully employed by the instrument, i.e., it should be matched to the accuracy of the rest of the instrument.

Magnifications up to 80 times have been employed, but generally they vary from 10 to about 40 times and, for the average theodolite, from about 24 to 30 times.

4.3 The theodolite (Tacheometer or transit or transit theodolite)

In the past, theodolites were relatively simple instruments, and the average surveyor was quite capable of carrying out the routine maintenance of cleaning and lubrication.

The modern theodolite, on the other hand, is a highly complex and delicately assembled mechanism, the repair and internal maintenance of which are quite beyond the scope of the average surveyor. It is, however, far more reliable and maintains its adjustments far better than the older models.

Nevertheless, periodic inspection and overhaul are essential to maintain its high standards of efficiency and ease of handling, and to prevent excessive wear due to grit collecting in the working parts. This work should be entrusted only to suitably qualified instrument mechanics, as any attempt by the surveyor to carry out this work will, almost certainly, lead to serious damage.

It is, however, essential for the surveyor to have some knowledge of his instrument, the names of its more important parts and the basic ways in which it functions, so that he will better appreciate its capabilities and its limitations and be capable of caring for it properly and carrying out certain tests and adjustments.

4.3.1 General description

The following description should be read in conjunction with **Figure 4.5**. This shows a sectional view of a very simple theodolite, and will serve to illustrate the basic principles and acquaint the student with the names of the various parts.

The other illustrations show general and sectional views of some typical modern theodolites, and should be thoroughly studied after the more basic type is understood, and also in conjunction with the detailed descriptions of some of the more important parts.

The theodolite consists primarily of a horizontal circle, a vertical circle and telescope mounted centrally relative to both circles.

The horizontal circle can be rotated in a horizontal plane, together with the telescope and can be clamped in any desired position.

The telescope can also be rotated independently of this circle, so that it can be set in any desired position relative to the circle, or the direction in which the telescope is pointing may be accurately read in relation to the horizontal circle.

The vertical circle is normally rigidly fixed in relation to the telescope, which can be rotated in a vertical plane, and a reading device is controlled by means of a level tube or spirit level, commonly referred to as a bubble, so that accurate vertical angles may be read.

The telescope is fitted with fine cross hairs, so that the object may be accurately sighted. In most modern theodolites, additional hairs are fitted, known as stadia hairs, which enable the observer to deduce distances when sighting a suitably graduated staff. When so fitted the instrument is known as a tacheometer.

Leveling screws, usually referred to as foots crews, are provided so that the instrument may be accurately leveled, with the aid of the bubbles.

Clamps and setting screws, often referred to as fine adjusting screws or slow motion screws, but more commonly, as tangent screws, are provided, so that the line of sight may be accurately set on the object and clamped in position.

Various adjustments are provided, so that the essential geometrical relationship between the components of the theodolite may be maintained.

If these were not provided, the instrument would soon permanently lose the perfection of adjustment, made at the factory, through normal handling and usage. This would result in avoidable errors being introduced into the measurements made with it.

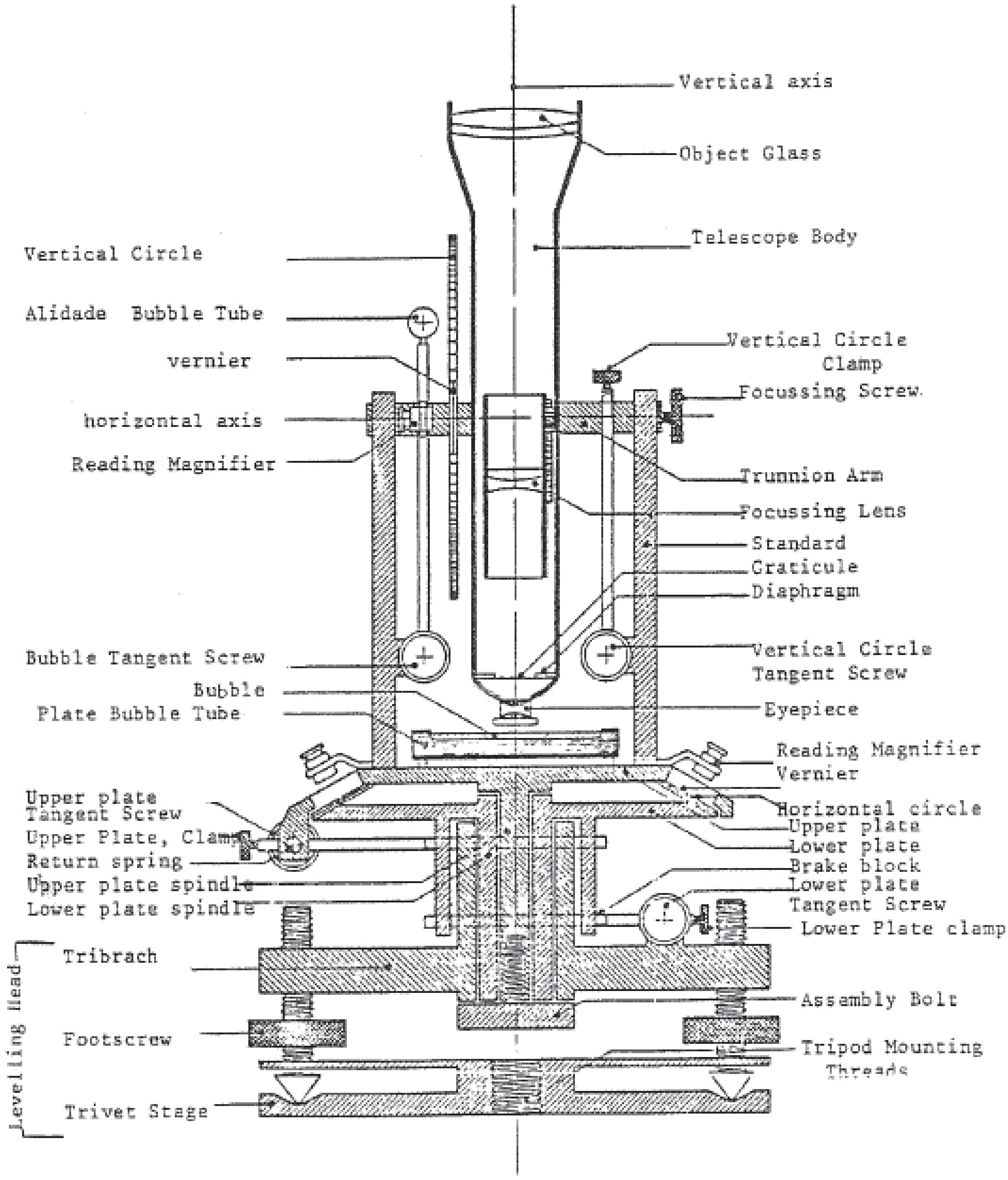


Figure 4.5 Diagrammatic section of simple vernier theodolite

Reading devices, either of the micrometer, vernier or optical scale type, allow both horizontal and vertical circles to be read with great precision. In many modern theodolites, optical devices are employed to make the reading more convenient and accurate.

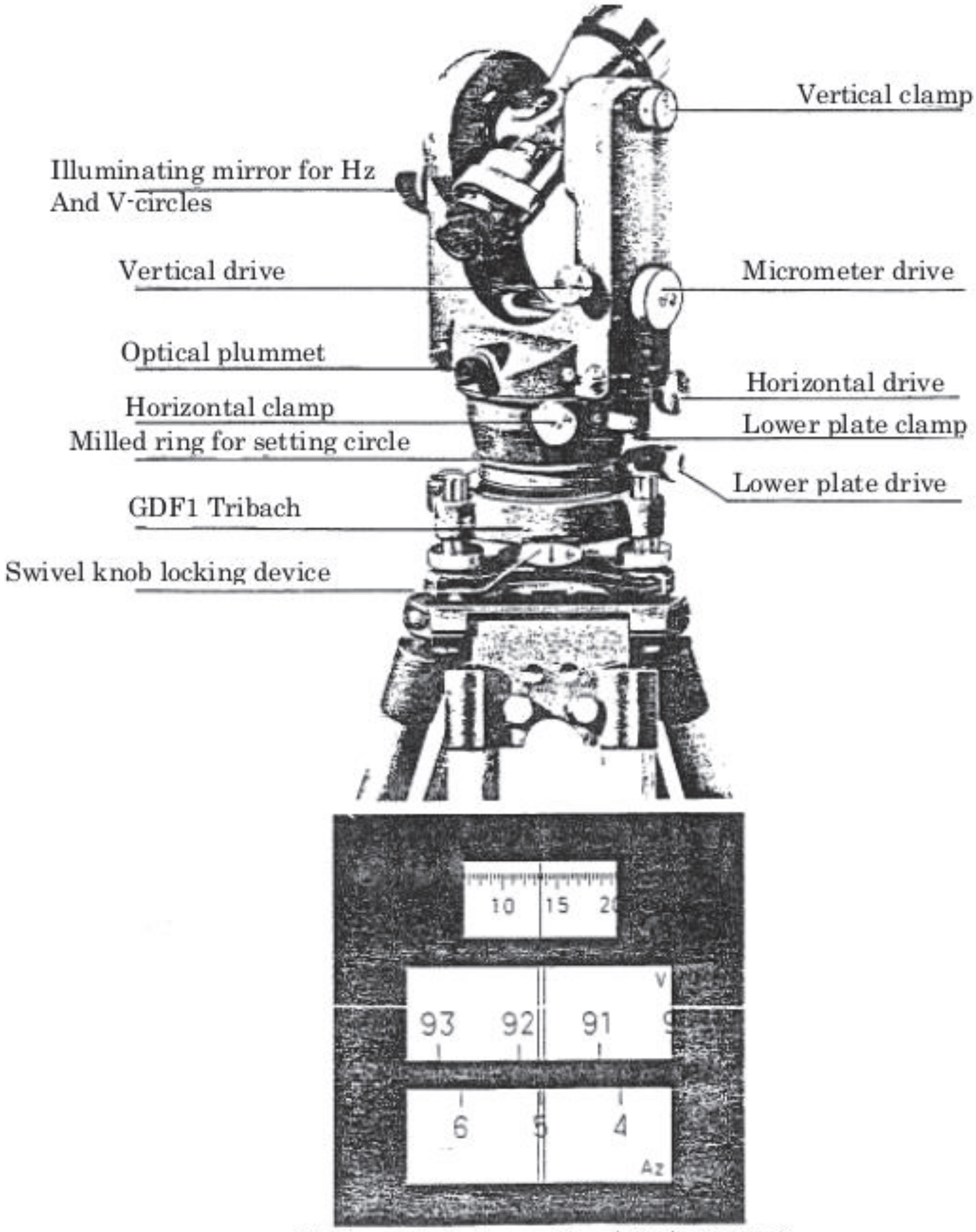
When in use, the instrument is firmly mounted on a tripod or, when observing from a beacon or the top of a wall, on a wall tripod or beacon plate, which is simply a tripod head mounted on three short metal pegs.

A few terms not mentioned in the above description may be of value.

- The *lower plate* is the mounting for the horizontal circle.
- The *upper plate* is the mounting for the reading device, and rotates horizontally with the telescope at all times.
- The *lower plate clamp* clamps the lower plate immovably, except for the movement allowed by the lower plate tangent screw.
- The *upper plate clamp* clamps the upper and lower plates rigidly together, except for the movement allowed by the upper plate tangent Screw.
- The *upper plate spindle* rotates within the lower plate spindle and these spindles have a common axis known as the vertical axis. In some theodolites, particularly the more precise models, the upper and lower plate bearings are entirely independent of each other.
- The *tribrach* is the mounting for the footscrews, whose lower ends rest in the trivet stage, and the whole leveling device is called the leveling head.
- The standards are the two vertical supports for the upper portions of the instrument. They are rigidly attached to the upper plate.
- Two *trunnion* arms support the telescope, their ends rotating in bearings fitted to the standards. The axis about which the telescope and vertical rotate is known as the horizontal axis or trunnion axis or transit axis.
- The *telescope clamp* or vertical circle clamp clamps the telescope and vertical circle immovably, except for the. movement allowed by the telescope tangent screw or vertical circle tangent screw.
- The *plate bubble* is attached to the upper plate, and its function isto control the verticality of the vertical axis.
- The *alidade bubble* is usually attached to the reading device for the vertical circle. Its purpose is to ensure that true vertical angles are read. It can be adjusted to the horizontal by means of the bubble setting screw or bubble tangent screw.
- The *telescope bubble* is attached to the telescope. It enables the telescope to be set truly horizontal. On most modern instruments it is not fitted.
- The *object glass* or objective is the large forward lens through which rays of light enter the telescope.
- The *eyepiece* is the rear end of the telescope, through which the observer views the sighted object. It consists of a series of lenses which provide the main magnification of the telescope and is capable of rotation, within a threaded sleeve, so that the cross hairs may be clearly focussed.
- A separate *focussing screw* enables the sighted object to be clearly focussed.
- The *diaphragm* is an adjustable ring just forward of the eyepiece, within which are fixed the cross hairs and stadia hairs. In earlier instruments. soider webs were glued directly onto the diaphragm ring as sighting marks, but, in modern instruments, fine lines are etched onto a thin glass disc which forms

part of the diaphragm. In some instruments this glass disc, or spider webs, are mounted in a separate cell, which may be readily withdrawn from the diaphragm ring and replaced. The glass disc with its arrangement of hairs or wires, as they are often called, is known as the graticule or reticule.

- The alidade is the whole upper part of the instrument and includes the upper plate, standards, verticle circle and telescope.



Horizontal circle reading (360°) 5°13'35"

Figure 4.6

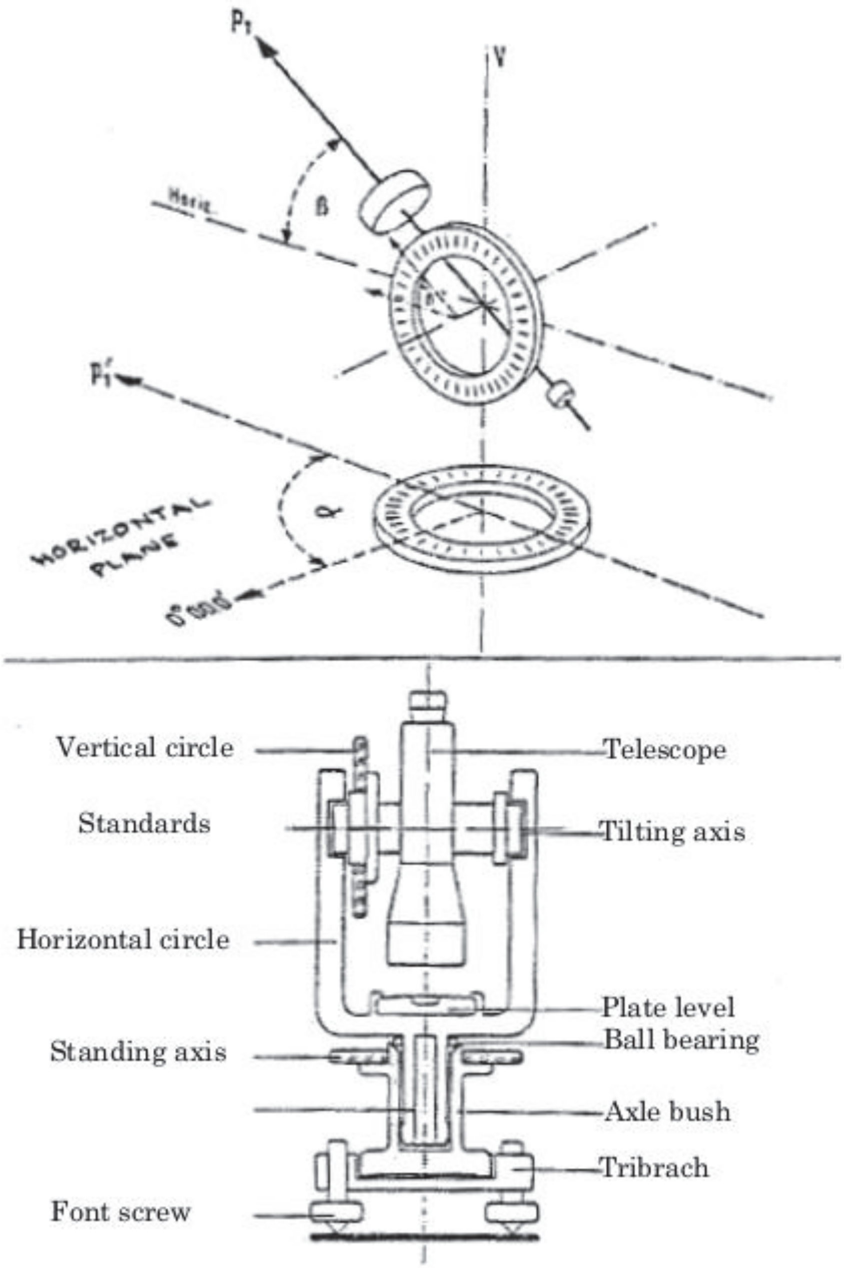


Figure 4.7 Essential features of a theodolite

4.4 Tacheometry

Tacheometry is that branch of survey work in which distances are measured directly from the instrument station, by measurement of the distance intercepted at the far point by a known angle at the instrument, or by measurement of the angle which intercepts a known distance at the far point.

The simultaneous measurement of the direction and vertical angle, allows both the horizontal and vertical positions of the far point to be fixed by purely optical means.

As the tedious methods of chaining and intersection, and the uncertainty of the range finder, are entirely eliminated, it is obvious that this is the ideal method of mapping areas within the limits of distance inside which plotting accuracy can be maintained.

In view of the fact that these distances are strictly limited, and that each point fixed must be visited by a member of the field party, its use is confined to large-scale mapping, the outside limit being a scale of about 1:10 000.

Recent developments in photogrammetry have brought aerial survey methods well within this range, and it can now safely be said that tacheometry is only an economical proposition at scales larger than 1:500, except in the case of strip surveys for road and railway location etc., where tacheometry may still be economical at smaller scales.

Where fairly large areas are concerned, photogrammetry is used at scales as large as 1:500, and may be regarded as having supplanted tacheometry almost entirely.

4.5 Theory of the tacheometer

A tacheometer is a theodolite fitted with two horizontal stadia hairs which are equidistant from the horizontal cross wire.

The instrument is used in conjunction with a graduated staff. A distance is obtained by multiplying the intercept made on the staff by these hairs, by a constant-usually 100. Height differences and directions are obtained from vertical and horizontal angles.

Tacheometry is divided into two main systems, each of which may be subdivided into subsidiary systems.

- The stadia system, in which the distance is measured by means of a single setting of the telescope.
- The Tangential or Subtense System, in which the distance is measured by means of two settings of the telescope.

4.6 The tangenital method

This method is based upon two paintings of the telescope, and may be employed either with a subtense bar or measured base for horizontal angular measurements or with a staff to which targets or marks are fixed, for vertical angle measurements.

The method is commonly known as the "Subtense Method" as it relies upon the accurate measurement of the subtense angle, i,e. the angle subtended, at the instrument station, by a base of known length.

The subtense bar (see **Figure 4.8a**) is a bar, usually two metres long, which is mounted on a tripod. Two targets, one at either end of the bar, are kept at a constant distance apart by an invar rod. The angle subtended by the targets is measured. As this is a short base, the angle must be measured with extreme care.

The object, again, is to find the horizontal distance from a known point: to the new point, as well as the elevation of the new point which is required.

The work may be classified under two headings.
1. Cases where the horizontal subtense angle(s) is (are) measured. Here, the subtense bar is used,
2. Where the vertical subtense angles are measured. Here, the staff is used.

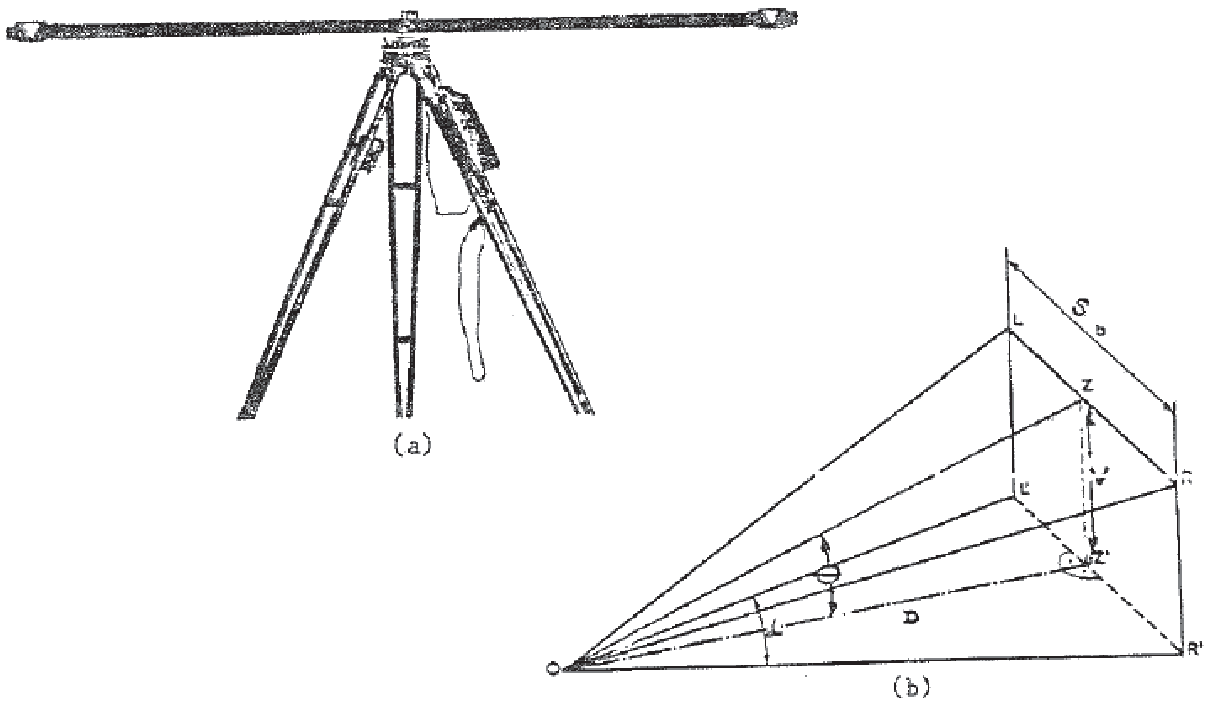


Figure 4.8

1. **Subtense bar with horizontal subtense angles**
   In **Figure 4.8b** LZR (=S) represents the subtense bar and telescope positioned at '0'; L' 0 R' represents an isoceles triangle on the horizontal plane through the trunnion axis of the telescope.

Let the horizontal angle L' 0 R' = $\propto$ °; and the vertical angle Z' 0 Z = θ°

Distance LZ = RZ = $\frac{LR}{2} = \frac{S}{2}$
Angle 0 Z' L' = 90° = angle 0 Z'R'
$\therefore$ Angle L'0Z' = angle R'0Z' = $\frac{\propto°}{2}$
$\therefore$ D = $\frac{S}{2}$ cot $\frac{\propto°}{2}$ …………………………….. (NB)

It is therefore clear, when S = 2 metres :

$D = \cot \frac{\propto°}{2}$ …………………………….. (NB)

The vertical distance from 0 to Z is represented by distance ZZ' (-V), which is a side in vertical right angled ZOZ', therefore:

$V = D \tan \theta$ …………………………. (NB)

2. Staff with vertical subtense angles

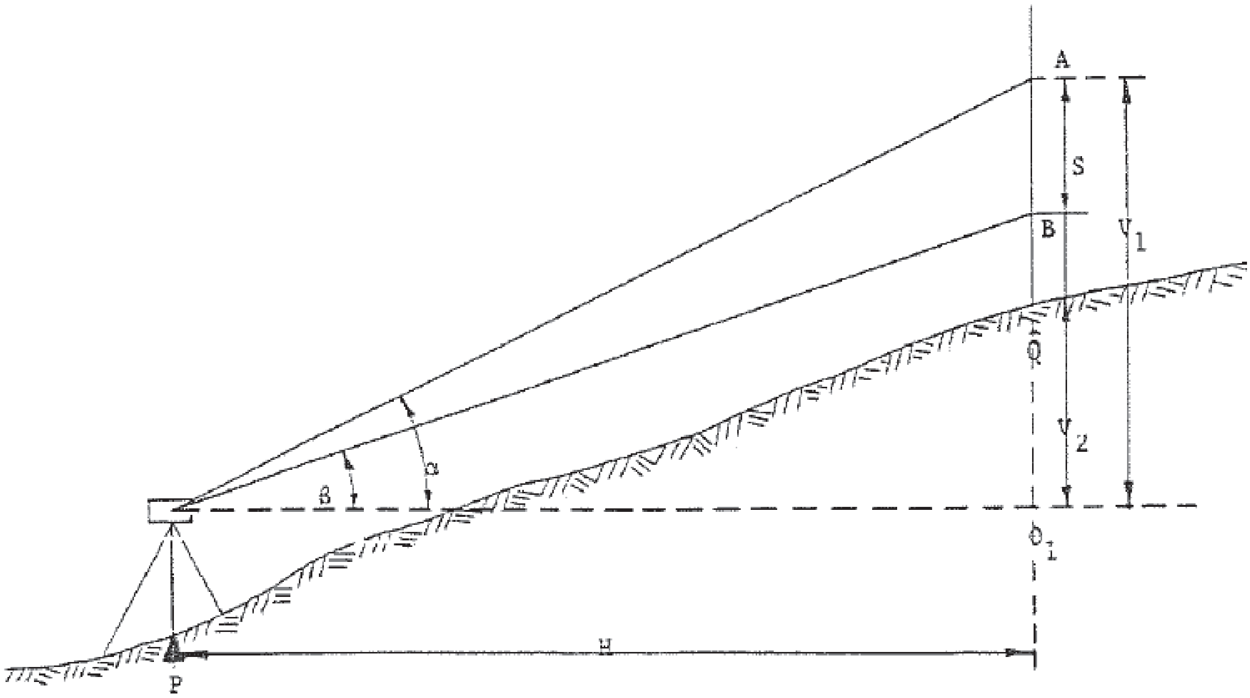


Figure 4.9

In figure 4.9, the upper reading is taken at A "" the staff and the lower reading taken at B on a staff held at Q. Let the difference between the upper and lower readings be 'S'. Let $\propto$ and β be the vertical angles observed from the instrument position at P.

The horizontal distance P - Q = H units.

V_1 and V_2 are the vertical distances from positions A and B on the staff, to the height of the instrument at P.

$V_1 - V_2 = S$

$\therefore V_1 = V_2 + S$ ………………………. (1)

$V_2 = H \tan \beta$ ………………………. (2)

Also $(V_2 + S) = H \tan \propto$

$\therefore V_2 = h \tan \propto - S$ …………………. (3)

$\therefore$ H tan $\propto$ - S = H tan β
$\therefore$ H tan $\propto$ - H tan β = S
$\therefore$ H (tan $\propto$ - tan β) = S
$\therefore$ H = $\frac{S}{(\tan \propto - \tan \beta)}$ .... (NB)

The above formula is used when $\propto$ and β are ***both*** angles of elevation or both angles of depression. In the case where β is an angle of depression and $\propto$ an angle of elevation the formula becomes:

H = $\frac{S}{(\tan \propto - \tan \beta)}$ .... (NB)

Having calculated the horizontal distance H, V_1 or V_2 may be calculated using equation (1) or (2).

Also, elevation Q. = elevation P + inst. ht. + V_1 - AQ.

Worked Example 4.1

Calculate the horizontal distance K-T, and the elevation of T, given the following data:

(a) Subtense bar is horizontal and 1,000 m above T.
(b) Bar length = 2 metres.
(c) Instrument height above K = 1,500 m.
(d) Horizontal readings on targets
  (i) 22.00.00
  (ii) 25.06.00
(e) Vertical angle K-T = +10.00.00
(f) Elevation of K = +2 133,50

Solution:

Horizontal distance, D $= \frac{S}{2}\cot\frac{\propto°}{2}$
$= \frac{2}{2}\cot\frac{(25°06' - 22°00')}{2}$
$= \cot\frac{03°06'}{2}$
$= \cot 01°33'$
$= 36{,}956\ metres$

Vertical length V $= \mathrm{D}\tan\theta$
$= 36{,}956\ tan\ 10°$

log 36,956 = 1,567 685 0
+ log tan 10° = 9,246 318 8
log V = 0,814 003 8
V = 6,516

Elevation K	= +2 133,500
Instr. Ht. at K	= + 1,500
Ht. of instr.	= +2 135,000
V	= + 6,516
Bar elevation	= +2 141,516
Bar height	= + 1,000
Elevation T	= +2 040, 516

Answer: Elevation T = +2 040,516

Horizontal distance K-T = 36,956

Worked Example 4.2

Reduce the following tacheometric observation:
Instrument height at K = 1,380 m
Readings on vertical staff at R.

POSITION	AXIAL READING	VERTICAL ANGLE
Upper mark	3,000	+ 09° 16’ 00”
Lower mark	1,000	+ 08° 02’ 00”

Table 4.1

Solution:

$$H = \frac{3{,}000 - 1{,}000}{\tan 09°16' - \tan 08°02'}$$
$$= \frac{2{,}000}{0{,}163\ 159\ o - 0{,}141\ 134\ 2}$$
$$= \frac{2{,}000}{0{,}022\ 025}$$

Using logs to simplify, H = 90,806

Let V_1 be the vertical distance from the instrument axis to the upper mark and V_2 be the vertical distance from the instrument axis to the lower mark.

Thus,

$$V_1 = H \tan \propto \qquad V_2 = H \tan \beta$$
$$= 90{,}806 \tan 09°16' \qquad = 90{,}806 \tan 08°02'$$
$$= 14{,}816 \qquad = 12{,}816$$

Check $V_1 - V_2 = S$
$\therefore 14{,}816 - 12{,}816 = 2{,}000 \rightarrow$

Elevation of K = +1 435,000

	Instr. height at K	= + 1,380
	Height of instr. at K	= +1 438,380
	V_1	= + 14,816
	Elev. of upper mark	= +1 451,196
	Axial reading	= - 3,000
Answer:	Elev. of R	= +1 448,196
	Horizontal distance K-R	= 90,806 m

4.7 Staves

A great variety of staves bas been produced, some good, some bad, and the choice is usually a matter of personal preference or availability.

For comparatively short distances, which are required to maximum accuracy, the telescopic leveling staff is undoubtedly I the most convenient, and should be used for taché surveys at scales up to about 1/1200.

For smaller scale surveys, however, distances are not required to the same degree of accuracy, and are often too great to be resolved successfully on the leveling staff with its small graduations.

In such cases, a special taché staff, with a clear white face and black graduations, is preferable. Better still, every second metre-length may have a clear yellow background.

The graduations are usually in the form of blocks of 0,1 m spaced 0,1 m apart, so that the graduations are alternately black and white. The blocks are often stepped to denote 0,05 m. This is a most useful refinement, as it assists estimation.

Graduations in the shape of diamonds, comb teeth etc, usually entail sloping sides, which make estimation difficult, and do not find general favour in South Africa.

The value of each metre-length should be repeated at intervals to facilitate reading at close distances or where portions of the staff are obscured by bush or other obstructions. The numbering of each alternate graduation also assists materially in this direction.

Usually, single-section rigid staves of 3 or 4 metres length are favoured. Transport difficulties may, however, dictate the use of a folding staff, but these are usually heavy and generally unsatisfactory.

The staff should be light, and of narrow section to minimize the effects of wind, but at the same time, rugged and rigid. To this end, very light sections are often reinforced by ribbing, which complicates transport, but can be the most satisfactory in certain.

As these staves are usually locally manufactured to a specified pattern, the user is able to design a staff best suited to his own needs, and some time spent on designing a really good staff, as regards both construction and graduation, will be amply repaid in added efficiency.

The staff must be held vertically, and the use of a staff level bracket as in leveling is recommended. Staff men should be trained to get the staff into a vertical position, and bold it there with only an occasional glance at the bubble, as it is most exasperating to the observer if the staff man fails to see his hand signals, due to watching the bubble too closely.

4.8 Reading stadia distances

Due to the fact that many instruments now employ erecting telescopes, while others still have an inverted image, some confusion may arise in the use of the words "upper" and “lower” in connection with the stadia hairs.

It should, therefore, be remembered that, in the following explanations, the upper hair is understood to be the upper-reading hair, and the lower hair is the lower-reading hair. Ie, in a telescope having an inverted image, the "upper hair" is seen in the lower half of the field of view, and vice versa.

The copy-book method of reading stadia or taché distances is to set the horizontal cross hair at a staff reading equal to the height of the instrument above the peg, and to book the readings of each stadia hair.

If the HI reading is obscured, the horizontal hair may be set to any full-metre reading, or to a reading a full number of metres above or below HI. This provides a check on the three staff readings and on the deduction of the distance, in the following manner:

1. Add together the upper and lower stadia readings. The total should equal double the reading of the centre hair. This checks the actual staff readings.

2. Find the difference between the upper stadia and centre hairs, and between the centre and lower stadia hairs, and add the two results together.

3. Find the difference between the upper and lower stadia readings. The result should be the same as that obtained in (2) above, and the two determinations of the distance check each other.

For reasons which will be clarified later, this laborious process is quite unnecessary for spot shots, but may well be adopted for taché traversing where the additional labour is justified.

For spot shots, the following simple routine is more practical and far less time-consuming.

Firstly, it must be remembered that the staff intercept must be multiplied by 100. The facility for doing this quite unconsciously is soon developed by regarding each metre on the staff as 100 metres, each 0,1 as 10 metres and each 0,01 as one metre.

The procedure is as follows:

1. Select a position on the staff such that the lower of the two stadia rays does not travel nearer to the ground than one metre, for the unequal refraction near the ground may cause distortion of this ray and give an incorrect reading. This is a counsel of perfection and is not, by any means, always possible. The reading position on the staff should also not be too high, as the upper portion of the staff is seldom steady, and accuracy is impossible.

   Under extreme conditions, especially in flat country it may be impossible to overcome the effect of unequal refraction, and distances may be shortened by as much as 1%. Under such circumstances, traverse distances should be taped.

2. Set the lower stadia hair, ie, the lower reading on the staff, onto a full metre division, say 1 metre.

3. Read the position of the upper, ie, higher reading hair, say 2,38, but mentally convert this to 238 as described above. Book the upper and lower hair readings.

4. Mentally subtract 100 from 238 to give metres. Book it in the "distance" column.

5. Read the horizontal cross hair, and book it. (This will be the axial reading.)

6. Signal staff man to proceed to the next point.

7. Read and book the vertical angle.

4.9 Stadia traversing

Stadia or Taché Traversing differs little from tape traversing, and the basic details must be followed, As distances cannot be measured very accurately, excessive refinement of directional observations would be a waste of time, but maximum accuracy, consistent with no wasted time, should be maintained.

The accuracy to which distances should be read depends upon the scale of' the final plan and the required traverse closures, Distances rood to the nearest metre are only acceptable for small scale work.

For scales between 1/2 500 and 1/5 000, distances might be read to the nearest half metre, while for scales between 1/1 000 and 1/2 500, an attempt should be made to estimate the nearest tenth of a metre. Distances will not necessarily be accurate to 0,1 metre, but results should be considerably improved.

For scales 1/1 000 and larger, stadia traversing is not suitable, and tape traversing should be employed.

The procedure at each instrument station is as follows:

1. The instrument is set up accurately over the peg and leveled, and the IH, is measured and booked in the appropriate column. The back (orienting) direction is booked in the horizontal angle (direction) column.

2. The instrument is oriented on a ranging rod held on the back station peg by the rear staff man. The staff man is signaled to exchange his rod for the staff. After orientation the direction is again read and checked against that already booked.

3. The distance is carefully read to the required accuracy. The stadia readings are booked in the appropriate column in the form $\frac{238,5}{100}$, and the deduced distance, 138,5 m, is booked in the "direct distance" column. This is actually the procedure which should be adopted in reading the distance to the front station.

   As we are, at present, checking the distance previously read from the back station to the present occupied station, a variation of procedure is desirable. This may be attained by setting the upper-reading hair an a full metre, say 3, and noting the reading of the other hair, say 161,5. The stadia readings are booked in the form $\frac{300}{161,5}$.

   In this case, 161,5 is deducted from 300 to give 138,5 m, which is booked as before.

4. The axial reading is next set and booked. In traversing this must always equal the IH, so that, in the calculations, these two factors are not taken into consideration, the line of sight being parallel to the line joining the tops of the two pegs.

In addition, a check on the reading of vertical angles is provided by the fact that the vertical angles from the two terminal pegs of each leg eg must be equal, but of opposite sign. If a different axial is used , the calculations are considerably more involved.

5. After ensuring that the altitude bubble is central, the vertical angle is read and booked and compared with that read from the previous station. If they do not agree with 20" – 40" a mistake has been made or the instrument has an index error.

   Index error and the effects of curvature on are cancelled out by using the mean of the angles read from either end of the leg, in the calculations.

   It is unnecessary to take face left and face right observations, as the observations from the two pegs have much the same effect.

6. The rod held by the front station staff man is now sighted, and the direction is read and booked.

7. Steps (3) to (5) are repeated for obtaining the distance and vertical angle to the front station, but the procedure outlined in the first part of step (3) is adopted for reading the distance.

4.10 Spot shots

The ability to take spot shots quickly and efficiently, using all the little time-saving methods which each surveyor must develop for himself, and economizing as much as possible in the number of spot shots, while, at the same time, omitting no necessary shots, is the hallmark of the good tacheometrist, and is developed only by years of experience.

The help of an assistant, known as a booker, who enters all the readings in the field book , greatly accelerates the work.

Wide experience by a great many surveyors has proved the following order of reading to be the most efficient.

1. Read the distance.
2. Set the axial.
3. Signal the staff man to proceed to the next position.
4. Book distance and axial.
5. Read and book the direction.
6. Read and book ~be vertical angle, making sure the altitude bubble is reasonably central.

⚠	**Note:** Some surveyors read both angles and then book them, but this is extremely dangerous. The altitude bubble must be kept sufficiently central to obviate vertical angle errors greater than 30°.

7. Book the spot description neatly and concisely, using standard abbreviations wherever possible.

Perhaps the greatest time-saver of all is the level spot shot. This is used when the difference in height between the instrument and the staff is such that the staff may be observed with the telescope set horizontal.

There being no vertical angle, the height of the point is derived by the collimation method. This saves a great deal of time, and with practice, the field work takes no longer.

The method of reading is as follows:
1. Set the telescope horizontal (90° 00') in most modern theodolites) if not already in this position from the previous shot.
2. Note the axial reading, and keep it in mind.
3. Read the distance, and keep it in mind.
4. Re-set the horizontal hair to the axial reading, so that the telescope is ready for the next shot.
5. Signal the staff man to proceed to the next position.
6. Book distance and axial.
7. Read the direction, and check that the vertical circle reading is 90° 00'. This entails only a glance.
8. Book the direction and vertical circle reading. The latter may be booked as 90,00 or merely L for "1evel".
   This step must not be omitted, since it prevents the formation of a habit which may lead to the omission of the vertical angle in ordinary spot shots.

With optical micrometer instruments, a great deal of time is saved, if the micrometer is set to zero, and left in this position.

The directional readings are easily estimated to the nearest 10 minutes, which gives ample accuracy. With a little trouble, most micrometer adjusting screws may be set to read exactly zero when the screw is turned to its stop position. This greatly facilitates setting, and is well worth while.

Some surveyors find the field work of this method more rapid than ordinary spot shots, and consider it worth-while for as few as two consecutive spot shots.

⚠	**Note:** When the staff cannot be held on the required spot, eg when locating a tree, it should be held next to the point of detail, so that the distance will be correct.

The line of sight is shifted to the correct position before the direction is read. If this is not possible, the staff is held on the correct line of sight, and the distance is adjusted by estimation.

4.11 Placing of spot shots

This is probably the most important aspect of tacheometry, and while a great deal may be learned from experience, it is a fact that some surveyors are more gifted in this respect than others, Good staff men make a tremendous contribution to the ease of work and the quality of the job.

As contour positions are interpolated between adjacent spot shots, the ground between the shots being regarded as evenly sloping, it is absolutely imperative that shots be taken at every change of slope, otherwise a completely misleading picture of the country may be produced.

If shots were taken haphazardly, a piece of ground might be shown as evenly sloping, whereas, in fact, it may have a gentle slope for some distance, and then drop sharply.

Tops of hills might be omitted altogether, and valleys shown shallower than they are, or omitted altogether, or changes of slope may be shifted and distorted. The rule, therefore, is that shots must be taken at the of hills, bottoms of valleys, and at all intermediate changes of slope.

This argument should be obvious, but it is surprisingly often disregarded, so that it cannot be stressed too strongly.

On evenly sloping or relatively level ground, a safe, if sometimes overdone rule, is to space spot shots so that they will plot approximately 25 mm apart on the plan, irrespective of scale, ie, for a scale of 1/5 000, they should be about 125 m apart, whereas for a scale of 1/500, they should be about 125 m apart.

The location of detail is a more involved process, requiring experience, common sense and patience, and cannot be reduced to a set of rules.

The first necessity is a clear appreciation of what detail is to be shown, and what is to be omitted. This is governed by the scale of the plan and the purpose for which it is required.

Obviously it would be a waste of time to pick up a bay window on a house for a scale of 1/5 000, but on a plan of 1/500, required for the planning of road widening in a city, it might be a significant feature, affecting the eventual position and cost of the road.

If, however, the plan is required purely for road works, and must simply indicate the positions of' adjacent houses, it would be a waste of time to show the bay window. This difficulty is ever present, particularly when working to large scales.

For city work, even a scale of 1/500 is often too small, and full detail can only be shown at scales of 1/250 or even 1/100. Taché distances are hardly accurate enough for the latter scale, and chaining or intersection should be resorted to for important points of detail.

A common mistake is the omission of necessary detail, particularly features flush with the ground, such as manholes, and the surveyor should train himself' to notice all detail, and his chainman to draw his attention to anything he has overlooked.

Perhaps the most common of' all mistakes, and one not necessarily corrected by experience, is the inability to make one spot shot perform several functions, with the result that far too many spot shots are taken.

The great majority of shots used for locating detail can also be used for contouring, but often they are not placed so as to serve both purposes. A fence may be picked up with a slavish adherence to the spacing of the shots, quite missing an important change of slope, so that at least one additional shot has to be taken, In this way the surveyor can waste a great deal of time and effort.

Much time can be saved by intelligent descriptions such as "Telephone pole 1 m W of edge road, 1 12m E of fence". Abbreviations would, of course, be used and it is seldom necessary to insert the directions so that the entry in the book would be, "TP 1m fr. ed. rd. 1,2m fr. F".

This spot shot then does the work of four. Cases have been observed, where four spot shots have been used to locate a single manhole on a large scale, where one shot and a remark such as "SW cr. MH 0,6 m square 1 m fr. and parallel to K", would help to locate the kerb and enable the manhole to be plotted quite satisfactorily.

Note:
It is often forgotten that the instrument station can be used for location of detail, and peg descriptions such as “1 m fr. ed. rd.”' are most useful.

Simple houses can be located by taking three corners, and taping the lengths of all the walls. A small sketch must be drawn. Where the detail is more complicated, one or two additional corners will be found useful.

Close detail may also sometimes best be located by having two spot shots serve as the terminals of a small chain survey, the detail being located by offsets.

It must also not be forgotten that a multitude of spot shots on the plan often leaves little room for showing the detail clearly, while providing an unnecessary abundance of height data.

One way of overcoming this, is to omit the height of the shot, only the distance and direction being taken in the field.

Note:
Remember to take the vertical angle for steep shots.

Often too, heights are scrapped in the office, if they are only then found to be unnecessary. Banks probably give more trouble than all other forms of detail put together, and it would be surprising to find exactly the same interpretation of a really complicated system of banks by two independent surveyors.

They are often broken and indeterminate, and largely a matter of individual interpretation, and although the two plans might differ considerably in appearance, they might be equally good, the main details having been shown correctly.

The points of difference are usually unimportant. One point which is often not appreciated, is that a bank shown by staggered shots is practically worthless,

The "top bank" and "foot bank" shots must be shown one directly above the other, so that the height of the bank at that particular point is accurately known. Where a system of banks is at all complicated, careful sketches should be made, showing all shots duly numbered.

The above are only a few ideas on which to build good habits, and each surveyor should strive to become an expert in placing his shots, He should also try to learn from others, by comparing methods, for every surveyor has his own pet ideas, many of' them very good.

4.12 Accuracy of reading

Economy of accuracy is another hallmark of the good surveyor.

In all branches of survey work it is a waste of time to make readings to an accuracy which cannot be used, or to attempt to take readings beyond the capabilities of the equipment being used.

The following absurdities have been noticed.

Reading taché distances to 0,1 m, for scales having a plotting error of a metre or more, reading angles to single seconds on a 20" micrometer theodolite, where the utmost accuracy of setting is 5" to 10", and reading directions for spot shots to single seconds.

These readings are quite solemnly booked and vigorously defended as being accurate and useful. They are neither, and simply indicate lack of experience and reasoning.

⚠	**Note:** In taché work the limits of accuracy required are dictated by the office work.

Directions are plotted by protractor, and except with specially manufactured polar plotters which are seldom used, cannot be set off to a greater accuracy than ten minutes of arc.
In any case, at the distances normally used, the displacement caused by an error of 10 minutes is not plottable. It would, therefore, be a waste of time to read directions for spot shots to a greater accuracy than the nearest 10 minutes, and this is easily estimated on the majority of instruments, thus saving the time taken up by reading verniers or micrometers.

The degree of refinement of distances is dictated by the scale of the plan. The limits of error should be such as to cause an error no greater than ¼ millimetre on the plan.

This would indicate accuracies such as the following examples:

For a scale of 1/5 000, accuracy to nearest 1,25 m.
For a scale of 1/2 000, accuracy to nearest 0,5 m.
For a scale of 1/500 1 accuracy to nearest 0,125 m.

This standard is, however, more idealistic than practical for tacheometry, and refers more specifically to plane tabling. For contouring shots and minor detail, the limits may safely be doubled, and the following examples are more realistic.

Scale 1/5 000, nearest 2 m, and important detail nearest 1 m.
Scale 1/2 000, nearest 1 m, and important detail nearest 0,5 m.
Scale 1/500, nearest 0,5 m, and important detail nearest 0,1 m.

The past case presents a problem, in that it is not possible to stipulate anything practical which really suits this very commonly employed scale. Half-metre accuracy is not really accurate enough, except for spot shots used only for contouring, and then only for comparatively gently sloping ground, while an accuracy of a tenth of a metre is not attainable with any degree of certainty.

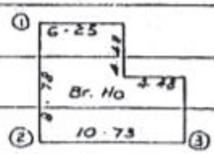
Anything falling between these two limits is not really practical. However, 0,2 m is about the all-round ideal, and it is recommended that distances be estimated to 0,1 m, on the principle that, in the majority of cases, they will be accurate to 0,2. For contouring spot shots, provided the ground is not too steep, half-metre accuracy could be used.

Vertical angles are normally read to the nearest minute. This is ample for calculation of heights to single decimals. Two decimals of a metre may sometimes be required, but it should be remembered that a quite unplottable movement of the staff will often make an appreciable difference to the height, so that the use of two decimals is misleading, except at the largest scales and on very definite points of detail.

Two decimals should, therefore, only be used in special cases, and then vertical angles could be read to 10" or 20".

DATE 19/1/70

Station From	Station To	Stadia	H.I. or Axial	Angles Hor.	Angles Vert.	Distances Direct	Distances Hor.		Height Diff.	Reduced Level Inst.	Reduced Level Point	Remarks
P12			1 58							144·8	143·26	W.P. 1 m fr. cd. rd 1·5 m fr F.
	P11	200/112·2	1·58	221·03·00	92·17·50	~~87·8~~	87·7					
	P13	185·3/100	1·58	73·08·20	86·20·00	~~85·3~~	85·0					
	R.O.			221·03 10								
	P 13		0·0		87 23 40	85·80 ~~85·83~~	85 71					Tape dist. (2 bays)
			1 2	70·00	90·00	18·5					143·6	K 2 m fr F.
			1·9	68·10	"	7·0					142·9	E.L.P. 0 3 m fr. K 2·2 m fr. F
			2 8	82·10	"	20·7					142·0	TP. bk of bn. 2 m fr. F
			3 0	94·00	95·09	~~20·1~~	19·9		-1·8	141·8	140·0	Ft " " "
			"	110·00	99·01	~~10·1~~	10·1		-1·6	"	140·2	Ft. bk.
			2 0	94·40	92·12	~~8·5~~	8·5		-0·3	142·8	142·5	Tp. "
			"	1·40	81·48	~~34·7~~	34·0		+4·9	"	147·7	Cr. Ho. ①
			"	7·30	83·32	~~26·2~~	25·9		+2·9	"	145·7	" " ②
			"	25·30	84·24	~~32·3~~	32·0		+3·1	"	145·9	" " ③
			"	47·00	82·39	~~51·2~~	50·4		+6·5	"	149·3	S.S.



Note: The distance P. 12 to P. 13 has been taped in catenary. This is booked on a separate line, with the relevant vertical angle and a note of the number of bays used. The sag correction is first deducted as shown, and then the distance is corrected for slope.

Figure 4.10

Axials are normally set or read to a single decimal, except in the special cases mentioned above, when two decimals would be used.

When traversing:
(a) The degree of accuracy required for distances is discussed later in this module.
(b) Directions and vertical angles are read to the maximum accuracy of the instrument, with the proviso that a greater accuracy than 10" is not warranted, and simply complicates the calculations.
(c) Axials are read or set to two decimals,

4.13 The field book and booking

The layout of the field book will probably always be cause for argument, as individual preferences vary considerably, and most surveyors will defend the layout to which they are accustomed.

Many different layouts have been designed, but most of them are thoroughly Bad. The specimen page shown in **Figure 4.10** is certainly convenient and efficient, and is probably as near ideal as possible. It is also very suitable for adaptation to many other types of survey work.

Booking should always be neat and clear, the figures being formed simply and unambiguously, line conventional figures, which can deteriorate into confusing or unreadable hieroglyphics should, under no circumstances, be used, but they are all too common.

Note:
It is good practice for the surveyor occasionally to examine his writing critically and to take any remedial action which may be necessary to ensure that all his figures are simple and clear.

When a booker is employed, he should be comfortably seated, if necessary on the ground, sufficiently close to the instrument, so that the surveyor does not have to shout. On the other hand, the surveyor must call out the readings in a firm clear voice, with careful enunciation.

Booking by the surveyor himself, may be considerably facilitated by grasping the top edge of the book, and pressing it firmly against the stomach. In this way, the book is firmly held for properly controlled writing.

Both hands should be used for manipulating the instrument, and the book should be slipped into a pocket or under the belt when not in use. Clasping the book under the left arm is fairly satisfactory, but does not leave the left hand entirely free.

Placing the book on the ground, and squatting down to book are inefficiencies only practiced by beginners.

The book should be kept clean and in good condition. A hard paper cover, which can be easily slipped on and off is a great help. The book should never be folded backwards, as this results in remarks being booked on the wrong line, and ruins the binding.

Think about it!
A dirty and dilapidated field book is usually a fairly good indication of slovenly and unreliable work.

The question of sketches raises the vexed question of whether plotting should be done by a draughtsman, or by the surveyor who bas done the field work.

A satisfactory plan can be produced by a good draughtsman, provided he is supplied with something more than mere sketches, but rather, preliminary field plans, showing the pegs and all spot shots taken for locating detail, plotted,' in the field, by protractor and scale, and the detail carefully filled in.

These are best done on separate A4 size sketch blocks of squared paper, All the relevant sketches are stapled into the back of each field book. Such sketches, of course, take up a great deal of the surveyor's time, whereas, if he is to plot the final plan himself, he will require only simple and purely indicatory sketches of the most complicated detail.

The greater part of his work will be done entirely without sketches for he has been over the ground in detail, and provided he has a good eye for country, retains a surprisingly complete mental picture of the detail and topography.

He is also personally acquainted with each spot shot, and can picture its situation, so that he is in a position to recognise an incorrectly taken spot shot.

As this is the only check on the majority of spot shots, it is a very material consideration. His contouring too, is more likely to interpret the character of the country correctly.

The conclusion is that the surveyor himself is in a position to produce a better plan, in less time (he spends less time referring to sketches), than the draughtsman.

Note:
The pencilled plan will, no doubt, be less pleasing to the eye, but not necessarily of inferior quality.

For a first class job, the inking-in must, of course, be done by a competent draughtsman. As salaries do not differ greatly, it is evident that the plotting of the plan by the surveyor himself is the more economical and satisfactory method.

Under camp conditions, however, satisfactory drawing is seldom practicable, and under these circumstances, it is better that plans be produced by draughtsmen, in properly equipped offices, and the lesser efficiency of this method be condoned.

The number, type and scope of sketches, therefore, depends upon the method of plotting the final plan, but whatever the method, the sketches should always be neat and clear, and the spot shots shown should be numbered for identification.

4.14 Procedure

This section will deal with the planning and organization of the job.

The first step is to determine the exact requirements, no surveyor should tolerate vague or incomplete instructions.

Instructions should include:

1. Locality and exact boundaries of job. Too many good plans are ruined by additions, which could have been foreseen in the original instructions, if a little more thought had been given to the matter. Similarly, vast sums of money are wasted on the unnecessary surveying of areas which cannot possibly affect the job in hand. More general appreciation of the cost and labour involved in even a small survey, would be welcome.

2. Purpose of the job. This the surveyor should know, so that he can exercise his own discretion where necessary.

3. Required scale of the final plan.

4. Limits of accuracy required.

5. Contour interval required.

6. Whether full detail is required, and if not, a clear statement, in writing, of the requirements.

7. Whether details of underground services, if any, are required.

8. If no triangulation and level data are available, whether time should be spent in carrying these into the area, or whether assumed data should be used. The latter should, however, never be used, unless it is quite unavoidable, except for very small surveys. Even in such cases, the use of the correct level datum is often essential.

9. Whether property boundaries are required or not.

10. The urgency of the job, which, of course, often dictates the quality of the work.

Instructions sometimes include details control available, cadastral (boundary) data and ground services. Where this information is not must, himself, institute the necessary research. Certain specialized instructions are also sometimes given.

When all the necessary information has been obtained, a careful reconnaissance of the area should be carried out. Some time spent in planning the conduct of the survey is amply repaid by added efficiency. A small scale plan of the area is most useful.

In the country, all farmers affected should be visited, and permission obtained to work on their land. Their help is often invaluable, and their co-operation should always be sought. For city work, it is better to obtain permission from property owners, as the work proceeds.

Trig control is located, and sites for additional points fixed, These should be so distributed that ample and rigid control for the traversing is provided.

Level bench marks are located, if any exist in the locality, and future lines of levels are planned.

Main and even subsidiary traverses are planned, and a through insight into the best possible approach to the job is obtained.

The next step is the building of permanent or semi-permanent marks for any control stations which may be necessary and the building of bench marks, if relatively long lines of levels are to be run.

Note:
If possible, the leveling programme should be planned to follow the main traverse lines, and the leveling carried out after these traverses have been run.

The basic requirement is to establish accurate levels at all control stations, so that these also serve the purpose of bench marks, or have a bench mark close by. This is, however, not always convenient or necessary, particularly if all traverse pegs are to be leveled.

Where taché levels are to be used this is most desirable, so that level control is available where required, If the leveling is carried out after the traverse pegs have been placed, these may be accurately leveled, with practically no additional labour.

It may be that the job is to be based upon trig levels. It is doubtful whether these would be sufficiently accurate to warrant a leveling programme, and taché levels could be relied upon for the purpose of the survey. As a rule, however, levels will later be required for construction purposes, and these should be accurate within themselves, irrespective of the accuracy of the datum, so that a leveling programme is normally carried out.

The observing programme for the establishment of control, is next carried out, and the co-ordinates of the control points calculated. At the same time, orienting directions for the control points are calculated.

The next step is again dictated by circumstances. If the plotting is to be done by the surveyor as the job proceeds, experience has proved that each traverse should be completed before the location of detail from it is commenced, so that office work may commence as soon as possible.

The reason for this is twofold. If a traverse has been completed, calculated and plotted, the plotting of detail may be carried out while it is still fresh in the surveyor's mind, and ample office work is provided for periods of inclement weather.

If, on the other band, plotting is to be carried out in some distant Head Office, or for some reason, commenced only after the field work has been completed, the traversing and location of detail should be done at the same time, as then the instrument is set up only once at each station.

In the former case, it is, of course, not necessary to run all the traverses initially. Plotting may be commenced as soon as the first traverse has been completed.

Note:
The position of each traverse peg merits careful thought, and it is only in the most straight forward country that this responsibility can safely be left to the front station staff man.

Pegs should be so placed that setting up the instrument is no problem, and if necessary, bush and high grass or other obstructions should be cleared from the site, so that movement around the instrument is unobstructed and safe.

Visibility between pegs, and of detail to be located, must also be considered most carefully. It often happens that a shift of a few ems could make a tremendous difference to the amount of detail visible.

Experience will very soon teach the surveyor that this consideration is of prime importance, The length of the traverse legs and the position of the next peg must also be borne in mind. It is, unfortunately, quite common to see pegs placed with little or no thought.

Wooden pegs are usually used, and these should be sturdy and durable. When working in termite-infested country, they should be treated with a repellent.

Leveling should be carried out immediately after the traverse has been completed, and before the location of detail, where these two operations are carried out separately.

Note:
Proficiency in this direction can only be attained by experience and observation of the methods of others.

The surveyor must ensure that all the required detail has been located, with sufficient spot shots, clarity of descriptions and sketches, to ensure that plotting can be carried out without confusion, and that shapes are not distorted. The entire area must also be covered with sufficient spot shots to ensure that the contours will be accurate and reliable.

When several surveyors are engaged on the same job, several of the processes may be done at the same time, and location of detail can begin virtually immediately. In such cases it is sometimes necessary to work on assumed directions until true directions become available.

4.15 Taché levels

The term "Taché Levels" is applied, rather loosely, to the levelling of traverse pegs, by means of distances and vertical angles, and does not necessarily refer only to those where the distances are obtained tacheometrically.

Whether taped or tacheometric distances are used the calculations vary only in the method of deducing the height differences.

For stadia distances, differences should be obtained from Jordan's Tacheometrical Tables, which are based on the stadia formulae. As two decimals are required, graphs and slide rules not sufficiently accurate, except for very small differences.

Distances derived by tangential tacheometry and horizontal taped distances are multiplied by the tan of the angle. Sloping taped distances are multiplied by the sin of the angle.

A fair degree of accuracy can be attained provided the field work is carefully done and the calculations are properly set out and adjusted. No attempt should be made to do these calculations in the field book, as this leads to confusion and inaccuracy.

Limits of error are rather fluid, depending largely upon the nature of the job, but an accuracy of 0,005 /Dist. in metres should be maintainable without trouble. A considerably higher degree of accuracy than this is usually attained with careful work.

4.16 Reducing spot shots

Many aids to reducing heights and horizontal distances from stadia distances have been devised. The best known of these are Cox's Stadia Computer and Jordan's Tacheometrical Tables.

The Cox's computer is quite satisfactory for most purposes but some people find it tiring to the eyes. Tacheometrical Tables are most useful. The pages give height differences and reduced distances for unity or 100 metres.

These factors, combined with a calculating machine give satisfactory results while a considerable amount of time may be saved.

Figure 4.11 is an extract from the tables. If, for example, the stadia distance ‘s' = 120, and the vertical angle + 11° 14’, then:

H = 120 x 0,962 1 = 115,45 m
V = 120 x 0,191 1 =   22,93 m

The above factors are underlined in the table.

The following sequence should be adopted when reducing the observations:

1. Write the peg height in the first line of the "Point" column, ie the last narrow column.

2. To the peg height, add the HI to determine the collimation level of the instrument, and enter this in the "Inst.” column, ie the penultimate narrow column.

3. For each spot shot, subtract the axial reading from the collimation height.

4. (a) For level shots, the result is entered in the "Point" column, as it is the final required height.
   (b) For normal shots, the result is entered in the “Inst.” column.

5. Apply the height difference to the figure in the "Inst.“ column, to obtain the final height, and enter this in the “Point” column.

α	10°		11°		12°		13°		14°	
	½ sin 2α	cos²α	½ sin 2α	cos²α	½ sin 2α	cos²α	½ sin 2α	cos²α	½ sin 2α	cos²α
0'	0,1710	0,9698	0,1873	0,9636	0,2034	0,9568	0,2192	0,9404	0,2347	0,9415
1'	0,1713	0,9697	0,1876	0,9635	0,2036	0,9567	0,2194	0,9403	0,2350	0,9413
2'	0,1716	0,9696	0,1878	0,9634	0,2030	0,9565	0,2197	0,9401	0,2352	0,9412
3'	0,1718	0,9695	0,1881	0,9633	0,2042	0,9564	0,2200	0,9400	0,2355	0,9411
4'	0,1721	0,9694	0,1884	0,9632	0,2044	0,9563	0,2202	0,9489	0,2358	0,9409
5'	0,1724	0,9693	0,1887	0,9630	0,2047	0,9562	0,2205	0,9488	0,2360	0,9408
6'	0,1726	0,9692	0,1889	0,9629	0,2050	0,9561	0,2203	0,9486	0,2363	0,9407
7'	0,1729	0,9691	0,1892	0,9628	0,2052	0,9559	0,2210	0,9485	0,2365	0,9405
8'	0,1732	0,9690	0,1895	0,9627	0,2055	0,9558	0,2213	0,9484	0,2368	0,9404
9'	0,1735	0,9689	0,1987	0,9626	0,2058	0,9557	0,2215	0,9482	0,2370	0,9402
10'	0,1737	0,9688	0,1900	0,9625	0,2060	0,9556	0,2218	0,9481	0,2373	0,9401
11'	0,1740	0,9687	0,1903	0,9624	0,2063	0,9555	0,2221	0,9480	0,2376	0,9400
12'	0,1743	0,9686	0,1905	0,9623	0,2066	0,9553	0,2223	0,9479	0,2378	0,9398
13'	0,1745	0,9685	0,1908	0,9622	0,2068	0,9552	0,2226	0,9477	0,2381	0,9397
14'	0,1748	0,9684	0,1911	0,9621	0,2071	0,9551	0,2228	0,9476	0,2383	0,9395
15'	0,1751	0,9683	0,1913	0,9619	0,2073	0,9550	0,2231	0,9475	0,2386	0,9394
16'	0,1754	0,9682	0,1916	0,9618	0,2076	0,9549	0,2234	0,9473	0,2388	0,9393
17'	0,1756	0,9681	0,1919	0,9617	0,2079	0,9547	0,2236	0,9472	0,2391	0,9391
18'	0,1750	0,9680	0,1921	0,9616	0,2081	0,9546	0,2239	0,9471	0,2393	0,9390
19'	0,1762	0,9679	0,1924	0,9615	0,2084	0,9545	0,2241	0,9469	0,2396	0,9389
20'	0,1765	0,9678	0,1927	0,9614	0,2087	0,9544	0,2244	0,9468	0,2399	0,9387
21'	0,1767	0,9677	0,1930	0,9613	0,2089	0,9543	0,2247	0,9467	0,2401	0,9386
22'	0,1770	0,9676	0,1932	0,9612	0,2092	0,9541	0,2249	0,9466	0,2404	0,9384
23'	0,1773	0,9675	0,1935	0,9610	0,2095	0,9540	0,2252	0,9464	0,2406	0,9383
24'	0,1776	0,9674	0,1938	0,9609	0,2097	0,9539	0,2254	0,9463	0,2409	0,9382
25'	0,1778	0,9673	0,1940	0,9608	0,2100	0,9538	0,2257	0,9462	0,2411	0,9380
26'	0,1781	0,9672	0,1943	0,9607	0,2103	0,9536	0,2260	0,9460	0,2414	0,9379
27'	0,1784	0,9671	0,1946	0,9606	0,2105	0,9535	0,2262	0,9459	0,2416	0,9377
28'	0,1786	0,9670	0,1948	0,9605	0,2108	0,9534	0,2265	0,9458	0,2419	0,9376
29'	0,1789	0,9669	0,1951	0,9604	0,2110	0,9533	0,2267	0,9456	0,2422	0,9375
30'	0,1792	0,9668	0,1954	0,9603	0,2113	0,9532	0,2270	0,9455	0,2424	0,9373
31'	0,1795	0,9667	0,1956	0,9601	0,2116	0,9530	0,2273	0,9454	0,2427	0,9372
32'	0,1797	0,9666	0,1959	0,9600	0,2118	0,9529	0,2275	0,9452	0,2429	0,9370
33'	0,1820	0,9665	0,1962	0,9599	0,2121	0,9528	0,2278	0,9451	0,2432	0,9369
34'	0,1803	0,9664	0,1964	0,9598	0,2124	0,9527	0,2280	0,9450	0,2434	0,9367
35'	0,1805	0,9663	0,1967	0,9597	0,2126	0,9525	0,2283	0,9448	0,2437	0,9366
36'	0,1808	0,9662	0,1970	0,9596	0,2129	0,9524	0,2285	0,9447	0,2439	0,9365
37'	0,1811	0,9661	0,1972	0,9595	0,2132	0,9523	0,2288	0,9446	0,2442	0,9363
38'	0,1814	0,9660	0,1975	0,9593	0,2134	0,9522	0,2291	0,9444	0,2444	0,9362
39'	0,1816	0,9658	0,1978	0,9592	0,2137	0,9520	0,2293	0,9443	0,2447	0,9360
40'	0,1819	0,9657	0,1980	0,9591	0,2139	0,9519	0,2296	0,9442	0,2449	0,9359
41'	0,1822	0,9656	0,1983	0,9590	0,2142	0,9518	0,2298	0,9440	0,2452	0,9357
42'	0,1824	0,9655	0,1986	0,9589	0,2145	0,9517	0,2301	0,9439	0,2455	0,9356
43'	0,1827	0,9654	0,1988	0,9588	0,2147	0,9515	0,2304	0,9438	0,2457	0,9355
44'	0,1830	0,9653	0,1991	0,9586	0,2150	0,9514	0,2306	0,9436	0,2460	0,9353
45'	0,1833	0,9652	0,1994	0,9585	0,2153	0,9513	0,2309	0,9435	0,2462	0,9352
46'	0,1835	0,9651	0,1996	0,9584	0,2155	0,9512	0,2311	0,9434	0,2465	0,9350
47'	0,1838	0,9650	0,1999	0,9583	0,2158	0,9510	0,2314	0,9432	0,2467	0,9349
48'	0,1841	0,9649	0,2002	0,9582	0,2160	0,9509	0,2316	0,9431	0,2470	0,9347
49'	0,1843	0,9648	0,2004	0,9581	0,2163	0,9508	0,2319	0,9430	0,2472	0,9346
50'	0,1846	0,9647	0,2007	0,9579	0,2166	0,9507	0,2322	0,9428	0,2475	0,9345
51'	0,1849	0,9646	0,2010	0,9578	0,2168	0,9505	0,2324	0,9427	0,2477	0,9343
52'	0,1851	0,9645	0,2012	0,9577	0,2171	0,9504	0,2327	0,9426	0,2480	0,9342
53'	0,1854	0,9644	0,2015	0,9576	0,2174	0,9503	0,2330	0,9424	0,2482	0,9340
54'	0,1857	0,8642	0,2018	0,9575	0,2176	0,9502	0,2332	0,9423	0,2485	0,9339
55'	0,1860	0,9641	0,2020	0,9574	0,2179	0,9500	0,2335	0,9422	0,2587	0,9337
56'	0,1862	0,9640	0,2023	0,9572	0,2181	0,9499	0,2337	0,9420	0,2490	0,9336
57'	0,1865	0,9639	0,2026	0,9571	0,2184	0,9498	0,2340	0,9419	0,2492	0,9334
58'	0,1868	0,9638	0,2028	0,9570	0,2187	0,9497	0,2342	0,9417	0,2495	0,9333
59'	0,1870	0,9637	0,2031	0,9569	0,2189	0,9495	0,2345	0,9416	0,2497	0,9332

Figure 4.11

4.17 Plotting the plan

The introduction of dimensionally stable transparent drawing foil bas made the use of paper, for plotting plans, practically obsolete.

This material, of which several types are available, is virtually free of distortion, and its use obviates tracing or photographic reproduction, as printing may be done directly from the finished plan.

Where paper is used, the most generally satisfactory is good quality linen-mounted drawing paper. Unmounted paper is fairly satisfactory, but it is less durable, and is likely to suffer from much handling.

Cartridge paper should, if possible, not be used as pencil marks are easily engraved into the paper and erasure of ink work almost invariably damages the surface. The use of the work "paper" in the following pages also covers synthetic drawing foil where applicable.

Note:
For work of the highest quality, where freedom from paper distortion is required, special sheets of high quality paper, mounted on either side of sheets of aluminium, are sometimes used.

These aluminium-mounted sheets are, however, too expensive for general work and not always convenient, as the size is limited, and transportation and filing can be a problem.

Flat sheets are more convenient for working than rolls, but for large jobs, roll plans are usually unavoidable.

If possible, the job should be plotted on one piece as this is more convenient for those who have to use the plan later, and the labour of matching is avoided.

Paper is available in the following international (ISO) sizes:

A0 - 841 x 1 189 mm
A1 - 594 x 841 mm
A2 - 420 x 594 mm
A3 - 297 x 420 mm
A4 - 210 x 297 mm

Note:
Each paper size is half the area of the size preceding it, and its length is the width of the next larger size.

The most generally useful width for roll plans is 841 mm.

Very large jobs can often not be plotted as a single plan, and in this case, the area should be divided into convenient sized rectangles, each of which is plotted on a flat sheet.

The boundaries of the sheets should be even-numbered grid lines. A satisfactory system of numbering the sheets must be devised, and a key plan prepared and kept.

If the job is to be plotted on one sheet, the first consideration is to position the grid in such a way that the work will be neatly positioned on the paper, with reasonable margins and slightly below centre.

Note:
If possible, the direction of North should be towards the top or right hand side of the paper, but this is not essential and it is often necessary to have the grid lines running at an angle to the edge of the paper.

The most satisfactory method is to plot from co-ordinates, a sufficient number of points, on or near the perimeter of the job, on a piece of squared paper, and to a scale convenient to the squares, so that the perimeter may be sketched fairly accurately.

The position of the paper to be used, in relation to the work, can be drawn, and the size required can be scaled, The angle of the grid to the paper, and the approximate position of one long grid line, together with the position of one grid intersection upon the chosen line, are then easily transferred to the drawing paper as a base from which the rest of the grid can be plotted.

If the grid is drawn without careful consideration, the work will, almost certainly, run off the paper or, at best, be poorly positioned on the paper.

A grid drawn from a fixed base line is best constructed with the help of a beam compass if no template is available.

The plan will require a suitable heading and details of scale, height datum, number, etc, These are usually added after the rest of the plan has been completed, but should be kept in mind, as the space they are to occupy should be left free of inked grid lines.

4.18 Plotting spot shots

The traverse pegs are carefully plotted from co-ordinates, and the plotting is checked by scaling between successive points.

The pegs are sometimes joined by fine lines to indicate the traverses, but this serves no very useful purpose, and particularly in areas of close detail, tends to make the plan more confusing.

The spot shots are plotted by protractor and scale, but where there are many, something more convenient than an ordinary protractor is required.

Probably the easiest and most: rapid plotting is by means of a special plotting protractor, made of celluloid or stiff drawing paper, a few pieces o!' which may be laminated together for rigidity.

An opening, not quite extending to the perimeter, is cut out carefully along the zero line, and this line is graduated with the scale to be used, with the centre as zero.

The protractor is graduated in reverse, ie, anti-clockwise. A very fine needle, which may be attached to the centre of the protractor or inserted through a very fine hole in the centre, is used to centre the protractor over the plotted position of the peg, so that it is free to rotate.

A line through the peg, parallel to the X axis in the zero direction, usually Southwards, is used as a reference mark on which to set the direction.

The zero of the protractor, and hence the scale, now point in the required direction, and the distance is marked off on the scale.

Note:
The main disadvantage of this type is that it smears any dirt which may get under it onto the plan, and that it does not lend itself to filling in detail as the plotting proceeds. It is quite unsuitable for use on transparent drawing foil.

A more satisfactory, but less rapid instrument, consists of a very large protractor, usually about ½ m diameter, graduated in the conventional manner, on a large piece of drawing paper. The figures are written outside the graduations, and the centre is cut out up to the graduations, leaving a large circular opening.

This is centred by matching the four quadrant directions with lines through the peg position, and parallel to both grid lines. An ordinary boxwood scale is used for plotting, and this may be anchored over the peg by means of a thin metal or celluloid plate glued to the scale and having a fine hole at the zero mark, or the scale may be used unattached.

In the latter case, a very fine nick at the zero helps considerably in setting the scale against a fine needle at the peg. The word “fine” is stressed, as anything but the very finest needle will make unsightly holes in the plan.

Manufactured versions of the above types of protractor, and variations, are obtainable.

If assumed directions have been used, the reference marks are shifted by the amount of the correction required, and the spot shots are plotted from the observed directions.

> **Note:**
> Great care must be taken that the shift is in the correct direction.

Plotting should be carried out with a fine pencil, and the spot marked by a dot. The height of the point is written fairly lightly so as not to engrave the paper, but also to be legible and not easily obliterate. The best grade of pencil is about H or 2H. The dot should be used as the decimal point.

When two shots plot in the same position, as often happens with vertical banks and retaining walls, the two heights may be written $\begin{array}{ccc}78 & & 9\\ & \cdot & \\ 83 & & 4\end{array}$ so the spot is common to both.

The relative positions of the figures indicate which side is 'the higher ground. The figures should never be written in the form of a cross.

Some surveyors number all spot shots, and enter these numbers on the plan. This should be quite unnecessary, and causes confusion to the draughtsman unless the numbers are properly erased.

4.19 Filling in detail

This may be done as plotting proceeds, or after all the shots from a station have been plotted according to preference.

4.20 Contouring

Contours are lines of equal elevation, and are probably best understood by imagining a piece of country surrounded by still water, at a given elevation of say 120 metres.

The edge of the water will coincide with the 120 metre contour. It will follow all the folds of the country, reaching far into valleys, and being divided by bills and ridges. High points, with lower ground all around, will be completely surrounded.

If the water now rises to 130 m, the edge will represent the 130 m contour. A larger area will be covered, the water reaching farther into the valleys, and the portions of the hills above it will be smaller. More islands "'ill be formed, while some disappear.

On steep ground, the distance between the previous and the new edges will be small, but on gently sloping ground it will be greater, so that the contours are close together on steep ground and farther apart on gentle slopes. The steepness of the ground can thus be judged from the distance apart of the contours.

In practice, contours are often broken where they cross or coincide with features of detail such as banks, houses, roads, etc.

This is, however, only done to avoid confusion, and any contour entering a detail feature, must again emerge from it, unless carried off the job within the detail feature.

Contours can thus only end at the edge of the job. This rule is sometimes altered, where contours of closer vertical interval than used for the bulk of the area, are shown over comparatively level areas where the normal interval would be inadequate.

Where four additional one metre contours are seen beginning at the widening of the space between two five metre contours, and ending at the narrowing of the space, it does not mean that they actually begin and end where shown, but that they are required only where shown.

Correctly drawn contours will give an indication of the character of the country by the way they are drawn. In rugged, broken country the contours will be angular, and the spacing uneven, while an area of rolling hills will be shown by sweeping curves and even spacing.

At the large scales used for tacheometry, however, this distinction is not so noticeable, and, while it may be employed effectively, it is more usual to show even spacing and flowing curves.

Contours having a more rugged appearance are more often the result of poor workmanship than of deliberate intent, and are almost invariably judged as such. If, therefore, the tacheometrist does not wish to create a bad impression of his work, he should not attempt to draw rugged looking contour.

Note:
The ground between spot shots is presumed to slope evenly, and the positions of the contours are determined by interpolation.

The first step, therefore, is to carry out all the interpolations in a given area, 'the size of which is governed largely by inclination.

The cardinal rule is never to interpolate past a spot shot, ie do not interpolate along a line between two shots which passes close to a third shot, as a change of slope will probably be missed.

The spaces to be interpolated are largely a matter of common sense, Interpolation by eye may be quick and easy, but leads to shoddy work, and the surveyor should never indulge in this easy way out.

The most accurate, but slowest method, is to measure the distance between spot shots, and to calculate the positions of the contours, but this is extremely tedious, and some mechanical aid should be used.

A scale and set square may be used to set off the proportional parts. A fine line is drawn between spot shots A and B (see **Figure 4.12**), and the scale (line BC) is set against one shot (B, in **Figure 4.12**) at a reading representative of its height.

ABC may be any convenient angle. Along BC mark off the required contour values (1 metre intervals in this case) as well as the height of A, at a point D using any convenient scale. Join AD and draw parallels through the other points on line BC and mark points e, f and g on line AB. The heights of e, f and g are J7, J8 and J9 respectively.

In **Figure 4.13** the elevations of K and Q. are 179,25 and 176,50 respectively.

Suppose contour lines at 1 m intervals are required. Draw line KR to form any convenient angle with KQ. Using any convenient scale allow for 0,25 m by marking off ¼ unit from K, then mark off two full units, then a ½ unit to allow for the 0,5 fraction of the elevation of Q. Join PQ and draw parallels to give positions t, n and m with elevations 179, 178 and 177 respectively.

A convenient method of interpolation is to use a strip of graph paper which will take up the positions of the random lines BC and KR in **Figures 4.12** and **4.13**.

The set square is now positioned over the graph strip at a reading representative of the elevation of the end spot shot (A and Q), after the graph strip was positioned at a reading representative of the elevation of the other spot shots (B and K).

The set square is now easily shifted over the graph paper into the required positions. This method has the advantage that the plan remains clean in that no unnecessary lines are drawn on it, and it also obviates confusion.

A series of numbered equidistant parallel lines, on tracing paper or cloth, may be set to such an angle chat the spot shots fall under their respective values.

Contour positions are then pricked through.

Another method, making use of radials and parallels, can be used, but is not recommended.

Methods which rely on pricking through, and the marking of interpolations by dots, are not recommended, as inerasable marks are made on the plan.

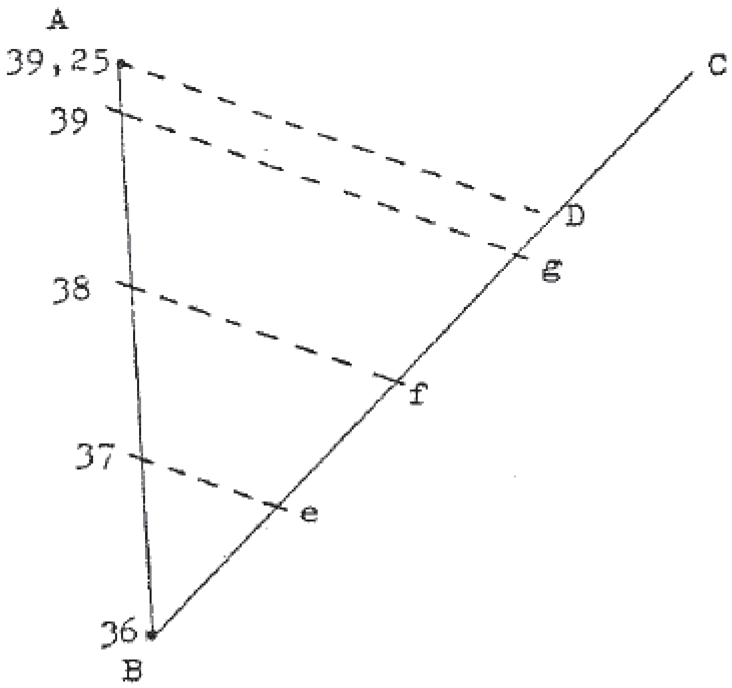


Figure 4.12

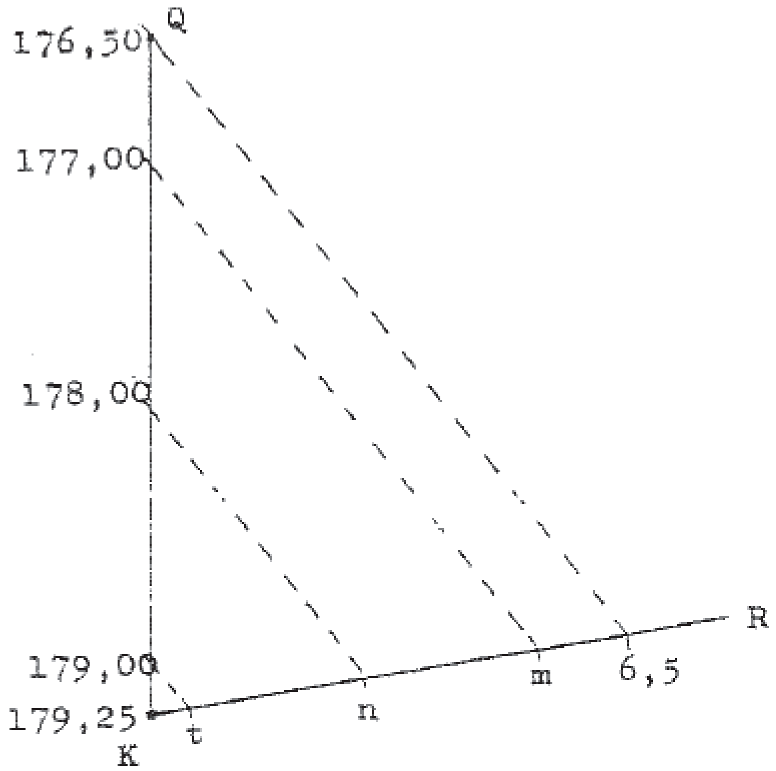


Figure 4.13

Having completed interpolations for the chosen area, one contour value is selected, and the marks representing this value are joined by a smoothly flowing line. All other contour values are treated similarly.

The drawing of the contours may be stated in a few words as above, but it is by no means a simple matter, and requires a great deal of practice.

A sharp pencil and a soft eraser are the essential tools. No attempt should be made to draw the contour with one continuous motion, as this results in wavering lines and incorrect curves.

The line should rather be drawn with short, very light, "painting" strokes, which can be controlled far better. It is often helpful to draw all the contours roughly but lightly, and to neaten them later.

Contours must not be drawn with railway or French curves, or other mechanical aids, as this gives them an artificial appearance.

A few hints and common pitfalls may be of help.

1. Normally it is not permissible to interpolate past a spot shot. By this is meant that the line joining two spot shots must not pass a third shot. The reason for this is that the third spot shot is, very likely, placed in that position to show a

change of slope, and if its influence is ignored, the ground will be shown as evenly sloping.

It is often difficult to decide whether the line "passes" a third spot shot, but the following simple test will usually give a reliable indication.

In certain cases it is permissible to interpolate past a spot shot. If it is known that the ground slopes evenly between two spot shots, as along the edge of a stream, interpolation should be carried out, even though the line joining the two shots passes close to another, a little distance away from the stream.

2. Where a stream, or other evenly sloping detail, curves between spot shots, interpolat1on must be carried out along the detail feature. It is usually sufficient to interpolate along the line between the shots, and then to move the interpolation marks across, onto the detail feature.

3. The apex of each curve should fall on an interpolation mark. Very often a contour will be drawn as a series of curves between the interpolations, when it should have been virtually straight or one sweeping curve. This is possibly the commonest cause of incorrectly drawn contours.

4. A contour cannot change its value, but very often is made to do so, and not only beginners are guilty of this mistake.

5. Very often the spacing of two contours which lie on either side of a spot shot, is either narrower or wider than that of the contours on either side of them. This would indicate a sudden and localised steepening or flattening of the ground, of which the spot shots give no evidence.

6. Short sections of contours often curve in the reverse direction to that in which they should.

If tackled in the correct frame of mind, contouring can be intensely fascinating and in the nature of an art.

4.21 Matching

When a survey has been produced on more than one sheet, it is essential to match all the adjoining sheet edges, to ensure that all detail, contours and boundaries meet exactly if the sheet edges are placed side by side, and that the lines flow smoothly from one sheet to the next, without unnatural bends at the sheet edges.

This is best done by completing one sheet up to its edge or, better, slightly over the edge. The detail is then carefully traced onto a strip of tracing paper, together with the boundary grid line and all intersections. This strip is then carefully positioned on the next sheet, and the detail is joined up on the tracing paper. It is then transferred to the sheet by means of pencil carbon paper.

It is as well to check the matching, by preparing a fresh strip, but this time, the edge of the tracing paper should coincide with the sheet edge. This is then positioned outside the other sheet, but just touching its edge, so that any small final adjustments may be made directly on the sheet.

When detail is inked before the contours are drawn, it will be necessary to carry out the above process twice, once before each inking.

Matching should again be checked after inking.

Activity 4.1

1. Sketch a neat cross-section of a theodolite showing and labeling the main components.
2. Name and describe the component of a telescope that holds the crosshairs in place.
3. What are the functions of a telescope?

Activity 4.2

1. Name two main systems of tacheometry.
2. Reduce the following tacheometric observations (ie, horizontal distance BC and elevation B):
   (a) subtense bar is set up horizontally and 1,250 metres above C;
   (b) bar length = 2 metres;
   (c) instrument height above B = 1 ,525 m;
   (d) horizontal subtense angle= 03°12'00";
   (e) vertical angle B to C = + 03 °22 1 00";
   (f) elevation of- C = + 1 450,000.
   A neat, clear sketch should accompany your answer.
3. Reduce the following tacheometric observations:
   Instrument height at K = 1,530.
   Readings on staff at M.

POSITION	AXIAL READING	VERTICAL ANGLE
Upper mark	3,000	+ 09° 44’ 00”
Lower mark	1,500	+ 06° 24’ 00”

Tabble 4.2

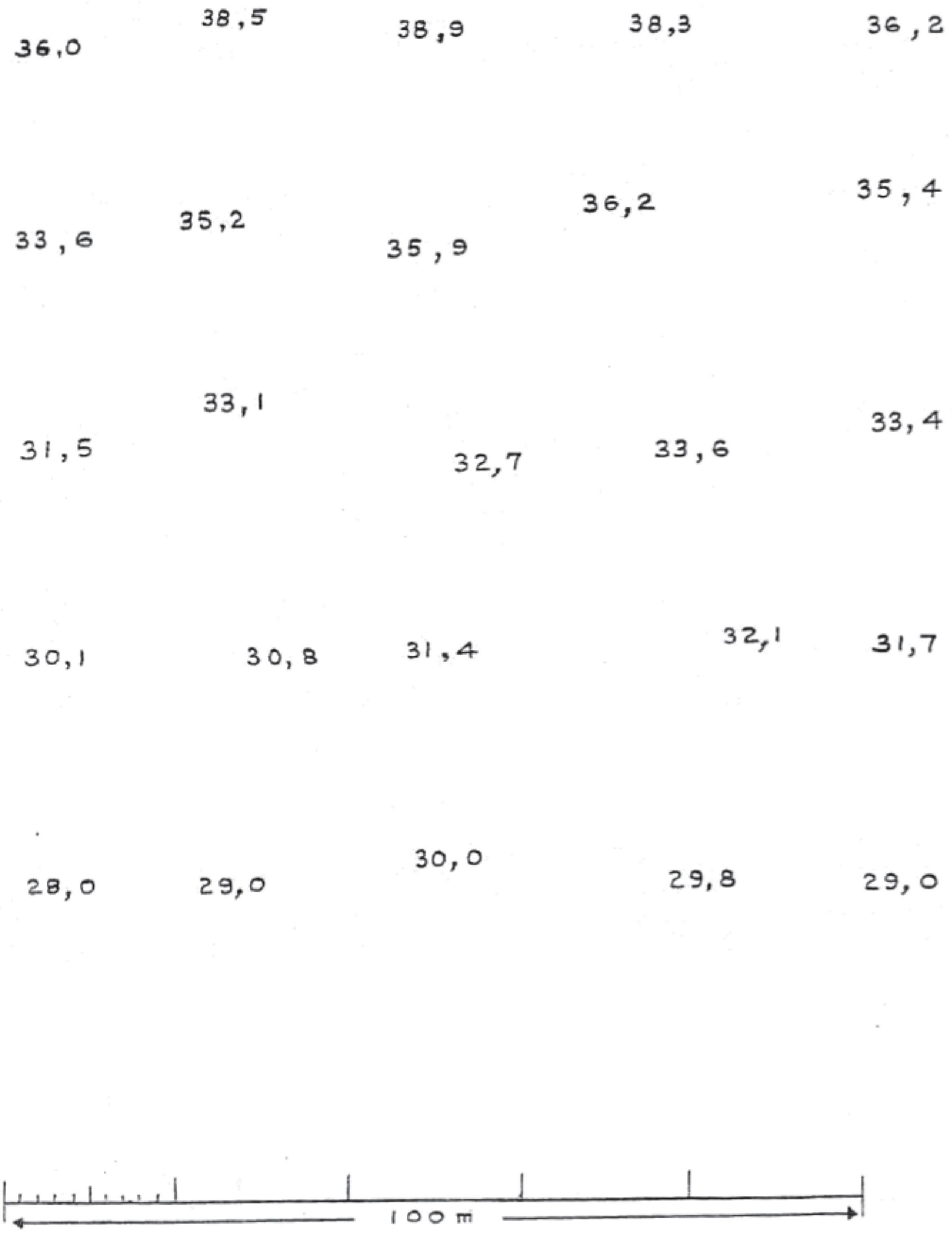
Elevation of K = + 1 690,750
+ 1 500,500

Activity 4.3

1. Draw a specimen field book page and on this page, enter the following observations and reductions:
   Set-up at A IH = 1,475
   To B. Top hair = 3,500
   Lower hair = 0,700
   Axial reading = 1,500
   Vertical angle A-B = + 12° 23' 00''
   Elevation of A = + 361,20
   Multiplying constant = 100
2. Write short notes on the use of the field book.
3. Write down the sequence of steps that should be used for reducing spot shot observations.

Activity 4.4

1. On the attached plan draw in contour lines at 1 metre intervals.



Self-Check		
I am able to:	**Yes**	**No**
• Describe the basic construction of the theodolite.		
• Describe the various types of theodolite.		
• Describe the use of the theodolite to measure horizontal and vertical angles.		
• Demonstrate recording the readings form the theodolite.		
o Explain tacheometry, calculation of horizontal and vertical components.		
If you have answered 'no' to any of the outcomes listed above, then speak to your facilitator for guidance and further development.		

Module 5

Building surveys

Learning Outcomes
On the completion of this module the student must be able to:

- Demonstrate surveying a small building.
- Describe instruments used for taking internal and external dimensions.
- Demonstrate measuring and recording a building and its site.
- Demonstrate running internal and external measurements taken horizontally and vertically.
- Demonstrate a plotting survey from field measurements.
- Do the following calculations:
  - Cut and fill eg for a road,
  - Plotting vertical sections, roads, drains, etc.

5.1 Introduction

This modules describes the instruments used for taking internal and external dimension when surveying a small building. You will also learn how to do calculations for a road.

5.2 Setting out small buildings

Before any setting-out can be done the stand boundaries and building lines must be established, if there is any doubt whatsoever in this connection the Local Authority should be contacted and it may be necessary to employ a qualified registered land surveyor to establish the necessary reference points and lines, in order to avoid any legal disputes after commencement or completion of the construction.

Once this step is completed the actual framework may commence, corners are established first by using triangulation, one method is shown in **Figure 5.1**.

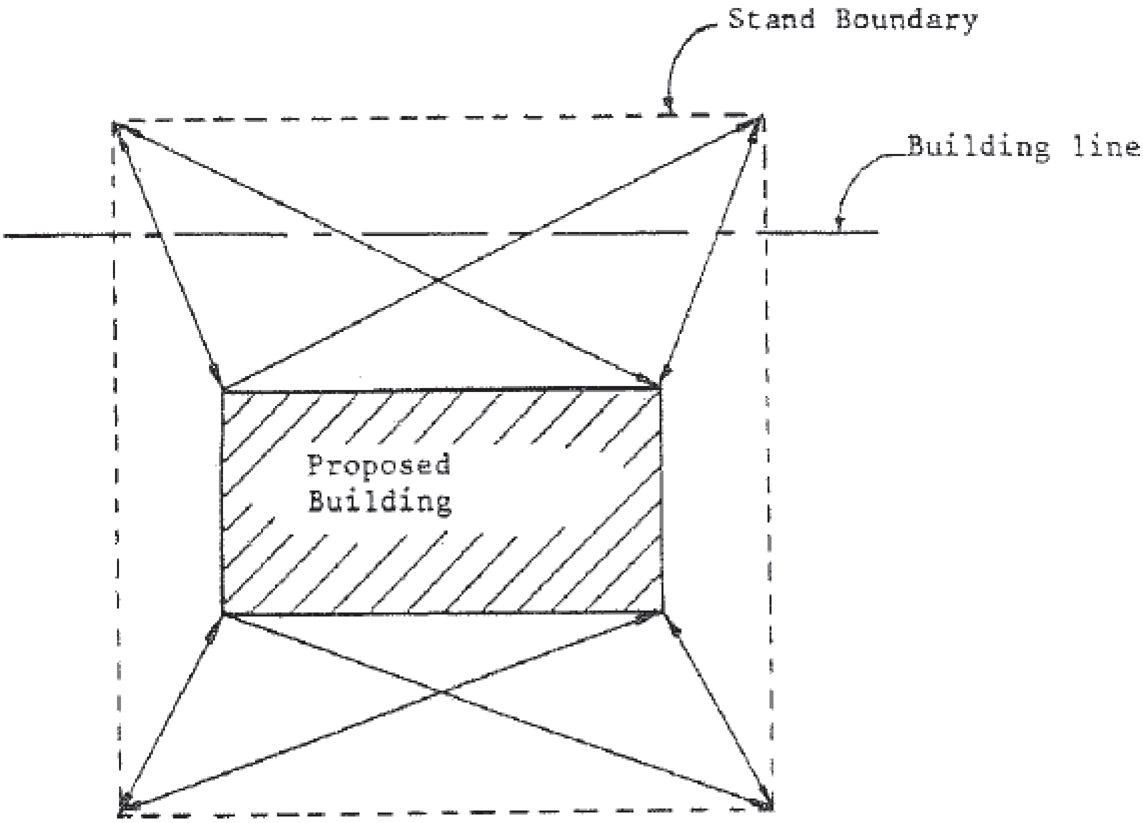


Figure 5.1

As long as each corner is fixed by measuring from two or more established reference points, and all the calculations and measurements are accurate, no problems should arise.

When the corners of the proposed building have been fixed, then the framework of the building may be set out using profile boards.

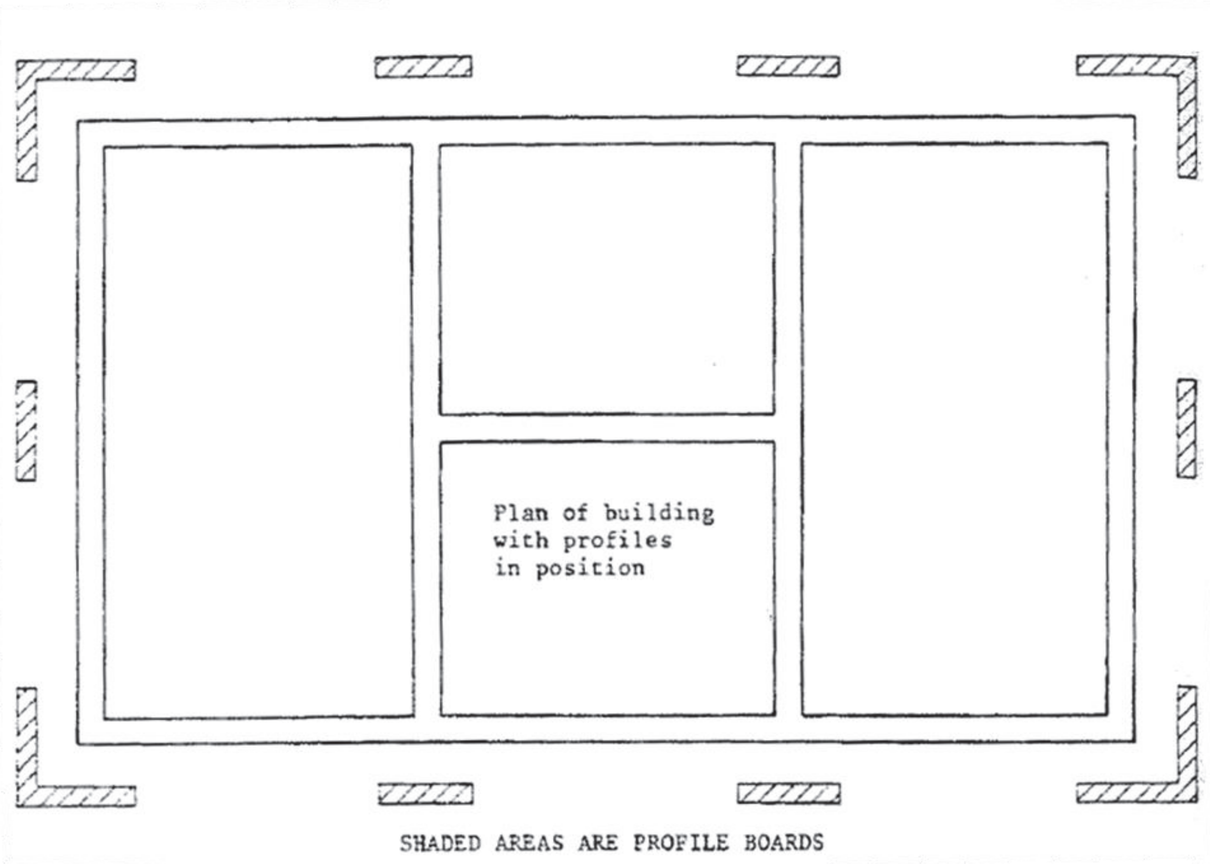


Figure 5.2

5.3 Instruments required

The obvious equipment to start with would be:

- a spade;
- scythe or similar grass cutting tool;
- a club hammer; dog spikes;
- 50 x 50 wooden pegs from 200 to 1 000 mm long;
- boarding usually 20 x 100 to 150 mm in various lengths;
- gum poles 75 to 100 mm in diameter of lengths to suit the terrain and the work;
- an axe and a wood saw;
- cement, sand and stone or concrete;
- nails of various lengths and a claw hammer;
- heavy duty nylon fish-line;
- an optical square or a cross-staff (see **Figure 5.3**);
- a measuring tape to suit the job on hand;
- a spirit-level;
- paint-white;
- red or yellow brushes;
- chalk or crayons;
- ranging rods;
- plumb bob;

- the type of level will depend on the work, the site and the required accuracy, on some buildings (eg a dwelling on level ground) it may not be necessary to use a level at all, naturally a spirit level is a basic requirement.

The only instrument, listed here, not previously covered in the course (N4 and N5) is the cross-staff.

5.3.1 The cross-staff

A typical cross staff is shown in **Figure 5.3a** and consists essentially of an octagonal brass box with slits cut in each face so that opposite pairs form sight lines.

The instrument may be mounted on a short ranging rod and, to set out a right angle, sights are taken through any two pairs of slits whose axes are perpendicular. The other two pairs then enable angles of 45° and 135° to be set out.

An alternative type of cross staff is shown in **Figure 5.3b** It is cheaper than an optical square and less likely to be damaged.

5.4 Setting out of drainage

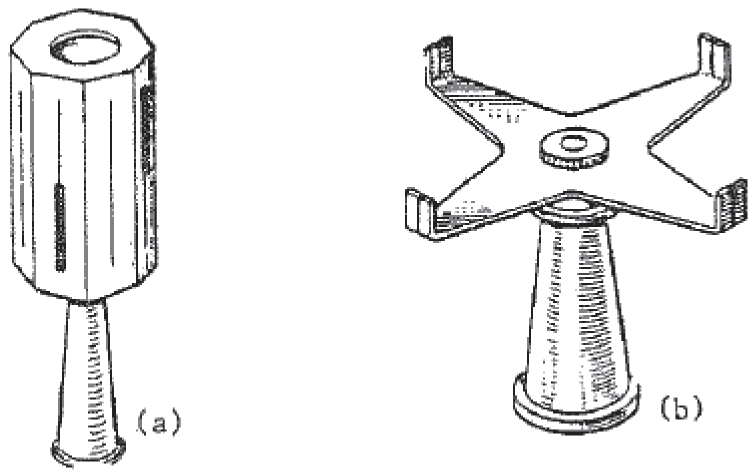


Figure 5.3

When setting out drains, sewers and roads many contractors still prefer to fix sight rails to predetermined levels, set at the gradients of the construction work to be built some short distance below.

Figure 5.4 illustrates a sight rail; a horizontal wooden rail nailed to two posts situated on either side of a drain trench. Variations of this design may be seen in various parts of the country but, whatever form they take, the objective remains to fix a certain reduced level on the top edge of the rail. In drainage work, where an excavation is being carried out with a mechanical excavator, it may be necessary at some stage of the work to remove the rail to permit the free working of the excavator.0

⚠	**Note:** Various devices may be used with success. One method is to fix a light steel bracket on to each post and drop the rail into position; the rail is simply lifted out when the excavator is working there.

Boning rods (see **Figure 5.4**) have to be used in conjunction with sight rails and the operation is shown in **Figure 5.5**. One operative looks along the line of the sight rails while another holds a boning rod vertically at the base of the excavation; if the top of the rod coincides with the sighting between two successive sight rails the base of the rod is at the required level and at the predetermine gradient.

Sight rails should be positioned so that the operative can comfortably bend down to bring his eye level with the top edge. For easy use a relatively high or low rail position must be avoided.

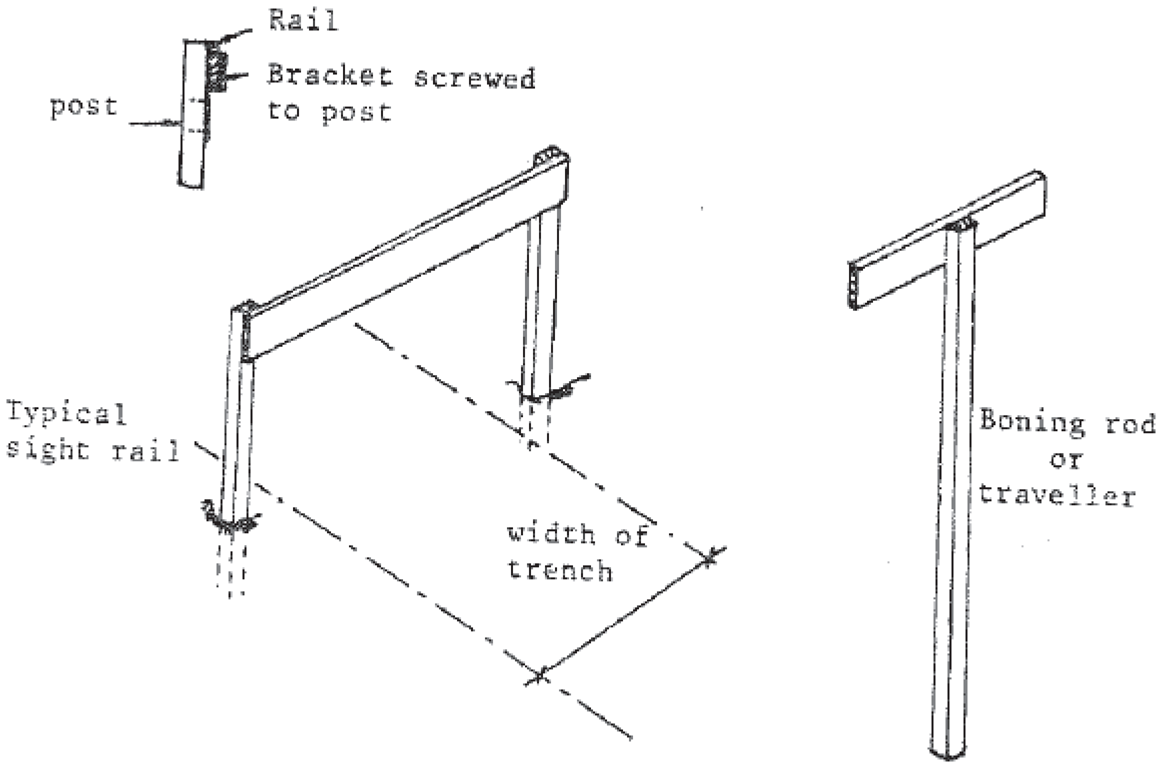


Figure 5.4

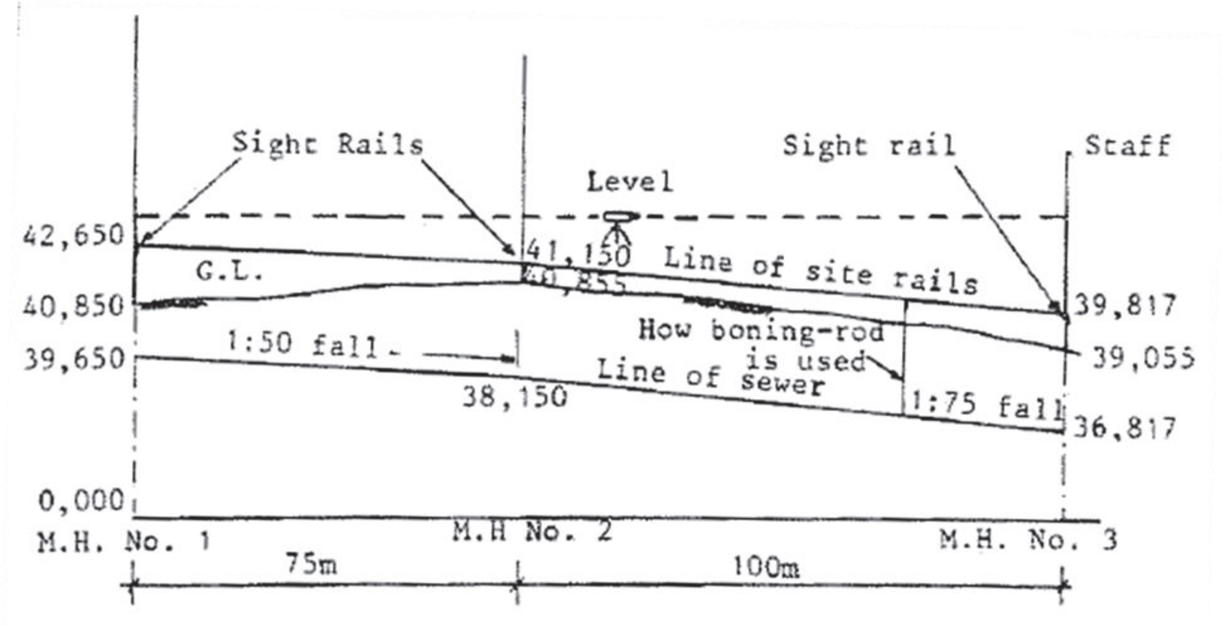


Figure 5.5 Longitudinal section of proposed drain

Example

Consider a length of sewer being laid from manhole A, with an invert level of 30,02m, to manhole B, 60 m away, the gradient from A to B being 1 in 100 and falling from A to B.

Figure 5.6 shows the proposed sewer, and it will be seen that the general depth of the sewer is below 3m.

Thus if two rails are fixed over stations A and B about 1 m above ground level, and each a fixed height above invert level, then an eye sighting from rail A to rail B will be sighting down a gradient equal to that of the proposed sewer.

In the example, a convenient height above invert would be 3,75 m, so that a boning rod (a plate with a sight bar across one end, and looking like a T-square) of this length, held vertically so that its sight bar just touched the line of sight between sight rails A and B, would give at its lower end a point on the sewer invert line.

(Note: the invert of a sewer is the lowest point on the inside surface.)

Staff readings might well be required to 0,001 m when establishing sight rails even though the nearby bench mark heights are known only to 0,01 m.

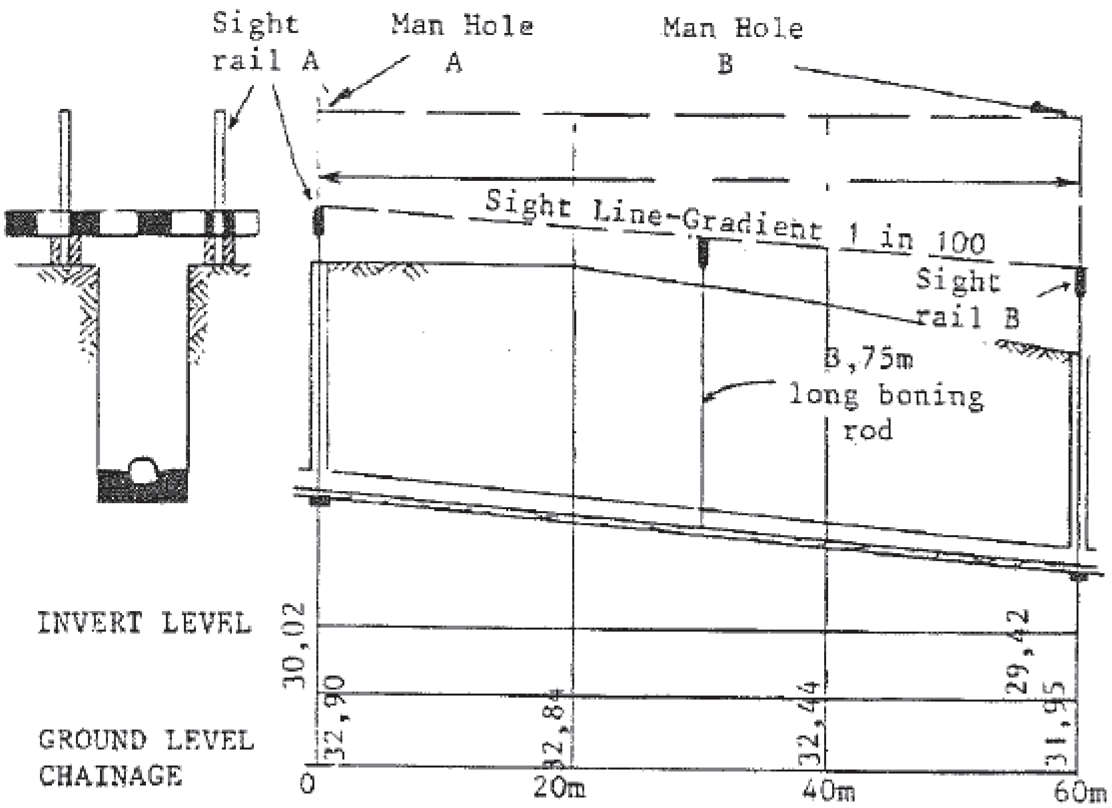


Figure 5.6

To fix these sight rails for use with a 3,75 m-long boning rod, therefore, we drive two posts on either side of the manholes and nail the rails between these at the following levels:

	Sight rail A, RL	= 30,02 + 3,75
		= 33, 77 m
	Distance AB	= 60 m
Therefore		
	Fall	$= \frac{60}{100} = 0{,}60$m
	∴ Invert level B	= 30,02 - 0,60
		= 29,42 m
and	Sight rail B, RL	= 29,42 + 3,75
		= 33, 17 m

RL = reduced level.

If a level set up nearby has a height of collimation of, say 34,845 m, then the staff is moved up and down the posts at MHA until a reading of 34,845 - 33,770 = 1 ,075 m is obtained.

Pencil marks are made on each post and the black and white sight rail is nailed in position as shown in **Figure 5.6**. For rail B, the staff reading would, of course, be

34,845 - 33,170 = 1 ,675 m

Frequent checking of sight rails is required, as they are liable to be disturbed by excavators, dumpers, lorries, etc.

In this example the length of the boning-rod could be calculated in the following way.

At MHA assume sight rail to be 1,50 m above ground level, then the length of boning rod required is the ground level plus the sight rail height minus the invert level ie 32,90 + 1,50 - 30,02 = 4,38m.

Then the sight rail at MHB would have to be, the difference between the boning rod and the difference between the ground and invert levels, above the ground, ie

4,38 – (31 ,95 - 29,42) = 1 ,85 m above the ground

5.5 General

5.5.1 Small Sites

An accuracy of about 2 mm in 30 m is desirable; errors of over 5 mm in 30 m should be rectified. The setting out is done from a given base line by use of the theodolite and a steel tape.

All distances must be measured horizontally and a plump bob used to transfer horizontal distances down vertically. The appropriate time for precise work is when the blinding concrete to foundations has been placed.

At this stage centre lines of columns, centre lines and outside or inside lines of walls and their footings must be precisely fixed. The main centre, or reference line, for the building should already have been fixed by pegs sited well away from the work, and it is usual to work from co-ordinates along this line and right-angle distances off them.

Note:
The main base line, or some other line parallel to it, may have to be transferred down into the foundations by theodolite sighting and by careful steel-tape and plumb-bob work.

When the theodolite is set over any given co-ordinated point, care must be taken to set it precisely over the given point to an accuracy of 1 mm or less. If this is not done errors will be magnified.

It is possible to set a theodolite in line with two points, between them, by siting first on one point and transitting to sight on the other, shifting the theodolite until it transits accurately to each point. Sighting points for theodolite work should be nails with their heads field to a point.

For the assistance of bricklayers and shuttering carpenters, sight boards can be provided, with the top of the cross arm fixed a given amount above formation level and a saw cut made exactly on the precise line of sight. A string can then be fastened through such a saw cut.

5.5.2 Setting out larger sites

Triangulation from a measured base line is the usual method adopted; the triangles being as well proportioned as possible.

This method is usually better than a lengthy closed traverse when the weather is changeable, since a closed traverse represents more work to be done at one time and may be interrupted by bad weather. If survey of a closed traverse has to be interrupted there is always a danger that one or the other of the last two traverse pegs will be disturbed by plant.

Note:
Even if the pegs have not been disturbed, a large closing error may cause the surveyor to think this has happened, and he will feel it necessary to do the whole job over again.

It is worth going to some trouble to find a suitable base line which, for preference, should be level. A level tarmac road is ideal; if this is not available it is worthwhile cutting the grass and removing any humps from another piece of level ground so that the tape may be laid flat and given the standard pull required by means of a spring balance.

A new tested steel tape should be used, the maker's corrections being known and allowed for. A thermometer is also necessary. The temperature should be fairly steady, eg the work should not be undertaken if there is intermittent hot sunshine between periods of cold.

A quiet, still, and cloudy day is best. Accurate measurement of angles with a theodolite is easier than accurate measurement of distances by tape, so it is worth finding a good base line at the expense, possibly, of not having the best angled triangles for setting out the other points.

Note:
Preliminary calculations before deciding on the base line will indicate whether a satisfactory degree of accuracy will be obtained.

Measurement of other distances is usually undertaken by using the linen tape. When measuring distances on steep slopes it is a good deal easier to use a 3 m piece of timber than to try to pull the tape out horizontally. A 3m interval is accurately marked on the timber, a plumb bob being suspended from one end-mark.

Work proceeds downhill, the timber being kept horizontal by means of a small builder's level. Pegs are put in at the 2 m intervals, the exact 2 m distance - as shown by the point of the plumb bob - being marked off on each new peg.

5.5.3 Leveling

A carpenter's spirit level should not be used for setting out the level of anything more than incidental work. It is not sufficient in conjunction with a straight edge, for instance, for getting a floor screed level.

Note:
It is difficult to get concrete floors uniformly level to an accuracy less than 5 mm, and a contractor should always be warned when greater accuracy than this must be obtained with concrete.

Usually discrepancies of this magnitude are taken up in the floor finish, which is either ground down to the desired smooth finish (as with terrazzo) or is placed with especial care, as in floor tiling.

To get tiling accurately laid, small pieces of tile are mortared on to the floor base at intervals across it, their level being fixed precisely to the correct finished level by use of the instrument level. A straight edge is then used to keep the finished tiling at the right level between the set levels, the pieces of tile being chipped off as the finished tiled area approaches.

5.6 Setting out roadworks

5.6.1 Determination of levels

One of the major purposes of the longitudinal section and cross section diagrams is to show the proposed levels or heights at which a road is to be constructed and also the existing levels of the ground. It is therefore of the utmost importance that the road constructor ensures that the road is constructed at the correct height or level.

An added complication is that a road consists of several layers, for example, subgrade, sub-base, base-course and wearing course. The road constructor must ensure that each layer of the road is built at the correct height or level.

The same principle will apply irrespective of whether a road has a cross fall or even camber. Because of the tremendous importance of accuracy it 1s the surveyor's task to provide the levels which the road constructor must use later for constructing the road.

Consequently, although a project may be allocated to a particular road constructor, who will be responsible for the correctness of construction, the initial setting out of levels will be done by the surveyor before the road constructor moves onto site.

	Note: The Surveyor is responsible for ensuring that the instrument he uses is functioning correctly, and he must check it for collimation errors at least once a fortnight.

5.6.2 Setting out

The surveyor must set out the levels for the road construction using the following procedures:

1. Place two reference pegs at each chainage peg, one on each side of the road at right angles to the centre line and at a fixed distance of 1,5 metres behind the edge of black top. All reference pegs at both horizontal and vertical curves as well as at any change of grade to be concreted in.

2. Place a gumpole or fencing standard at about 200 mm behind each reference peg.

3. After all the reference pegs and poles have been placed, the level of each reference peg must be determined by the surveyor by levelling from a bench mark with a surveyor's level. Forward and return levelling must be done to eliminate errors in levelling.

4. Levels one metre higher than the final centre line, quarter point, and gutter levels at each particular chainage are then marked on each gumpole as follows:

   (i) Cambered road
   (a) The level of the reference peg is known (see 3. above).
   (b) Final road level at the centre line, quarter point, and gutter are read off from the cross-section.
   (c) Add 1 metre to the levels obtained in (b).
   (d) The difference in height between the top of the peg and the levels obtained in (c) are obtained by subtracting (a) from (c).
   (e) The levels determined in (c) are then marked on the pole by placing a staff on the peg, measuring up the difference calculated in (d) and making a pencil mark on the pole at the required height with the help of a set square (see **Figure 5.7**).

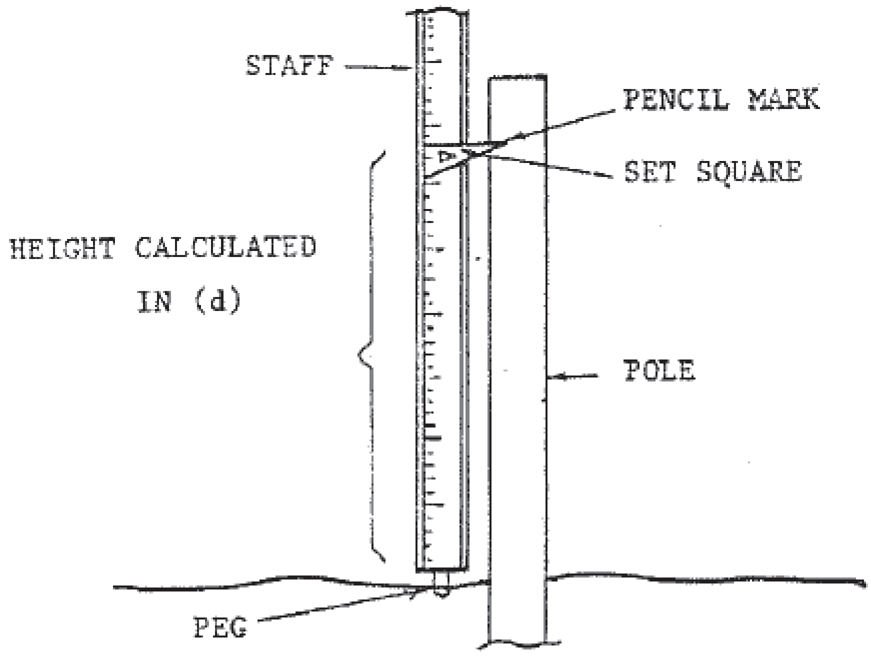


Figure 5.7 Marking levels on pole

Alternatively, the levels determined in (c) may be marked on the pole directly using the surveyor's level.

(f) A nail should be knocked into the pole at each pencil mark to provide a more permanent reference.

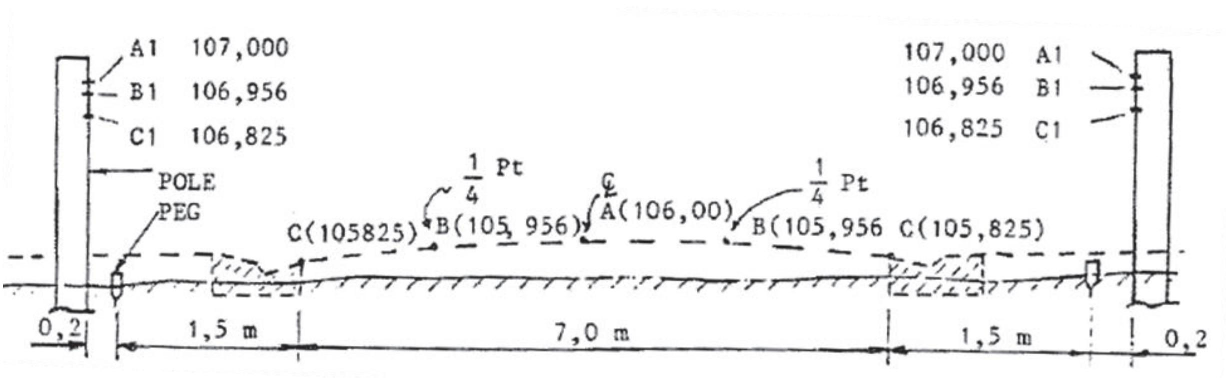


Figure 5.8 Typical layout for a cambered road

(ii) Road with straight cross fall

(g) Calculate the levels of the imaginary points p and q (see **Figure 5.9** and model calculation).

(h) Calculate the levels of P1 and Q1 by adding 1 metre to the levels of p and q.

(i) The levels of the reference pegs are already known from previous levelling (see 3.).

(j) The difference in height between the levels of the tops of the pegs and the levels of P1 and Q1 are obtained by subtracting (i) from (h).

(k) The levels of P1 and Q1 are then marked in pencil on the poles using the method described in (e) and nails are knocked into the poles at these points.

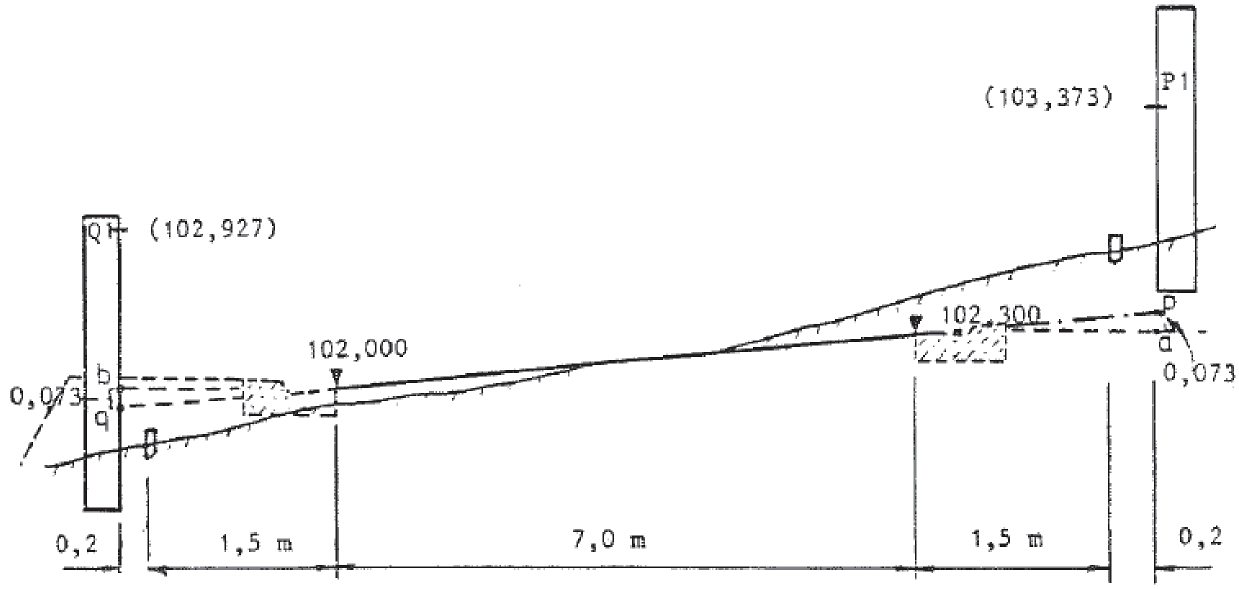


Figure 5.9 Typical layout for a road with crossfall

To calculate ap and qb

In 7,0 m change in level of road is 0,3 m

In 1,'7 m change in level is $\frac{1{,}7}{7} \times 0{,}3 = 0{,}073$

∴ ap = qb = 0,073 m

To calculate P1 and Q1

Level of P1 = 102 300 + 0,073 + 1 000 = 103 373

Level of Q1 = 102 000 - 0,073 + 1 000 = 102 927

5.6.3 Transfer of levels from the poles to the road

The road constructor can now set out the centre line, quarter points or gutter levels at any chainage by adopting the following procedure:

1. Locate the position of the centre line, quarter points and gutters by measuring horizontally the required distance from one of the reference pegs in a straight line between them.
2. Stretch a line tightly from one pole to the other, holding each end on the relevant nail and then measuring down from the line a distance of 1 metre at the required point. This will give the final road level at that point.
3. Surface, base, sub-base and subgrade levels may conveniently be determined with the help of a wooden gauge rod, marked up as shown in **Figure 5.10**. The levels referred to are always those of the tops of the layers in question.

Note:
The markings on the gauge rod are dependent on the road structural design and will vary from road to road in accordance with the thicknesses specified for the various layers of road construction.

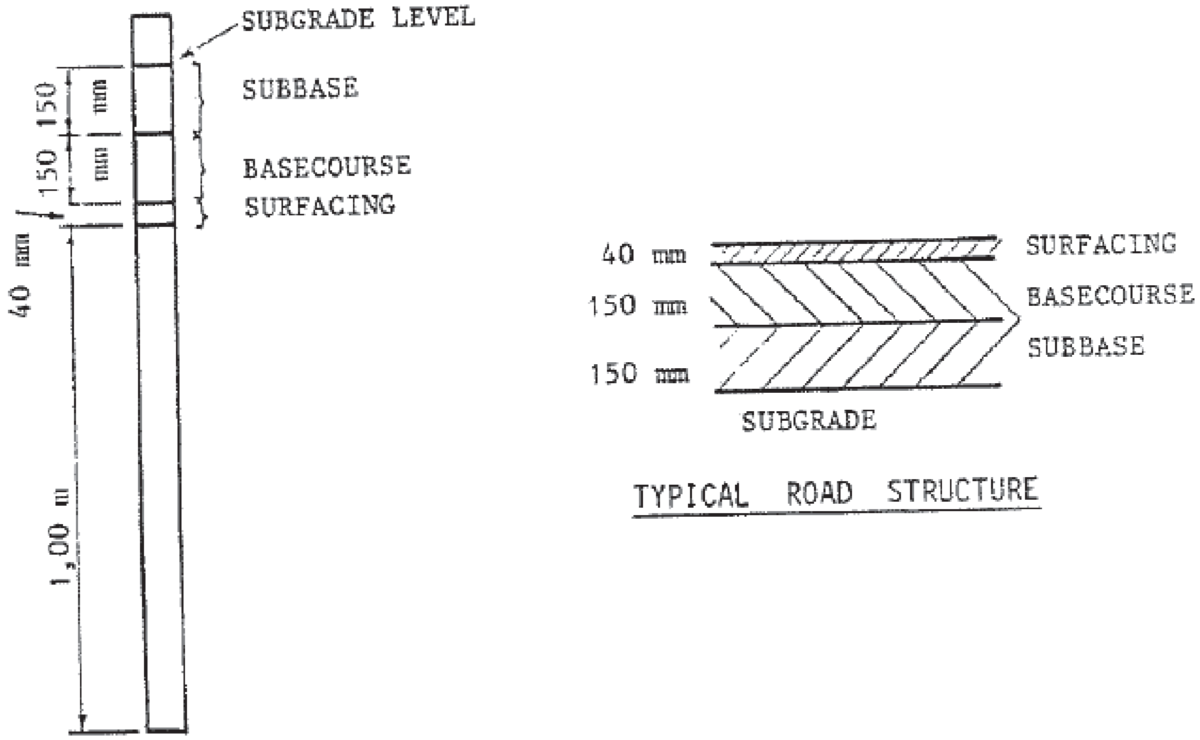


Figure 5.10 Gauge rod for road structure shown in **Figures 5.11** to **5.13**

4. If, say, the subgrade level at the centre line is required to be set out on a cambered road, the string is stretched between points A1 on opposite poles and the gauge rod is held at the centre line with the subgrade mark against the string. The bottom of the rod will then be at the proposed subgrade level.

If the subgrade level at the gutter line is required, the string is stretched between points C1 on the poles and the gauge rod is held at the gutter line position with the sub-grade mark against the string.

The bottom of the rod will again denote the subgrade level at that point.

The subgrade level at the quarter point can similarly be found by stretching the string between points B1 and holding the gauge with the subgrade mark against the string.

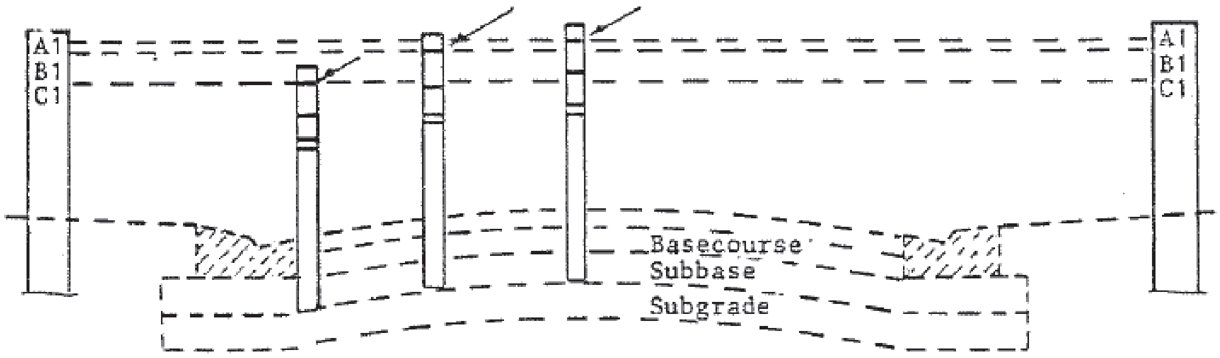


Figure 5.11 Determining sub-grade levels

Similarly if the sub-base, base course or surface level is required, the gauge rod is held with the sub-base, base course or surface mark against the string.

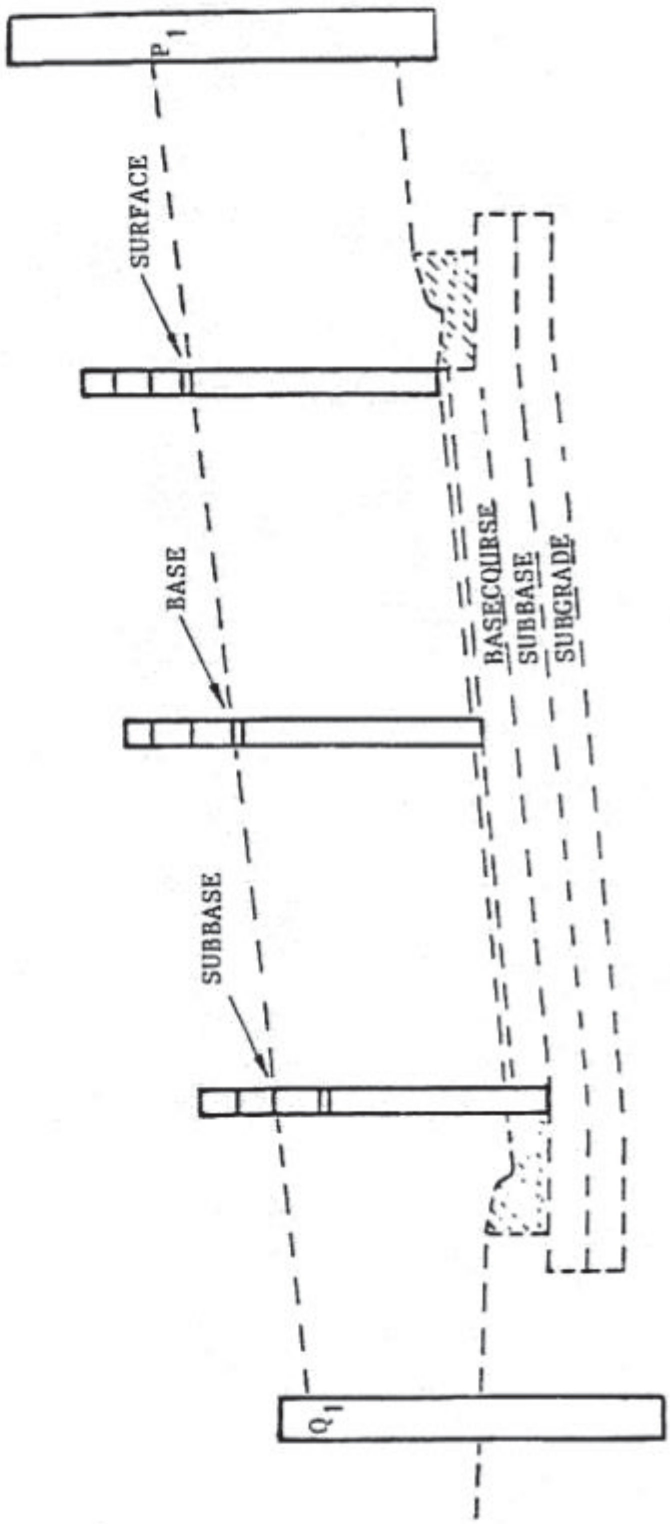


Figure 5.12

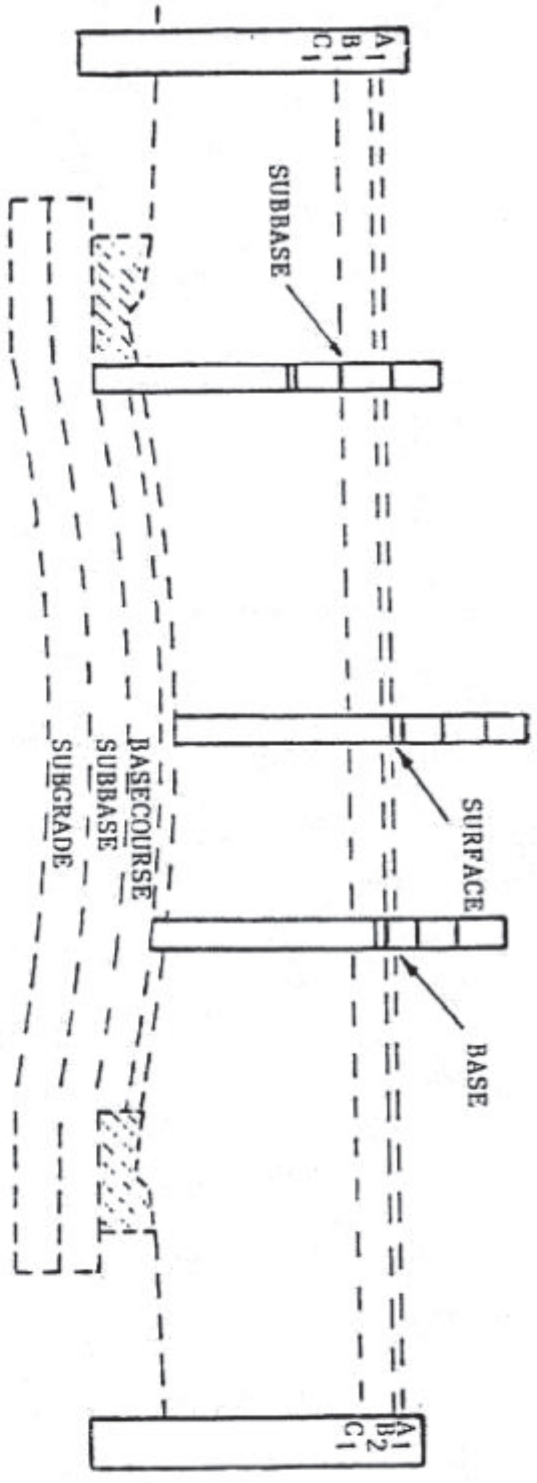
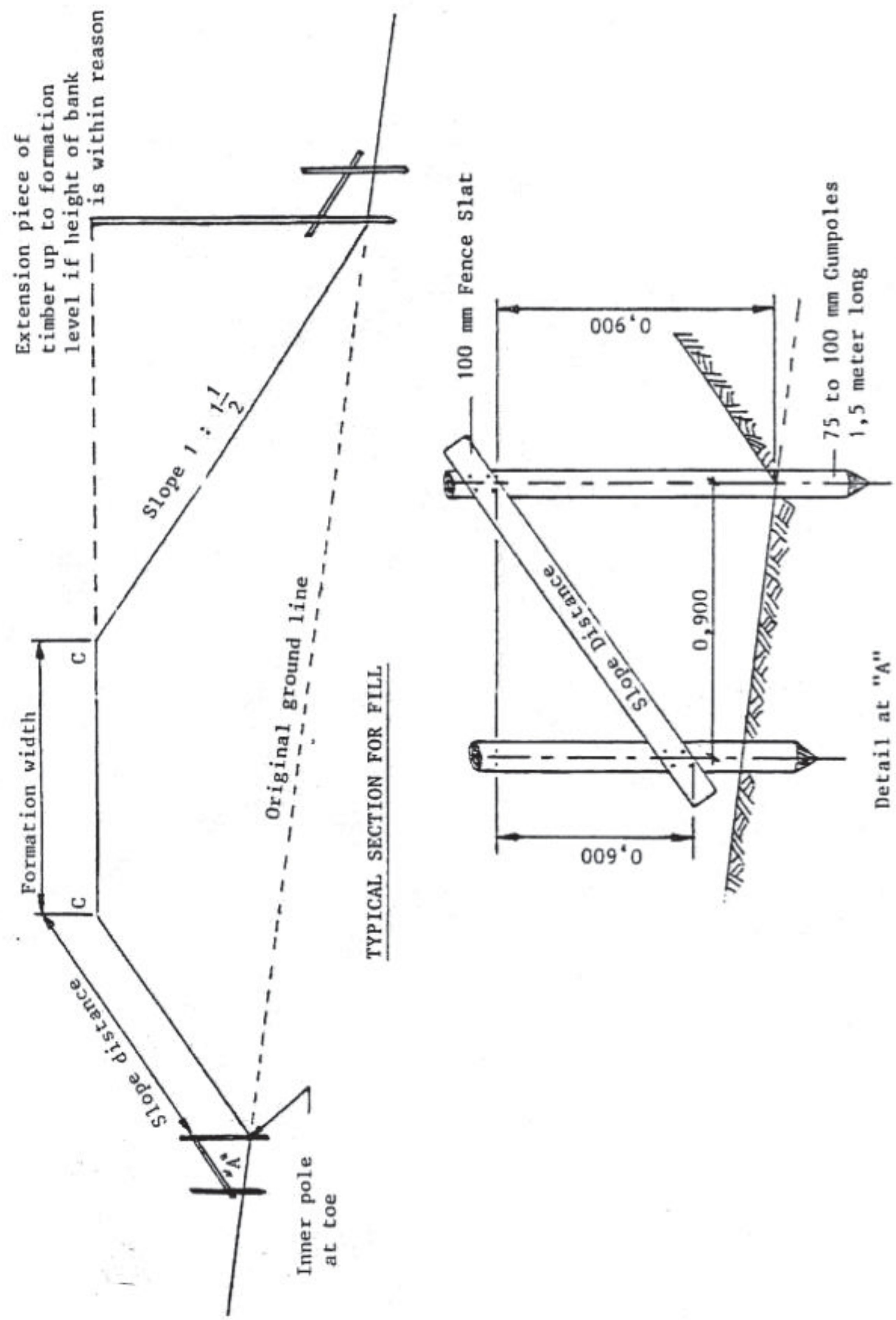


Figure 5.13

5.6.4 Setting out of batters for cuts and fills

Batter pegs are normally set at each 20 metres distance NORMAL to the centre line. They may be required at shorter intervals where the radius of the curve is small. **Figures 5.14** and **5.15** show methods of setting out.

1. Poles to be painted white and distance written on front and back of poles sufficiently large to be discernable at a distance.
2. As batter boards are erected it is unnecessary to use red paint to indicate cuts.
3. Use Company standard colours, and if fills are not excessive erect auxiliary poles showing formation levels.
4. Slope distances are normally taken of cross-sections.
5. Each cross-section should be carefully considered and a sketch made and taken into the field to aid in setting out. A separate file or field book should be used for this and distances from the centre line carefully recorded to assist in approximate relocation of the centre line. These distances should preferably be written on the poles.
6. Information such as slope distances and distances from the centre line may be conveniently written on metal tags.
7. In fairly shallow cuts, levels may also be given on poles on the centre line. These poles may remain until required depth is obtained, and then cut out.
8. Fills should be re-centred and new batters placed either monthly or at each 2 metre lift. This needs particular attention.
9. Coloured insulating tape should be used for standard levels on poles rather than paint, which fades rather quickly.
10. Care should be taken in setting batters to note non-standard slopes, which often occur in freeway work.
11. Centre-line poles an high fills should be extended vertically as the layers are added to maintain the centre line.
12. When cut or fill has reached sub-grade level (approximately) pegs should be placed at position C (**Figures 5.14 & 5.15**) to ensure that the cut (or fill) is wide enough.



Extension piece of timber up to formation level if height of bank is within reason
Formation width
C
C
Slope 1 : 1½
Slope distance
Original ground line
"A"
Inner pole at toe
TYPICAL SECTION FOR FILL
100 mm Fence Slat
0,900
0,900
0,600
Slope Distance
75 to 100 mm Gumpoles 1,5 meter long
Detail at "A"

Figure 5.14

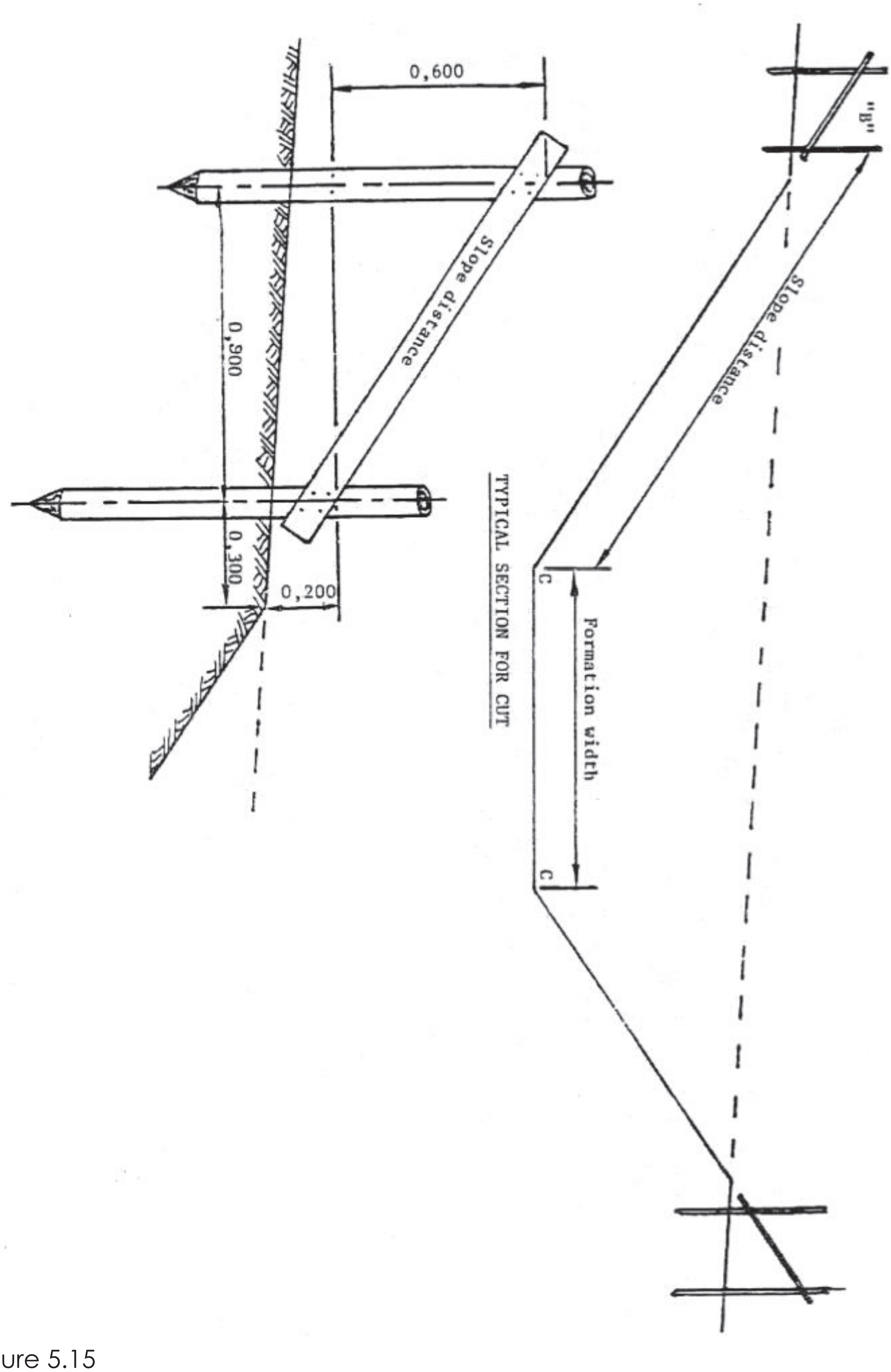

0,600
Slope distance
0,900
0,300
0,200
TYPICAL SECTION FOR CUT
C
Formation width
C
"B"
Slope distance

Figure 5.15

Activity 5.1

1. Draw a longitudinal section of a 150 mm ø sewer pipe with a 1 : 100 fall from A to B, 20 m away. GL at A = 46,24 m, invert at A = 44,65 m, GL at B = 46,55 m. The sight rail at A is 1,00 m above GL; calculate the length of boning rod, the height of the sight rail above GL at B and the invert level at B. Use a 1 : 100 scale and show all details.
2. What degree of accuracy is acceptable on small construction sites?

Activity 5.2

1. Distinguish between crossfall and camber.
2. Draw a detailed cross-section to a scale of 1 : 100 of a road with a formation width of 8 mm, in a cutting at chainage 1 530 where the left side slope is 1 : 2 and the right side slope is 1 : 1½, the left slope distance is 3 m and the right slope distance is 3,5 m. The road is level. Show how the batter boards would be placed and draw a 1 : 10 detail of the left side batter board.

Self-Check

I am able to:	**Yes**	**No**
• Demonstrate surveying a small building.		
• Describe instruments used for taking internal and external dimensions.		
• Demonstrate measuring and recording a building and its site.		
• Demonstrate running internal and external measurements taken horizontally and vertically.		
• Demonstrate a plotting survey from field measurements.		
• Do the following calculations:		
o Cut and fill eg for a road,		
o Plotting vertical sections, roads, drains, etc.		

If you have answered 'no' to any of the outcomes listed above, then speak to your facilitator for guidance and further development.

Module 6

Setting Out

Learning Outcomes
On the completion of this module the student must be able to:

- Describe the setting out procedure for a simple rectangular building.
- Describe the equipment required.
- Describe possible constraints in setting out a building.
- Describe the positioning of profiles and datum for a building.
- Discuss how profiles are used with a traveller to control excavation and foundation levels.
- Describe setting out and levelling of drainage work.
- Explain how to invert
  - a drain
  - a sight rail
  - a traveller
- Demonstrate calculating a suitable length of traveller and reduced levels of sight rails from given drawings.
- Establish sight rails for horizontal position and depth control of a straight drain between manholes.

6.1 Introduction

Setting-out is to stake out reference points and markers that will guide the construction of new structures such as roads or buildings. These markers are usually staked out according to a suitable coordinate system selected for the project.

6.2 Setting out foundations

When a site is handed over to the contractor it is necessary to give the positions of:

(a) the "building line" along one frontage at least;
(b) a point on the latter marking the position of a particular feature, say, one corner: and
(c) a datum to which all foundation, floor and other levels are referred.

Setting out the works is one of the chief duties of the builder or his foreman, and in order to carry out this operation efficiently, he must be fully conversant with the architect's plans and details. He should be able to use the level to ensure the accurate execution of foundation and floor levels.

> **Note:**
> A knowledge of simple surveying is also an asset when the foreman is called upon to set out buildings placed obliquely to the site.

Before the actual building operations are commenced, the walls and trenches to receive the foundations must be set out on the site, the exact position of the corners, division walls, etc, being indicated.

The first line to be set out is generally that of the main frontage of the building. Its relative position must be carefully fixed and its length accurately measured.

The remaining walls of the building should be set out by using the 3 : 4 : 5 right angle, or the builders' square, which is a large wooden square used for setting out and checking right angles.

> **Note:**
> The best and most efficient method of setting out a building is to employ a professional engineer or surveyor to set out all the corners and levels with a theodolite, in any event, all foremen should at least be able to work with a dumpy level for setting out and levelling purposes.

It is proposed to set out a small building to illustrate the basic methods of setting out foundations.

The first pegs should be inserted at the external corners of the building. In good practice the corner pegs are driven down until their tops are at ground floor level, ie with the top of the ground floor decided upon. This is done by using a dumpy level and leveling staff.

When the pegs are inserted, and before the profiles are erected, it may be necessary to do some site clearing or rough leveling to ensure that rough taping, with its resultant loss in accuracy, is obviated.

In any event, regular checking of measurements is essential, as a small mistake may become cumulative if not discovered in time, and this may lead to a loss for the builder.

The next step is to insert the profiles at convenient distances from these pegs, on which the foundation and wall width should be marked. If the building is small the profiles should be kept well clear to allow the free passage of wheelbarrows around the excavated trenches, as the profiles remain in position until the walls are set out on the foundations.

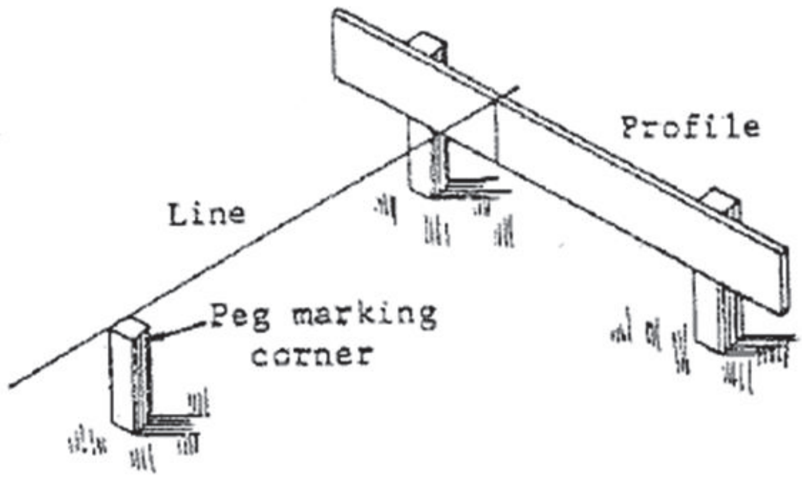


Figure 6.1 Marking wall line on profile

The wall thickness and foundations are marked on the profiles and to be sure permanence a saw cut is made at each of these pencil marks.

Strain a stout ranging line to coincide with the pegs at the corners.

If the walls are at right angles, the ranging may be performed with a builder's square, or by the construction of a right-angled triangle by the 3, 4, 5 method.

Mark off along the frontage line a distance of 4 m and make a distinct mark on the line (usually a short length of string tied to the line). The measurement is taken from the point where the lines intersect.

From the same point, along the line which is to be squared, mark off 3 m (mark with string).

Now fix the position of this line so that the distance across the angle included by the two lines, and taken form the two measured positions (strings tied to ranging lines) will be 5 m.

This procedure is followed until all the corners have been squared.

As a final check, tape across (diagonally) from corner to corner. If the building has been set out correctly the measurements will correspond. Lines representing the width of the concrete foundation are then stretched between the profiles. The trenches are then excavated.

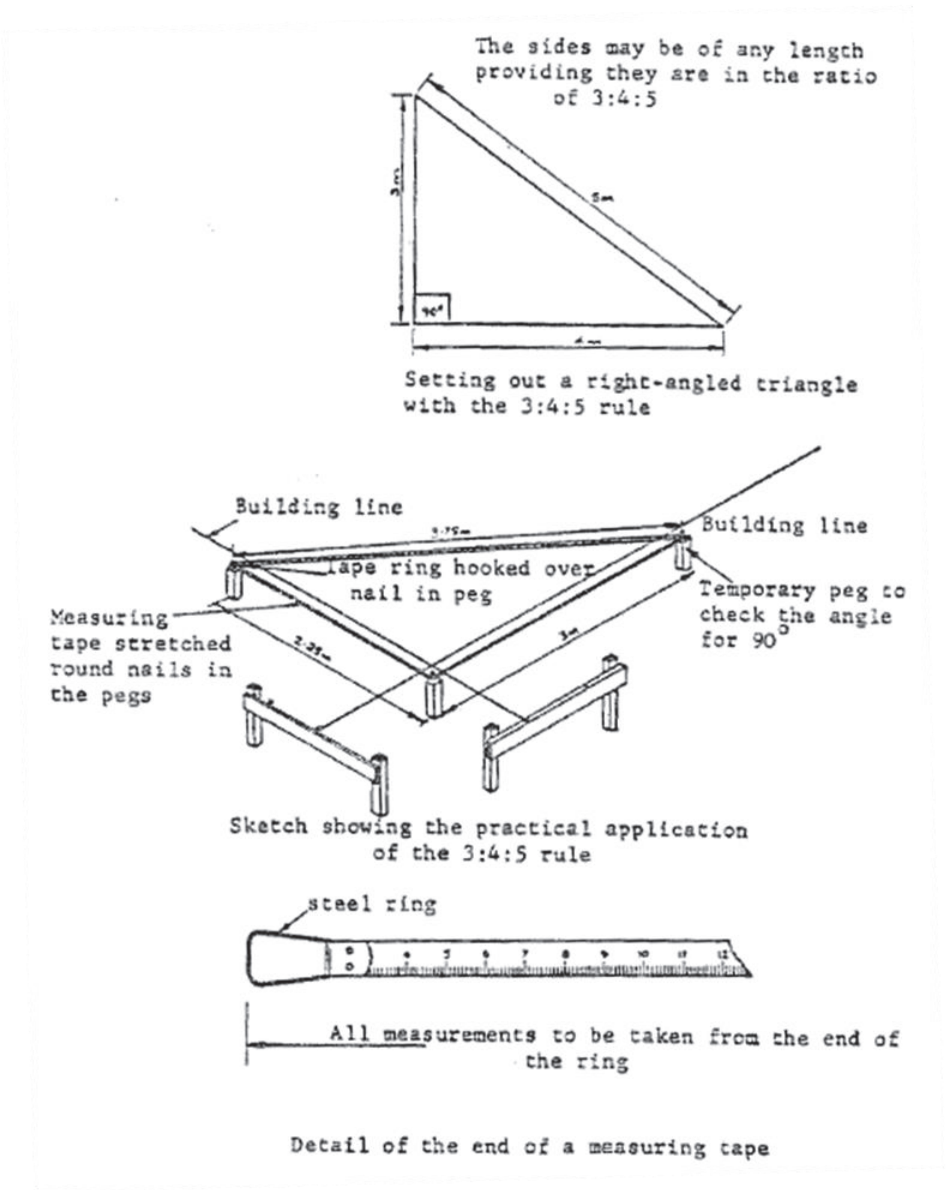


Figure 6.2

6.3 Levelling trenches and putting in of level pegs with a spirit level and a straight edge

Wooden pegs or short lengths of mild steel rod are hammered in at intervals along the bottom of the trenches. These pegs show the person doing the casting how deep the foundations are to be, eg if the depth of the concrete is to be 228 rnm, the pegs are hammered into the bottom of the trench so that they protrude 228 mm.

During casting a long straightedge is used to level off the concrete using the guide depth pegs.

When putting in the pegs a long straightedge together with a spirit level is used as shown in **Figure 6.3**.

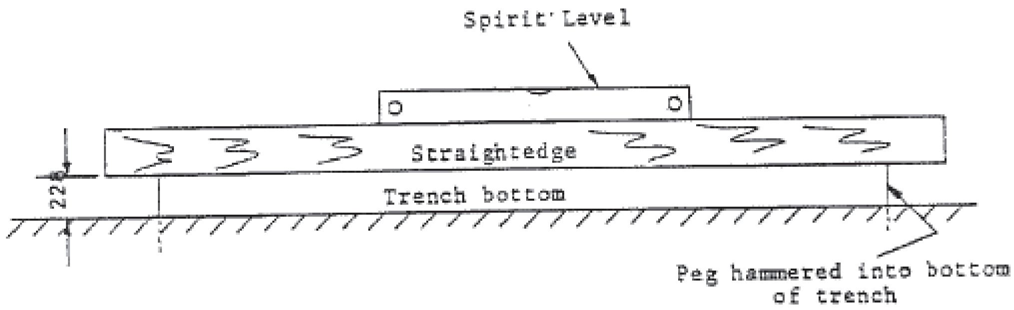


Figure 6.3

Illustrates the use of straightedge and level to put in level pegs
Begin at the lowest point (ground level) in the excavations. Hammer in a peg and make sure that the peg protrudes 228 mm. Place the straightedge on the peg and hammer in another peg approximately 70 mm short of the end of the straightedge.

Check for level. If adjustment is necessary, give the second peg a few blows with the hammer, whilst lifting up straightedge and level, until the second peg is level with the first. As the first peg is set to the required depth, adjustment in level can only be done on the second peg.

This performance is then repeated until all the level pegs have been hammered in.

To allow for any inaccuracy in the level we must turn the straight-edge (with level in position on top) through 180°, so that: the end of the straightedge which rested on first peg now rests on peg number 3. This turning of the straightedge and level must be done every time before a new peg is hammered in.

When all the pegs are in position the bricklayer can easily see where further excavations are necessary.

When settings out walls, etc, in basement excavations, the profiles should be set at ground level and strong building lines stretched between the profiles. A plumb line can be hung from these lines to transfer the markings to the bottom of the excavations.

Note:
This method is also us for column foundations, etc.

Care should be taken that sufficient working space is allowed for in marking out the profiles, so that they are not in the way when work commences on the foundation walls, e.g. if a vertical DPC is required on the outside of the foundation walls.

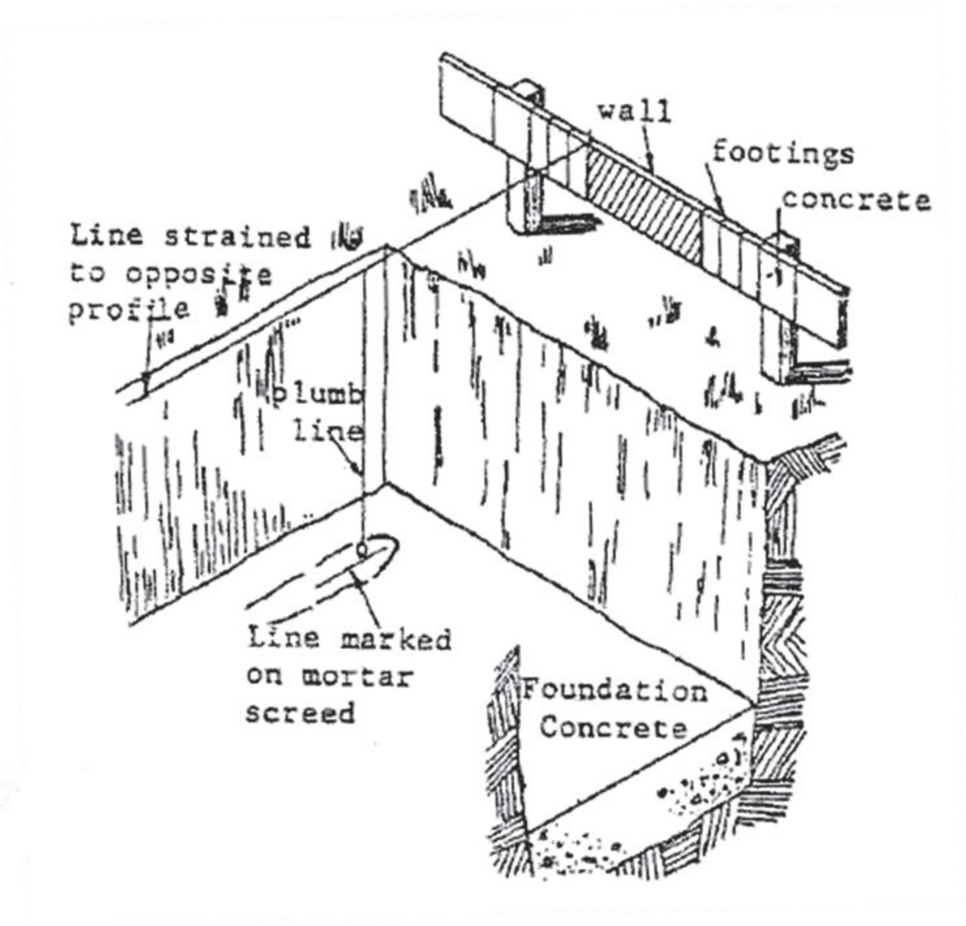


Figure 6.4

The brick footings and walls are then built to a line between profiles.

Great care should be taken with regard to the accuracy with which the footings are laid, as it is on these footings that the main structure of the building is to be erected.

6.4 Setting out on an inclined site

A building is not always level, therefore, we shall also consider a site with a sharp incline.

Here we cannot use the profile method described above, but we use a method where the dumpy level is used to set up the profiles.

The corner pegs of the building is put in as shown in **Figure 6.1** checking diagonally for square. Wooden poles are planted at intervals about 1,5 m clear of the building line, so that these poles are not too near when excavation is done as shown in **Figure 6.1**.

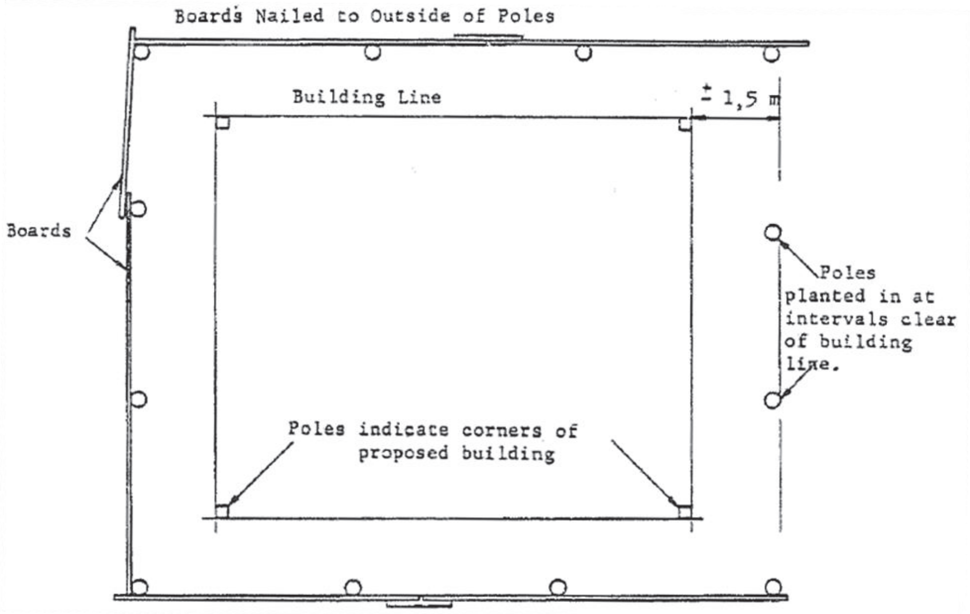


Figure 6.5

The dumpy level is set up on the site as shown in **Figure 6.6**.

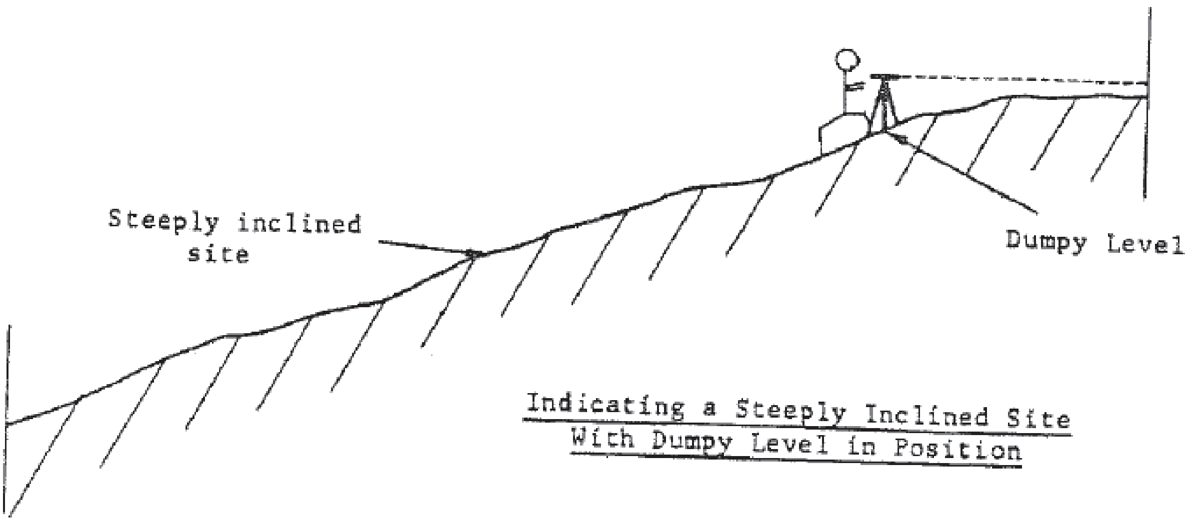


Figure 6.6

We now proceed to nail scaffold boards or other straight planks to the poles starting at the highest point (boards just clear of ground) continually checking with the dumpy level to make sure that the boards are nailed level horizontally.

We will end up having a ring of boards around us with their top edges absolutely level as shown in **Figure 6.7**. We now proceed to set out the building as described. The marks on the boards are projected down by means of a line and plumb bob.

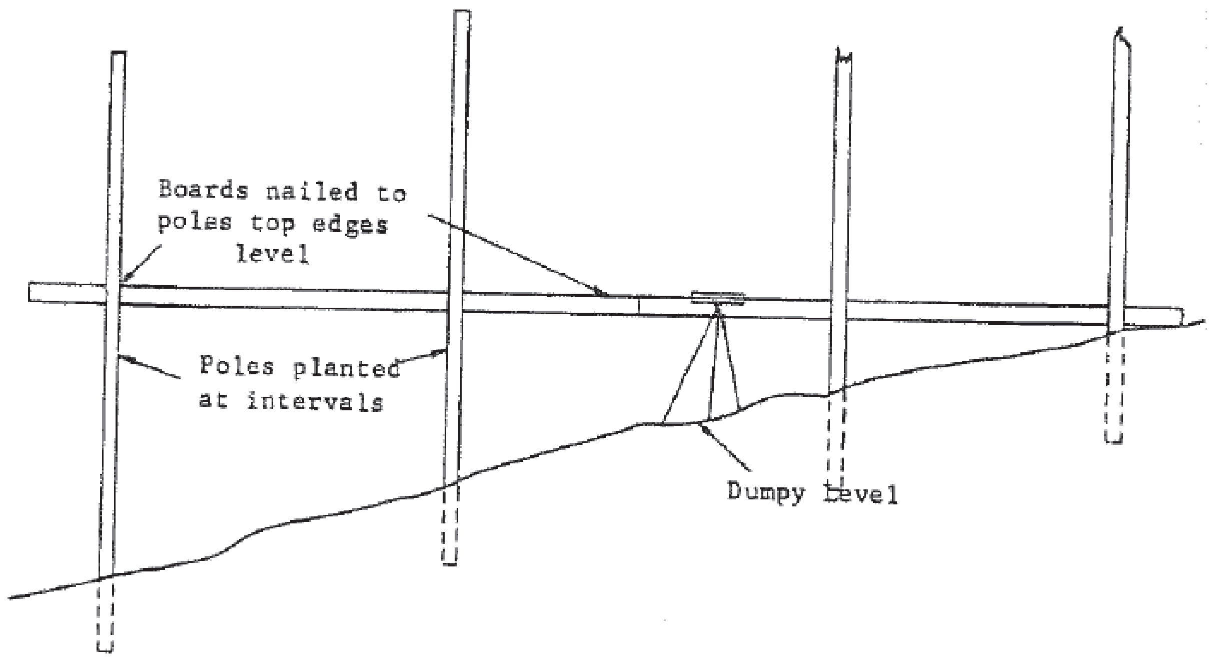


Figure 6.7

6.5 Setting out of columns

The principles of setting out as described in the following example are generally applicable to most structures, although complications arise when buildings are not square on plan or on sites that are not level.

Most of the difficulties encountered in setting out are solved by methods evolved by careful consideration of the problem on the site, and, since responsibility for the accuracy of the setting out is usually laid upon the contractor, the services of a foreman experienced in site surveying is advantageous.

If possible, the setting out of the principal lines should be checked by the resident engineer or clerk of-works, before actual construction begins.

Figure 6.8 shows the first stage in setting out the column foundations of a rectangular building. The given point A on the building line represents one corner of the structure.

The first step is to establish the centre point of each column foundation and to place marks from which the column centre lines can at any time be picked up. The plan of the building will show the distances a, b, c, d, etc, from point A to column B and between the centre lines of each two columns.

These distances are set out with a steel tape along the building line to establish the points, B, C, D, etc, at each of which a peg is driven into the ground. Points B and E are the points where the centre line through the end rows of columns cuts the building line, and through these points lines must be set out at right angles to the building line.

If a theodolite is not available this can be done by using the 5 method (or variations thereof) as follows:
Using a tape, set out from B towards C a distance of 3 metres and drive in a peg at F. Now locate the point G which is 5 metres from F and 4 metres from B.

This can be done by attaching a piece of string, 5 metres long, to a nail in the peg at F, and another piece, 4 metres long, to a nail in the peg at B; point G will be where the free ends of the strings meet when stretched taut, and the angle at B will be a right angle.

Another method is to hold the end of a tape at B and the 9-metre mark at F. Pull the tape taut while holding this length of tape at the 4-metre mark; the position of the 4-metre mark will be the point G. A linen tape should be used.

Having established the point G, stretch a line from the nail in the peg at B over the top of the peg at G to some extreme point H. In a similar manner establish a point J, corresponding to H, on a line through E at right angles to the building line.

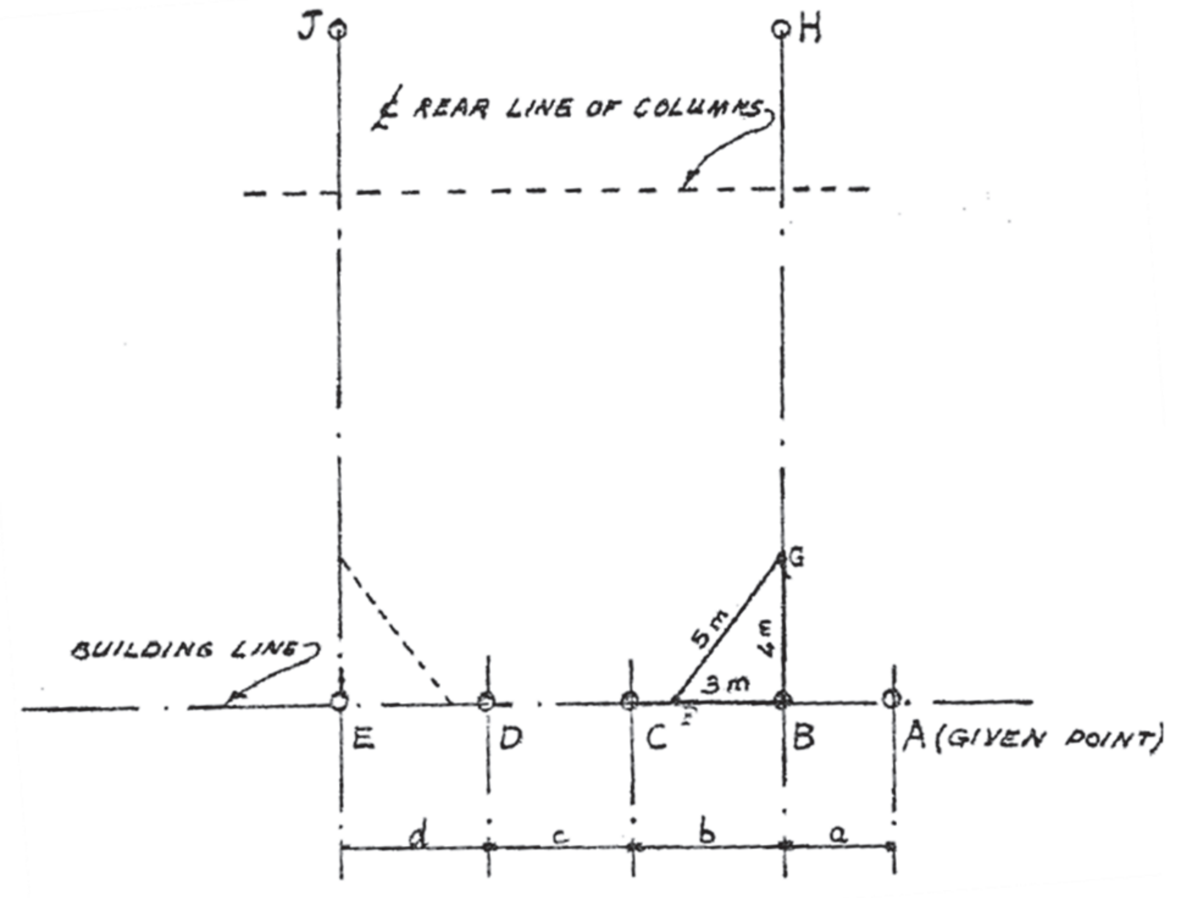


Figure 6.8 First stage in setting out columns

Second stage:
Along each of the lines BH and EJ (**Figure 6.9**) drive in pegs at the distances e, f, g and h taken from the drawings, that establishing the points K, L, M, N and P, Q, R and S which are the positions of the columns.

Stretch lines from K to P, L to Q, M to R, and N to S. These lines represent the centre lines of each transverse row of columns. With the distances b, c and d the points T and U on line NS can be established and lines stretched from T to C and from U to D will represent the centre lines of intermediate rows of columns.

The intersections at V, W, X and Y will establish the centres of the remaining columns and pegs should be driven in at these points.

At this stage the setting out should be checked. This is best by comparing the measured distances KP, PS, SN and NK with the corresponding dimensions given on the drawings.

If these agree, the accuracy of the angles can be checked by the diagonal method, comparing the distance PN with KS; these two dimensions should be identical; if not, the whole of the setting out should be adjusted.

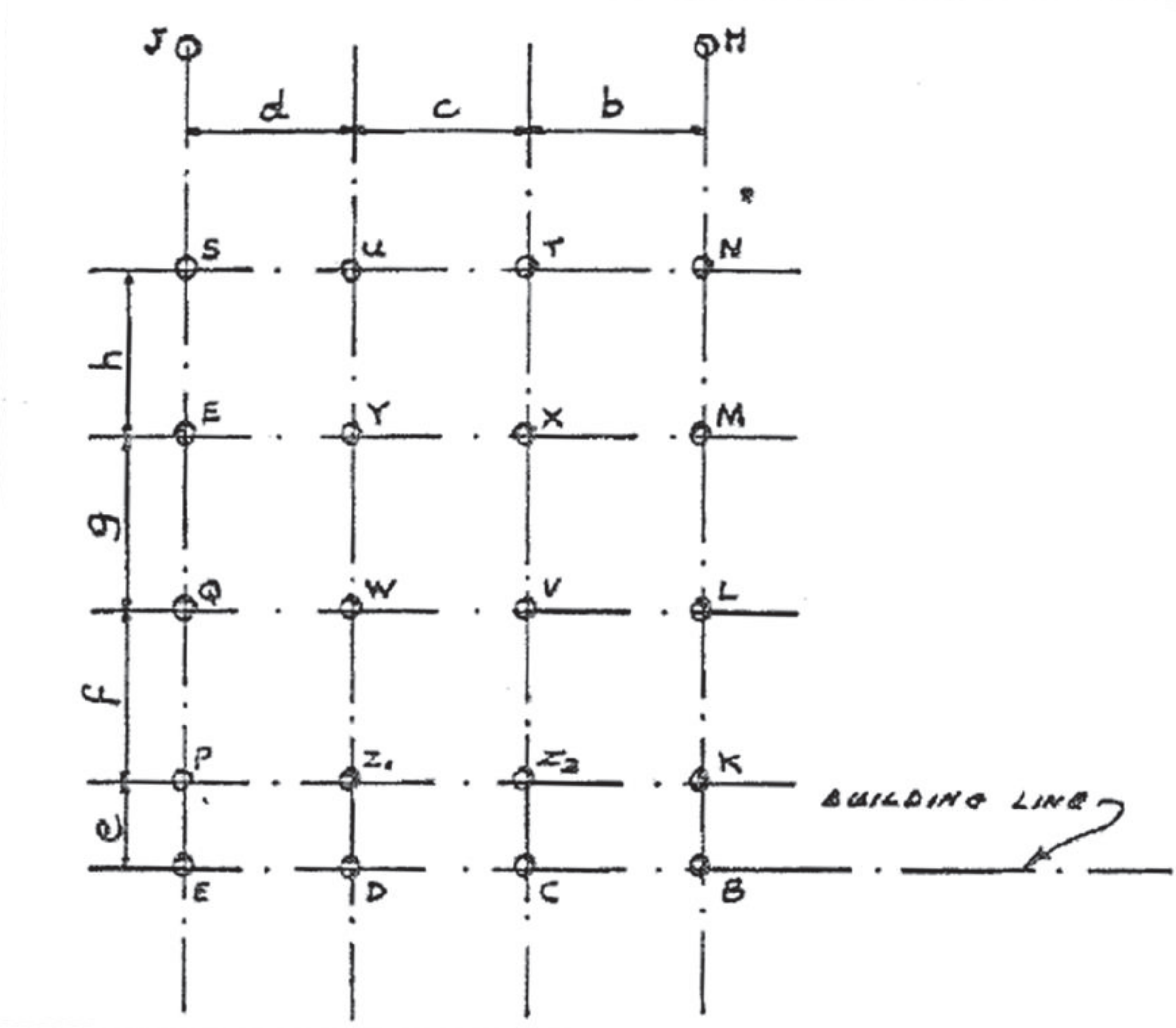


Figure 6.9 Second stage in setting out columns

The third stage is to set up marks (**Figure 6.10**) outside the area to be excavated so that when the pegs representing the column centres are removed during digging, the column centre lines can easily be picked up again.

The usual method is to drive a line of stakes parallel to each side of the building and attach horizontal boards called profiles (or battens), to the stakes, approximately 300 millimetres above the ground.

If these boards are not continuous they are no obstacle to free access to the site. Each of the column centre lines can now be extended to the corresponding profiles, a nail being driven into the latter at the exact point of intersection. The lines can now be removed.

If it is necessary to locate any column centre it can now be simply done by stretching transverse and longitudinal lines between the appropriate pairs of nails on the profiles, The lines may be of cord, but for the main setting out lines of major structures piano wire or nylon fishing line stretched taut is more durable and accurate.

Pegs are usually cut from 50 x 50 timber with one end sharpened and driven well into firm soil. For more durable marks the peg should be embedded in a small concrete block.

Marks outside the building area for the purpose of picking up column centre lines can often be conveniently established by scribing a line on adjacent brick walls, stone kerbs, steel rails, or elsewhere as convenient.

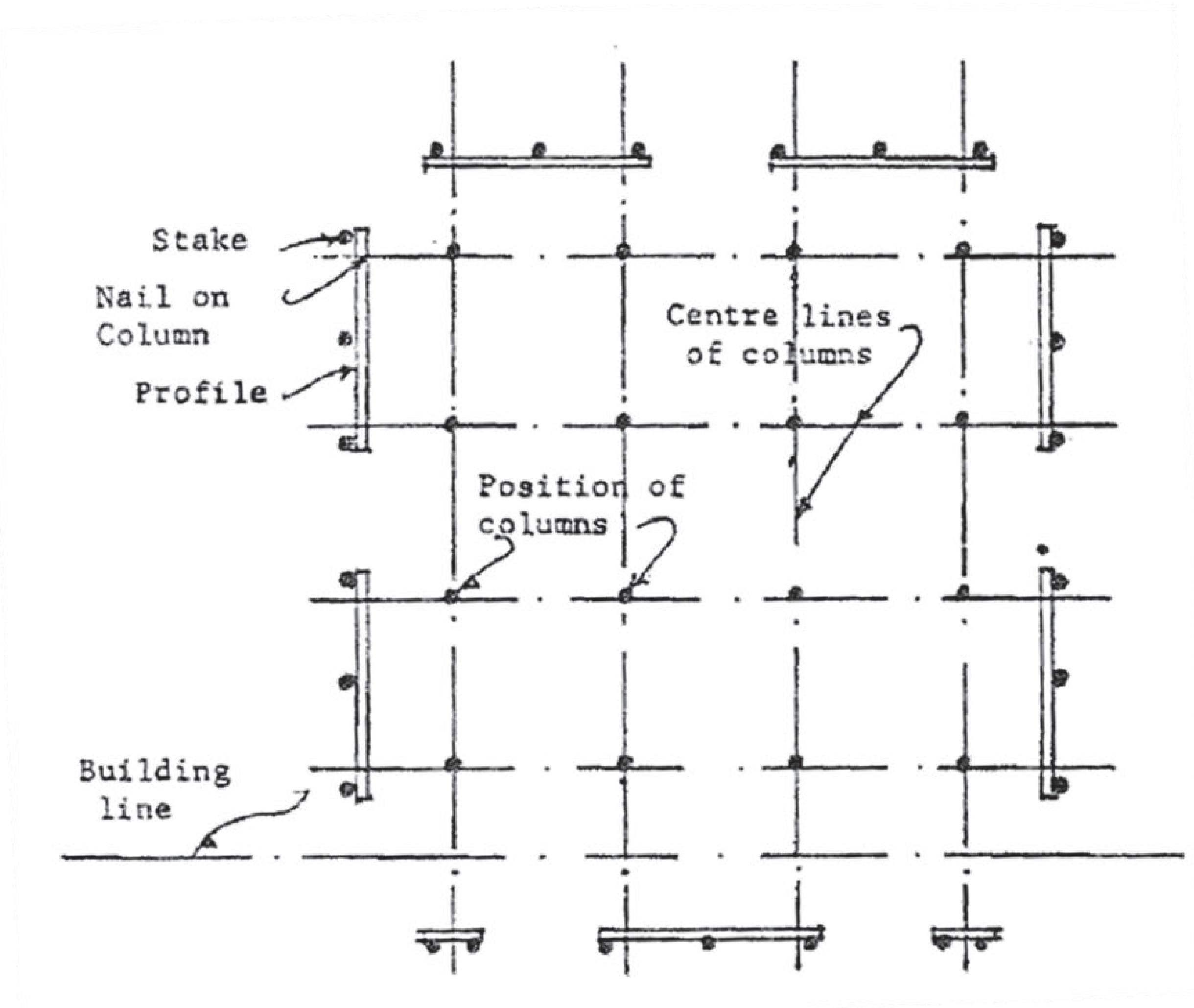


Figure 6.10 Final stage in setting out columns

6.6 Taking levels

All levels which are taken are always assumed to be in relation to a fixed level. For example, whenever the height of hills or mounts is calculated, it is determined by the distance in metres above sea level.

Similarly, the depth of a mine will be calculated by its distance in metres below sea level.

The fixed level is called a DATUM. One the site it is usually to set a datum level and all other levels are reckoned in relation to that level.

This datum will generally be predetermined by the architect or engineer responsible for the site. It is often taken from a local bench mark, which is a Trigonometrical Survey Point at a known height above sea level.

These bench marks (**Figure 6.11**) are frequently seen on the sidewalks of city streets, to use the point the cover is removed.

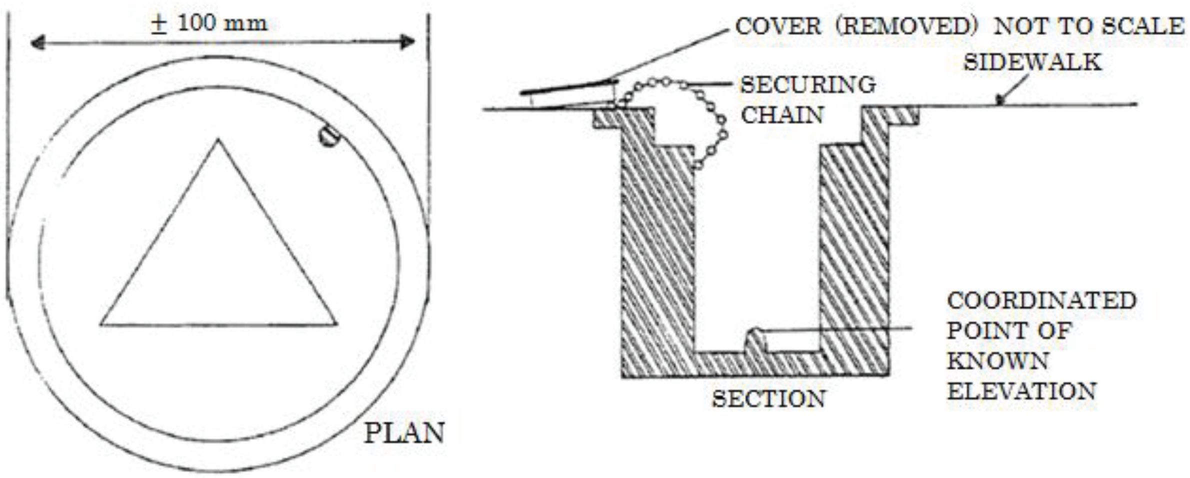


Figure 6.11

For convenience on site a datum peg is usually set so that it can readily be seen from most parts of the site.

It should preferably be placed somewhere near the site office so that it can easily be checked if it is knocked out of position by a machine.

The datum peg should be bedded in concrete and a timber framework should be built around it to prevent it, as far as possible, from becoming disturbed (**Figure 6.12**). Quite often the datum on site is the ground floor or damp-proof course, but any similar fixed point could jus as easily be used.

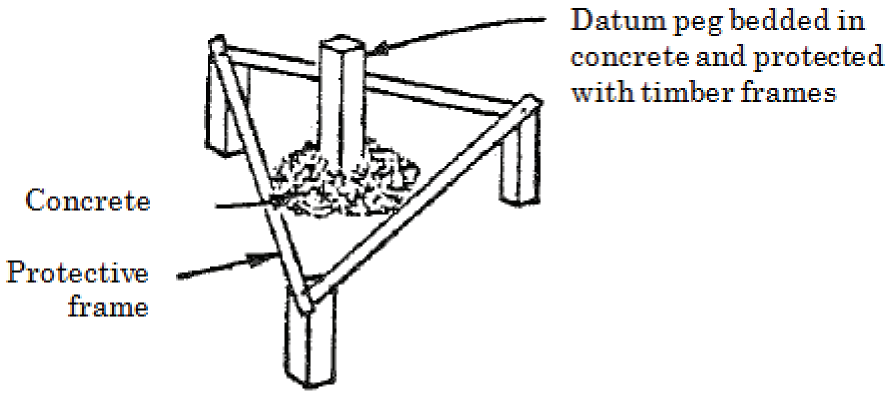


Figure 6.12

A "dog-spike" is commonly used in urban areas, this is simply a nail (See **Figure 6.13**) hammered into the tar or paving.

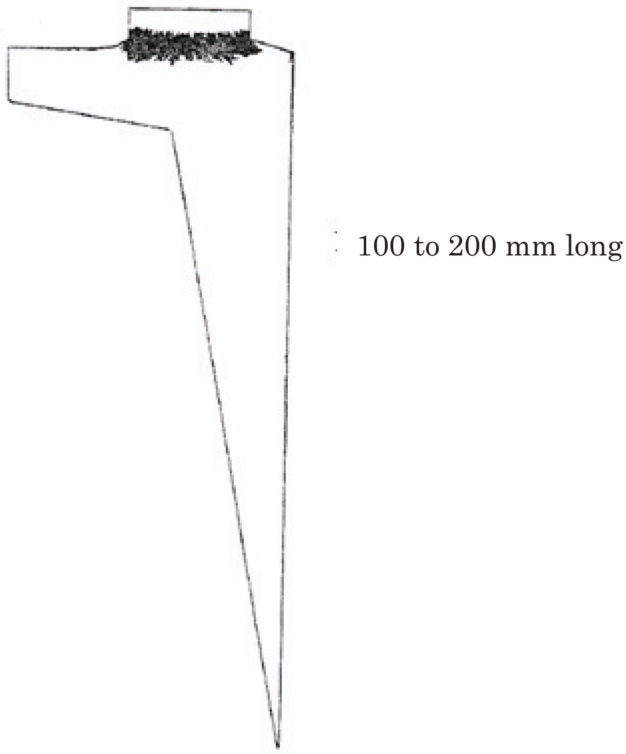


Figure 6.13

Activity 6.1

1. Briefly explain how you would go about setting out a small building.
2. Briefly describe how you would put in the concrete guide depth pegs with a straight-edge and level.
3. You have to set out a building on an inclined site. By means of neat sketches illustrate the procedure you would follow.

Activity 6.2

1. Briefly describe the method of setting out the column foundation for a rectangular building. Illustrate your answer with sketches where possible.
2. Define the following terms:
   2.1 Datum
   2.2 Bench mark

Self-Check

I am able to:	Yes	No
• Describe the setting out procedure for a simple rectangular building.		
• Describe the equipment required.		
• Describe possible constraints in setting out a building.		
• Describe the positioning of profiles and datum for a building.		
• Discuss how profiles are used with a traveller to control excavation and foundation levels.		
• Describe setting out and levelling of drainage work.		
• Explain how to invert		
o a drain		
o a sight rail		
o a traveller		
• Demonstrate calculating a suitable length of traveller and reduced levels of sight rails from given drawings.		
• Establish sight rails for horizontal position and depth control of a straight drain between manholes.		

If you have answered ‘no’ to any of the outcomes listed above, then speak to your facilitator for guidance and further development.

Past Examination Papers

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Department:
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REPUBLIC OF SOUTH AFRICA

APRIL 2013

NATIONAL CERTIFICATE

BUILDING AND STRUCTURAL SURVEYING N5

(8060045)

2 April 2013 (X-Paper)
09:00 – 12:00

Nonprogrammable calculators are allowed.

This question paper consists of 5 pages, a addendum and a formula sheet.

TIME: 3 HOURS
MARKS: 100

INSTRUCTIONS AND INFORMATION

1. Answer ALL the questions.

2. Read ALL the questions carefully.

3. Number the answers according to the numbering system used in this question paper.

4. Sketches should be neatly and clearly labelled.

5. Your understanding of the subject is what is important NOT reproduction of the study material.

6. Start every question on a NEW page.

7. Write neatly and legibly

QUESTION 1

Indicate whether the following statements are TRUE or FALSE. Choose the answer and write only 'true' or 'false' next to the question number (1.1-1.5) in the ANSWER BOOK.

1.1 A surveyor should be a registered professional.

1.2 An alternative term for a chainage is survey station.

1.3 Angular measurement is a horizontal or vertical measurement in degrees, minutes and seconds between two points from a third point.

1.4 Setting out is placing of pegs in the ground to mark out limits for a structure, foundation, road earthwork and road final levels.

1.5 Plane survey covers a relatively small area to the extent that the curvature of the earth is ignored.

(5 x 2) **[10]**

QUESTION 2

A line A-D was measured in three sections:
A-B: 92,288 mat slope of 3°44'20"
B-C: 82,408 mat a slope of 4°32'59"
C-D: 57,652 m at a slope of 2°09'07"
Find the horizontal distance A to D.

[10]

QUESTION 3

3.1 The GIVEN FIGURE 1 (ADDENDUM A, attached) of REESTON INTERNAL SERVICES AREA C'- SEWER LAYOUT details junction C1- C7
(NOTE: There are five sections of the pipeline to be considered.)
Calculate the total length of the pipe work C1 to C7.

[25]

QUESTION 4

4.1 Explain, how a horizontal distance is measured using step chaining. (5)

4.2 Two elevations are given in the same straight line, namely; 277,25 m and 277,01 m. (5)
The slope distance measured between the two points is 10 m.
Calculate the difference in elevation and thereafter, the horizontal distance between the two points.

4.3 Explain how the 3, 4, 5 method of setting a point is executed on a site. (5)

[15]

QUESTION 5

5.1 A square plot has an area of 25m². If the land is to be represented on a plan of 1 : 200, find the length of a side in millimetres. (6)

5.2 Give TWO errors that are most common when reading a staff. (4)

5.3 5.3 A slopping rectangular site has been set out. As a site surveyor, you are required to put profiles for excavation so as to level the site.

Explain how you would go about transferring your formation levels onto the profiles based on the length of your traveller. (10)

[20]

QUESTION 6

6.1 Describe the uses of the following surveying instruments.

6.1.1 Optical square

6.1.2 Ranging rod

6.1.3 Levelling instrument

6.1.4 Theodolite

6.1.5 Measuring tape (5x2)

6.2 Briefly explain the following terms:

6.2.1 Contours

6.2.2 Cadastral survey

6.2.3 Geodetic survey

6.2.4 Level line

6.2.5 Survey station (5x2)

[20]

TOTAL: 100

ADDENDUM A
REESTON INTERNAL SERVICES
AREA C - AS-BUILT SEWER LEVELS

FROM	TO	LENGTH	AS-BUILT INVERT LEVEL	AS-BUILT OVER LEVEL	DEPTH	AS-BUILT GRADE	COORDINATES	
							Y	X
	C2		198,543	199,953	1,310		-75263,37	3649934,32
C2	C1	79,300	194,415	195,500	1,085	5,33	-75243,10	3650011,00
C1	C3	49,729	186,341	189,501	1,160	12,21	-75292,88	3650010,66
C3	C7	24,033	183,434	185,080	1,846	20,42	-75316,13	3650018,79
	C4		193,192	194,750	1,558		-75309,79	3649948,55
C4	C3	66,300	188,341	189,501	1,160	7,32	-75292,88	3650010,66
	C6		188.024	188,344	1,320		-75333,84	3649988,33

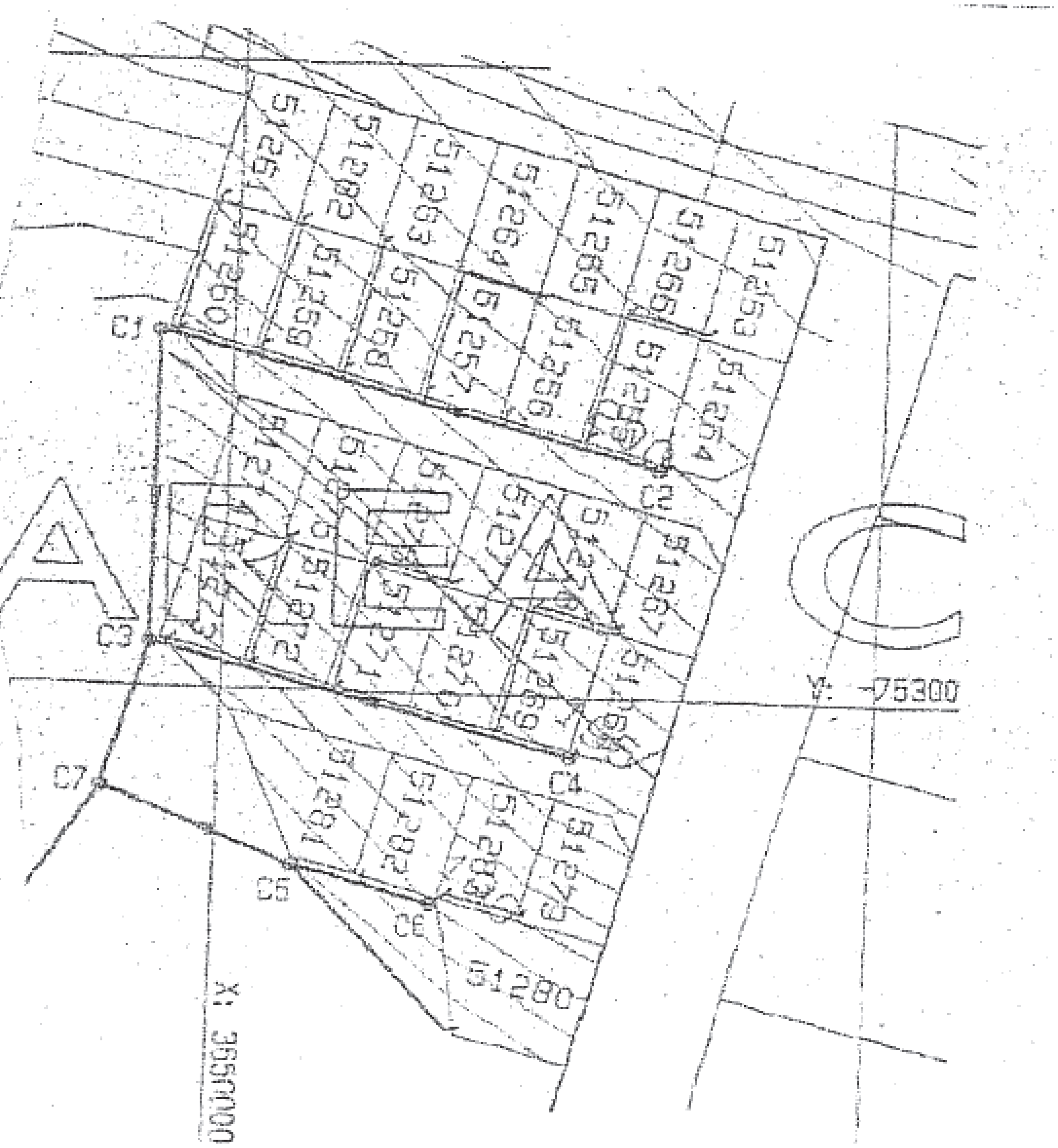

FIGURE 1

BUILDING AND STRUCTURAL SURVEYING NS

FORMULA SHEET

Any applicable formula may also be used.

$$\Delta h = 50I \sin 2\theta + HI - MH = 100I \sin\theta \cos\theta + HI - MH$$

or

$$V = -KS\ Cos\theta\ Sin\theta$$

$$HD = 100I \cos^2\theta \text{ of } KS \cos\theta$$

$$Ct = L.e.(Tm - Ts)\ ;\ Ct = L.e\ (Tm - Ts)\ of\ L[1 + e\ (Tm - Ts)]$$

$$\alpha = \tan^{-1} \frac{\Delta y}{\Delta x}$$

$$\alpha = \tan^{-1} \frac{\Delta x}{\Delta y} + 90°$$

$$\alpha = \tan^{-1} \frac{\Delta y}{\Delta x} + 180°$$

$$\alpha = \tan^{-1} \frac{\Delta x}{\Delta y} + 270°$$

$$S = \frac{\Delta y}{\sin\alpha}$$

$$S = \frac{\Delta x}{\cos\alpha}$$

$$\Delta y = s.\sin\alpha$$

$$\Delta x = s.\cos\alpha$$

$$)h = 50I \sin 22 + HI - MH = 100I \sin 2 \cos 2 + HI - MH$$

$$V = \frac{d}{3}[(y_1 + y_n) + 2(y_3 + y_5 + ... + y_{n-2}) + 4(y_2 + y_4 + ... + y_{n-1})]$$

Marking Guidelines

higher education & training

Department:
Higher Education and Training
REPUBLIC OF SOUTH AFRICA

APRIL 2013

NATIONAL CERTIFICATE

BUILDING AND STRUCTURAL SURVEYING N5

(8060045)

2 APRIL 2013

This marking guideline consists of 4 pages.

QUESTION 1

1.1 True

1.2 False

1.3 True

1.4 True

1.5 True

(5 x 2) **[10]**

QUESTION 2

AB = 92,288 Cos 3:44:20
= 92,092 m
BC = 82,408 Cos 4:32:59
= 82,148 m
CD = 57,652 Cos 2:09:07
= 57,611 m
AD = AB + BC + CD
= 92,092 + 82,148 + 57,611
= 231,851 m

[10]

QUESTION 3

3.1 C1 - C2 = [(-75 263,37-75 243,16)² + (36 499 34,32- 36 500 11 ,00) ²]
= -1 [(-20,21) ² + (-76,68) ²
= 79,300 m

C1 - C3 = [(-75 292,89- 75 243, 16) ² + (36 500 10,32- 36 500 11 ,00) ²]
= [(-49,73) ² + (-0,34) ²]
= 49,73 m

C3- C4 = [(-75 309, 7 9- 75 292 89) ² + (36 499 46,55-36 500 1 0,66) ²]
= -1 [(-16,90) ² + (-64,11) ²]
= 66,30 m

C3- C7 = [(-75 316,13- 75 292,6) ² + (36 500 16,79- 36 500 1 0,66²]
= -1 [(-23,24)2 + (-6, 13)2]
= 24,033 m

C3 - C7 = [(-75 333,84 - 75 316, 13) ² + (364 996,33 - 36 500 16, 79²]
= [(-17,71) ² + (-50,46) ²]
= 53,477 m

TOTAL = 79,30 + 49,73 + 66,30 + 24,033 + 53,477
= 272,84 m

[25]

QUESTION 4

4.1 Step chaining is conducted on site using the following instruments: tape, plumb bob, peg and ranging rods. From the peg on the ground the tape is stretched out horizontally and the distance to the tape being comfortably held at waist height a peg is punched in the ground by using a plumb-bob suspended from the horizontal distance being measured. The same operation is re-done up until the required total horizontal distance is measured. (5)

4.2 Difference in elevation = 277,5- 277,01 (5)
= 0,24 m
Horizontal distance = $[(10)^2-(0,24)]^2$
= 9,997 m

4.3 This method of setting out a right angle from a point on a given straight line is executed by three people, the first person holding the tape on the point from which the perpendicular is required at 0 and 12 m. The second person holds the tape 3 m away from the first person and the third person holds the tape at 8 m in the direction in which the perpendicular line is required and the whole system makes a right angled triangle when pulled tight by the third person. (5)

[15]

QUESTION 5

5.1 Each side = 25 m^2 (6)
= 5 m
= 5 000 mm
= 5 000 mm/200
= 25 mm

5.2 Reading the staff upwards instead of downwards (4)
Reading an inverted staff downwards instead of upwards

5.3 From the site boundaries measure set out the proposed building increasing the area by plus or minus 1 m. Punch in two pegs (plus/minus 2 m long pegs) 1m away from each corner in line with the building line in all four corners. Because of the length of the pegs a traveller of 1,5 m would be appropriate. The formation level plus/minus the benchmark, plus the length (10) of the traveller will give the staff reading on all the eight pegs sight rails.

[20]

QUESTION 6

6.1.1 Optical square is a hand held instrument used to set out lines at right angles to each other.

6.1.2 Ranging rod is used as sighting marks, intermediate markers, or markers for terminal points on a taped line.

6.1.3 Levelling instrument is mainly used to find differences in height/elevation on the surface of the earth.

6.1.4 Theodolite is used to measure vertical and horizontal angles on the surface of the earth and it can only be used for levelling work.

6.1.5 Measuring tape is mainly used to measure horizontal distances on the surface of the earth. (5x2)

6.2.1 Contours are lines on the surface of the earth joining points of the same height above mean sea level.

6.2.2 Cadastral survey is that branch of surveying concerned with property boundaries.

6.2.3 Geodetic surveying concerned with the large areas on the surface o the earth to the extent that the curved nature of the earth cannot be ignored.

6.2.4 Level line is a line which lies in the level surface and is therefore normal to the direction of gravity at all times.

6.2.5 A peg/marker in the ground to be used as a boundary peg, control point, benchmark, etc. (5x2)

[20]

TOTAL: 100

Past Examination Papers

higher education & training
Department: Higher Education and Training
REPUBLIC OF SOUTH AFRICA

NOVEMBER 2012

NATIONAL CERTIFICATE

BUILDING AND STRUCTURAL SURVEYING N5

(8060045)

22 November 2012 (X-Paper)
09:00 – 12:00

Non-programmable calculators are allowed.

This question paper consists of 4 pages, a table, 1 addendum and a 1-page formula sheet.

TIME: 3 HOURS
MARKS: 100

INSTRUCTIONS AND INFORMATION

1. Answer ALL the questions.
2. Read ALL the questions carefully.
3. Number the answers according to the numbering system used in this question paper.
4. Sketches should be neatly and clearly labelled.
5. Your understanding of the subject is what is important NOT reproduction of the study material.
6. Start each question on a NEW page.
7. Write neatly and legibly

QUESTION 1

Choose the correct word(s) from those given in brackets. Write only the word(s) next to the question number (1.1 - 1.5) in the ANSWER BOOK.

1.1 An instrument is a common site name given to any (surveying instrument on a tripod/hand-held instrument/tape measure).

1.2 A traveller is mainly used to control (an excavation/a pipe/laying/water mains hydraulic pressure).

1.3 One of the basic requirements when setting up a levelling instrument is to (adjust the circular bubble to be in its centre/make sure that the tripod feet are not firmly forced on the ground /make sure that the top of the tripod is sloping gently).

1.4 A theodolite can be used to set out (vertical angles/horizontal angles/levelling/all three).

1.5 Errors are classified as (gross errors/systematic errors/accidental errors/all three).

(5 x 2) **[10]**

QUESTION 2

Indicate whether the following statements are TRUE or FALSE. Choose the answer and write only 'true' or 'false' next to the question number (2.1 - 2.5) in the ANSWER BOOK.

2.1 Distometer is mainly used to measure distance and is mounted on a theodolite.

2.2 Dumpy is a common site name given to any levelling instrument.

2.3 Some of the uses of the theodolite is to measure horizontal and vertical angles.

2.4 A plumb bob is used for all levelling geodetic surveys.

2.5 A plumb is one of the instruments used when conducting step chaining.

(5 x 2) **[10]**

QUESTION 3

3.1 The scale on a plan has been omitted. A line on the plan, measuring 15 cm, is exactly 300 m long on the ground. Determine the scale of the plan. (5)

3.2 The distance between two pegs M and N is 7,063 mas measured on a slope.
The height measured on the pegs are M = 1,814 m and N = 0,587 m.
Find the horizontal distance. (5)

[10]

QUESTION 4

Briefly explain each of the following terms:

4.1 Chaining (3)

4.2 The site (3)

4.3 Chainage (3)

4.4 Level line (3)

4.5 Angular measurement (3)

[15]

QUESTION 5

5.1 The following notes were taken during a levelling survey between A and B. Reduce the readings using the rise and fall method. Use the TABLE 1 (attached) to answer the question and submit it with the ANSWER BOOK. (20)

5.2 Explain, with the aid of a sketch, how one would chain if the chain line is obstructed by a building. (10)

[30]

QUESTION 6

6.1 A line E- H was measured in three sections:
E- F: 90,288 mat a slope of 3°44'20"
F- G: 72,408 mat a slope of 4°32'59"
G- H: 47,652 mat a slope of 2°09'07"
Calculate the horizontal distance E to H. (10)

6.2 Explain why we make temperature correction on a steel tape. (5)

[15]

QUESTION 7

7.1 A sewer's internal reticulation longitudinal section MH020-MH026 is shown

in FIGURE 1 ANNENDUM A (attached).
Calculate each of the following:

7.1 The length of the pipeline, between MH023 and MH025 (5)

7.2 The slope of the terrain between MH024 and MH025 (5)

[10]

TOTAL: 100

EXAMINATION NUMBER:

TABLE 1

POINT	BACK SIGHT	INTER SIGHT	FORE SIGHT	RISE	FALL	REDUCED LEVEL	REMARKS
A	0,49						
B	0,27		3,29				
C	0,39		3,77				
D	3,72		3,59				
E		1,11					
F	3,56		0,82				
G	3,89		1,36				
H	3,72		0,99				
I	3,69		1,02				
J	3,86		1,31				
K	3,90		1,56				BM 1275,00
L			2,40				

ADDENDUM A

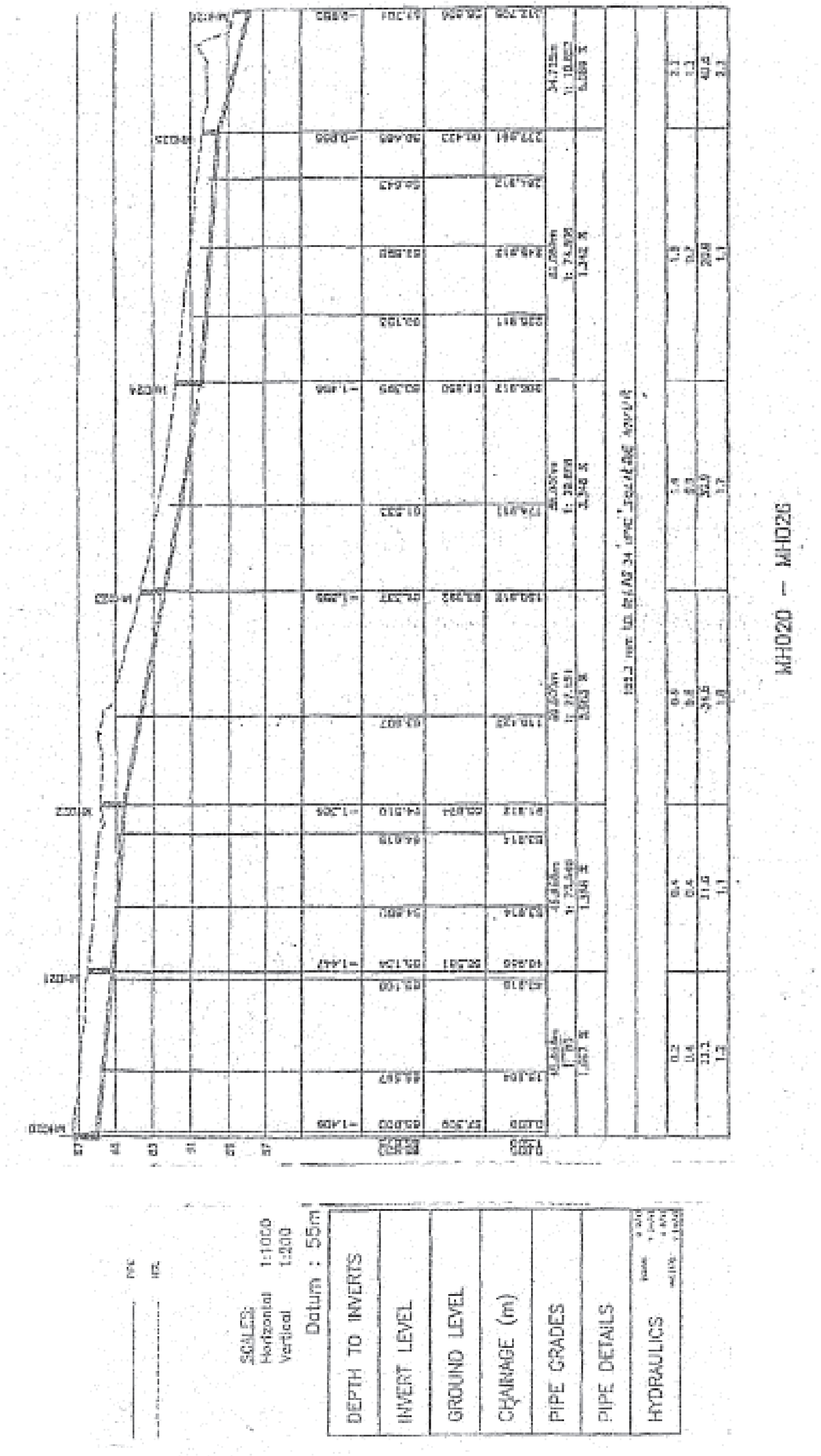

FIGURE 1

BUILDING AND STRUCTURAL SURVEYING NS

FORMULA SHEET

Any applicable formula may also be used.

$$\Delta h = 50I \sin 2\theta + HI - MH = 100I \sin\theta \cos\theta + HI - MH$$

or

$$V = -KS\ Cos\theta\ Sin\theta$$

$$HD = 100I \cos^2\theta \text{ or } KS \cos\theta$$

$$Ct = L.e.(Tm - Ts)\ ;\ Ct = L.e\ (Tm - Ts)\ of\ L[1 + e\ (Tm - Ts)]$$

$$Cs = L.(1 - \cos\theta)$$

$$C\varepsilon = \frac{L\ .\ H}{R}$$

$$\text{Slope} = \frac{\Delta h}{HD}$$

$$V = \frac{d}{3}[(y_1 + y_n) + 2(y_3 + y_5 + ... + y_{n-2}) + 4(y_2 + y_4 + ... + y_{n-1})]$$

$$\alpha = \tan^{-1}\frac{\Delta y}{\Delta x}$$

$$\alpha = \tan^{-1}\frac{\Delta x}{\Delta y} + 90°$$

$$\alpha = \tan^{-1}\frac{\Delta y}{\Delta x} + 180°$$

$$\alpha = \tan^{-1}\frac{\Delta x}{\Delta y} + 270°$$

$$S = \frac{\Delta y}{\sin\alpha}$$

$$S = \frac{\Delta x}{\cos\alpha}$$

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NOVEMBER 2012

NATIONAL CERTIFICATE

BUILDING AND STRUCTURAL SURVEYING N5

(8060045)

22 NOVEMBER 2012

This marking guideline consists of 5 pages.

QUESTION 1

1.1 Surveying instrument on a tripod

1.2 Control an excavation

1.3 Adjust the circular bubble to be in its centre

1.4 All three

1.5 All three

(5 x 2) **[10]**

QUESTION 2

2.1 True

2.2 True

2.3 True

2.4 False

2.5 True

(5 x 2) **[10]**

QUESTION 3

3.1 300 m on the ground and 15 cm on the plan.
= 300 m x 100 cm/15 cm (5)
= 2 000 cm
Thus Scale on the plan is 1:2000

3.2 3.2 Difference in height = 1.814-0.587
= 1,227 m
Horizontal Distance = $[(7,063)^2-(1,227)^2]$
= 6,956 m (5)

[10]

QUESTION 4

4.1 Chaining is the common site name given to all taping work. (3)

4.2 The site is the area for a plot or terrain to build on (3)

4.3 Chainage is the position of a point from the origin. (3)

4.4 Level line is a line which lies in the level surface and is therefore normal to (3) the direction of gravity at all points.

4.5 An angular measurement degree amounts from contour as beacon line and the unit of measurement is 0°0'0" (3)

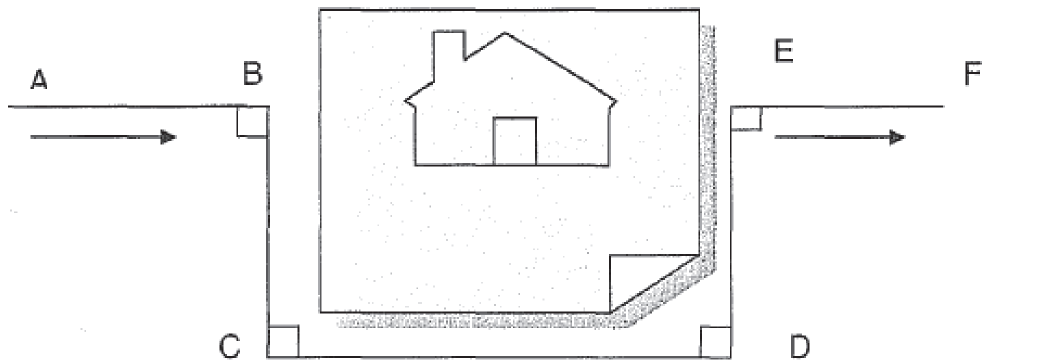
[15]

QUESTION 5

5.1 (20)

POINT	BACK SIGHT	INTER SIGHT	FORE SIGHT	RISE	FALL	REDUCED LEVEL	REMARKS
A	0.49					1280.880√	
B	0.27		3.29		2.800√	1283.680√	
C	0.39		3.77		3.500√	1287.180√	
D	3.72		3.59		3.200√	1290.380√	
E		1.11		2.610√		1287.770√	
F	3.56		0.82	0.290√		1287.480√	
G	3.89		1.36	2.200√		1285.280√	
H	3.72		0.99	2.900√		1282.380√	
I	3.69		1.02	2.700√		1279.680√	
J	3.86		1.31	2.380√		1277.300√	
K	3.90		1.56	2.300√		1275.000√	BM 1275.000
L			2.40	1.500√		1275.000√	

5.2 (10)



At two points A and B erect perpendicular line BC. At line BC erect another perpendicular line CD, to clear the obstacle. At CD erect a perpendicular line DE and equal in length to BC and at DE set up a right angle EF. The direction EF is the extension of the survey line and distance CD=BE.

[30]

QUESTION 6

6.1 E - F = 90,288m x Cos 3°44'20"
= 90,096m
F - G = 72,408m x Cos 4 °32'59"
= 72, 175m

G - H = 47,652m x Cos 2°09'07" (10)
= 47,618m
E - H = 90,096m + 72, 175m + 47,618m
= 209,889m

6.2 The contraction of steel tapes in very cold weather and expansion on very hot days causes the steel tape to become short or long. This can cause error on the reading taken, therefore corrections have to be made. (5)

[15]

QUESTION 7

7.1 MH023- MH024 (5)
Length = $58^2 + 1{,}942^2$
= 58,032 m
MH024- MH025
Length= $69{,}069^2 + 0{,}927^2$
= 69,075 m
Total Length= 58,032 + 69,075
= 127,112 m

7.2 Slope (5)
$\text{Tan}^{-1}\,\theta$ = 1 ,427/69,069
= 1°11 '00"
OR
Slope= 1 ,427/69,069
= 0,08066 x 100
= 2,07%

[10]

TOTAL: 100

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Department:
Higher Education and Training
REPUBLIC OF SOUTH AFRICA

AUGUST 2012

NATIONAL CERTIFICATE

BUILDING AND STRUCTURAL SURVEYING N5

(8060045)

27 July 2012 (X-Paper)
09:00 – 12:00

Non-programmable calculators are allowed.

This question paper consists of 4 pages, 2 diagrams and a 1-page formula sheet.

TIME: 3 HOURS
MARKS: 100

INSTRUCTIONS AND INFORMATION

1. Answer ALL the questions.
2. Read ALL the questions carefully.
3. Sketches should be neatly and clearly labelled.
4. Your understanding of the subject is what is important NOT reproduction of the study material.
5. Start each question on a NEW page.
6. Number the answers according to the numbering system used in this question paper.
7. Write neatly and legibly

QUESTION 1

Scrutinise the attached cadastral diagram, FIGURE 1, ANNEXURE A (attached) and indicate whether the following statements are TRUE or FALSE. Choose the answer and write only 'true' or 'false' next to the question number (1.1 - 1.5) in the ANSWERBOOK.

1.1 ALL the private roads are 10 m wide. (2)

1.2 The total number of erven illustrated is thirty. (2)

1.3 MHOZO-MH037 are illustrated manholes/inspection chambers. (2)

1.4 The scale is 1:1 000. (2)

1.5 Erf 191 is illustrated as a private space (POS). (2)

[10]

QUESTION 2

2.1 The co-ordinates of points A and B are:

	Y	X
A	+ 10174,36	+ 10149,75
B	+ 10000,00	+ 10000,00

Calculate direction and distance AB. Any method is acceptable as long as it will give the correct answer. (15)

2.2 On site, explain how to set out the points A and B stationed on a survey station of known co-ordinates. (Explain do NOT show any calculations). Assume that point B is also in existence. (5)

2.3 State FIVE requirements to obtain sufficient accuracy when taping. (5)

[25]

QUESTION 3

3.1 A square plot has an area of $16m^2$. If the land is to be represented on a plan 1:150, find the length in millimetres. (6)

3.2 State TWO characteristics of contours according to the slope to the terrain. (4)

3.3 A sloping rectangular site has to be set out. As site surveyor, you are required to put profiles for excavation so as to level the site. Explain how you would go about transferring your information levels onto the profiles based on the length of your traveller. (10)

[20]

QUESTION 4

The given FIGURE 2, ANNEXURE 8 (attached) of Reeston Internal Services Area C'SEWER LAYOUT gives details of junction C1- C7.
NOTE: There are FIVE sections of the pipeline to be considered.

4.1 Calculate the total length of pipe work C1 to C7. (25)

4.2 The standard temperature of the tape is 16 °C and the coefficient of expansion is 0,00012/°C. What is the reduced horizontal distance if the measured distance is 348 m in a slope of 5°50' and a temperature of 32 °C? (5)

[30]

QUESTION 5

5.1 The following off-sets were taken from baseline to the shoreline of a site that is along the coast at 10 m intervals for a distance of 1 00 m. The offset were 75 m, 85 m, 95 m, 105 m, 110 m, 100 m, 98 m and 80 m. Calculate the area of the site. (5)

5.2 Explain how we go about setting out a rectangular building site for the removal of topsoil. Include, in your explanation, any FIVE instruments that can be used. (10)

[15]

TOTAL: 100

ANNEXURE A

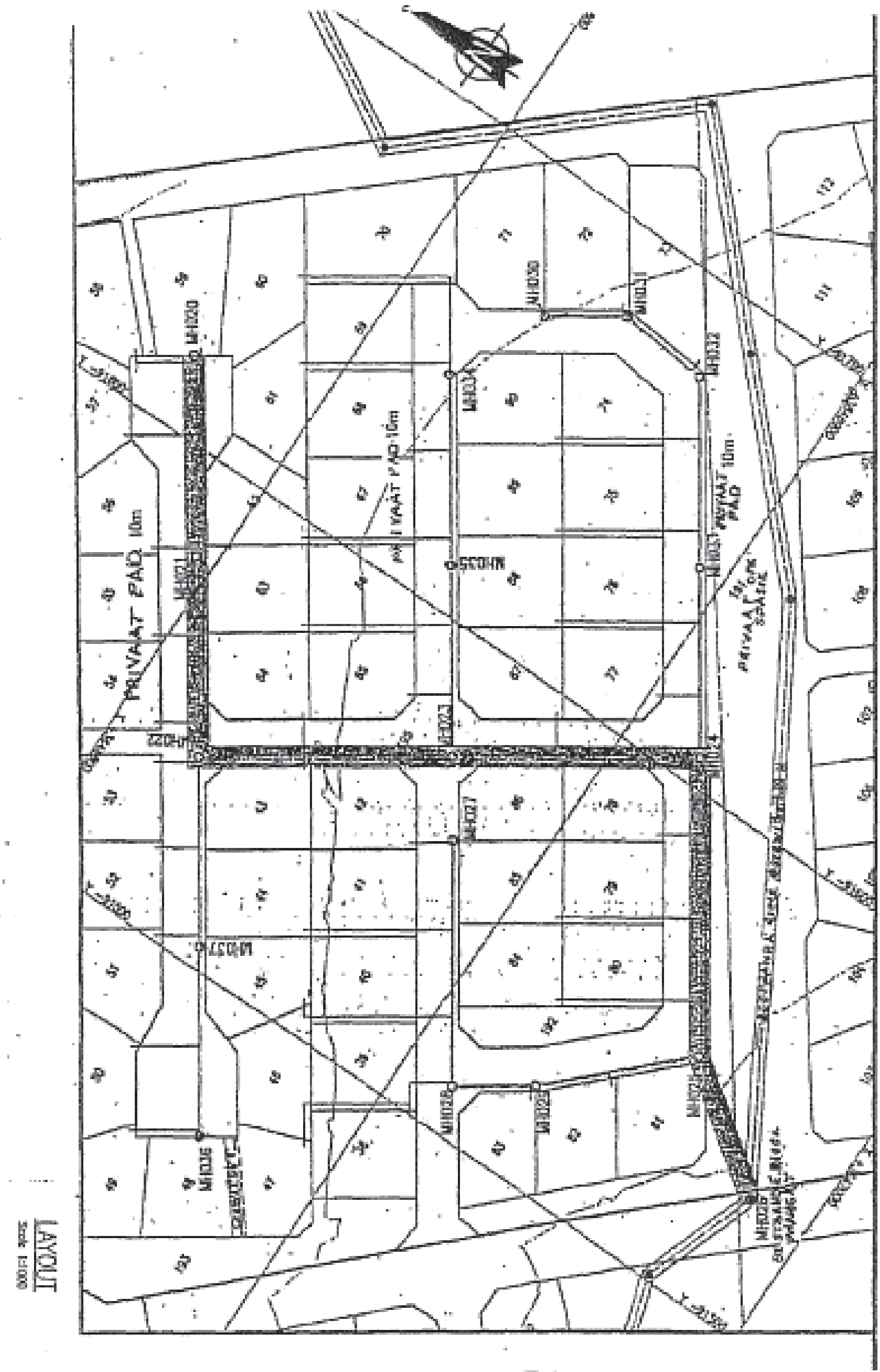

FIGURE 1

REESTON INTERNAL SERVICES

AREA C – AS-BUILT SEWER LEVELS

FROM	TO	Length	As-built Invert Level	As-built Cover Level	Depth	As-built Grade	Co-ordinates Y	Y
	C2		198.643	199.953	1.310		-75263.37	-3649934.32
C2	C1	79.300	194.415	195.500	1.085	5.33	-75243.16	3650011.00
C1	C3	49.729	188.341	189.501	1.160	12.21	-75292.89	3650010.66
C3	C7	24.033	183.434	185.080	1.646	20.42	-75316.13	3650018.79
	C4		193.192	194.750	1.558		-75309.79	3649946.55
C4	C3	66.300	188.341	189.501	1.160	7.32	-75292.89	3650010.66
	C6		188.024	189.344	1.320		-75323.84	-3649966.33

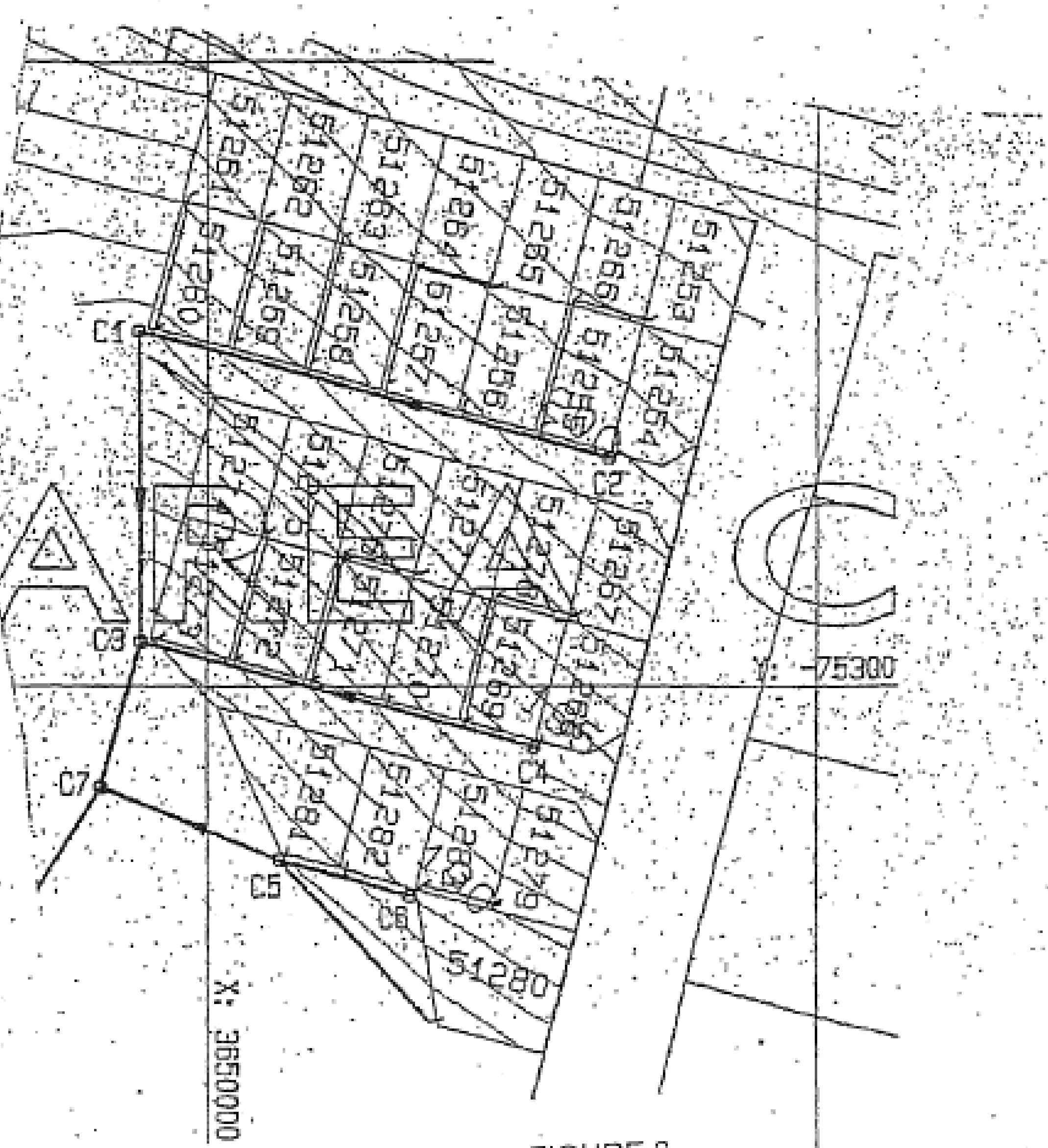

FIGURE 2

BUILDING AND STRUCTURAL SURVEYING NS

FORMULA SHEET

Any applicable formula may also be used.

$\Delta h = 50l\sin 2\Theta + HI - MH = 100l\sin\Theta\cos\Theta + HI - MH$

Or

$V = - KS\cos\Theta\sin\Theta$

$HD = 100/\cos^2\Theta$ of $KS\cos\Theta$

$Ct = L.e.(Tm-Ts)$, $Ct=L.e(Tm-Ts)$ of $L[1+e(Tm-Ts)]$

$Cs = L.(1-\cos\Theta)$

$Cs = H(\sec\Theta-1)$

$Ce = {}^{LH}/_{R}$

$Slope = {}^{\Delta h}/_{HD}$

$V = {}^{d}/_{3}[(y_1+y_n) + 2(y_3+y_5+\ldots+y_{n-2}) + 4(y_2+y_4+\ldots+y_{n-1})]$

$\alpha = \tan^{-1}{}^{\Delta y}/_{\Delta x}$

$\alpha = \tan^{-1}{}^{\Delta x}/_{\Delta y}+90°$

$\alpha = \tan^{-1}{}^{\Delta y}/_{\Delta x}+180°$

$\alpha = \tan^{-1}{}^{\Delta x}/_{\Delta y}+270°$

$S = {}^{\Delta y}/_{\sin\alpha}$

$S = {}^{\Delta x}/_{\cos\alpha}$

Marking Guidelines

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This marking guideline consists of 4 pages.

QUESTION 1

1.1 True (2)

1.2 False (2)

1.3 True (2)

1.4 True (2)

1.5 True (2)

[10]

QUESTION 2

2.1

JOIN CALCULATION				
Station	Y	X	Calculations	Direction/D Distance/S
A B	+10 147,36 +10000,00	+10 149,75 +10000,00	=tan^{-1} -147,363/ . 149,750 √ = 44°32'21" √	D = 180° + 44°32'21"√ = 224°32'21"√
	-147,360 √√	-149,750 √√		
			Check	S = $\sqrt{\Delta y^2 + \Delta x^2}$ √ = √ $[(-147{,}360)2 + (-149{,}750)2]$ =210,095m √√
			210,095Cos 224°32'21"	
			= -149, 75 m√	
			210,095 Sin 224°32'21" = -147,36 √	

(15)

2.2 Set up the theodolite on the survey station S. Calculate the direction SA and SB and find the angle between SA and SB. Orientate zero degree on ranging rod A and hence swing to unknown B by directing the assistant with a ranging rod. (5)

2.3 Tape must be held horizontal.
Tape must be held on its correct zero mark.
The correct/sufficient tension must be applied to the tape.
Tape must be held on the correct peg.
View tape vertically over the peg.
Measure to the centre of a ranging rod. (5)

[25]

QUESTION 3

3.1 Each side = $16m^2$
= 4m (6)
= 4000mm
Thus 4000/150
= 26,667mm

3.2 The contours are further apart when the terrain is gentle. (4)
The contours are close together when the terrain is a steep slope.

3.3 From the site boundaries measure set out the proposed building increasing the area by plus or minus 1 m. Punch in two pegs (plus/minus 2m long pegs) 1m away from each corner in line with the building line in all four corners. Because of the length of the pegs a traveller of 1,5 m would be appropriate. The information level plus/minus the benchmark, plus the length of the traveller will give the staff reading on all the eight pegs sight rails. (10)

[20]

QUESTION 4

4.1 C1-C2= [(-75263,37 -75243,16)² + (3649 934,32-3650 011,00) ²] (25)
= [(- 20.21) ² + (-76,68) ²]
= 79,300m
C1-C3= [(-75292,89 -75243,16) ² + (3650010,66-3650011,00) ²] C1-C2=
= [(-49,73) ² +(-0,34) ²]
= 49,73m
C3-C4= [(-75309,79 -75292,89) ² + (3649946,55- 365001 0,66) ²]
= [(-16,900) ² + (-64,11) ²]
= 66,30m
C3-C7= [(-75316,13-75292,89) ² + (3650016,75 -3650010,66) ²]
= [(-23,24) ² + (6,13) ²]
= 24,033m
C7-C6= [(-75333,84-75316,13) ² + (3649966,33- 3650016,79) ²]
= [(-17,71) ² +(-50,46) ²]
= 53,477m
TOTAL= 79,30 +49,73 + 66,30 +24,033 + 53,477
= 272,84M

4.2 Correction = MD x coe x (T_2 - T^1)
= 348 x 0,00012/°C x (32°C -16°C)
= 0,668m (5)
Correction Slope = MD (1- Cosθ)
= 348 (1- Cos5°50')
= 1,802 m
Correct Distance = 348 + 0,668- 1,802

= 346,866m

[30]

QUESTION 5

5.1 Area= $^{10}/_{2}$ [($^{75+80}$)/$_{2}$ + 85 + 95 + 105 + 125 + 110 + 110 + 100 + 98
= 4527,50 m^2 (5)

5.2 Measure the distance of the proposed structure from all four corners and make it plus/minus 1 m less ach side of the building. Put the steel pegs or droppers on these new found points and mark them for a 2m traveller and take the 150 mm depth of top soil into consideration.

Equipments :+- 2m Traveler
:Levelling instrument
:Fish line
:Lime
:+4 x 2m steel pegs or droppers
:+-100m tape (10)

[15]

TOTAL: 100

N5 Building and Structural Surveying is one of many publications introducing the gateways to Civil Engineering Studies. This course is designed to develop the skills for learners that are studying toward an artisanship in the Building and Civil Engineering fields and to assist them to achieve their full potential in a building construction career.

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